Prepared By

United States
Department of the Interior
Bureau of Reclamation

for the

Foreign Operations Administration

DEVELOPMENT PLAN FOR THE LITANI RIVER BASIN REPUBLIC OF LEBANON

VOLUME III

GENERAL PLAN

ULTIMATE DEVELOPMENT

Litani River Basin Investigation Staff

Beirut, Lebanon

June, 1954

General View--Upper Nabatiye Plateau
Village of Nabatiye in Middle Background

FOREWORD

The Litani River Basin in Lebanon is a rural region of rugged mountains and fertile plains. The people live in villages along the lower mountain slopes and tend their crops and pasture their herds on the valley lands. Generally, agricultural methods are primitive, the land is plowed by ox team or dug by hand, and harvesting is commonly done with hand sickle and threshed with an ox-drawn wooden sled. For the most part the standard of living is very low.

Although the region through which flows the Litani is one of the principal lifelines of the country, the river's potentialities have remained neglected and little attempt has been made to conserve its resources. Numerous superficial studies of the river's possibilities have been made during past years, but these have provided no adequate basis for full development. The current report, submitted herewith, is not only detailed but also eminently practicable and feasible.

In April 1951 under the sponsorship of the Technical Cooperation Administration (now the Foreign Operations Administration), a reconnaissance of the Litani River Basin was undertaken by the Bureau of Reclamation to investigate the feasibility of a basin-wide development, and, if warranted, to formulate a program for a more detailed investigation. A report of this survey entitled, "Reconnaissance Report of the Litani River Project, Lebanon," was submitted to the Technical Cooperation Administration in June of that year. This report proposed a program for further investigations to be directed toward a plan for integrated basin development with specific emphasis on certain features which could be scheduled for early construction. Because of the great urgency and importance attached to this project, the Administrator of the United States Technical Cooperation Administration approved the program as proposed by the Bureau, and requested it to proceed with arrangements for continuation of the investigations.

Under authority of Project Authorization No. 106-446, approved November 6, 1951 by the Administrator, Technical Cooperation Administration, Department of State, the detailed investigations were undertaken by the Bureau. A comprehensive Basin Plan is now developed, preconstruction studies are completed for units recommended for initial financing and construction, and the planning investigations are completed for the remaining units comprising the basin plan. The staff of the Assistant Commissioner and Chief Engineer has contributed to the designs of the various features of the initial project, and has reviewed this report and concurs in the general conclusions and recommendations. The report covering these investigations is presented herewith in three basic volumes and three volumes of appendices of substantiating material as follows:

Volume I--Is a general description of the results of the investigation and studies

Volume II--Is a detailed presentation of the results of the planning and preconstruction investigations of the units selected for initial financing and construction

Volume III--Is a presentation of the results of the planning investigations of the units remaining in the project other than those covered by Volume II

Appendix to Section III--Hydrology

Appendix to Sections IV and VII--Geology and Ground Water Investigations

Appendix to Section VI--Power.

The intent of these investigations is to make the maximum use of water for irrigation purposes to supplement the meager food and forage supply and thereafter to produce much-needed electrical energy.

To achieve maximum power benefits it will be necessary to coordinate the operation of the proposed hydroelectric plants with thermal electric capacity in order to avoid the production of large quantities of lower-valued dump or secondary energy. There is a wide fluctuation between wet and dry seasons and between average and critically dry years. The financial analysis deals with the coordinated system but the details of the project as covered in this report deal only with the hydroelectric power and irrigation facilities.

The report includes a power market study which shows that Lebanon would easily absorb, within 25 years, the entire hydroelectric output of the project together with the thermal electric energy necessary to coordinate the hydro output.

The financial analysis for the project, including both the proposed hydroelectric power and irrigation facilities, indicates that the entire construction cost can be repaid in 40 years with interest on the unamortized balances at 6 percent per annum, including all operation, maintenance, and replacement costs with no increase, and possibly some decrease, in the power rates prevailing in Beirut.

The recommended plan for development provides for the construction of the Karaoun Dam and Reservoir on the Litani River, the diversion of a portion of the Litani water through the Lebanon Mountain Range into the Bisri River Basin, and the multiple-purpose use of the water in both basins for irrigation and power development. The plan calls for the construction of a low dam on the Bisri River to regulate the flow from Karaoun, and the construction of the Khardale Dam on the lower Litani to regulate the releases from Karaoun Reservoir and to store the water accruing to the Litani below Karaoun Dam. The hydroelectric power is to be coordinated through an interconnected transmission system with existing and future thermal power developments in Lebanon.

Provision is made in the plan for the installation of 171,000 kilowatts in the power units and the construction of a transmission system for operation at 69,000 volts. It is estimated that 626 million kilowatt-hours of energy will be produced by the completed hydroelectric project in an average year. The plan calls for the construction of irrigation works which will furnish a full water supply to approximately 18,600 hectares (46,000 acres) of land and a partial supply to an additional 2900 hectares (7,200 acres).

The units recommended for initial construction covered by this report consist of the first stage of Karaoun Dam and Reservoir, the Sohmor Tunnel and Power Plant immediately below Karaoun Dam, the Bisri Tunnel through the Lebanon Mountain Range, the Bisri Power Plant with two units below Bisri Tunnel, the necessary transmission facilities for these units, and the Bekaa Gravity Irrigation Unit consisting of 5700 hectares (14,000 acres) in an area above Karaoun Reservoir. The total installed capacity of the hydroelectric units to be constructed initially will be 38,000 kilowatts.

The total estimated cost of the project, taking into consideration the Lebanese price levels prevalent in December 1953, is 341,920,000 Lebanese Pounds ($97,800,000). After allocating the joint facility costs, the cost of the power facilities is 250,920,000 Lebanese Pounds ($71,800,000) and the cost of the irrigation facilities is 91,000,000 Lebanese Pounds ($26,000,000). The cost to construct the initial features (Phase "A") is estimated to be 117,071,000 Lebanese Pounds ($33,500,000), of which 5,500,000 Lebanese Pounds ($1,570,000) is the estimated cost to construct the works for the Bekaa Gravity Irrigation System. The reimbursability of the irrigation costs depends upon determination by the government of Lebanon as to whether they will assess the costs of construction against the land or attempt to cover the costs by other means. In this report these costs are considered to be in excess of those which can be repaid by the farmer, and in the financial analysis this excess is assumed to be repaid out of power revenues.

As a result of these investigations and studies it is concluded in summary that the plan of development presented in this report is physically feasible and economically justified.

The report recommends:

1. That the plan of development of the water resources of the Litani and Bisri River Basins described herein be adopted by the government of Lebanon;

2. That this plan be executed by the government of Lebanon at approximately the rate of development shown and that the several units be constructed in the sequence indicated;

3. That construction be undertaken on the units included in the initial phase as soon as their financing can be arranged and the detailed plans and specifications can be prepared;

4. That construction and operation of the thermal power, required to supplement and make economically feasible the irrigation and hydro power development included in this plan, be developed in accordance with the schedule presented.

[redacted]

R. F. Herdman
Project Engineer

Beirut, Lebanon
June 15, 1954

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TRANSLITERATION

Certain inconsistencies in the spelling of place names may be noted on maps and in the text. Because of the difficulty in transliterating Arabic words into exact French or English equivalents, a wide variation of spellings occur in the original basic maps and documents used for this report. Because many of the maps presented have been reproduced in their original form, it has been impractical to use the same spelling for every place name in all instances. It will be noted, however, that the phonetic pronunciation of names is similar regardless of the spelling.

Other inconsistencies occur in the use of certain Arabic words which are often retained in the French and English versions of proper names. For instance, the word "Nahr" is the Arabic word for "river", "Jebel" is "mountain", "Ain" is "spring", etc. Local usage has often dictated the use of certain of these Arabic words on maps and in the text.

Local usage has also required the use of the names "Awali" and "Bisri" for the same river. This river is known as the Bisri River from the confluence of the Barouk and Bhanine Rivers down to about 23 kilometers from the sea, from this point to its mouth it is known as the Awali River.

CONVERSION FACTORS

Metric - English Systems

1 meter (m) = 39.4 inches = 3.28 feet

1 kilometer (km) = 0.621 miles

1 square meter (sq m) = 1.20 square yards = 10.8 square feet

1 hectare = 10,000 square meters = 2.47 acres

1 cubic meter = 1,000 liters = 1.31 cubic yards = 35.315 cubic feet

1 liter = 0.264 gallons

1 million cubic meters = 810.72 acre-feet

1 kilogram = 2.20 pounds

1 metric ton = 2205 pounds

1 kilogram/square centimeter = 14.2 pounds/square inch

1 cubic meter per second by 1 meter head = 10 kilowatts at 100 percent efficiency

1 million cubic meters by 1 meter head = 2,700 kilowatt-hours at 100 percent efficiency

Lebanese - United States Monetary Values

1 dollar ($) = 3.50 pounds (L. L.)

1 pound (L. L.) = 100 piastres = 0.286 dollars

1 pound per cubic meter = 22 cents per cubic yard

1 pound per kilogram = 13 cents per pound

1 pound per hectare = 12 cents per acre

SECTION XX

INTRODUCTION

The recommended plan of development for the Litani and Bisri River Basins and its general engineering features are contained in Volume I and the basic data upon which it is based are included in the appendices accompanying this report. The general data and discussion included in Volume I form the basis for the recommended plan for development.

This volume contains a project planning report, covering the investigations, designs, plans, construction and operation schedules, and cost estimates for all the units in the recommended plan of development after selection of those units recommended for initial or Phase "A" construction included in Volume II. The plans and details contained herein are the basis for the determination of over-all project feasibility and economic justification. Further detailed studies will be required before their construction is undertaken.

Authority

The first stage of these investigations was carried out during Fiscal Year 1951 under Project Authorization No. (1) 1466931 and covered by a report entitled "Reconnaissance Report, Litani River Project, Lebanon," prepared by the Bureau of Reclamation in June 1951. The second stage of the investigations has been carried out under authority of Project Authorization No. 106-446 approved by Administrator Henry G. Bennett, Technical Cooperation Administration, United States Department of State, November 6, 1951. The nature of the assistance to be given to the Republic of Lebanon in the second stage investigation is indicated in the project authorization, briefly as follows:

> A directive to carry out the second stage of the detailed investigation of the economic and engineering feasibility of a basin-wide development of the Litani River Basin. These investigations to include other river basins in Lebanon where integrated development with the Litani was found desirable. A program of field operations to be directed toward an integrated basin plan with specific emphasis on those units which, having independent justification, might be scheduled for early construction. Preparation of detailed project reports and designs to serve as the basis for financing and for preparation of the detailed plans and specifications.

Plan of Development

The recommended plan of development includes a transmountain diversion of a portion of the Litani River waters into the Bisri River Basin, and the multiple purpose use of the water in both basins for irrigation development and the production of hydroelectric energy. The hydroelectric power is to be coordinated through an interconnected power transmission system with existing and future hydro and thermal power development in Lebanon, so as to provide the maximum amount of hydroelectric energy from the water available at the lowest cost from the combined system.

The over-all plan provides for the installation of 171,000 kilowatts of hydroelectric power and the construction of a transmission system for operation at 69,000 volts. The irrigation works included in the plan will furnish a full water supply to approximately 18,600 hectares (46,000 acres) of land and a partial supply to an additional 2,900 hectares (7,100 acres). The locations of the several units included in this recommended plan of development are shown on Plate XX-1 and their principal features on Plate XX-2.

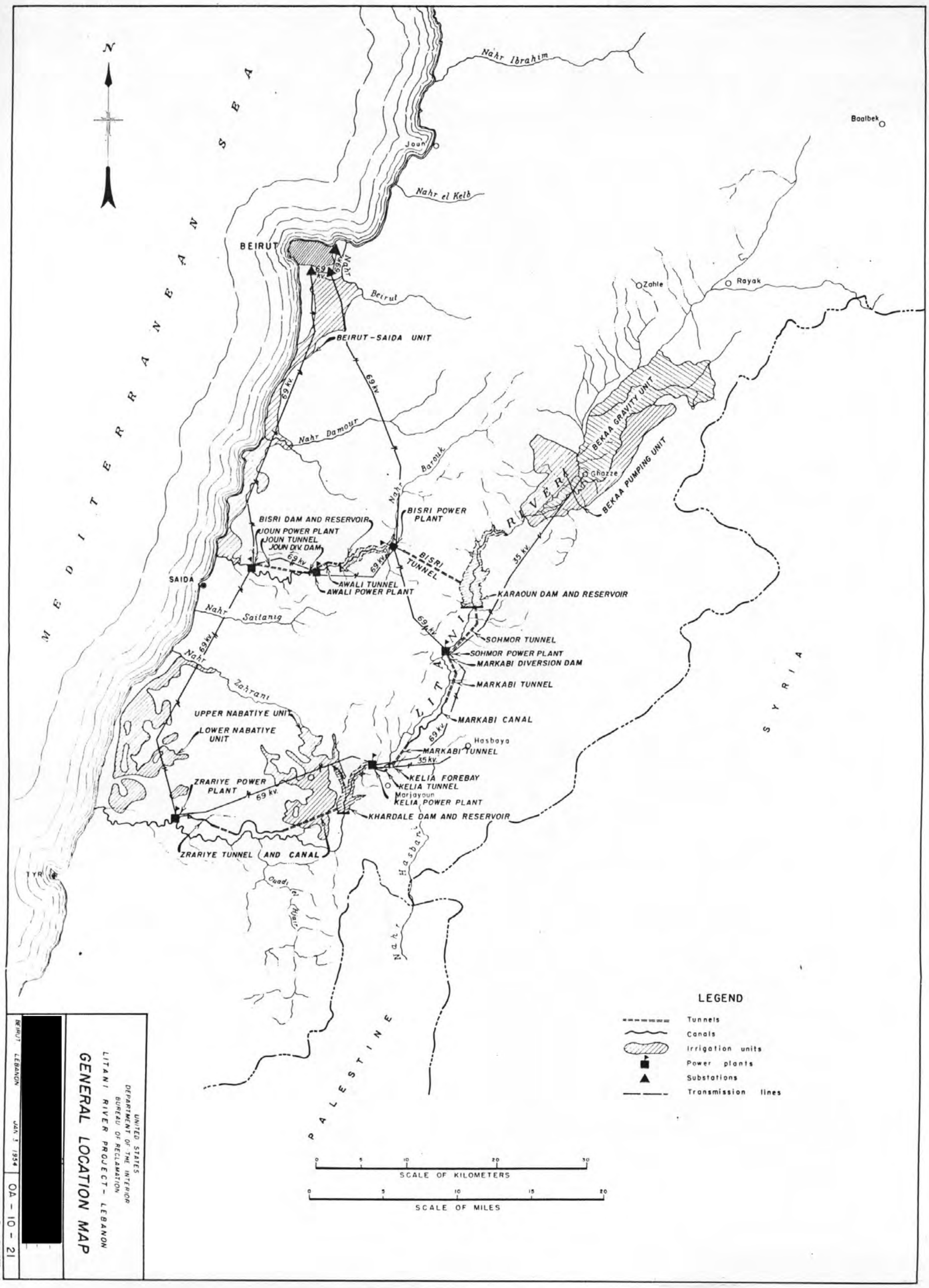
The units in the recommended plan of development, exclusive of those selected for early construction, and designated as Phase "A" units are sometimes referred to in this volume as Phase "B" units. These include the second stage construction of Karaoun Dam, to raise the reservoir to elevation 856 meters; the Bisri and Khardale Dams and

Reservoirs; the installation of the third unit in the Bisri Power Plant, the Kelia, Zrariye, Awali and Joun Power Units; the Markabi and Joun Diversion Dams; the necessary additional transmission facilities to connect these units to the power load center at Beirut and to the Phase "A" units; and the Bekaa Pumping, the Upper Nabatiye, the Lower Nabatiye, and the Saida-Beirut Irrigation Units. The locations of these units are shown on Plate XX-1.

These units will provide for an additional installation of 133, 000 kilowatts of hydroelectric power, and the production of about 400 million kilowatthours of additional electric energy in an average year. They will also provide for the furnishing of a full irrigaion water supply to about 15, 800 hectares of additional land included in the four irrigation units.

The general project planning investigations and the data collected for these units are contained in Volume I of this report and in accompanying appendices.

It is proposed that construction of this complete plan of development be completed within a 25-year period, between 1955 and 1980. The units included as Phase "A", and discussed in Volume II of this report are scheduled for construction during the 6-year period 1955-61. The installation of the third unit in the Bisri Power Plant, which is considered as a Phase "B" feature, would be delayed until about 1972, when its additional peaking capacity would be required. It is proposed that construction be started on the remaining units of the Phase "B" plan of development in about two years after the start of construction on the Phase "A" units, and continued throughout the remainder of the 25-year period. This construction schedule is shown on Plate XX-3. Kelia Power Unit and the Upper Nabatiye Irrigation Unit are recommended for such construction, to start in 1957. Since only project planning investigations have been completed on the units other than Phase "A", it is contemplated that this additional time will be required for preconstruction studies, the development of detailed designs and specifications and the arrangements for construction, after the plan of development is accepted by the Lebanese Government. This early start on construction of the next units in the plan will permit the production of additional energy at the time it will be needed to meet expected power load growth. The start of construction of the Upper Nabatiye Irrigation Unit at the same time will result in it being completed in time to utilize the water made available by construction of the joint water conveyance features of the Kelia Power Unit.



N
MEDITERRANEAN SEA
Nahr Ibrahim
Baalbek
Joun
Nahr el Kelb
BEIRUT
Nahr Beirut
69 kv.
Zahle
Rayak
BEIRUT-SAIDA UNIT
Nahr Damour
Nahr Barouk
BEKAA GRAVITY UNIT
BEKAA PUMPING UNIT
Ghazze
LITANI RIVER
BISRI DAM AND RESERVOIR
JOUN POWER PLANT
JOUN TUNNEL
JOUN DIV DAM
BISRI POWER PLANT
BISRI TUNNEL
35 kv.
SAIDA
AWALI TUNNEL
AWALI POWER PLANT
KARAOUN DAM AND RESERVOIR
Nahr Saitaniq
SOHMOR TUNNEL
SOHMOR POWER PLANT
MARKABI DIVERSION DAM
Nahr Zahrani
MARKABI TUNNEL
SYRIA
UPPER NABATIYE UNIT
MARKABI CANAL
LOWER NABATIYE UNIT
Hasbaya
MARKABI TUNNEL
KELIA FOREBAY
ZRARIYE POWER PLANT
KELIA TUNNEL
Marjayoun
KELIA POWER PLANT
KHARDALE DAM AND RESERVOIR
ZRARIYE TUNNEL AND CANAL
TYR
Ouadi el Hujeir
Nahr Hasbani
PALESTINE
LEGEND
Tunnels
Canals
Irrigation units
Power plants
Substations
Transmission lines
0 5 10 20 30
SCALE OF KILOMETERS
0 5 10 15 20
SCALE OF MILES
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
LITANI RIVER PROJECT - LEBANON
GENERAL LOCATION MAP
BEIRUT LEBANON
[redacted]
JAN 1 1954
OA - 10 - 21

PLATE XX - 1

UNITED STATES DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

LITANI RIVER PROJECT – LEBANON

CONSTRUCTION AND FINANCING SCHEDULE

PHASE B

Note: Monetary exchange assumed at $1 US = L.L.3.50

Procure Hydro-Electric equipments.

Beirut, Lebanon
March 25, 1954
Prepared by: A.Z.
Checked by: H.H.W.

URES OR UNITS	EST. COST L.L.	1956	1957	1958	1959	1960	1961	1962	1963	1964	1965	1966	1967	1968	1969	1970	1971	1972	1973	1974	1975	1976	1977
POWER UNIT	45,360,000		7,100,000	12,000,000	14,860,000								3,500,000	5,500,000		400,000	2,000,000						
M AND RESERVOIR	24,500,000				1,500,000	6,500,000	9,500,000	7,000,000															
POWER UNIT	21,150,000					2,750,000	6,000,000	7,000,000			500,000	2,200,000							500,000	2,200,000			
OWER UNIT	22,780,000					2,080,000	4,000,000	6,000,000	8,000,000													500,000	2,200,000
E POWER UNIT	23,540,000							2,100,000	6,440,000	7,000,000	8,000,000												
LE DAM AND OIR	19,500,000								3,500,000	9,000,000	7,000,000												
ABATIYE IRR. UNIT	5,500,000		500,000	1,000,000	2,500,000	800,000	700,000																
BEIRUT IRR. UNIT	6,500,000						500,000	1,000,000	1,000,000	1,000,000	1,000,000	1,000,000	1,000,000										
NABATIYE IRR. UNIT	7,000,000							500,000	1,000,000	1,000,000	1,000,000	1,000,000	1,000,000	1,000,000	500,000								
PUMPING IRR. UNIT	7,500,000									1,000,000	2,000,000	1,000,000	1,000,000	1,000,000	1,000,000								
N DAM & RESERVOIR STAGE 2)	18,232,000									9,000,000	9,232,000												
POWER UNIT	2,230,000																230,000	2,000,000					
ISSION LINES YARDS & SUBSTATIONS	21,057,000	309,000	500,000	448,000	1,100,000	700,000	2,500,000	2,300,000	2,000,000	1,700,000	1,500,000	1,500,000	1,400,000	800,000		300,000	600,000	700,000	1,600,000	500,000		300,000	300,000
TED COST L.L.	224,849,000	309,000	8,100,000	13,448,000	19,960,000	12,830,000	23,200,000	25,900,000	21,940,000	29,700,000	30,732,000	6,700,000	7,900,000	8,300,000	1,500,000	700,000	2,830,000	2,700,000	2,100,000	2,700,000		800,000	2,500,000

OA-10-274
PLATE XX-3

SECTION XXI

GENERAL BASIC DESIGN DATA

Surveys

Surveys made for this plan of development are discussed in detail in Section XIII of this report including the horizontal and vertical control adopted. Maps prepared for the Phase "A" units are also listed in that section.

Less extensive surveys have been made for the Phase "B" units covered by this volume since only project planning surveys have been made for the units included in this project planning report. It is anticipated that much more detailed surveys will be required for these units as a part of the preconstruction phase of their development. All surveys made as a part of this investigation are referred to the grid system known as the "Syrian-Lebanese Rectangular Coordinate System" and have been referenced to the Army Geographic Service mean sea level datum.

Available Maps

The following listed maps, profiles, and cross sections have been utilized in the investigations covered in this volume. A print of each of these maps and profiles is contained in a folio which can be made available in Lebanon or in the office of the U. S. Bureau of Reclamation in Denver, Colorado.

Karaoun Dam and Reservoir

(a) Topographic map - Reservoir de Karaoun Plans cotes - (Topographic map, Karaoun Reservoir) - Scale 1:5000; contour interval 5 meters; map in 3 sheets; undated.

(b) Topographic Map - Karaoun Dam Site - Scale 1:1000; contour interval 1 meter; in two sheets; revised August 1953.

(c) Cross Sections - Emplacement du Barrage de Karaoun, Profiles a Travers du Nahr Litani - (Cross Sections of Litani River at Karaoun Dam Site) - Horizontal Scale 1:500; vertical scale 1:125; in one sheet, undated.

(d) Cross Sections - Karaoun Damsite Profiles (Cross Sections). Horizontal scale 1:500; vertical scale 1:125; in one sheet; undated

Bisri Dam and Reservoir

(a) Topographic map - Emplacement du Barrage du Bisri (Location Map of Bisri Dam) - Scale 1:1000; contour intervals 1 and 2 meters; in two sheets; undated.

(b) Topographic map - Reservoir de Bisri (Bisri Reservoir) - Scale 1:5000; contour interval 5 meters; in one sheet; undated.

(c) Topographic map - Bisri Dam Site, "B" Axis - Scale 1:1000; contour intervals 1 and 2 meters; in one sheet; dated August 15, 1953.

(d) Cross Section - Nahr Bisri Cross Section (200 meters downstream from "B" Axis) horizontal scale 1:500; vertical scale 1:100; in one sheet; undated.

(e) Cross Section - Station de Jaugeage de Nahr Bisri, Profiles en Travers (Cross section of Bisri River at Gaging Station) horizontal scale 1:500; vertical scale 1:125; in one sheet; undated.

Khardale Dam and Reservoir

(a) Topographic map - Khardale Reservoir - Scale 1:5000; contour interval 5 meters; in one sheet; dated May 23, 1951; revised September 19, 1953.

(b) Topographic Map - Emplacement du Barrage de Khardale (Location Map of Khardale Damsite) - Scale 1:1000; contour intervals 1 and 2 meters; in two sheets; dated November, December 1952.

(c) Cross Section - Station de Jaugeage du Khardale sur le Nahr Litani Profils en Travers - (Cross Section of Litani River at Khardale Gage) Horizontal scale 1:500; vertical scale 1:125; in one sheet; dated December 12, 1952.

Markabi Diversion Dam

(a) Topographic map - Markabi Diversion Dam Site - Scale 1:500; contour intervals 1 and 2 meters; in one sheet; dated November 7, 1953.

Joun Diversion Dam

(a) Map - Pont de Baanoub, Elevation vue de L'aval (Plan of Bridge at Baanoub) Scale 1:100; in one sheet; dated July 1953.

(b) See also - Awali Power Unit - item (c)

Kelia Power Unit

(a) Topographic Map - Markabi-Kelia Tunnel Windows, Near Village of Bourghoz; Scale 1:500; contour interval 2 meters; in one sheet; dated August 21, 1953.

(b) Topographic Map - Markabi - Kelia Tunnel Windows, Wadi Zellaya - Scale 1:500; contour interval 2 meters; in one sheet; dated August 21, 1953.

(c) Topographic Map - Markabi - Kelia Tunnel Windows, Wadi After Zellaya - Scale 1:500; contour interval 2 meters; in one sheet; dated August 21, 1953.

(d) Topographic Map - Markabi-Kelia Tunnel Windows, Wadi Daraj - Scale 1:500; contour interval 2 meters; in one sheet; dated August 22, 1953; revised January 21, 1954.

(e) Topographic Map - Markabi-Kelia Tunnel Windows, Near Village of Kelia - Scale 1:500; contour interval 2 meters; in two sheets; dated September 5, 1953.

(f) Topographic Map - Markabi-Kelia Tunnel Windows, Wadi Malloul - Scale 1:500; contour interval 1 meter; in one sheet; dated October 14, 1953.

(g) Topographic Map - Markabi-Kelia Tunnel Windows, Near Village of Blate - Scale 1:500; contour interval 2 meters; in one sheet; undated.

(h) Profile - Tunnel de Markabi-Kelia, Profils en long Suivant l'axe du Tunnel (Profile along Proposed Axis of Markabi-Kelia Tunnel) - Horizontal scale 1:10,000; vertical scale 1:1000; in one sheet; undated.

(i) Profile - Markabi-Kelia Tunnel - Horizontal scale 1:5000; vertical scale 1:1000; in one sheet, undated.

(j) Topographic Map - Markabi-Kelia Canal - Scale 1:5000; contour interval 5 meters; in one sheet; dated November 17, 1953.

(k) Profile - Markabi-Kelia Canal - Flyline Profile - Horizontal scale 1:5000; vertical scale 1:500; in one sheet; dated November 17, 1953.

(l) Topographic Map - Markabi-Kelia Canal Outlet Portal Site - Scale 1:500; contour interval 1 meter; in one sheet; dated November 17, 1953.

(m) Topographic Map - Dibbine Reservoir - Scale 1:2000; contour intervals 1 and 2 meters; in one sheet; dated July 20, 1953.

(n) Topographic Map - Dibbine Dam Site - Scale 1:500; contour interval 1 meter; in one sheet; dated July 20, 1953.

(o) Topographic Map - Kelia Penstock - Scale 1:500; contour interval 1 meter; in three sheets; dated July 31, 1953, revised September 17, 1953.

(p) Topographic Map - Kelia Power Plant - Scale 1:500; contour intervals 1 and 2 meters; in two sheets; dated July 30, 1953.

Zrariye Power Unit

(a) Profile - Zaiye Tunnel Profile - Horizontal scale 1:5000; vertical scale 1:1000; in one sheet; dated February 10, 1954.

(b) Topographic Map - Zaiye Canal Flyline - Scale 1:5000; contour interval 5 meters; in one sheet; dated February 13, 1954.

(c) Map - Zrariye Tunnel, Canal, and Penstock Flyline Map - Scale 1:5000; in one sheet; dated February 13, 1954.

(d) Profile - Zrariye Tunnel Profile - Horizontal scale 1:5000; vertical scale 1:1000; in one sheet; dated February 5, 1954.

(e) Profile - Zrariye Tunnel Window Profile - Horizontal scale 1:1000; vertical scale 1:200; in 1 sheet; dated February 5, 1954.

(f) Topographic map - Zrariye Forebay map - Scale 1:500; contour interval 1 meter; in one sheet; dated December 7, 1953.

(g) Profile - Zrariye Penstock Profile - Horizontal scale 1:500; vertical scale 1:500; in one sheet dated February 11, 1954.

Awali Power Unit

(a) Map - Awali Tunnel - Scale 1:5000; in one sheet; undated.

(b) Profile - Awali Tunnel Flyline Profile - Horizontal scale 1:5000; vertical scale 1:1000; in one sheet; dated November 23, 1953.

(c) Topographic Map - Usine Hydro-electrique de Awali - Emplacements de la Conduite Forcee - (Awali Hydroelectric Plant, Location of Pressure Tunnel) - Scale 1:500; contour intervals 1 and 2 meters; in three sheets; revised April 1953.

(d) Map - Awali Power Plant and Penstock - Scale 1:500; in four sheets; dated July 1953.

Joun Power Unit

(a) Map - Joun Tunnel - Scale 1:5000; in one sheet; undated.

(b) Profile - Joun Tunnel Flyline Profile - Horizontal scale 1:5000; vertical scale 1:1000; in one sheet; dated November 18, 1953.

(c) Topographic Map - Usine Hydro-electrique de Joun - Emplacement de la Conduite Forcee - (Joun Hydro-electric Plant, Location of Pressure Tunnel) - Scale 1:500; contour intervals 1 and 2 meters; undated.

Bekaa Pumping Irrigation Unit

(a) Topographic map - South Bekaa Irrigation including (1) Pump House Site (2) Pump Discharge Pipe Outlet Site, and (3) River Crossing Pipe Inlet Site - Scale 1:500; contour interval 1 meter; in one sheet; dated October 10, 1953.

(b) Topographic map - South Bekaa Irrigation, River Crossing and West Branch Canal - Scale 1:5000; contour intervals 1, 2 and 5 meters; in 2 sheets; dated October 10, 1953.

(c) Topographic map - South Bekaa Irrigation Pump Discharge Line and Main Canal - Scale 1:5000; contour intervals 1, 2 and 5 meters; in one sheet; dated October 10, 1953.

(d) Topographic map - South Bekaa Irrigation East Branch Canal - Scale 1:5000, contour intervals 1, 2, and 5 meters; in two sheets; dated October 10, 1953.

(e) Profiles - South Bekaa Irrigation, Flyline Profiles - Horizontal scale 1:5000; vertical scale 1:500; in one sheet; undated.

(f) Profiles - South Bekaa Irrigation, Flyline Profile - Horizontal scale 1:5000; vertical scale 1:100; in one sheet; undated.

Upper Nabatiye Irrigation Unit

(a) Map - Main Canal Location - Scale 1:5000; in one sheet; dated December 12, 1953.

(b) Topographic map - Litani River Crossing Upper Head of Siphon - Scale 1:500; contour interval 1 meter; in one sheet; dated December 7, 1953.

(c) Map - Litani River Crossing, Lower Head of Siphon - Scale 1:500; in one sheet dated December 8, 1953.

Lower Nabatiye Irrigation Unit

(a) Profile - Lower Nabatiye Irrigation Canal Main Canal Tunnel Profile - Scale 1:1000; in one sheet; dated January 14, 1954.

(b) Topographic map - Lower Nabatiye Irrigation Canal, Tunnel Outlet Portal Site Map - Scale 1:500; contour interval 1 meter; in one sheet; dated January 14, 1954.

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(f) Profile - Lower Nabatiye Irrigation Canal West Canal, Profiles of Siphon - Scale 1:1000; in one sheet; dated January 14, 1954.

(g) Topographic map - Lower Nabatiye Irrigation Canal, West Canal Flyline map - Scale 1:5000; contour interval 5 meters; in one sheet; dated January 14, 1954.

Saida - Beirut Irrigation Unit

(a) Topographic map - Saida-Beirut Irrigation canal, Flyline Map - Scale 1:5000; contour interval 5 meters; in seven sheets; dated February 2, 1954.

(b) Profile - Saida-Beirut Irrigation Canal Profiles of Siphons - Scale 1:1000; in one sheet; dated February 2, 1954.

Regional Geology

Lebanon occupies a position on the eastern Mediterranean Sea. Its length in a north-south direction in about 190 kilometers, and its maximum width is about 75 kilometers. The Lebanon mountain range, paralleling the coast line, rises almost from the sea to elevations exceeding 2,250 meters. Along the eastern side of Lebanon another mountain range, the Anti-Lebanon, roughly parallels the Lebanon range at a distance of some 40 kilometers from the sea, and rises to about the same elevation. Eastward from these mountains is Syria. Between the two ranges is the Bekaa, a high, flat valley 10 to 12 kilometers in width, produced by faulting, which extends from the north of Lebanon southward to the vicinity of Karaoun, about three-fourths the length of the country. The Bekaa is separated near Baalbek by a low, flat divide. The North Bekaa is drained by the Nahr Orontes (Orontes River), which flows northward across the frontier through Syria to empty into the Mediterranean Sea in Turkey. The South Bekaa, about 66 kilometers long and 10 kilometers wide, is drained by the Nahr Litani (Litani River). Southwest of Karaoun, the mountains converge and the Litani River is confined to a deep, narrow, and tortuous gorge, except for a small basin near Khardale. The river continues to Deir Mimas, where it turns sharply westward, still in a canyon section, to the sea near Tyr.

Geologically the Lebanon and Anti-Lebanon Mountains and the intervening Bekaa section represent long, narrow crustal blocks, separated by faulting. These blocks have been up-lifted differentially by great tectonic movements. The faults along which these tectonic movements occurred are the northward continuation of major crustal breaks which form the African and Palestinian rifts. They extend from the north of Lebanon along the sides of the Bekaa southward through Jordan along the Dead Sea and into Africa. The major fault along the west side of the Bekaa is the Yamoune fault along which great massifs of Jurassic limestone have been uplifted. Along the east side there are several large faults such as the Hasbaya, the Rachaya, and the Serrhaya. At Khardale a major branch of the Yamoune fault passes along the course of the Litani River and affects the location of the Khardale dam site. Associated with the major faulting, which produced the present physiography of Lebanon, there was much flexing and minor faulting ~~of~~ formations. The result is seen in complicated structural conditions existing in many localized areas. The rock formations exposed in Lebanon represent the geologic age sequence from Jurassic up through the Cretaceous, Tertiary, and Quaternary. The greatest percentage of rocks is limestone, a condition which strongly influences the development of the Litani River Basin.

The condition and character of the various rock types is dealt with in more detail in the geologic treatment of the various units of the Litani development, but it is the structural position and the fissured, porous condition of the limestone formations in the Litani Basin which most profoundly affect the development of the Litani River. The elevation, the great area of surface exposure, the massive volume and the permeability of the limestone enable it to collect the torrential rainfall during the rainy season, store it and release it through many hundreds of springs to supply the Litani River in the dry season. At the same time it is the porous condition of the limestone which makes extremely difficult the location of reservoirs which will hold water and which are the keys to the project.

Seismology

The Litani River Reconnaissance Report[1] lists 117 earthquakes which occurred in Syria and the Middle East between the years 184 B.C. and 1873 A.D. Many destructive earthquakes occurred during this period of recorded history. The last destructive earthquake in Lebanon took place about 200 years ago and since then many small tremors have been noted. These numerous small movements have occurred at irregular intervals and their displacement has not been large enough to affect a dam structure to any extent.

In 1927 a major earthquake occurred near the Dead Sea but the intensity of the shock was small in the Karaoun area. In 1951 a small tremor, which amounted to about 2 millimeters vertical displacement was felt at the Ksara Observatory, about 40 kilometers north of Karaoun. Other small movements have occurred recently in northern Lebanon and to the north in Turkey.

Thus, it must be recognized that the large Yamoune fault has been relatively quiet in this area during the past 200 years. Also, smaller faults on the east side of the Bekaa plain have had very small movements during late years. However, destructive earthquakes in the near future cannot be ruled out in this area and dam designs should recognize this fact.

Hydrologic Design Data

Climate. The location of Lebanon on the eastern shore of the Mediterranean, between 33 and 34.5 degrees north latitude, results in it having a present-day climate of the Mediterranean type, that is moderately cold, windy, and wet in winter and warm and dry in summer and fall. The coastal area is semitropical, but the mountain slopes and interior valleys are cooler with frosts and some snow occurring during the winter in the higher elevations of both the Lebanon and Anti-Lebanon Mountains.

Temperature. The Ksara Observatory, located in the upper half of the Litani River Basin, has maintained temperature records from 1924 to date and these have been utilized in this investigation. A hydrothermograph was also established in 1952 at the College of Machmouche in the Bisri River Basin and its records will be available for use in operation of the Litani River development. The following table shows the monthly range of temperatures at Ksara based on the period 1940-1947:

1/ Reconnaissance Report, Litani River Project - Bureau of Reclamation, Beirut, Lebanon, June 1951.

TABLE XXI-1

MONTHLY TEMPERATURES AT KSARA

	Temperature - Degrees C.		
	Monthly	Daily Extremes	
Month	Mean	Maximum	Minimum
January	5.72	21	-17
February	7.13	27	-12
March	9.37	29	- 7
April	14.32	36	0
May	18.97	37	3
June	21.96	39	7
July	23.96	42	9
August	23.86	42	10
September	21.75	38	7
October	17.50	36	3
November	13.10	31	- 5
December	8.05	24	- 9

Annual Mean = 15.47°

Comparison of the above temperature data with that of the Willamette River Basin in Oregon, United States of America, indicates that the two areas have similar temperature variations. Consideration of the daily minimum extreme temperature of -17° C indicates that freezing weather occurs occasionally with temperatures approaching 0° F while the lowest monthly mean of 5.72° C indicates that such temperatures do not persist for long periods of time at Ksara. The station at Ksara is located at elevation 920 meters mean sea level, while the pool of Karaoun Reservoir will have a maximum elevation of only 856 meters. All of the other reservoirs considered are at still lower altitudes. It appears, therefore, that while thin ice may form occasionally around the edges of the proposed reservoirs it is unlikely to completely cover them. The occurrence of such ice formation is likely to be quite infrequent and its duration will be only for short intervals. Special precautions to provide for operation of any of these reservoirs or their appurtenances during icing conditions appear unnecessary.

Rainfall. Daily precipitation records have been maintained at American University at Beirut since 1876 and at Ksara Observatory since 1920. Records have also been maintained for varying periods at additional stations in Lebanon and its vicinity. Hourly records were available for the station at Ksara Observatory and have been utilized in this investigation. A comparison of the monthly precipitation as recorded at Ksara and Beirut is shown in the following table:

TABLE XXI-2

COMPARISON OF MONTHLY PRECIPITATION

All units in millimeters

Month	Ksara (1921 - 1951) Max.	Min.	Avg.	Beirut (1921 - 1951) Max.	Min.	Avg.	Beirut (1876 - 1951) Max.	Min.	Avg.
January	311	29	159.3	416	88	194.6	416	11	191.0
February	421	42	144.8	381	43	174.1	400	0	158.5
March	133	5	61.4	200	15	81.2	241	15	94.1
April	109	3	41.2	132	T	46.6	175	T	49.6
May	92	0	14.8	88	0	17.0	88	0	16.6
June	12	0	1.3	24	0	2.6	69	0	3.5
July	3	0	0.1	3	0	0.1	12	0	0.4
August	9	0	0.3	1	0	0.1	8	0	0.5
September	5	0	1.0	26	0	6.3	62	0	7.1
October	73	0	17.3	201	T	41.4	207	0	40.7
November	184	2	64.2	330	8	120.1	389	0	130.1
December	252	2	114.3	333	12	173.6	345	6	185.0
Full Year	960	322	640.8	1,198	393	857.7	1,209	393	877.1

Comparison of the monthly extremes of precipitation shown in the above table indicates that maximum precipitation is most likely to occur in January and February but that heavy precipitation may be expected during any of the months from November through April. Very light precipitation occurs during the months of June through September and during the 75-year period at Beirut there have been individual months where zero precipitation occurred in all months except December, January and March, with just a trace occuring in April. Fortunately there has been no year when all the minimum months occurred in combination at either station.

Snow. There are no records available to show either the depths or water content of the snow which falls and accumulates on both the Lebanon and Anti-Lebanon Mountains during some of the colder years. It is known that such accumulations do occur and in some cases on the highest peaks small areas of snow are still visible as late as August in some years. Stream-flow records available, however, do not indicate the occurrence of any snowmelt floods and there is little indication that snowmelt has contributed appreciably to the major flood occurrences of record.

Stream Flow. The following table shows the stream flow gages operated in the Litani River Basin and the length of record at each of these gages:

TABLE XXI-3

SUMMARY OF STREAM FLOW RECORDS

Stream	Station	Drainage area 1/ km2	Date Established
Litani River	Bar Elias 2/	849	January 1953
Litani River	Mansoura 3/	1323	November 1931
Litani River	Karaoun 3/	1545	September 1939
Litani River	Kelia 3/	1689	May 1949
Litani River	Khardale 3/	1822	September 1939
Litani River	Kasmie 3/	--	May 1944 4/
Nahr Bisri 5/	Near Bisri 2/	222	November 1952
Nahr Awali 5/	Near Saida 3/	301	January 1949
Nahr Bardoni 5/	Near Bar Elias 2/	77	January 1953
Nahr Ghazayel 5/	Near Bar Elias 2/	142	January 1953

1/ Drainage area shown is surface area and does not include any estimate of contributing ground-water area.

2/ Gages established and operated by the Litani River Investigation.

3/ Gages established and operated by the Ministry of Public Works.

4/ Discontinued in December 1946.

5/ Nahr is the local word for River.

All of the above stations were equipped with automatic water stage recorders, hence the gage height records are considered "excellent." However, only a limited number of measurements have been made at these stations and the majority of these at medium and low water stages. Therefore, it has been necessary to extend most rating curves, and discharge records are only of "fair" accuracy particularly for high flows.

Monthly Flows. Monthly discharges at Bisri gage as estimated for the 31-year period, 1921 through 1951 are shown on Plates XXI-1 and XXI-2 and reflect the irrigation development existing in the years shown.

Daily Flows. Daily discharge data for stream-flow stations in Lebanon have not been published but provisional records were obtained from the Ministry of Public Works and supplemented by additional measurements and computations made as a part of this investigation. Plate XXI-3 shows the daily hydrographs plotted from such data for 1949-1953 in the Bisri River Basin.

Flood Flows. Major floods were recorded in the Litani River Basin at Khardale gaging station with peaks on the following dates:

Station	Date	Discharge m^3/sec.
Khardale	January 31, 1940	185.0
	May 8, 1946	108.2
	February 2, 1947	145.6

Inspection of the hydrographs for these floods shows that floods rise relatively slowly considering the size of the drainage area indicating a very considerable lag between the occurrence of heavy precipitation over the basin and the occurrence of peak flows at Khardale. The recession side of each of these hydrographs is much longer than the rising side. This greater length may be due partially to a continuation of rainfall in the higher elevation sections of the drainage area where no rainfall stations exist, but is more likely due to the natural surface and channel storage in the Bekaa section of the valley above this gage.

The following table lists the estimated maximum flood peaks for each of the years 1931 to 1952 at Khardale gage. These estimates are based upon the best data available, but are necessarily approximate in a number of cases, due to lack of recorder charts and other recorded data. These data, however, have been utilized in frequency computations to determine the flows to be considered in determining diversion requirements during construction.

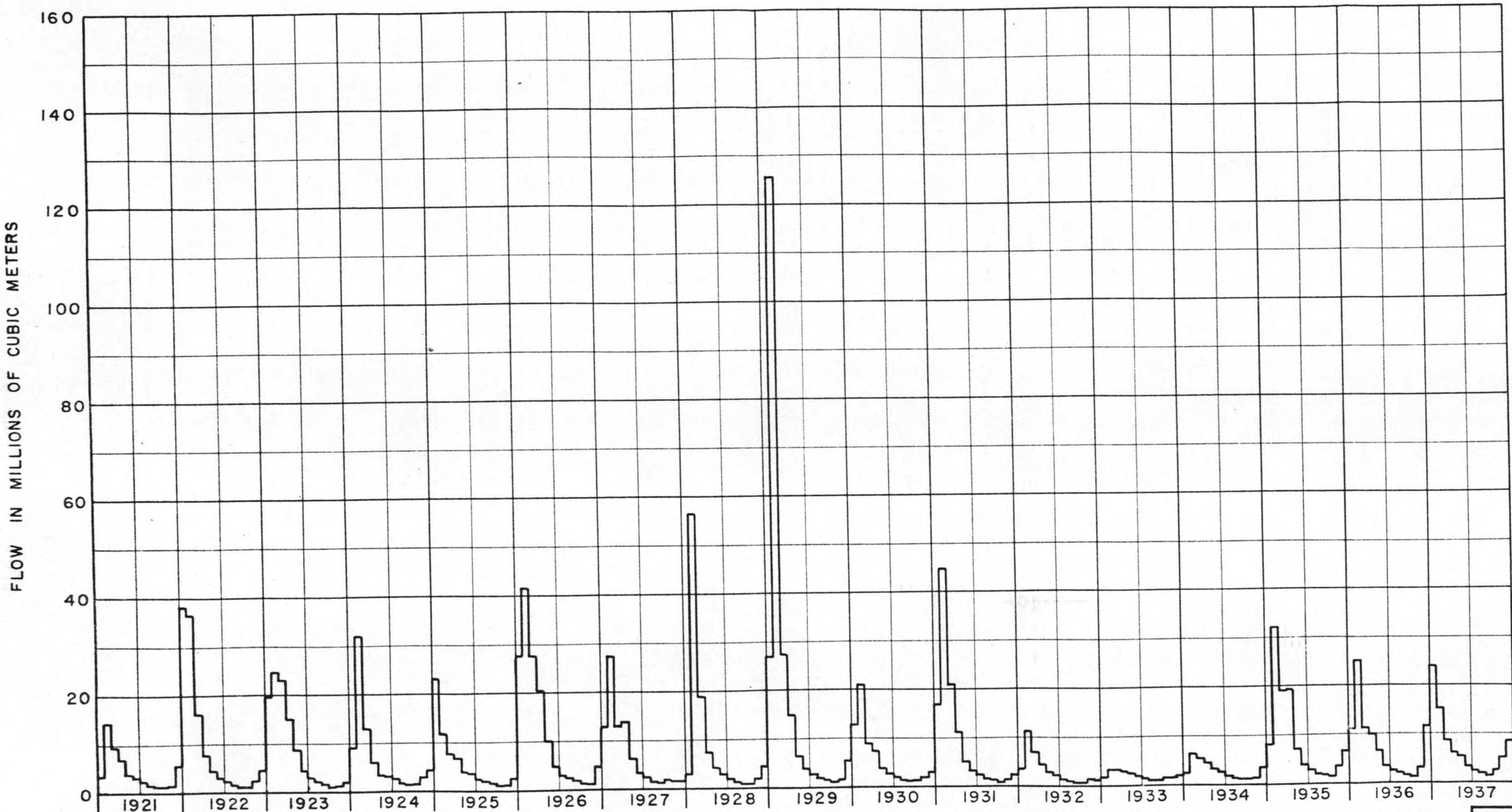
TABLE XXI-4

ESTIMATED MAXIMUM ANNUAL FLOODS
LITANI RIVER AT KHARDALE GAGE

Maximum Peak		Maximum Volume - DSM		
Date	Discharge m^3/sec.	1 - day	3 - day	5 - day
1931 - 32	110.8	97.5 (b)	256.0 (b)	399.0 (b)
1932 - 33	59.1	52.0 (b)	107.0 (b)	157.0 (b)
1933 - 34	101.4	89.2 (b)	235.0 (b)	355.0 (b)
1934 - 35	116.0	102.0 (b)	271.0 (b)	428.0 (b)
1935 - 36	109.6	96.4 (b)	242.0 (b)	348.0 (b)
1936 - 37	96.6	85.2 (b)	209.0 (b)	291.0 (b)
1937 - 38	118.2	104.0 (b)	267.0 (b)	413.0 (b)
1938 - 39	120.5	106.0 (b)	275.0 (b)	420.0 (b)
1/31/1940	185.0 (a)	164.0	434.5	621.0
1/15/1941	112.0	107.45	297.598	461.25
3/23/1942	163.9 (a)	155.9 (a)	339.5 (a)	483.0 (a)
1/29/1943	106.1	84.5	250.5	393.0
1/22/1944	114.6	89.0	249.0	383.0
1/21/1945	122.5	107.0	302.5	477.5
2/20/1946	95.5	85.0	239.0	370.0
2/ 2/1947	145.6	137.0	370.5	529.5
2/17/1948	123.0	106.7	301.413	462.489
2/ 8/1949	126.0	122.225	321.538	488.364
1/23/1950	85.1	79.0	219.188	338.289
1/23/1951	75.4	54.6	133.4	212.8
12/18/1952	127.9	114.5	307.2	467.7

(a) Estimated from Karaoun record.

(b) Estimated from Mansoura record.



NOTE

The estimated monthly discharges shown were determined by correlation methods from existing and computed records of the Awali River near Saida. The proposed diversion for the Aley Municipal Water Supply has been deducted from the values shown.

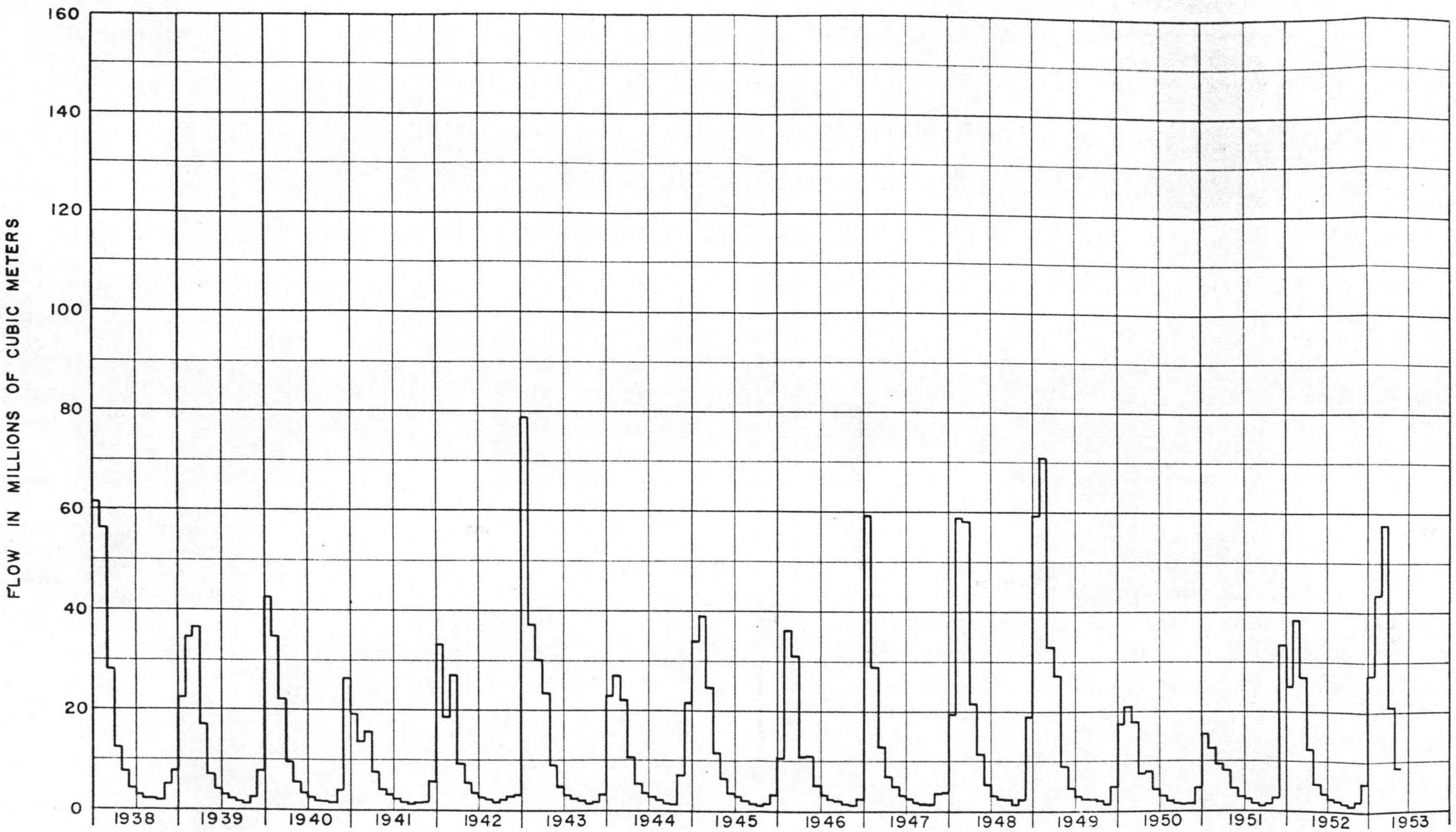
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
LITANI RIVER PROJECT — LEBANON

ESTIMATED MONTHLY DISCHARGE BISRI RIVER AT BISRI GAGE 1921 — 1937

[redacted]

BEIRUT, LEBANON JAN 7, 1954 SHEET 1 OF 2 | OA — 10 — 160

PLATE XXI-1



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UNITED STATES
DEPARTMENT OF THE INTERIOR
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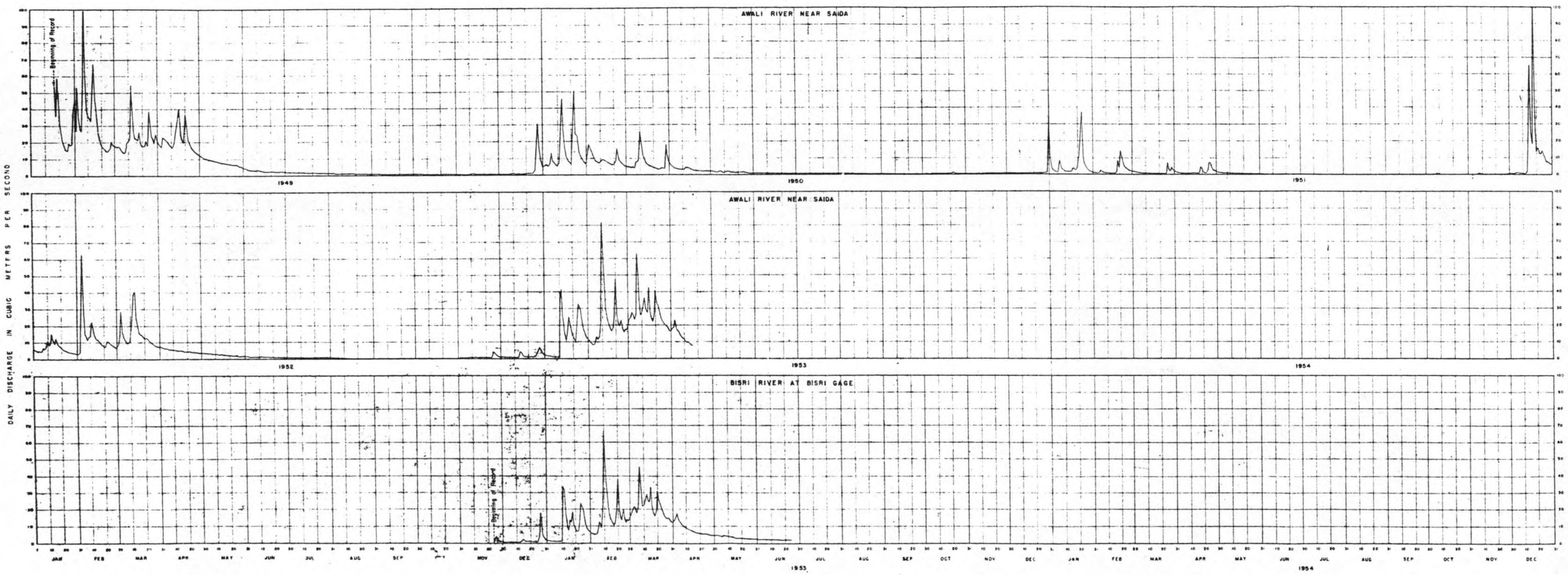
LITANI RIVER PROJECT—LEBANON

ESTIMATED MONTHLY DISCHARGE
BISRI RIVER AT BISRI GAGE
1938—1953

[redacted]

BEIRUT, LEBANON JAN 7, 1954 SHEET 2 OF 2 | 0A—10—161

PLATE XXI-2



NOTE

Gage on Awali River near Saida established in January 1949 and operated by Ministry of Public Works, Lebanon.

Gage on Bisri River near Bisri established in November 1952 and operated as a part of the Litani River investigation

All daily discharges shown were computed by the Litani River Investigation

UNITED STATES
DEPARTMENT OF THE INTERIOR
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LITANI RIVER PROJECT – LEBANON

BISRI RIVER BASIN

DISCHARGE HYDROGRAPH – 1949 THRU 1953

[redacted]

BEIRUT, LEBANON SEPT 14, 1953 | DA – 10 – 69

PLATE XXI-3

(c) Topographic Map - Lower Nabatiye Irrigation Canal, Bifurcation Site map - Scale 1:500; contour interval 1 meter; in one sheet; dated January 14, 1954.

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Annual Mean = 15.47°

Comparison of the above temperature data with that of the Willamette River Basin in Oregon, United States of America, indicates that the two areas have similar temperature variations. Consideration of the daily minimum extreme temperature of -17° C indicates that freezing weather occurs occasionally with temperatures approaching 0° F while the lowest monthly mean of 5.72° C indicates that such temperatures do not persist for long periods of time at Ksara. The station at Ksara is located at elevation 920 meters mean sea level, while the pool of Karaoun Reservoir will have a maximum elevation of only 856 meters. All of the other reservoirs considered are at still lower altitudes. It appears, therefore, that while thin ice may form occasionally around the edges of the proposed reservoirs it is unlikely to completely cover them. The occurrence of such ice formation is likely to be quite infrequent and its duration will be only for short intervals. Special precautions to provide for operation of any of these reservoirs or their appurtenances during icing conditions appear unnecessary.

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All units in millimeters

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Litani River	Kasmie 3/	--	May 1944 4/
Nahr Bisri 5/	Near Bisri 2/	222	November 1952
Nahr Awali 5/	Near Saida 3/	301	January 1949
Nahr Bardoni 5/	Near Bar Elias 2/	77	January 1953
Nahr Ghazayel 5/	Near Bar Elias 2/	142	January 1953

1/ Drainage area shown is surface area and does not include any estimate of contributing ground-water area.

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4/ Discontinued in December 1946.

5/ Nahr is the local word for River.

All of the above stations were equipped with automatic water stage recorders, hence the gage height records are considered "excellent." However, only a limited number of measurements have been made at these stations and the majority of these at medium and low water stages. Therefore, it has been necessary to extend most rating curves, and discharge records are only of "fair" accuracy particularly for high flows.

Monthly Flows. Monthly discharges at Bisri gage as estimated for the 31-year period, 1921 through 1951 are shown on Plates XXI-1 and XXI-2 and reflect the irrigation development existing in the years shown.

Daily Flows. Daily discharge data for stream-flow stations in Lebanon have not been published but provisional records were obtained from the Ministry of Public Works and supplemented by additional measurements and computations made as a part of this investigation. Plate XXI-3 shows the daily hydrographs plotted from such data for 1949-1953 in the Bisri River Basin.

Flood Flows. Major floods were recorded in the Litani River Basin at Khardale gaging station with peaks on the following dates:

Station	Date	Discharge m³/sec.
Khardale	January 31, 1940	185.0
	May 8, 1946	108.2
	February 2, 1947	145.6

Inspection of the hydrographs for these floods shows that floods rise relatively slowly considering the size of the drainage area indicating a very considerable lag between the occurrence of heavy precipitation over the basin and the occurrence of peak flows at Khardale. The recession side of each of these hydrographs is much longer than the rising side. This greater length may be due partially to a continuation of rainfall in the higher elevation sections of the drainage area where no rainfall stations exist, but is more likely due to the natural surface and channel storage in the Bekaa section of the valley above this gage.

The following table lists the estimated maximum flood peaks for each of the years 1931 to 1952 at Khardale gage. These estimates are based upon the best data available, but are necessarily approximate in a number of cases, due to lack of recorder charts and other recorded data. These data, however, have been utilized in frequency computations to determine the flows to be considered in determining diversion requirements during construction.

TABLE XXI-4

ESTIMATED MAXIMUM ANNUAL FLOODS
LITANI RIVER AT KHARDALE GAGE

Maximum Peak		Maximum Volume - DSM		
Date	Discharge m^3/sec.	1 - day	3 - day	5 - day
1931 - 32	110.8	97.5 (b)	256.0 (b)	399.0 (b)
1932 - 33	59.1	52.0 (b)	107.0 (b)	157.0 (b)
1933 - 34	101.4	89.2 (b)	235.0 (b)	355.0 (b)
1934 - 35	116.0	102.0 (b)	271.0 (b)	428.0 (b)
1935 - 36	109.6	96.4 (b)	242.0 (b)	348.0 (b)
1936 - 37	96.6	85.2 (b)	209.0 (b)	291.0 (b)
1937 - 38	118.2	104.0 (b)	267.0 (b)	413.0 (b)
1938 - 39	120.5	106.0 (b)	275.0 (b)	420.0 (b)
1/31/1940	185.0 (a)	164.0	434.5	621.0
1/15/1941	112.0	107.45	297.598	461.25
3/23/1942	163.9 (a)	155.9 (a)	339.5 (a)	483.0 (a)
1/29/1943	106.1	84.5	250.5	393.0
1/22/1944	114.6	89.0	249.0	383.0
1/21/1945	122.5	107.0	302.5	477.5
2/20/1946	95.5	85.0	239.0	370.0
2/ 2/1947	145.6	137.0	370.5	529.5
2/17/1948	123.0	106.7	301.413	462.489
2/ 8/1949	126.0	122.225	321.538	488.364
1/23/1950	85.1	79.0	219.188	338.289
1/23/1951	75.4	54.6	133.4	212.8
12/18/1952	127.9	114.5	307.2	467.7

(a) Estimated from Karaoun record.

(b) Estimated from Mansoura record.

Characteristics of Power Load

The energy produced at the power plants included in this plan of development will be for the most part transmitted to the load center of Beirut over 69,000-volt transmission lines. Kelia Power Plant will be initially operated in conjunction with Sohmor and Bisri Power Plants but later will be coordinated with the combined thermal hydro power system. The output of the installations at all plants will be firmed with thermal capacity and the system will be interconnected with other hydroelectric developments through Beirut before the completion of other Litani power features.

The characteristics of the over-all load are shown on Plates XXI-4 and XXI-5, and are generally those of a system with a predominant lighting load. The system demand peaks sharply during the four early evening hours starting at about 4:30 p.m. in winter afternoons and about 6:00 p.m. on a summer evening. There is a secondary peak during the morning hours reaching a maximum at about 7:00 during the winter and at about 9:30 in the summer. The magnitude of the secondary peak is about 50 to 60 percent of that of the evening peak. System power factors have been allowed to drop too on the order of 60 percent off peak and restored to 90 percent on peak. Monthly distribution of peak demands, load factors, and energy demands are shown on Plate XXI-6. Load duration curves are shown on Plates XXI-7 and XXI-8 and are included to show characteristics during high water months (March) and maximum demand months (December) for the initial period of operation of the power plant.

During critical water years and times when there is a choice between making releases either down the Litani or through the Bisri Tunnel from Karaoun Reservoir, releases through the Bisri Tunnel will result in more energy production on the ultimate scheme than releases down the Litani. The proposed operation of the hydro system will be in the peak of the load during periods of low energy production and in the base of the load during high water periods.

Typical Design - Minor Irrigation Structures

A number of factors affect the designs of irrigation systems and structures. The more important factors are the availability of labor and materials, and local practices, conditions, and customs. These factors are discussed in the irrigation portions of both Volumes I and II and will not be repeated here.

For estimating purposes, the general designs of minor irrigation structures adopted for the Bekaa Gravity Unit, as presented in Volume II, have been used for the remaining irrigation units. These designs have been selected for their simplicity and ease of construction and for their economy. Formed concrete surfaces have been held to a minimum and locally produced stone for masonry has been used to a large extent. Brief descriptions of typical minor structures which form the majority of those required follow:

Turnouts. Typical turnout designs are shown on Plates XXI-9 and XXI-10. The first has a single barrel for capacities up to 400 liters per second, and the second has double barrels for larger capacities. The turnouts are of conventional design and will consist of a masonry transition and headwall in the canal bank, precast concrete pipe barrels, and a masonry downstream stilling and weir pool. Control at each turnout will be provided by means of a simple, screw lift, slide gate.

Checks. Checks will usually be required immediately downstream from each turnout. A typical check will be constructed of masonry and will have adjustable stoplog control as shown on Plate XXI-11.

Bridges. Most bridges will be subjected to only infrequent, light traffic. A typical bridge design is shown on Plate XXI-12. Abutments will be masonry and the deck will be of reinforced concrete designed for H-10 loading. On most roads, only a single-lane width of about 2 meters will be required, and a few roads will require wider double-lane deck widths. On major roads carrying heavy traffic, special bridges or siphons designed for the heavier loadings will be required. Culverts instead of bridges will be used when the canal capacity is less than about 500 liters per second.

Footbridges. A great number of simple footbridges will be required across all canals regardless of their locations. These will generally be only one meter in width and constructed with reinforced concrete decks set on concrete or masonry footings outside the canal lining. The design of a typical footbridge is shown on Plate XXI-13.

Stock-Watering Ramps. In consideration of the need for watering livestock at the most accessible sources of water, special provision must be made for the use of canals for this purpose. By providing ramps to allow access to the canal, damage to the canal and its lining can be largely avoided. As shown on Plate XXI-14, a typical ramp will be constructed of masonry with vertical abutments and gently sloping floors to permit easy access to the water.

Siphons. Minor siphons of moderate heads and capacities will be constructed as shown on Plate XXI-15. Masonry transitions will be used and the barrels will be constructed of precast reinforced concrete pipe with mortared joints. Such siphons will be used for both roads and smaller stream crossings.

Drain Culverts. In cases where cross-drainage channels are not unduly deep and the volume of drainage is low, simple culverts as shown on Plate XXI-16 will be used. These will consist of a masonry inlet transition on the uphill side of the canal bank, a precast concrete pipe barrel, and a masonry outlet. Both inlet and outlet will be protected with riprap as required. In some cases, culverts having two or more barrels may be required.

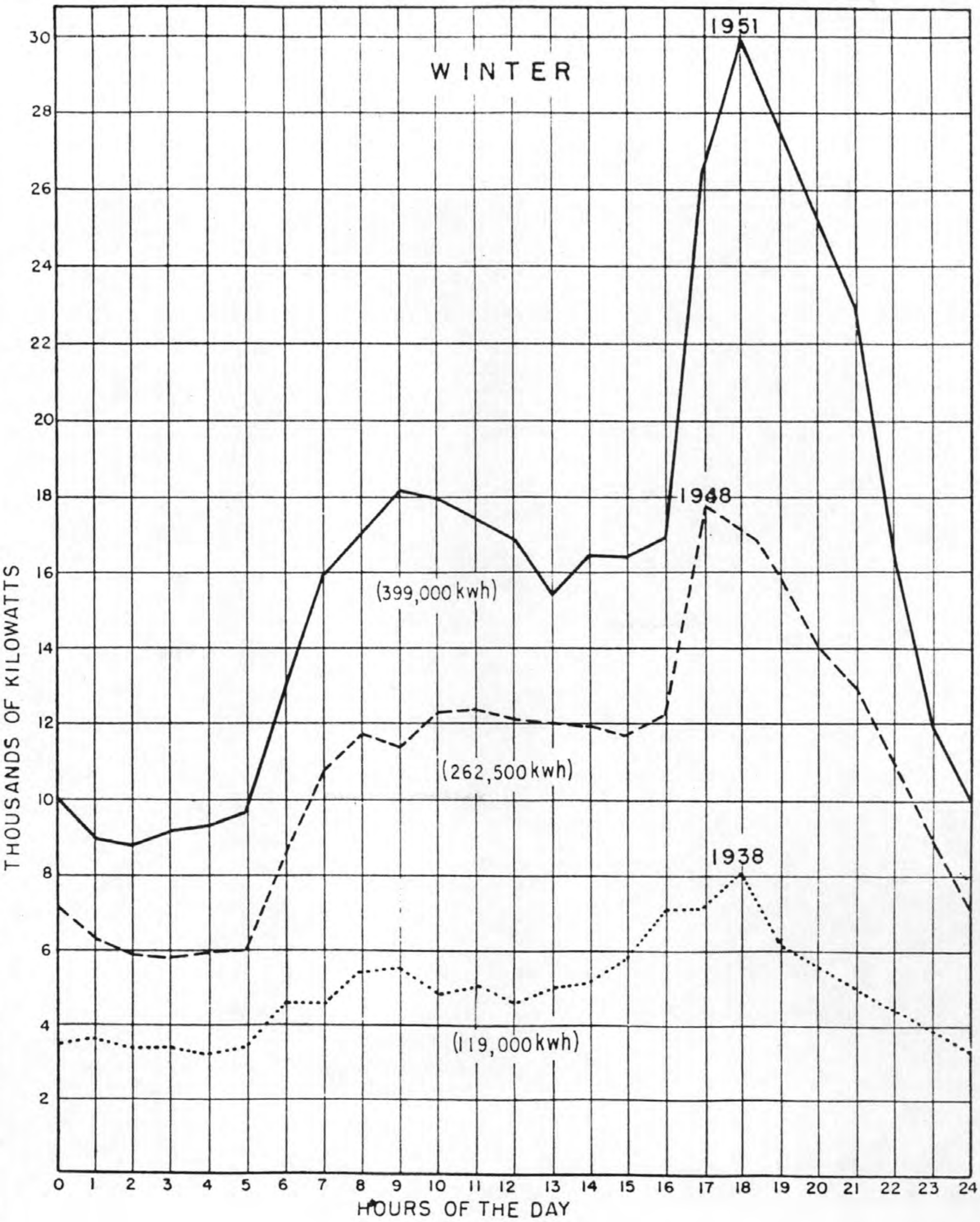
Transportation Facilities

Highways. The Lebanese highway network is quite extensive if compared to similar countries in Europe and the Middle East. Lebanon has a total highway kilometerage of about 3,000, of which 1,000 kilometers are asphalted roads, 1,300 kilometers of stone paved roads and 700 kilometers of dirt roads. In conjunction with attempts to encourage tourism, efforts are being made to increase and improve existing highway facilities. Mountain roads are tortuous and steep, very few bridges are found and there is practically no bridge clearance to consider in transportation problems. The main highways are between 15 and 20 meters wide, and the secondary highways are between 8 and 12 meters wide. All asphalted roads are designed to handle heavy traffic.

Railways. Three railroad lines serve Lebanon, and all are currently operated by the Damas-Homs and Branches Railway Company. A narrow-guage line links Beirut with Damascus, while standard-guage tracks connect Rayak, in the Bekaa, with Homs, Syria, from which point a line runs to Tripoli, thence south to Beirut and Nakoura, on the Israeli frontier. Total track in Lebanon is approximately 600 kilometers of standard-guage and 150 kilometers of narrow-guage.

Air Routes and Airports. Two Lebanese aviation companies and many foreign lines provide Lebanon with adequate air service from Beirut to the rest of the world. The New International Airport constructed just south of Beirut provides a major stop on international air routes. Another small airport is located near Tripoli. The Rayak airport is used by the Lebanese Army.

Shipping. Aside from being the most important port in Lebanese trade, Beirut derives a good proportion of its income from the heavy transit trade that annually passes through its modern harbor facilities.



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LOAD CHARACTERISTICS-LEBANON

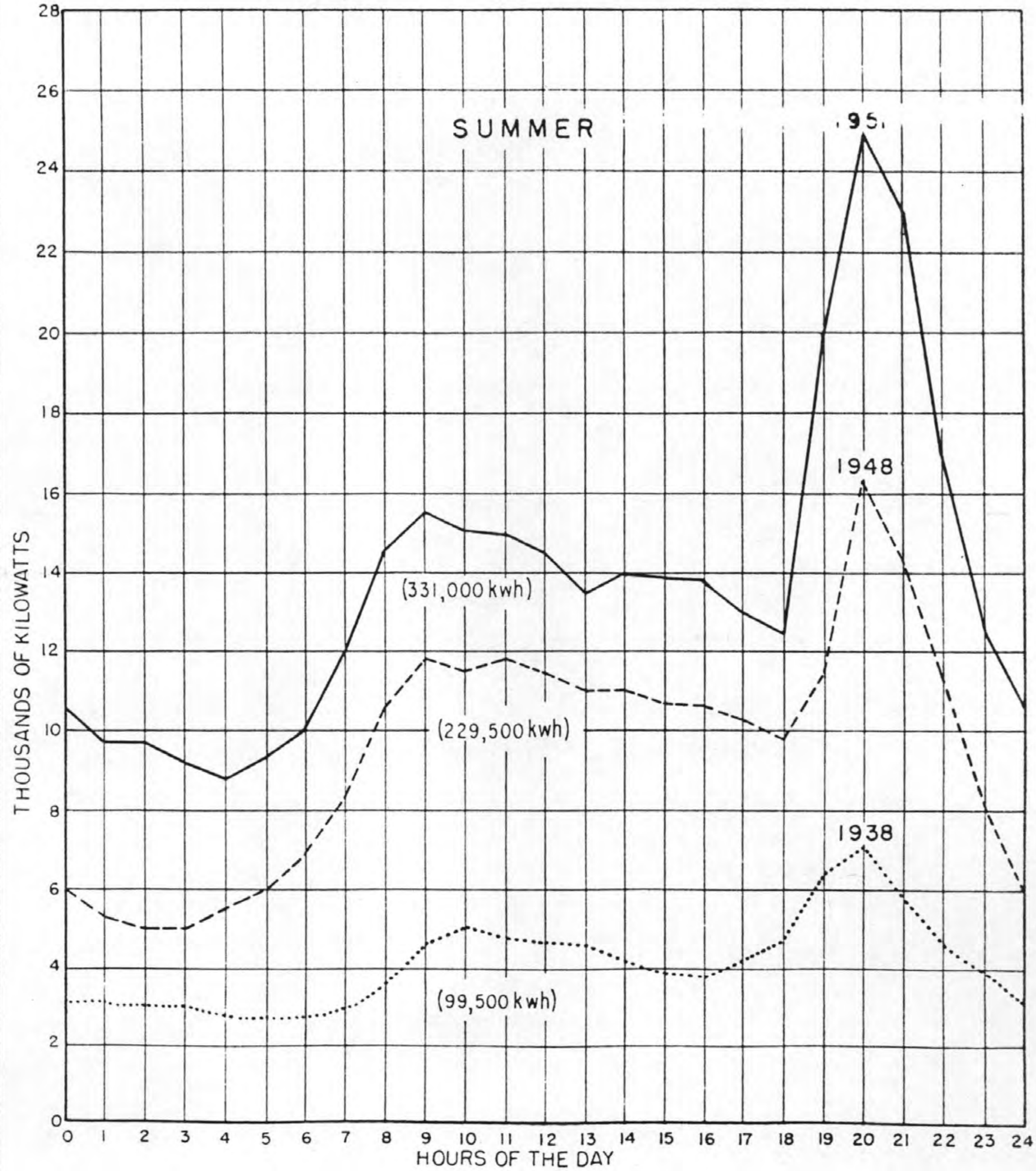
TYPICAL WINTER COMPOSITE

DAILY LOAD DIAGRAMS

[redacted]

BEIRUT, LEBANON 5-15-53 | OA-10-96

PLATE XXI-4



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LOAD CHARACTERISTICS-LEBANON

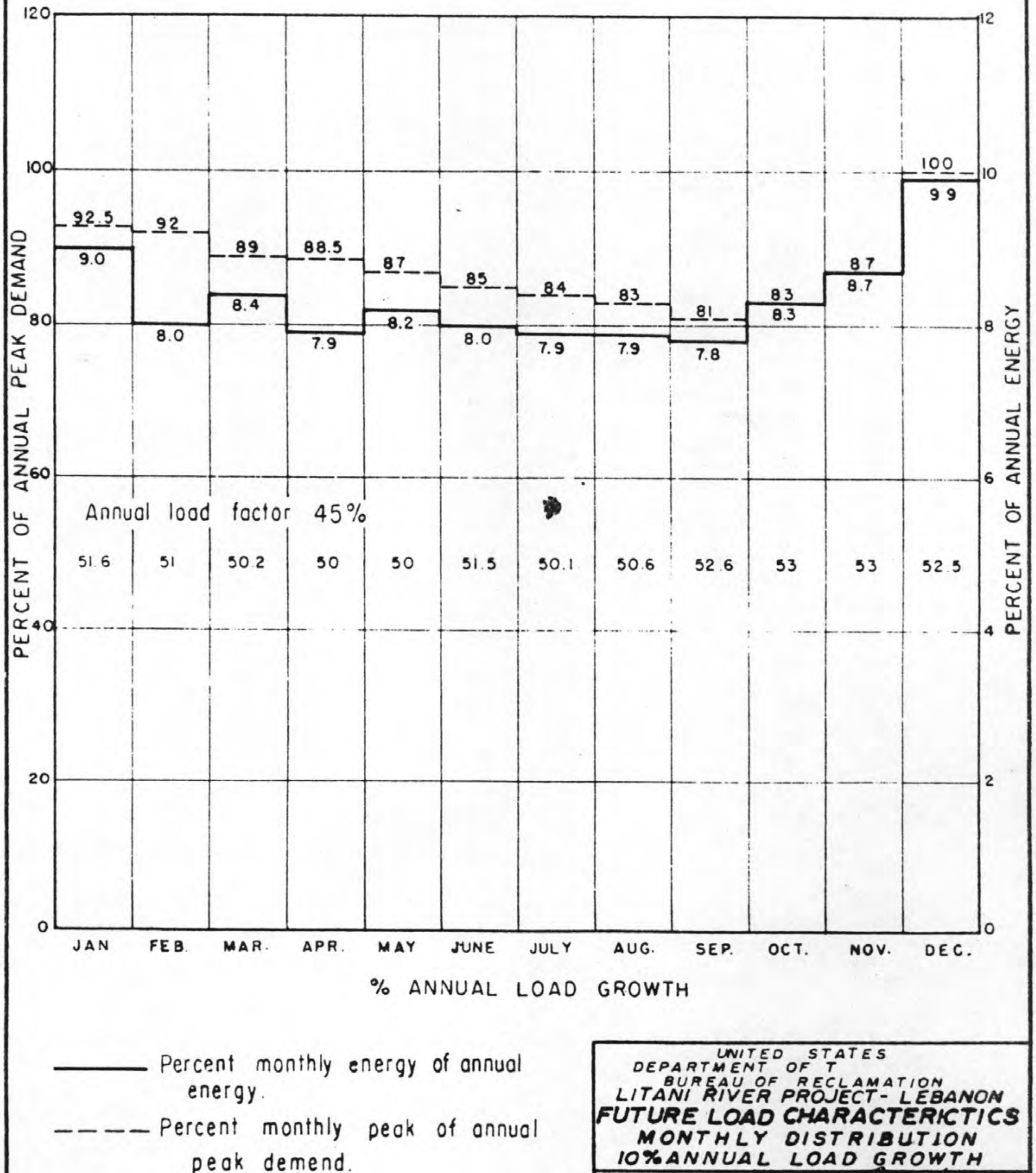
TYPICAL SUMMER COMPOSITE DAILY LOAD DIAGRAMS

[redacted]

BEIRUT, LEBANON 5-15-53 **OA-10-97**

PLATE XXI-5

1955 TO 1965 LOAD CHARACTERISTICS IN LEBANON



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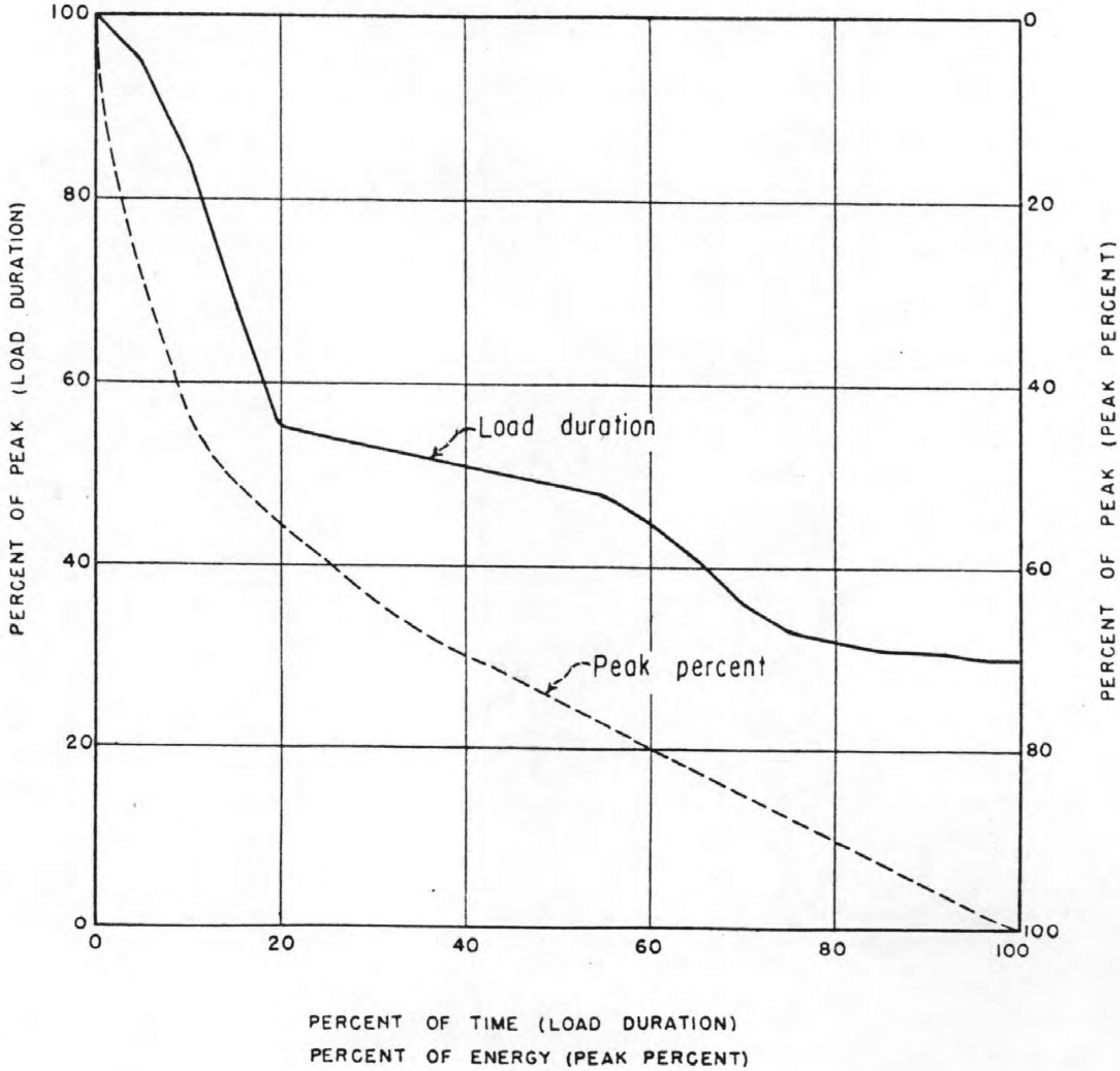
FUTURE LOAD CHARACTERICTICS
MONTHLY DISTRIBUTION
10% ANNUAL LOAD GROWTH

[redacted]

BEIRUT, LEBANON	5-15-53	0A - 10 - 98

MARCH CONDITIONS
1955–1965



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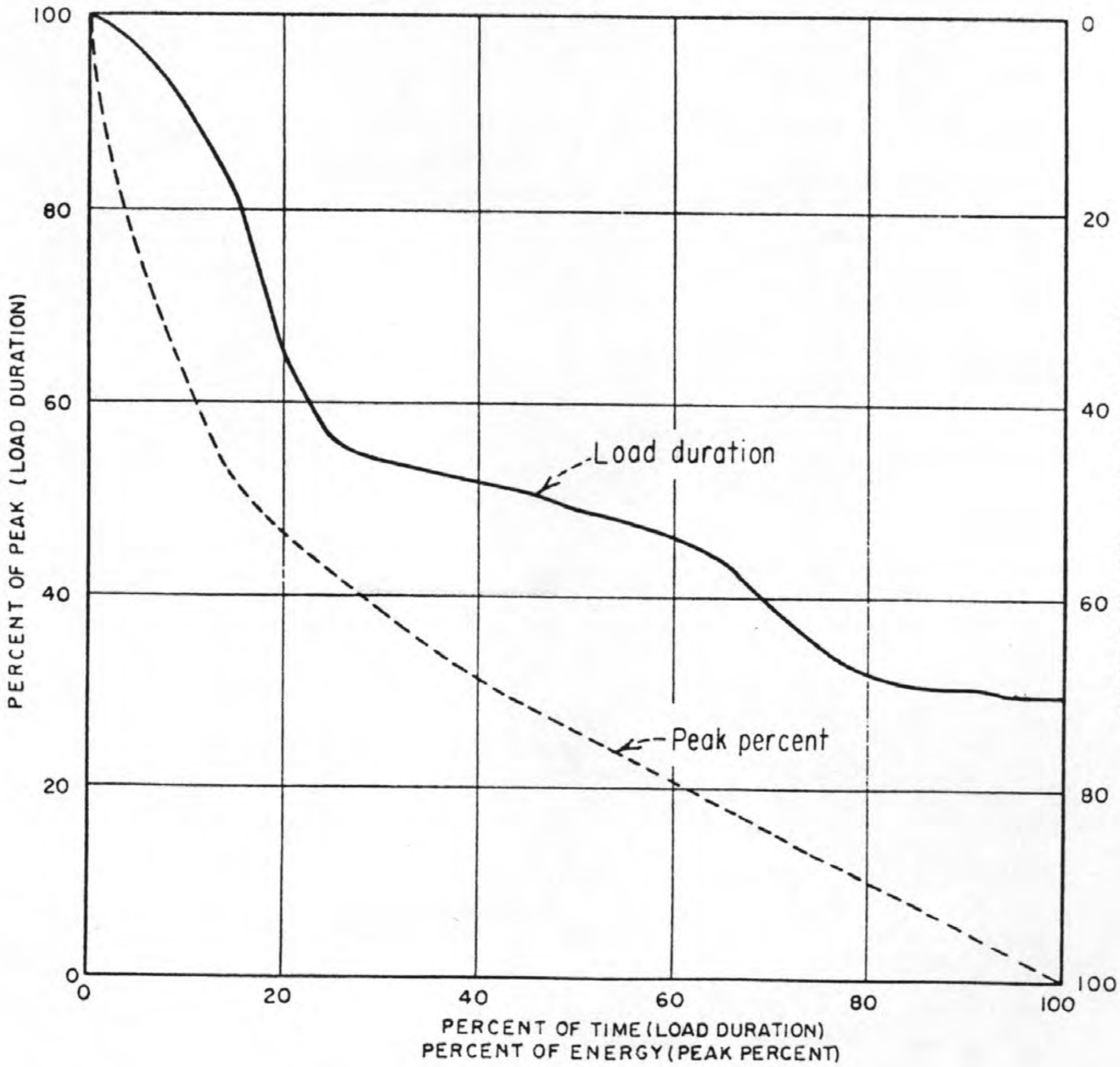
FUTURE LOAD CHARACTERISTICS
LOAD DURATION AND PEAK PERCENT CURVES
MARCH CONDITIONS-1955-1965
[redacted]
BEIRUT, LEBANON 5-15-53 | OA-10-100

PLATE XXI-7

DECEMBER CONDITIONS
1955 - 1965



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FUTURE LOAD CHARACTERISTICS
LOAD DURATION AND PEAK PERCENT CURVES
DECEMBER CONDITIONS-1955 - 1965

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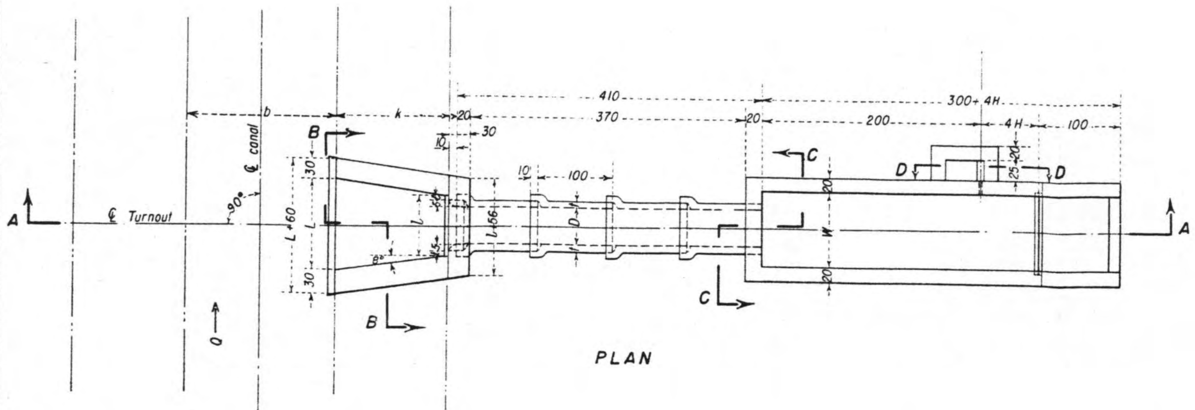
BEIRUT, LEBANON	5-15-53	OA - 10 - 101

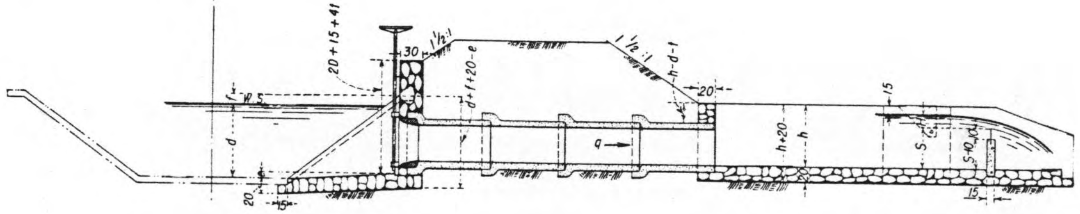
PLATE XXI-8



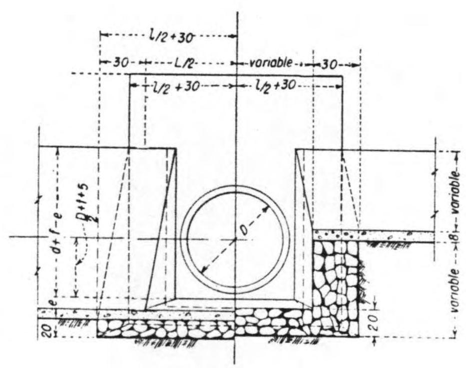
PLAN

ESTIMATED QUANTITIES

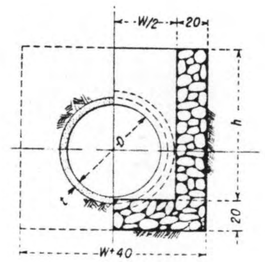
Flow q l/s	100	200	300	400
Excavation m^3	35	40	45	50
Masonry m^3	5.0	5.8	6.0	6.5
Plaster m^2	17	22	23	26
Concrete m^3	0.05	0.10	0.10	0.15
Pipes NoxD cm	4X50	4x60	4x60	4x60
Gates and Metal works Kg	130	180	180	180



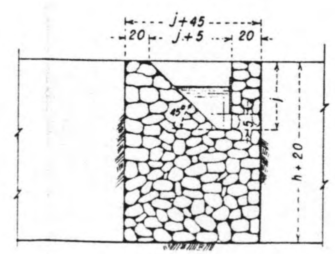
SECTION A-A



SECTION B-B



SECTION C-C



SECTION D-D

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LITANI RIVER PROJECT - LEBANON

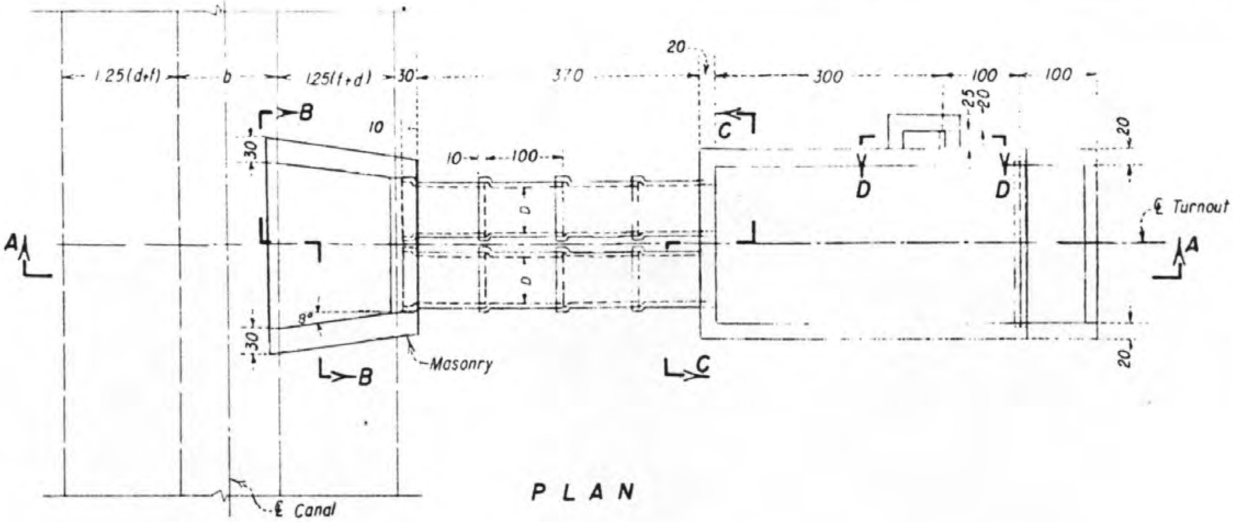
IRRIGATION UNITS

TYPICAL SINGLE BARREL TURNOUT

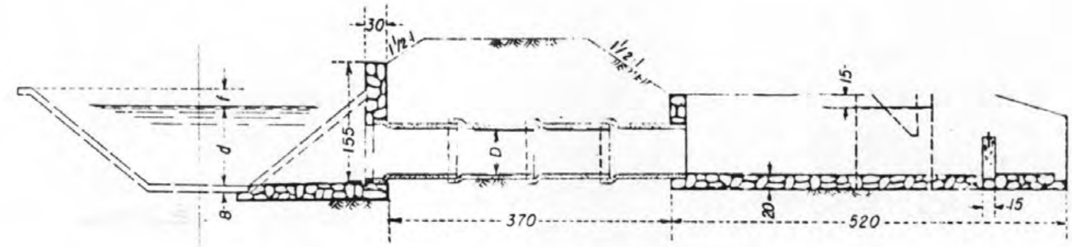
[redacted]

BEIRUT, LEBANON 4-23-1954 OA - 10 - 241

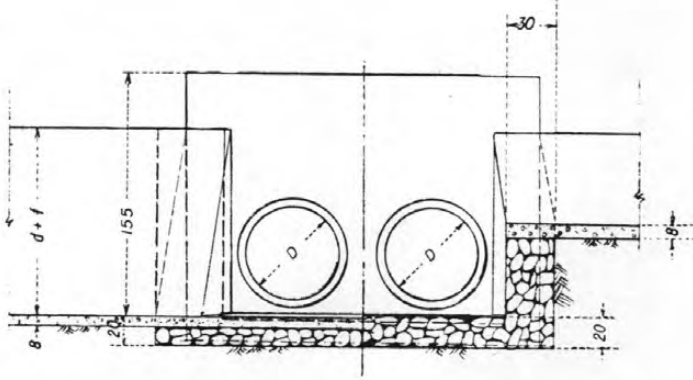
PLATE XXI-9



PLAN



SECTION A - A



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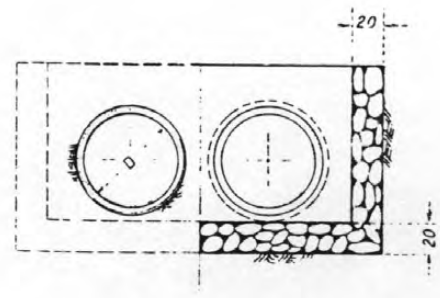
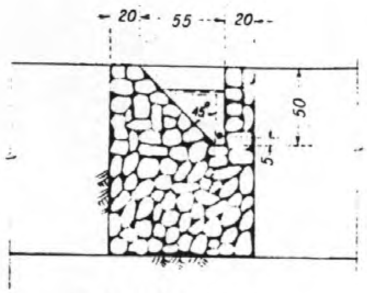


SECTION C - C



SECTION D - D

ESTIMATED QUANTITIES

Excavation	55 m^3
Masonry	7 m^3
Pipes	8 of 1 m. length
Concrete	0.2 m^3
Plaster	30 m^2
Gatesand Metal works	350 Kg.

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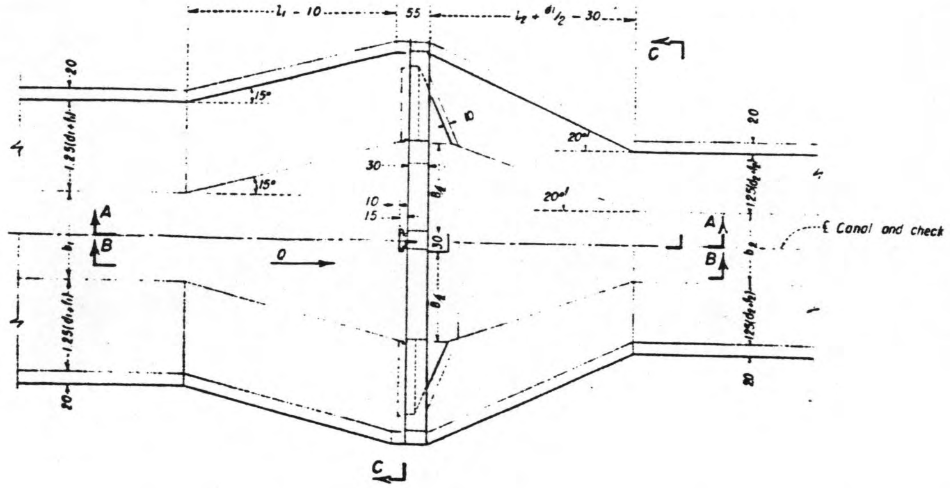
IRRIGATION UNITS
TYPICAL DOUBLE BARREL TURNOUT

[redacted]

BEIRUT, LEBANON | 4-23-1954 | OA - 10 - 242

PLATE XXI-10



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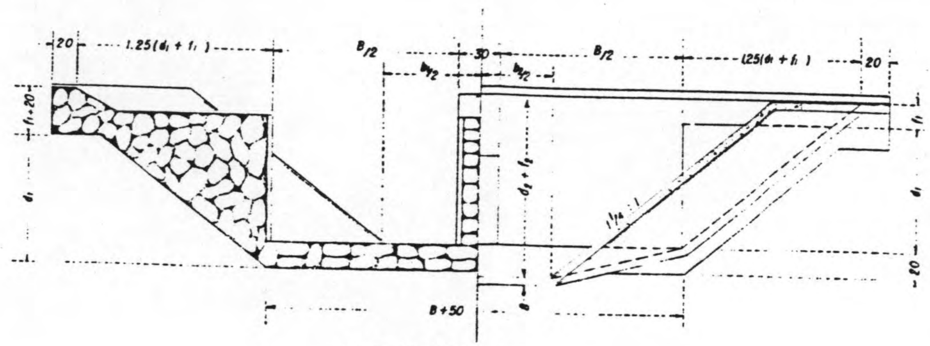
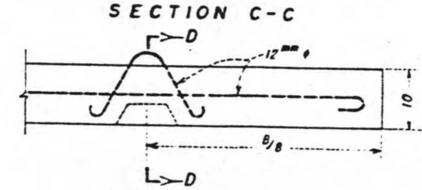
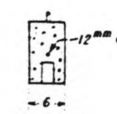


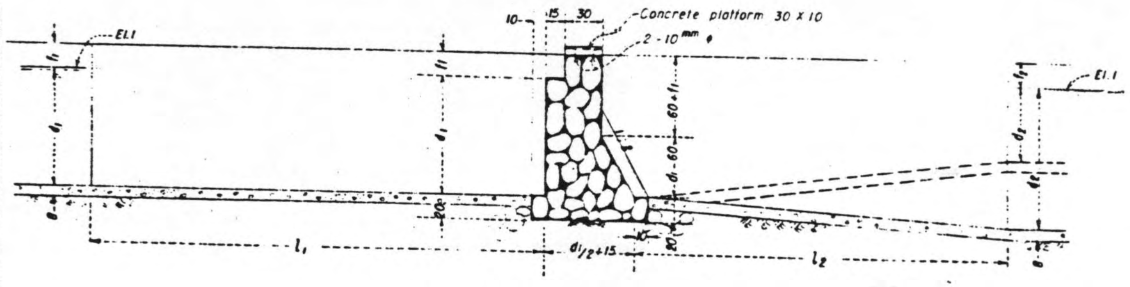
S E C T I O N C - C



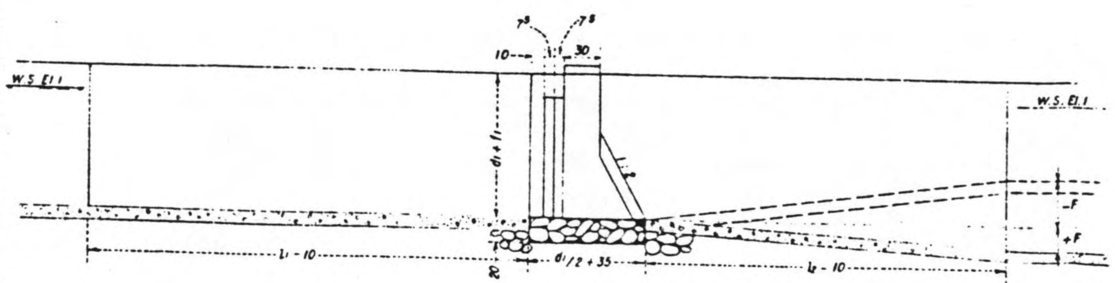
STOP - LOG DETAIL



SECTION D-D



S E C T I O N A - A



S E C T I O N B - B

ESTIMATED QUANTITIES

Q l/s	Excavation m³	Masonry m³	Concrete m³	Steel Kg
500	2.2	1.0	0.18	12
1000	4.8	1.4	0.26	20
1500	6.6	1.6	0.32	27
2000	8.6	1.9	0.36	33
2500	11.0	2.1	0.42	38

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LITANI RIVER PROJECT - LEBANON

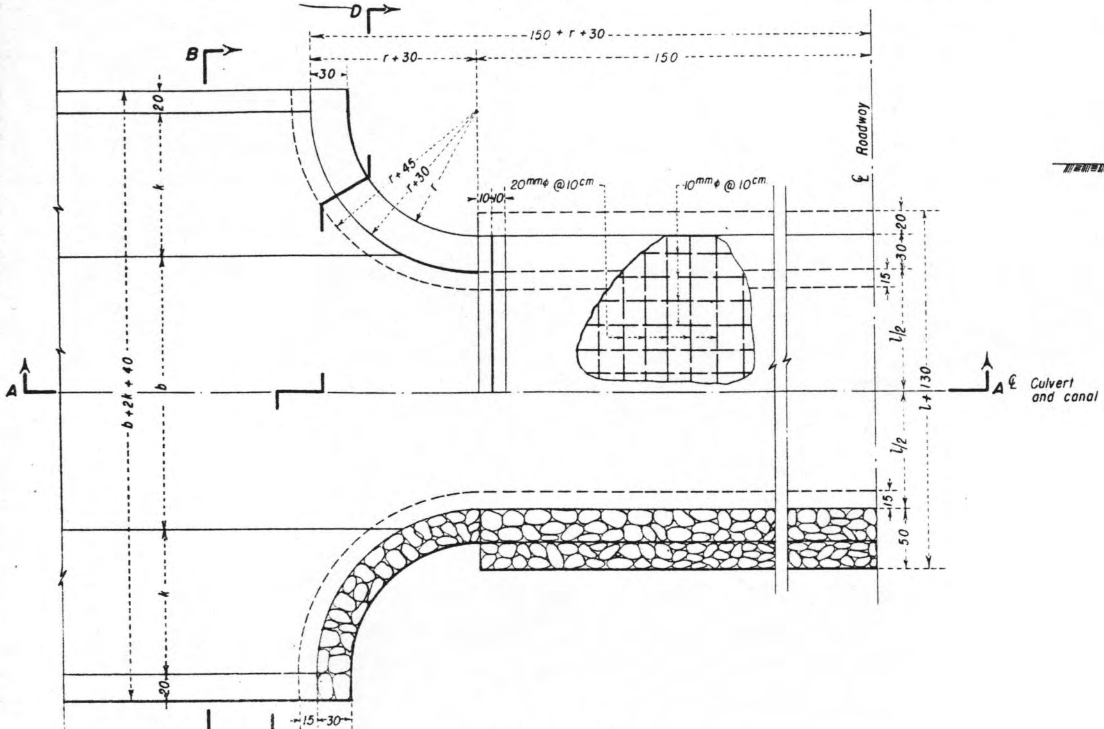
IRRIGATION UNITS

TYPICAL CHECK

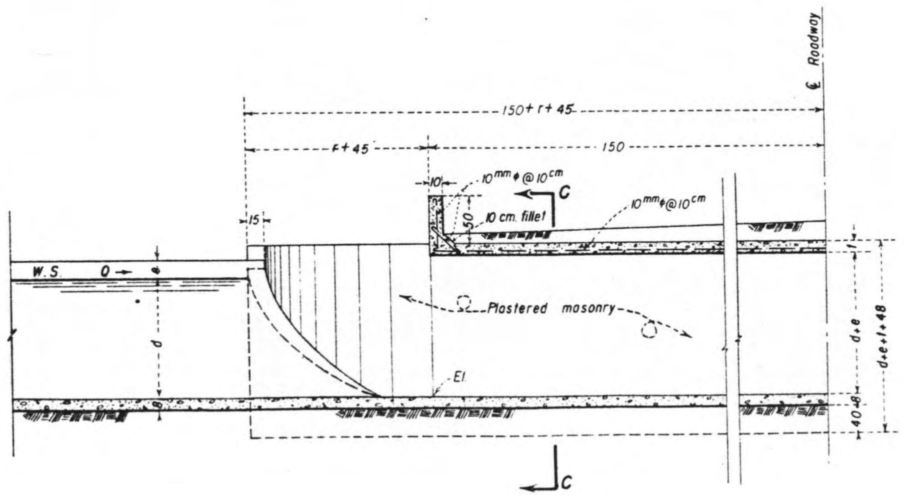
[redacted]

BEIRUT, LEBANON 4-23-54 OA - 10 - 243

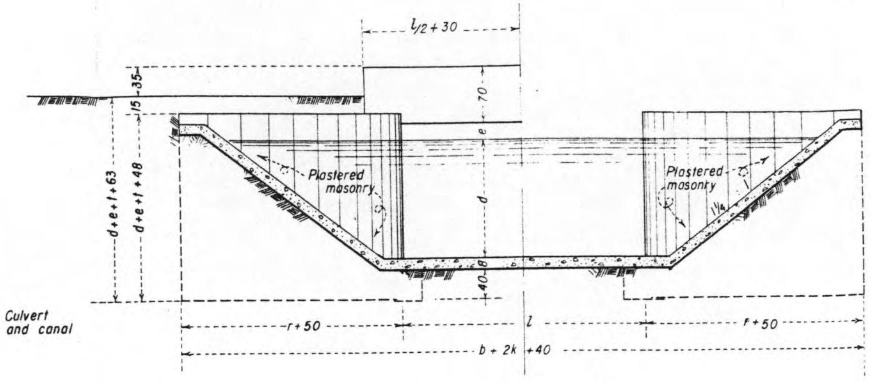
PLATE XXI-11



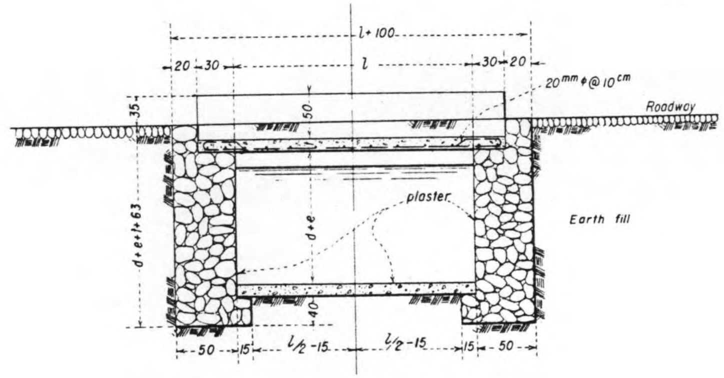
HALF PLAN



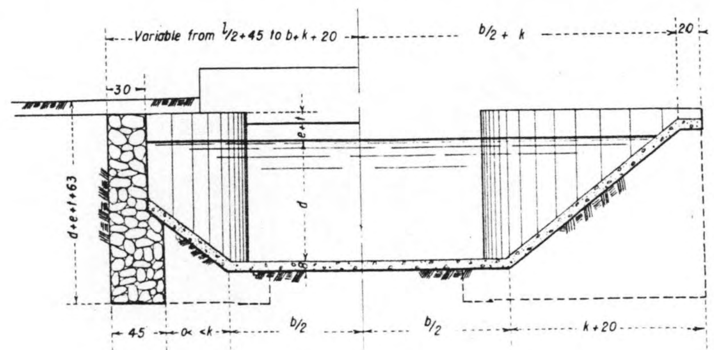
SECTION A–A



SECTION B–B



SECTION C–C



SECTION D–D

ESTIMATED QUANTITIES

Q l/s	Exc. m³	Masonry m³	Steel bars Kg.	Concrete m³	Plaster m²
500	30	7	160	1	6
1000	50	9	160	1	9
1500	60	10	160	1	11
2000	70	11	300	1 1/4	12
2500	85	13	300	1 1/4	14

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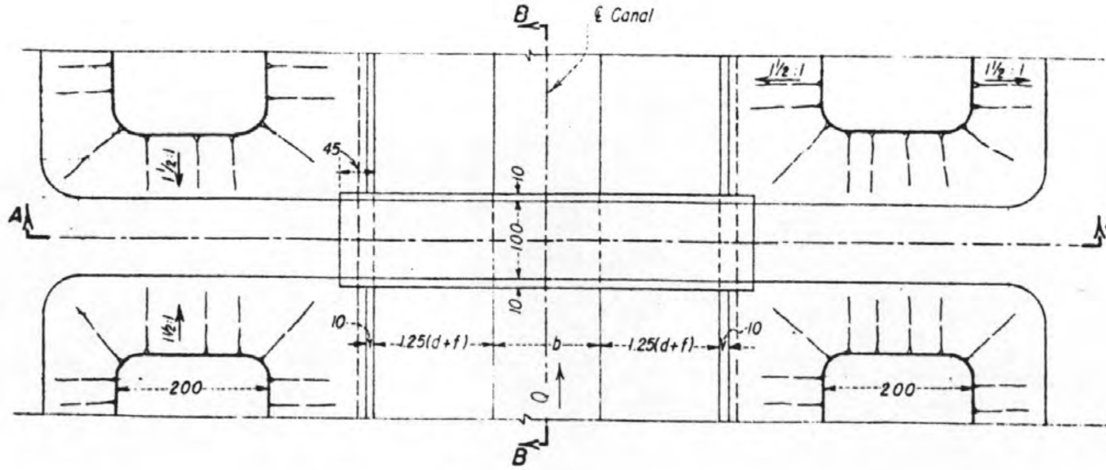
LITANI RIVER PROJECT - LEBANON

IRRIGATION UNITS
TYPICAL BRIDGE

[redacted]

BEIRUT, LEBANON SEPT 14, 1953 OA - 10 - 24

PLATE



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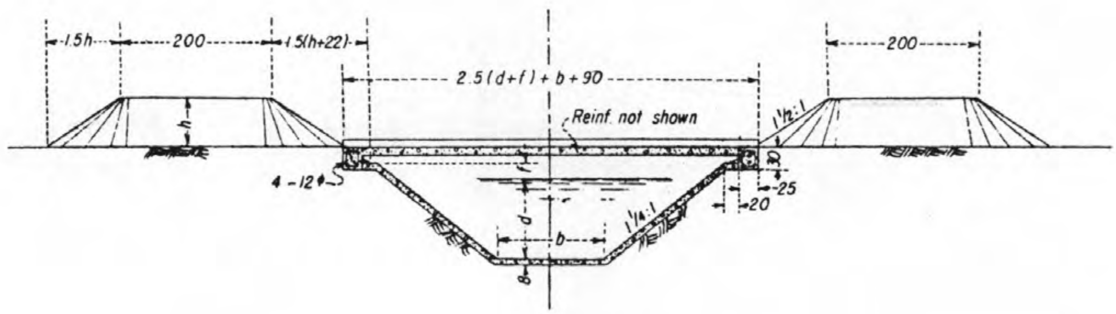
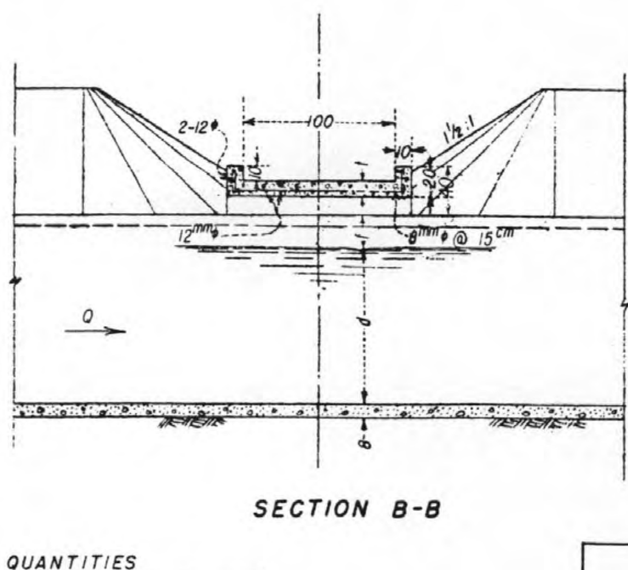


SECTION A-A



SECTION B-B

ESTIMATED QUANTITIES

Flow Q l/s	500	1000	1500	2000	2500
Concrete m^3	0.7	0.9	1.1	1.2	1.3
Steel bars Kg.	70	80	100	115	135

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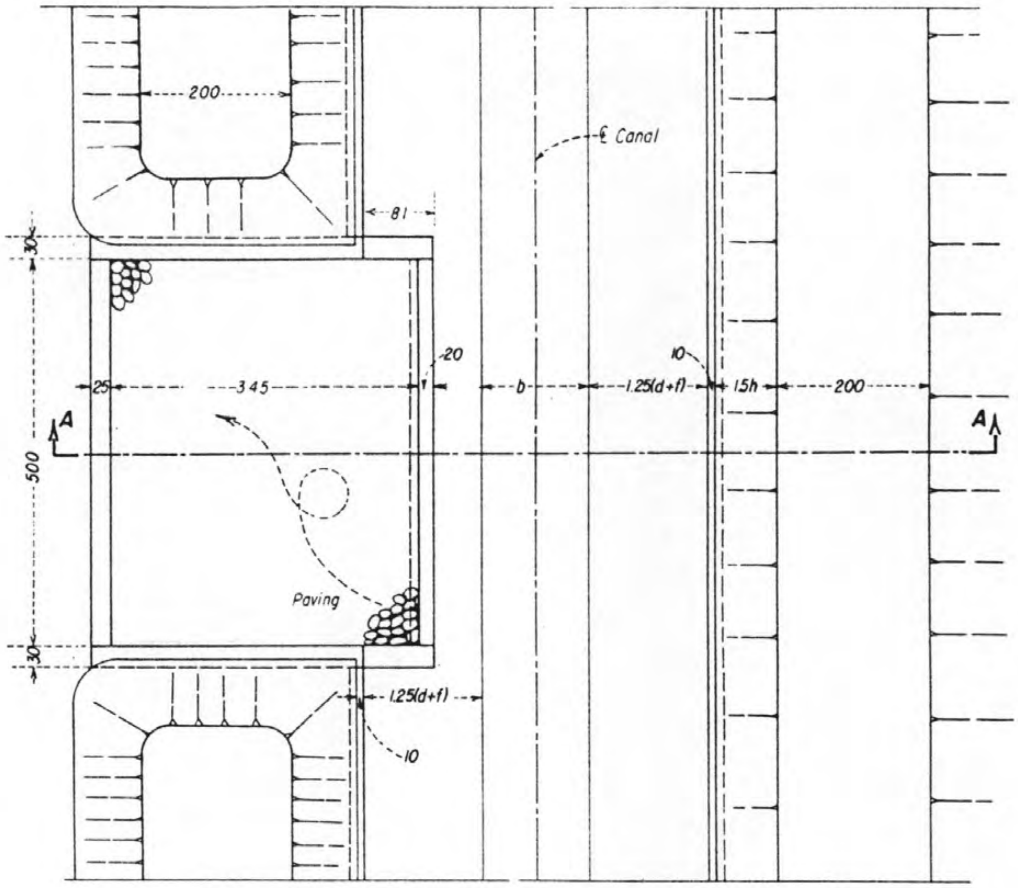
LITANI RIVER PROJECT – LEBANON
IRRIGATION UNITS

TYPICAL FOOTBRIDGE

[redacted]

BEIRUT, LEBANON. 4-23, 1954 OA – 10 – 245



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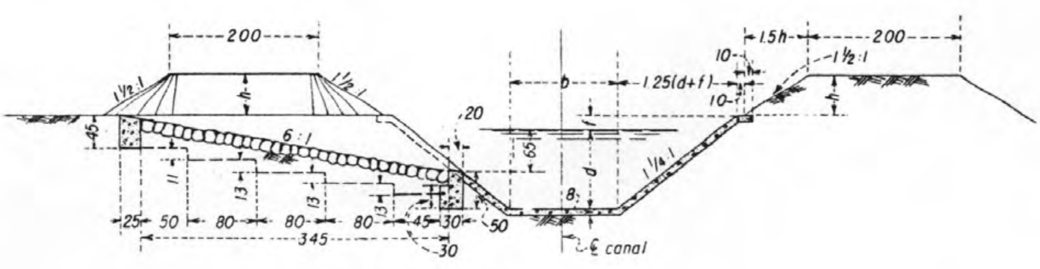

SECTION A–A

ESTIMATED QUANTITIES

Excavation	$20\ m^3$
Masonry	$1.7\ m^3$
Concrete	$1.5\ m^3$
Paving	$20\ m^2$

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IRRIGATION UNITS

TYPICAL STOCK WATERING RAMP

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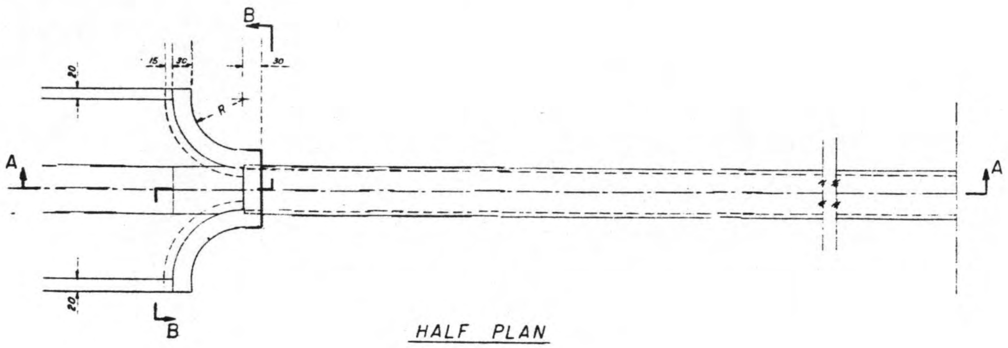
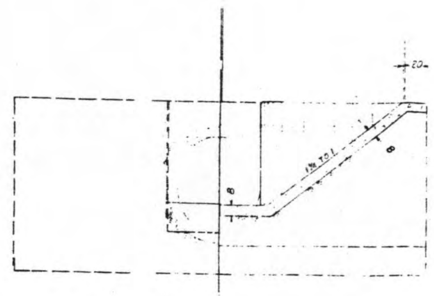
BEIRUT, LEBANON. 4-23, 1954 | OA – 10 – 246

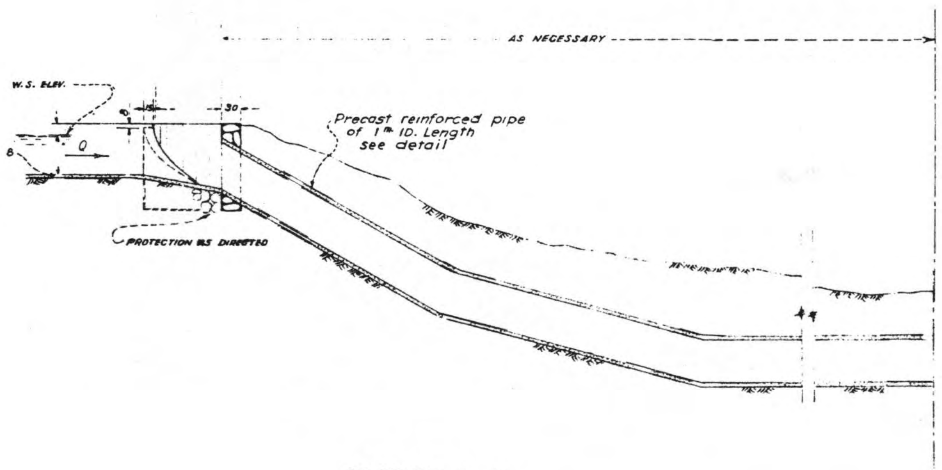
PLATE XXI-14



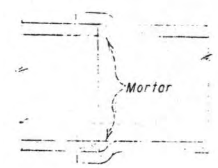
HALF PLAN



SECTION B-B



SECTION A-A



PIPE JOINT DETAIL

TRANSITIONS

Q l/s	Excavation m^3	Masonry m^3	Lining m^2
500	14	3.5	4
1000	22	5.0	5
1500	21	4.6	5
2000	28	5.7	7
2500	33	6.4	8

Note: Length of pipes and required excavation will be determined in the field.

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IRRIGATION UNITS

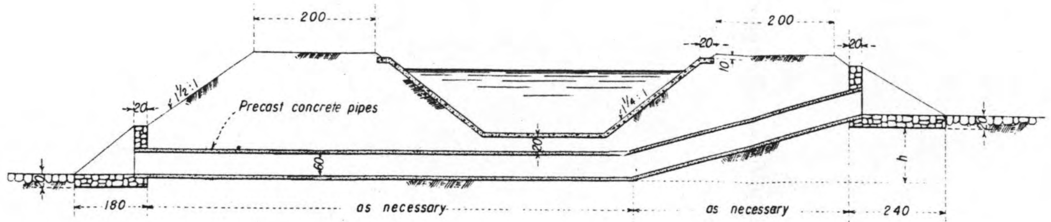
TYPICAL SIPHON

PLAN AND SECTIONS

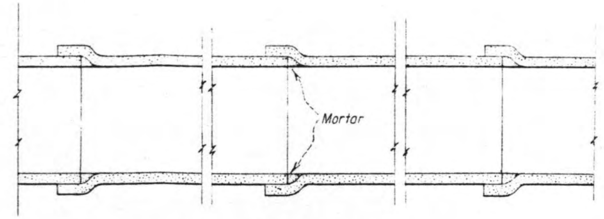
[redacted]

BEIRUT, LEBANON 4-23-1954 OA-10-247

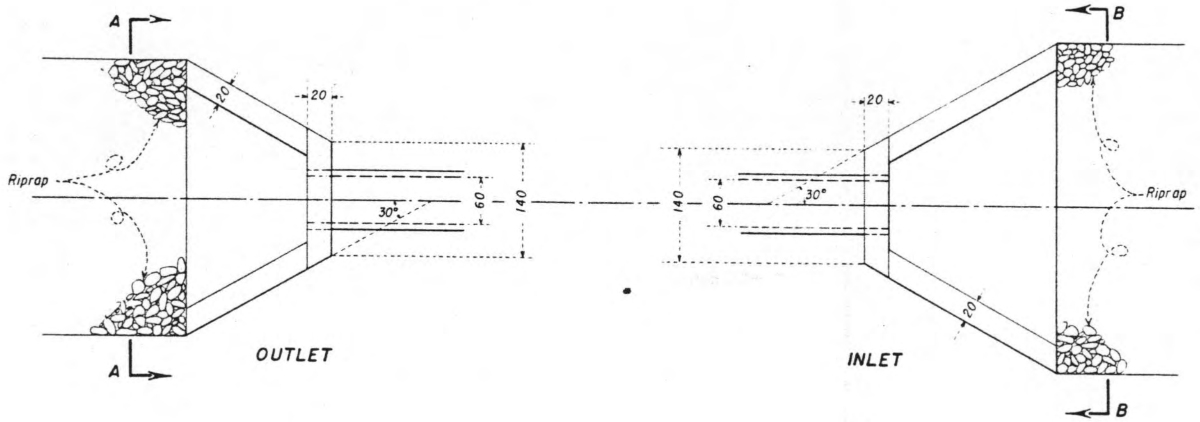
PLATE XXI-15



PROFILE
(Pipe joints not shown)



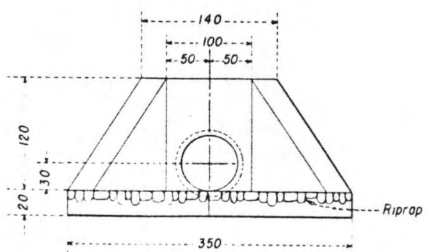
PIPE JOINT DETAIL



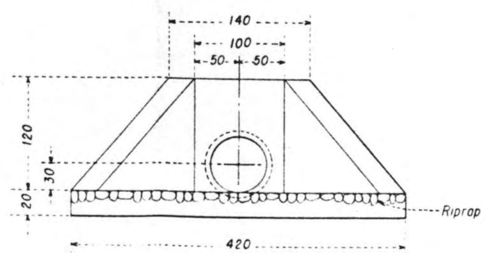
OUTLET **INLET**

ESTIMATED QUANTITIES

Masonry	4 m^3
Pipes D=60 cm	13 pipes of 1 m length
Excavation	10 m^3
Riprap	8 m^2



ELEVATION A-A



ELEVATION B-B

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LITANI RIVER PROJECT – LIBANON

IRRIGATION UNITS

TYPICAL DRAIN CULVERT

[redacted]

BEIRUT, LEBANON SEPT 14, 1953 OA – 10 – 248

Beirut's warehouse capacity is estimated at 80,000 metric tons in covered space and 200,000 metric tons in the open. It also possesses a free zone which was established to facilitate the transit trade.

Beirut Port Co. posseses a 50-metric ton crane mounted on a barge which can lift 30-metric tons at a span of 18 meters, in addition the Company owns several cranes from 5 to 15-metric tons.

Tripoli is the country's second port, but its shallow, undeveloped harbor makes it relatively unimportant except as a loading point for petroleum which transits Lebanon through the IPC pipeline to waiting tankers.

Labor Situation

Wealth and income are very unevenly distributed in Lebanon. Merchants, traders, capitalists, and professional men in the cities are prosperous. The economic status of the farming class varies markedly. Those who own their lands and have fair-sized units make incomes adequate for their needs. At the other end of the scale is the landless peasant who is very poor and whose productivity is very low. Despite the importance of trade, a great part of the population is rural and depends upon the land for livelihood. Unemployment has become a serious problem in Lebanon. It is estimated at between 30,000 to 40,000 for a working population not over 250,000.

The labor force of Lebanon is no doubt its chief industrial asset. The people of Lebanon are ahead of the rest of the Arab World in education and adaptability to industrial enterprise. Very few statistics are available on the number and geographic distribution of the labor force in Lebanon. Approximately 50 percent of the population in Lebanon can be classed as rural in 1951 as compared to 60 percent in 1939. Most rural areas showed a decline during the 1939-51 period due to the increase in the number of industries in urban areas, and the greater job opportunities arising in trade and services.

Many American engineers observing Lebanese construction methods are amazed by the amount of hand labor used almost every step of the way. Capital for construction machinery and equipment is scarce and interest rates are almost usurious. Contractors, having experienced high profits while using cheap labor are wary of investing money in machinery and equipment. Lebanese contractors are well aware of the value of mechanized construction, but there are many, and for them amply justified, reasons for not changing their method of operation. Even on such modern construction jobs as a 16-story building recently completed in Beirut, the only pieces of mechanical equipment used were concrete mixers and a construction elevator.

Lebanese laborers often work from 6:30 a.m. to 7:00 p.m. in the summer, but with short winter days it is usually too dark to accomplish much after 6 o'clock. There is no overtime. Separation allowances, distance from working site to the nearest town, relative cost of living in the area, number of children, age of workmen and years of experience add nothing to the wage of the unskilled laborer or skilled one. The average wage for an unskilled laborer varies between ₤L. 3 and 6 daily and for the skilled labor from ₤L. 7 to 14. The only factor which plays a part in determining the wage rate is the offer and demand of laborers. The Lebanese Government has set a minimum wage scale for monthly salaried employees and maximum working hours, but this law is silent about daily workers and their indemnities if fired.

Construction Power

There is no available power supply near any of the project features at this time. Initial construction power would have to be supplied by the contractor. After completion of the Sohmor Power Plant, ample construction power could be made available to any unit of the project by construction of suitable transmission lines.

Procurement of Materials and Equipment

Concrete aggregates are obtainable mostly in the limestone formations near all structure sites. Quarrying and crushing of these limestones will be necessary. Structures on the Bisri River can be built with natural aggregates which are available from the Bisri Power Plant site to the Bisri Dam site. Stone for riprap, masonry, etc., is available by quarry operations from limestone formations. A good grade of portland cement is produced at the Chekka Cement Plant which is located on the coast about 70 kilometers north of Beirut.

All wood and metal products, including reinforcing steel, form wood, wooden and steel poles and braces, fencing, penstock steel, etc., will have to be imported from Europe or the Americas.

All heavy construction equipment such as concrete mixers, bulldozers, trucks, earth moving equipment, and aggregate crushing equipment will have to be imported.

All electrical and hydraulic equipment, including generators, transformers, conductors, turbines, governors, radial gates, and high pressure gates, will also have to be imported.

Housing Requirements

At all of the construction sites it will be necessary to build temporary housing facilities for the contracting officer's engineers, technicians and inspectors. Although cost items for such facilities are included in the cost estimate of the various features, it is believed that the design and construction of the housing can be delegated to Lebanese engineers, who are familiar with local construction codes, methods, and materials.

It will be necessary for the contractor to construct housing facilities for his own key personnel. The majority of the local labor will come from the nearby villages and can be transported to and from the job by bus or truck. It may be necessary, in some locations, to provide low-cost living quarters and eating facilities for 30 percent of the labor force. The contractor should furnish all necessary warehousing, storage yards, and shop facilities. From 8 to 20 permanent houses will be required at the various sites for operating personnel after completion of construction. Undoubtedly, some of the temporary housing units provided for the contracting officer's personnel can be made to serve this purpose.

Construction Schedule

The time schedule proposed for construction of the various features of the Phase "B" units is shown on Plate XX-3. The plate also shows the estimated amount of money required each year for each feature plus the total required yearly for the units as presented in this report.

SECTION XXII

KARAOUN DAM AND RESERVOIR

Location

The second stage construction of Karaoun Dam and Reservoir will be accomplished by raising the initial structure. The dam will be located on the Litani River about 86 kilometers above the mouth. The location of the dam and reservoir is shown on Plate XX-1. The general plan, profile, and sections of the dam are shown on Plate XXII-1. A description of the first stage dam and reservoir is given in Volume II, Section XIV. The ultimate reservoir will extend 13 kilometers to the north of the dam. The reservoir will have an area of 1,100 hectares and an active capacity of 195 million cubic meters at the normal water surface elevation of 856 meters. The storage will be used for irrigation and hydroelectric power production. The maximum operating water surface elevation of this reservoir was fixed at elevation 856 meters to avoid flooding relatively large areas in the South Bekaa.

Geology

The general geology, reservoir geology, and ground-water tables are described in Volume II, Section XIV.

Hydrologic Data

Reservoir Levels. The ultimate maximum operating elevation to which this reservoir will normally be raised is fixed at 856 meters. The spillway and other outlets from the reservoir are designed for this level, except for short intervals during the passage of a major flood through the reservoir. For purposes of designs and estimates, the minimum reservoir water surface was assumed at elevation 820 meters. Plate XXII-3 shows the usable storage capacity of this reservoir and it was used in operation studies for this reservoir.

Tailwater Elevations. Approximate tailwater studies, as described in Volume II, Section XIV indicate that the tailwater elevation will be about 804.9 meters for the inflow spillway design flood of 785 cubic meters per second. During operation of the first stage dam and reservoir a stage-discharge curve for tailwater can be determined.

Fetch and Freeboard. Wind velocities and direction, wind set-up, and wave height are described in Volume II, Section XIV.

Ice Conditions. No special precautions, regarding either ice pressure or icing conditions, have been taken in the design of Karaoun Dam and Reservoir. Volume II, Section XIV contains a more detailed description of the ice conditions.

Sedimentation. Only a small amount of silt is carried by the Litani River above the dam site. The silt accumulation is estimated to be 2.3 million cubic meters during the first 100-years of reservoir operation. On this basis, the expected reservoir life exceeds 9,000 years. Sedimentation studies are described in Volume II, Section XIV.

Inflow Spillway Design Flood. The design flood hydrograph adopted for the inflow design flood has a peak flow of 785 cubic meters per second and a 20-day volume of 277.6 million cubic meters. This design flood is further discussed in Volume II, Section XIV.

Diversion During Construction. Hydrographs for the expected 5-, 10-, and 25-year floods at the Karaoun site show the expected maximum discharges for these frequencies as 103, 119, and 140 cubic meters per second, respectively. The hydrographs and a discussion of them is given in Volume II, Section XIV.

Reservoir Operation. Upon completion of the second stage construction of Karaoun Dam, the reservoir levels are assumed to be maintained so as to provide the minimum monthly storage capacities. These capacities were determined from studies of the complete project as discussed in Section XXXV. The capacities are the estimated minimum volumes to be held in the reservoir each month to reasonably insure adequate filling for a complete water supply during the following season. Observations during Phase "A" operation of the first stage dam may result in improvement of the operation curve presented.

Structural Design Data

General. The second stage construction of Karaoun Dam will involve the placing of earth and rock materials on the downstream face of the first stage dam. This will raise the top of the embankment from elevation 843.9 meters to elevation 859.9 meters. Also included in the work will be a new spillway and bridge structure, a new roadway over the dam, a new service road on the downstream side of the dam, and some minor highway relocations in the vicinity of the dam.

Volume II, Section XIV discusses the selection of type of dam, design stresses, earthquake design, stability, drainage, foundation grouting and reservoir treatment, sections of the dam, and other information applicable to both stages of the dam.

Foundation Grouting and Reservoir Treatment. Extensive grouting of the dam foundation during the construction of the first stage dam is expected to suffice for the second stage dam as well. Observations of the reservoir and dam leakage should be made during operation of the first stage dam. As mentioned in Volume II, Section XIV, this will enable a more accurate prediction of reservoir leakage when the dam is raised. To provide for possible reservoir and foundation treatment for the ultimate dam, contingent items for supplementary reservoir and foundation treatment in the amount of £L. 1,750,000 are included in the cost estimate for the second stage construction of the dam.

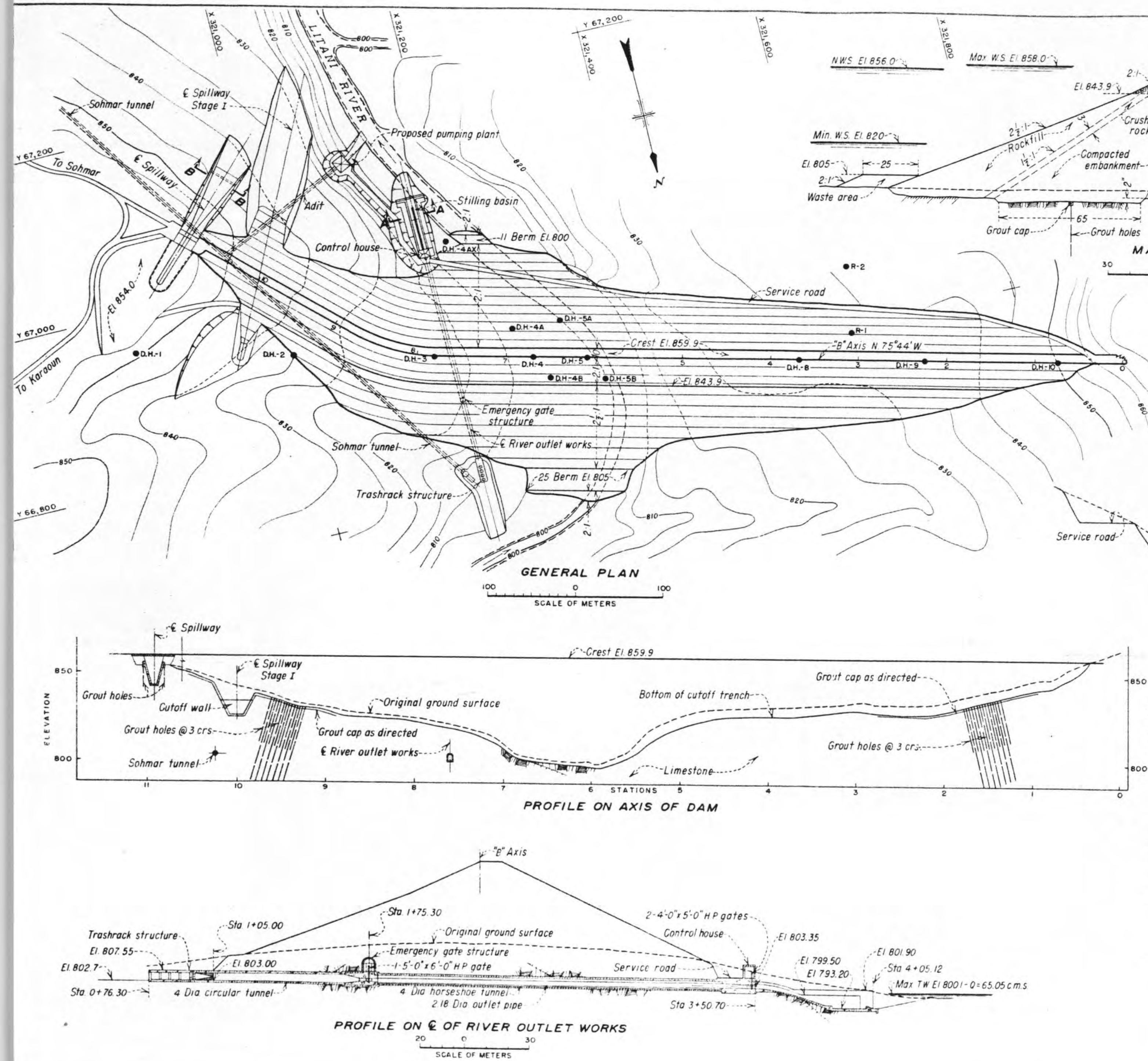
Sections of Dam. The second stage dam will be constructed by adding materials to the downstream face of the initial embankment, and will raise the crest of the first stage dam by 16 meters. The ultimate embankment will have a maximum height of 63 meters above stream bed. The crest will be 10 meters wide at elevation 857.4 meters. The Chtaura-Marjayoun highway will be relocated to cross the top of the dam. Sections of the second stage dam are shown on Plate XXII-1 and described in Volume II.

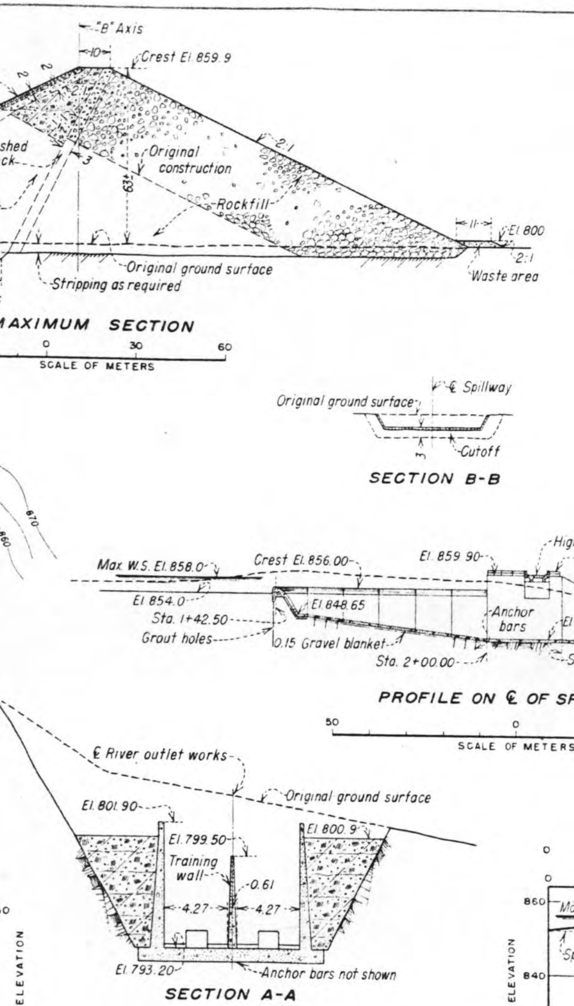
Spillway and Spillway Bridge. A new spillway structure with the crest at elevation 856.00, will be required adjacent to the original uncontrolled spillway. This ultimate spillway structure will be a double side channel overflow structure with an uncontrolled crest. The flood was assumed to start when the reservoir water surface was at the level of the spillway crest. The computations, show a maximum spillway discharge of 720 c.m.s. with a maximum reservoir water surface at elevation 858.00

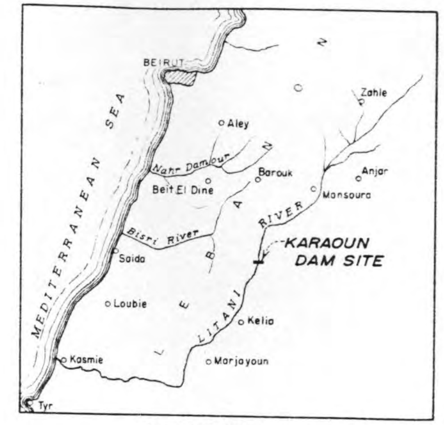
A reinforced concrete bridge will cross the spillway. This bridge will serve as a roadway. A 6-meter wide roadway is considered adequate for vehicular traffic. The bridge will be of reinforced concrete, continuously supported over four piers and the two abutments.

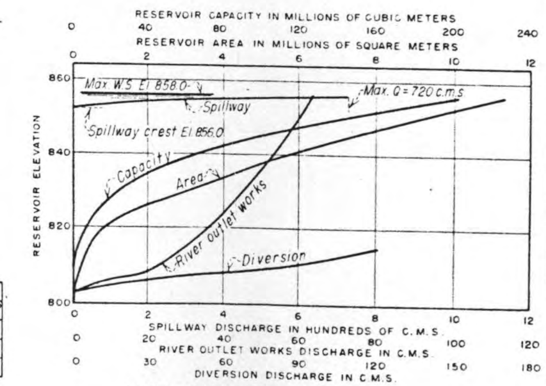
River Outlet Works. Completion of the stage one dam will include construction of the river outlet works as ultimately required. A general description of this structure is given in Volume II. During the second stage construction the outlet works may be used for river diversion. Provisions have been made to discharge water through the outlet works with the hollow jet valve removed.

Trashracks. The trashrack structure is discussed in Volume II. This structure will be included in stage one construction.









AREA-CAPACITY-DISCHARGE CURVES

RESERVOIR DATA

PURPOSE	ELEVATION	STORAGE MILLION CUBIC METERS
Power and irrigation	820 to 856	196.
Inactive and silt	800 to 820	6.4
	Total storage capacity	202.4

Surcharge storage of 21,000,000 C.M. (N.W.S. El. 856.0 to Max. W.S. El. 858.0) in combination with spillway releases is provided to protect against the inflow design flood. The spillway design flood has a peak of 785 C.M.S. and a 20 day volume of 277,620,000 C.M.

Note: All dimensions and elevations are in meters unless otherwise shown

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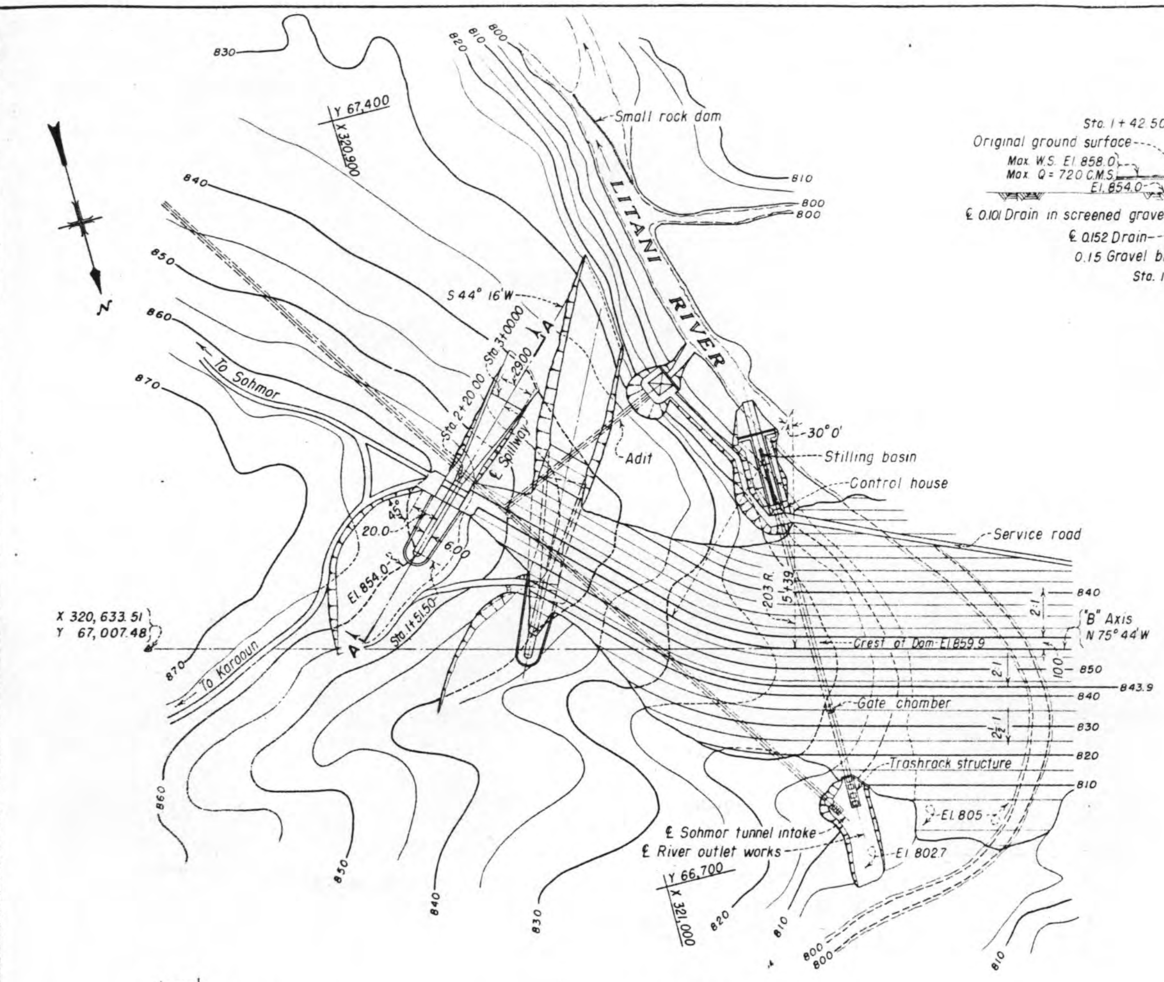
KARAOUN DAM
STAGE II
GENERAL PLAN AND SECTIONS

[redacted]

DENVER, COLORADO, MAR. 3, 1954

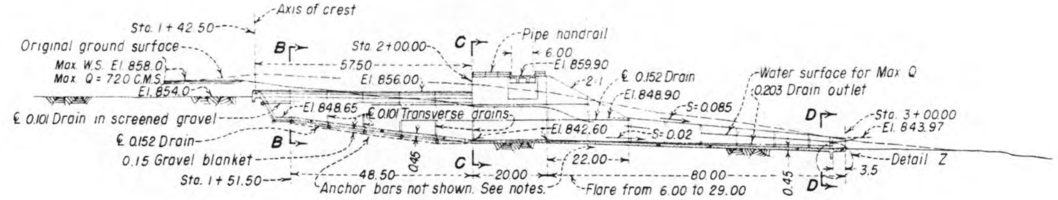
OA-10-304

PLATE XXII-1



PLAN

SCALE OF METERS

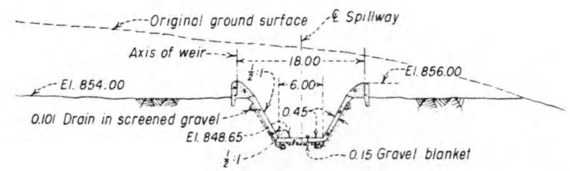


SECTION A-A

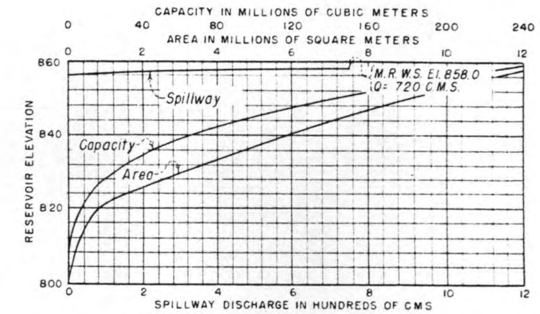
SCALE OF METERS

X	Y
0.000	0.000
0.500	0.066
1.000	0.253
1.500	0.546
2.000	0.937
2.500	1.421
3.000	1.997
3.500	2.671
4.000	3.450
4.500	4.349
4.627	4.598

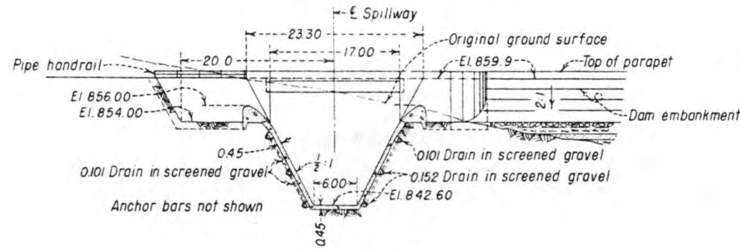
CREST COORDINATES



SECTION B-B



AREA-CAPACITY-DISCHARGE CURVES



SECTION C-C

NOTES

All dimensions and elevations shown are in meters.

All elevations shown are based on the French Army Geographical Service Datum of Mean Sea Level for 1926 at Beirut.

Spillway crest, sloping walls, and floors to be anchored with D25P, or equivalent, anchor bars spaced at 2 M horizontally and vertically and embedded 2.5 M.

Design flood taken from Dwg. OA-10-136 - Beirut, Lebanon, Sept. 21, 1953 - peak discharge, 785 m³/sec., R.W.S. at El. 856.0 at start of flood.

Keys and keyways in crest joints not shown.

Metal seals not shown.

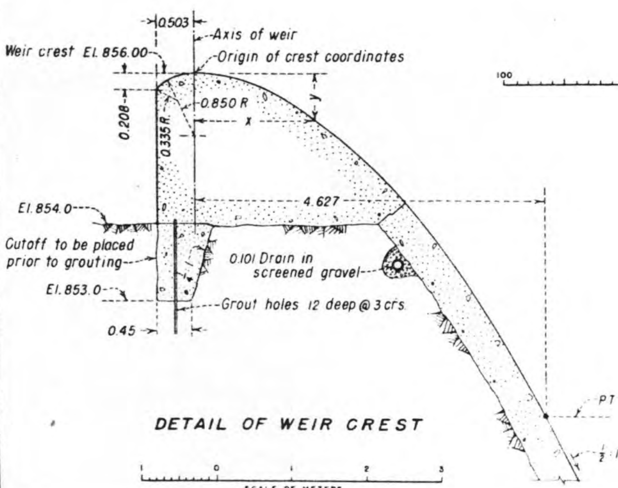
Reinforcement not shown.

Sizes of drains and reinforcement bars were converted to metric units from American standard sizes.

REFERENCE DRAWINGS

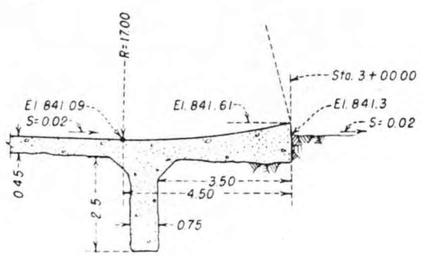
SPILLWAY - STAGE I - GENERAL PLAN AND SECTIONS OA-10-324

RIVER OUTLET WORKS - GENERAL PLAN AND SECTIONS OA-10-326

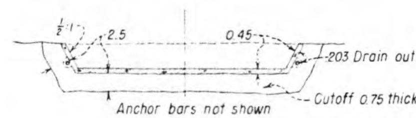


DETAIL OF WEIR CREST

SCALE OF METERS



DETAIL Z



SECTION D-D

SCALE OF METERS

UNITED STATES
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LITANI RIVER PROJECT - LEBANON

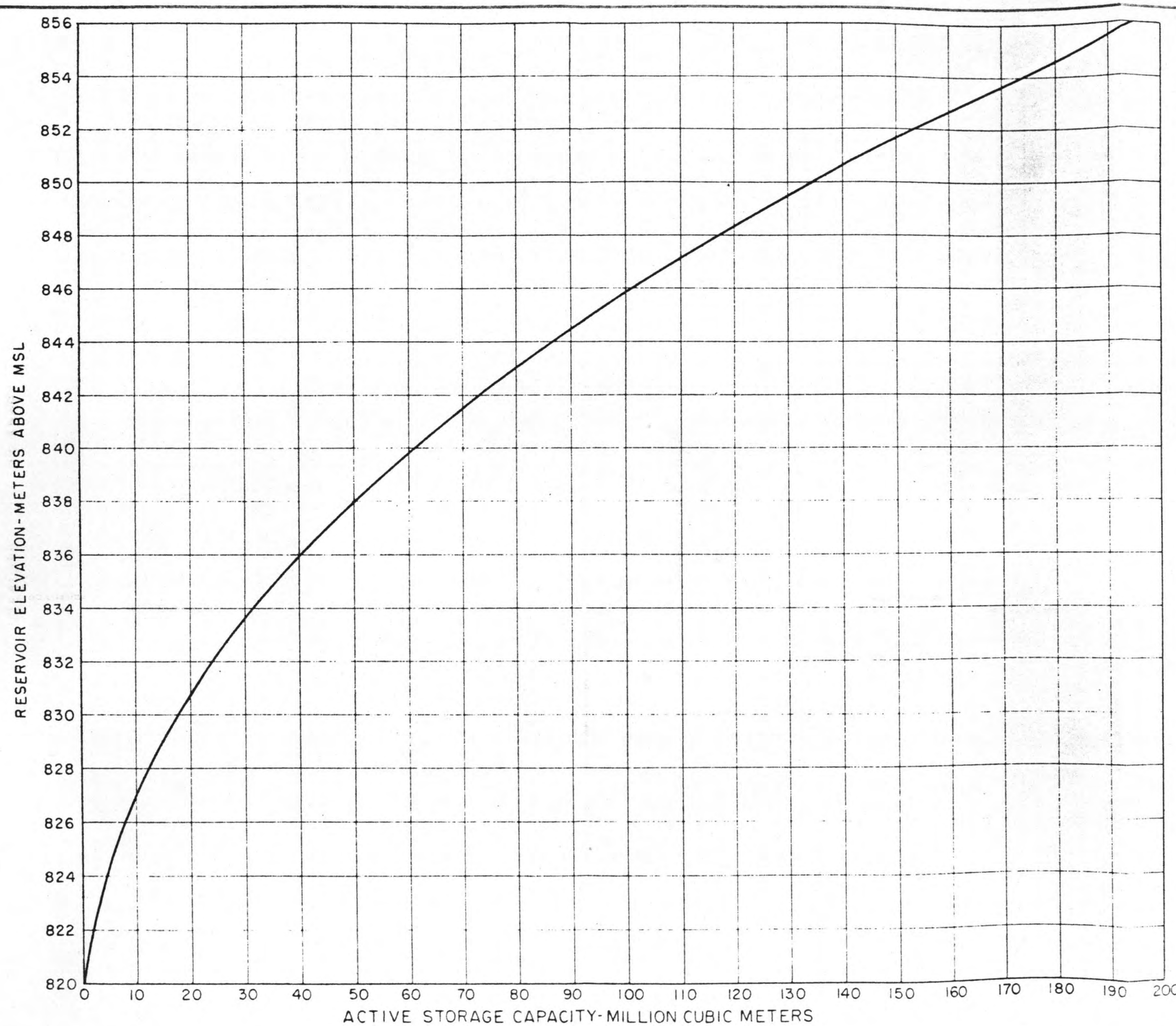
KARAOUN DAM
SPILLWAY
GENERAL PLAN AND SECTIONS - STAGE II

[redacted]

DENVER, COLORADO APRIL 12, 1954

OA-10-325

PLATE XXII-2



NOTE

The curve shown indicates the useable storage, estimated to be available at the end of a 100-year period, assuming a dam constructed at Axis B-B and the estimated sediment accumulation over the period deducted.

UNITED STATES
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BUREAU OF RECLAMATION

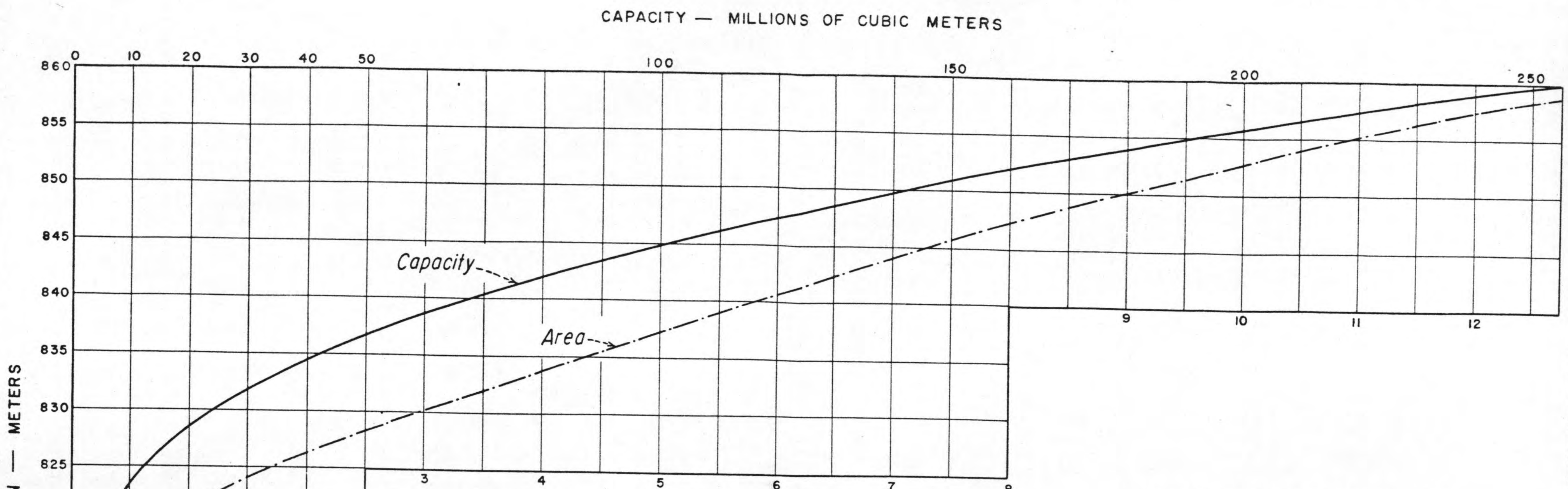
LITANI RIVER PROJECT — LEBANON

USEABLE STORAGE CAPACITY
KARAOUN RESERVOIR LITANI RIVER

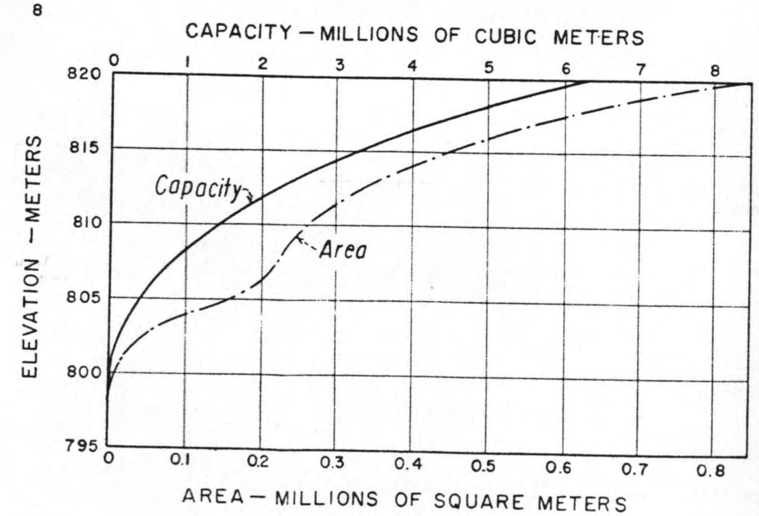
[redacted]

BEIRUT, LEBANON NOV. 13, 1953 | OA — 10 — 141

PLATE XXII - 3



ELEV. (METERS)	AREA (SQUARE METERS)	CAPACITY (CUBIC METERS)
798	0	0
800	6, 000	6, 000
805	160, 000	421, 000
810	263, 000	1, 477, 000
815	440, 000	3, 234, 000
820	828, 000	6, 403, 000
825	1, 711, 000	12, 751, 000
830	2, 971, 000	24, 455, 000
835	4, 369, 000	42, 804, 000
840	5, 820, 000	68, 275, 000
845	7, 291, 000	101, 053, 000
850	9, 002, 000	141, 787, 000
855	10, 907, 000	191, 560, 000
856	11, 340, 000	202, 684, 000
860	14, 270, 000	254, 501, 000



UNITED STATES
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BUREAU OF RECLAMATION
LITANI RIVER PROJECT—LEBANON
KARAOUN DAM—AXIS "B"
AREA—CAPACITY CURVES
[redacted]
BEIRUT, LEBANON 1-11-54 OA-10-44

PLATE XXII-4

Housing Requirements

It will be necessary to provide housing units for about 20 families near the Karaoun Dam site for the contracting officer's engineers, technicians, and inspectors. Some of these dwelling units may be used by the permanent operating personnel after completion of the construction. The design and erection of these houses can be delegated to local forces since they are familiar with the local building codes, labor, materials, and construction.

The contractors should provide adequate housing facilities for their supervisory personnel. Temporary quarters for laborers may also be required, however, experience during first stage construction may show that such provisions are not necessary. The majority of the laborers will come from neighboring villages and the contractor may need only to supply transportation. Warehouses, storage yards, and shop facilities should be furnished by the contractors.

Construction Schedule

The construction schedule for the second stage of Karaoun Dam is shown on Plate XX-3. For purposes of designs and estimates it is assumed that raising Karaoun Dam will commence 8-1/2 years after the first stage construction is started. The period of second stage construction assumes the use of rapid mechanized methods of construction, as mentioned in Volume II, Section XIV. Future investigation, however, may indicate the practicality of constructing the second stage embankment by the use of more primitive methods.

The cheap labor and the great number of unemployed persons in Lebanon may warrant a long-time construction program for raising the second stage embankment. Present local practices on construction jobs stress a liberal number of common laborers, a considerable number of draft animals, and a minimum amount of mechanical equipment. The long-time construction schedule can be adjusted so that additional reservoir storage can be provided only as the demand occurs. The height of the embankment required at any time can be determined from a time-storage curve of the combined power-irrigation demand.

Advantages of this slow schedule, over a more rapid mechanized methods are: (a) the reservoir and foundation treatment if required, will be simplified (more time for observation will allow a more beneficial treatment program), (b) the investiment loss will be minimized in the event of excessive leakage (progress can be halted until the leakage is safely reduced), (c) the reservoir storage can be more efficiently used, (d) only relatively small annual appropriations will be required of the financing agency, (e) repayments on the investment commence almost immediately (interest during construction is virtually eliminated), (f) the embankment settlement will be better controlled, and (g) the local labor force can be employed for a longer period.

A disadvantage of this slow construction method would be the necessity for passing flood discharges over the embankment during construction. This could be accomplished by providing a low saddle in the embankment at the first stage spillway area. Provision will be required for adequate protection of the exposed impervious zone against erosion during the passage of floods until such time as the ultimate spillway can be utilized.

SECTION XXIII

BISRI DAM AND RESERVOIR

Location and Description

Bisri Reservoir will be used as a single purpose reservoir for the storage of water for power production and will be formed on the Bisri River near Bisri village, and about 23 kilometers from the mouth of this river, by the construction of Bisri Dam. The reservoir will have a usable content of about 13.8 million cubic meters between the maximum reservoir elevation 430 and the minimum reservoir elevation of 422.55 meters. Bisri Dam will be a homogeneous earth fill type dam about 39 meters high above the streambed, and will require about 2,000,000 cubc meters of fill. It will require about 3 years to construct this dam. The locations of this dam and reservoir are shown on Plate XX-1.

Geology

The Bisri Dam site is located about 1-1/2 kilometers above a large landslide. This large mass of rock dammed the Bisri River many years ago when it moved across the river valley from its left side. Alluvial material was then deposited in the lake formed upstream from the slide area. At the present time the Bisri River is cutting through the landslide material. At the B-Axis the river is now flowing in a valley about 250 meters wide and is entrenched about 20 meters below the remnants of the alluvial terraces on either side.

It took several hundred years for this fill to accumulate behind the landslide. During this period leakage through this natural dam must have been small. As a result, the foundation material for the proposed dam appears tight and does not present a leakage problem.

The presence of four granite columns built by the Romans, in the upper end of the reservoir area helps date this alluvial fill. The columns are partially buried in at least 12 meters of alluvial silt and clay deposits and rise 5 meters above the ground surface.

The initial foundation investigations were made at the A-axis in an area where a talus and limestone ridge on the left abutment constricts the canyon. This location is about 1/2 kilometer upstream from the main mass of the ancient landslide. Here the alluvial fill was found to be about 115 meters deep in two drill holes near the center of the present canyon. The central portion of the proposed earth dam would rest upon this fill. At this axis the right abutment consists of marly dolomite which is massive and somewhat jointed and would require extensive grouting.

The left abutment was explored by nine drifts ranging from 10 to 70 meters in length. The drifts revealed a landslide and faulted mass of material varying from soft, wet clay to large blocks of limestone. These heterogeneous materials have slickensided surfaces, open seams, and cavities. The strike and dip of the rocks assume various positions. Blanketing along the upstream side of this abutment would be necessary to control leakage through this landslide mass. Even with blanketing the abutment would become saturated with water and its stability would be endangered.

The B-axis is about 1 kilometer above the A-axis in a narrow section along this reach of the river. Fourteen drill holes were drilled at this axis and the right abutment was swung up-river to avoid a badly jointed area disclosed in drill hole 1B.

Section A-A. This section is drawn between drill holes 1 B-B and 8B and is shown on Plate XXIII-1. In drill hole 1 B-B (45°) core losses were high, the drill water was repeatedly lost, and percolation tests showed high water losses in the Cretaceous (C4) marly dolomites in the upper 33 meters of the hole. Extensive grouting would be required from above the height of the alluvial terrace to near the top of the dam. Drill hole 2 B-B encountered limestone at 10 meters. Very little core recovery was made to the bottom of the hole at 36.7 meters. The overburden in the upper 9 meters of hole was not well consolidated and a core wall under the dam should be carried down to compact bedrock and some grouting will be necessary below the core wall.

The alluvial fill reached a maximum depth of about 94 meters as shown by drill hole 3 B-B, 4 B, and 5 B drilled near the center of the present canyon walls. The drill water was returned to the top of the hole in these 3 holes with 4-inch casing set to 38 feet showing that this alluvial fill is tight below 38 feet.

Drill hole 7 B and 7 B (45^{o}) were located at the foot of the left abutment in Jurassic limestones. Excellent core recovery was obtained in these two holes. Percolation tests showed high water losses due to the rock being fissured and jointed. A large regional fault (Khardale) exists in the river channel to the right of drill hole 7 B and the limestones are dipping steeply towards this fault beyond drill hole 7 B. This fault can be seen a short distance downstream from the B-axis.

Drill hole 8 B was drilled at an elevation about 30 meters above drill hole 7 B on the left abutment. The core recovery was good in the upper 35 meters of the hole and poor in the lower part. Percolation tests showed high water losses. Extensive grouting will be required at this abutment and some blanketing in the ravine immediately upstream from drill hole 8 B may be necessary to control leakage through the abutment.

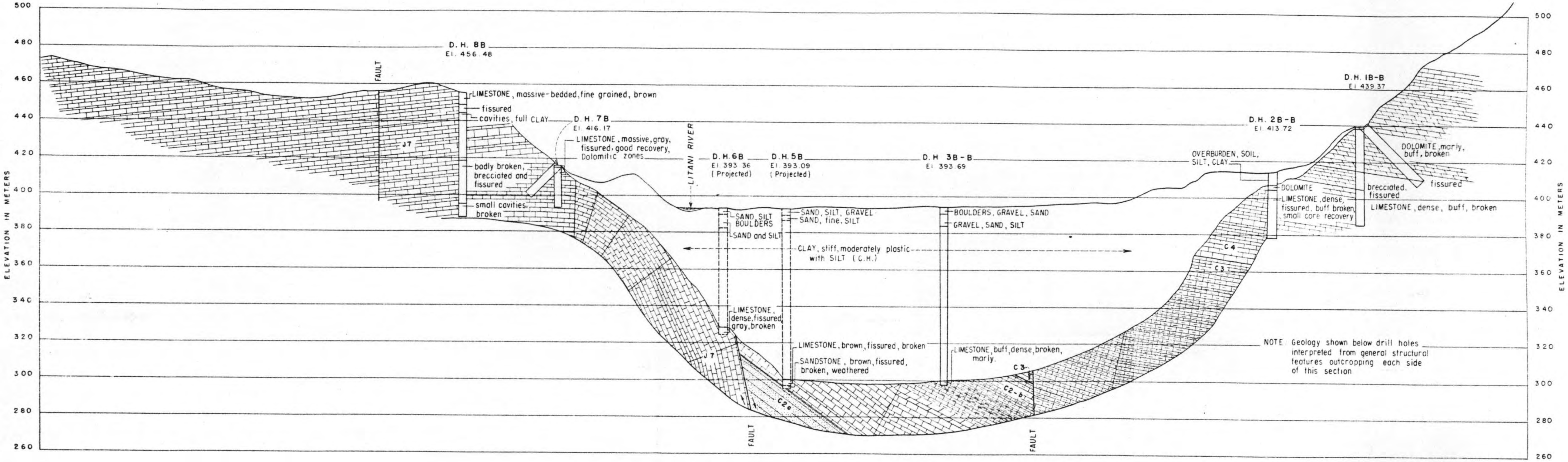
Reservoir Geology. The rocks in the reservoir area are composed of Cretaceous (C4, C3, C2b, C2a, C1) and Jurassic (J7, J6). The C1 and C2a series of Cretaceous rocks outcrop in the upper three-fourths of the reservoir area and are mainly composed of sandstones with interbedded marl and limestone. These rocks are fairly well blanketed and should not present a reservoir leakage problem.

The balance of the Cretaceous and Jurassic rocks outcropping in the vicinity of the dam site are all fairly tight and ground water would move very slowly through them.

Hydrologic Data

Reservoir Levels. Sedimentation studies for Bisri Reservoir indicate that the maximum sediment deposition in this reservoir in a 50-year period will be about 6,700,000 cubic meters of material. It is estimated that about 2 million cubic meters of this will be deposited in the top part of the reservoir and the remainder in the dead storage area. Existing silt terraces in the lower end of the reservoir are at about elevation 415 meters and the 50-year silt accumulation may be expected to partially fill the channel now existing between such terraces. It has been assumed therefore that the invert of the Awali Tunnel should be located 2 meters above this terrace level to insure future operations for this tunnel. This tunnel has been designed as a pressure tunnel with a diameter of 3.1 meters which added to the 417 meter elevation, selected as the entrance invert, results in an elevation of 420.1 meters for the top of this tunnel. Therefore a minimum pool elevation of 422.55 meters was selected for the Bisri Reservoir to provide sufficient water cover over the top of this tunnel for good operation. The maximum reservoir water surface was established at elevation 430 meters in order to equalize spillway discharges during the rainy season. Plate XXIII-2 shows the usable storage determined as being available after a 50-year period of operation and sedimentation. This curve has been used in operation studies for this reservoir.

Tailwater Curve. The Bisri gage was established on the Bisri River in November 1952 as a part of these investigations and has been operated since that time. It was located near the axis proposed at that time (A-axis) for Bisri Dam and was intended to serve as a tailwater gage for such structure. Later geologic investigations resulted in the selection of a dam site at the B-axis about one kilometer upstream from this gage.



BISRI DAMSITE (B-AXIS)

SECTION A-A

0 10 20 30 40 50 60 70 80 90 100 150

SCALE OF METERS

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BISRI DAMSITE

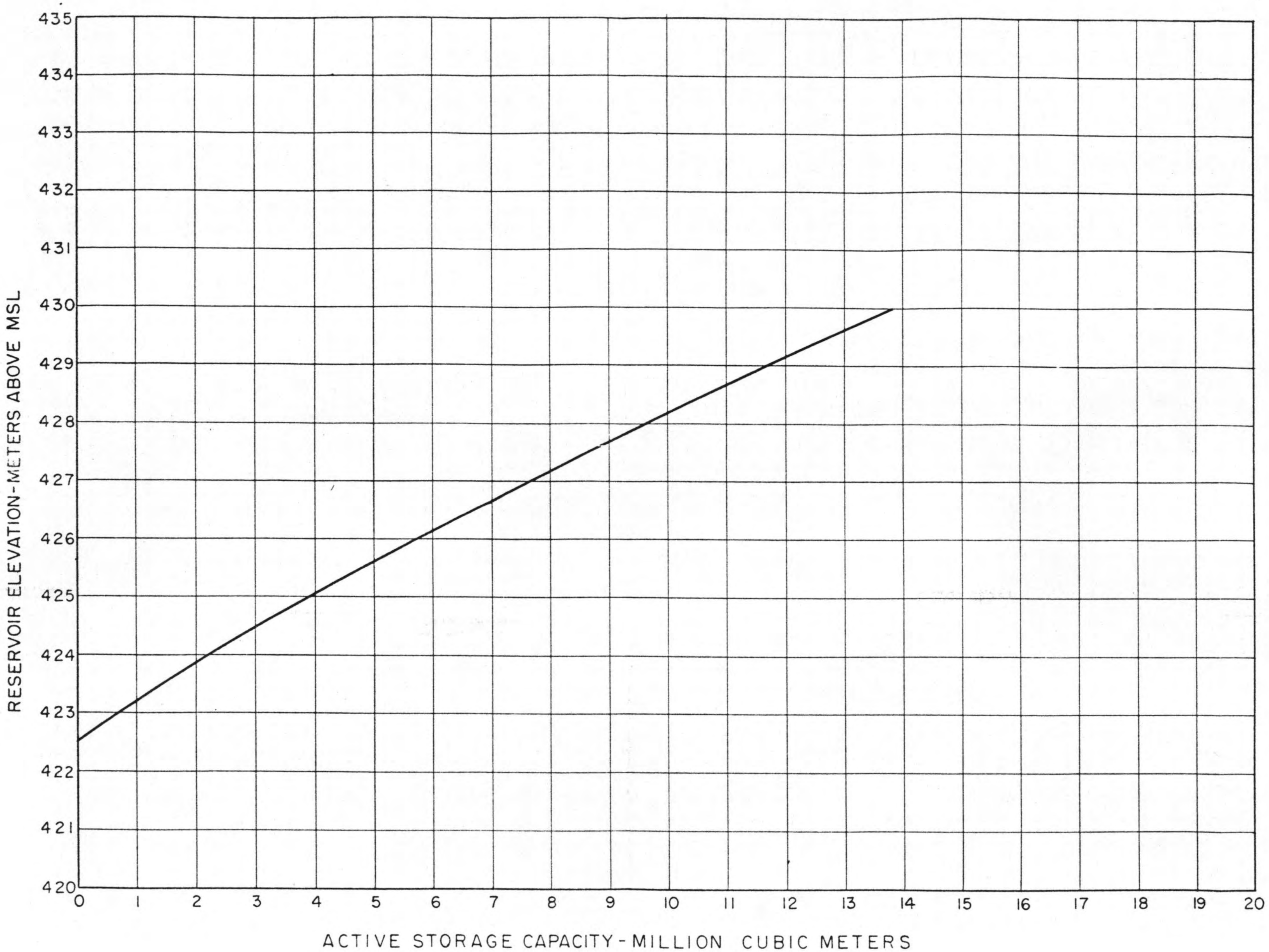
(B AXIS)

SECTION A-A

[redacted]

BEIRUT, LEBANON | SEP 15, 1953 | OA-10-35C

PLATE XXII



NOTE

The curve shown indicates the useable storage, estimated to be available at the end of a 50-year period, assuming a dam constructed at Axis B-B and the estimated sediment accumulations over the period, deducted.

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BUREAU OF RECLAMATION
LITANI RIVER PROJECT–LEBANON
USEABLE STORAGE CAPACITY
BISRI RESERVOIR–BISRI RIVER
[redacted]
BEIRUT, LEBANON 12-8-53 | OA–10–148

Channel conditions change so rapidly in the intervening reach of river from the gage to the selected dam site that it has not been feasible to transfer the stage-discharge relationship at the gage to the selected dam site. However, a tailwater curve has been estimated for the dam site by consideration of channel cross sections in this area and flows recorded at the gage. This curve is shown on Plate XXIII-3 and indicates that the tailwater below the dam may be expected to vary from an elevation of 391 meters at low water to 394 meters with a discharge of 775 cubic meters per second, estimated as the inflow spillway design flood. In this latter case it was assumed that the flood would occur when the reservoir was full to elevation 430 meters and that the peak discharge would remain the same as under the natural conditions existing prior to construction.

Fetch and Freeboard. The maximum wind velocity expected over the Bisri Reservoir was assumed to be the maximum gust velocity of 47 meters per second recorded at Ksara Observatory on January 22, 1951 from the west-southwest. This wind direction when applied to a line perpendicular to the axis of Bisri Dam results in an angle greater than 90 degrees and the wind component parallel to such a line acts in an upstream direction. It has been assumed therefore that no wind set-up will occur against Bisri Dam and that any minor depression that occurs at this point will be too small to be of any significance.

Wave height likely to occur at Bisri Dam has been determined by the Stevenson[1/] Formula as modified by Molitor[2/] which is as follows in English units for reservoirs whose fetch is less than 20 miles:

$$H = 0.17 \sqrt{VF} + 2.5 - \sqrt[4]{F}$$

where

H = wave height in feet

V = wind velocity in miles per hour

F = wind fetch in nautical miles

The fetch on Bisri Reservoir is estimated at 4.5 kilometers (1.9 nautical miles) and the wind velocity assumed at 47 meters per second. These values converted to English units and applied to the above formula result in a maximum wave height of 1.14 meters that may be expected at Bisri Dam. This wave height has been considered in determining the freeboard for this dam.

Ice Conditions. Temperature records indicate that this ice may form on this reservoir at times but its formation will be infrequent and its occurrence of short duration. No special precautions need be taken to provide for operation of this reservoir or its appurtenances during icing conditions.

Sedimentation. The reservoir area above Bisri Dam site is part of an old lake bed that was formed by an ancient landslide blocking the Bisri River Valley 1-1/2 kilometers below the dam site. This lake was filled with sediment. When the river finally cut a channel through the slide, it had already formed many meanders through such sediment deposits. Since that time it has deepened its channel along such meander lines and loose unconsolidated terraces remain along much of its channel through the reservoir area. At the present time high flows pick up and transport additional material from these terraces, but most of this movement will cease when the reservoir is constructed. Deposition will consist only of the material brought down from the river above the reservoir.

1/ Stevenson, Thomas - Design and Construction of Harbours; A Treatise on Maritime Engineering - Edinburgh, 1874.

2/ Molitor, D. A. - Wave Pressures on Seawalls and Break Waters - Trans. Am. Soc. Civil Engrs. Vol. 100, 1935, p. 984.

It was impractical to secure sediment samples at the head of this reservoir so they were taken at the Bisri gaging station about one kilometer below the proposed dam site and near the lower end of this area of heavy sediment deposits. Although the samples indicate present movement of material in the stream, they reflect much more transported material than will enter the completed reservoir. Hence the results of these measurements have been heavily discounted in determining the deposition likely to occur in the Bisri Reservoir.

The results of 24 sediment samples secured at the Bisri gage indicate a movement of about 177,000 metric tons annually at this gage under present conditions. Assuming a density of 1.325 metric tons per cubic meter, this would result in an annual deposit of about 133,500 cubic meters, or about 6.7 million cubic meters in a 50-year period. Since the measured movement is much higher than will actually occur after the reservoir is constructed, it has been assumed that the 50-year accumulation will be only 2 million cubic meters. This value has been used in determining the usable storage curve shown on Plate XXIII-2. Assuming that this same rate would continue, the reservoir would have a total life of 300 to 400 years.

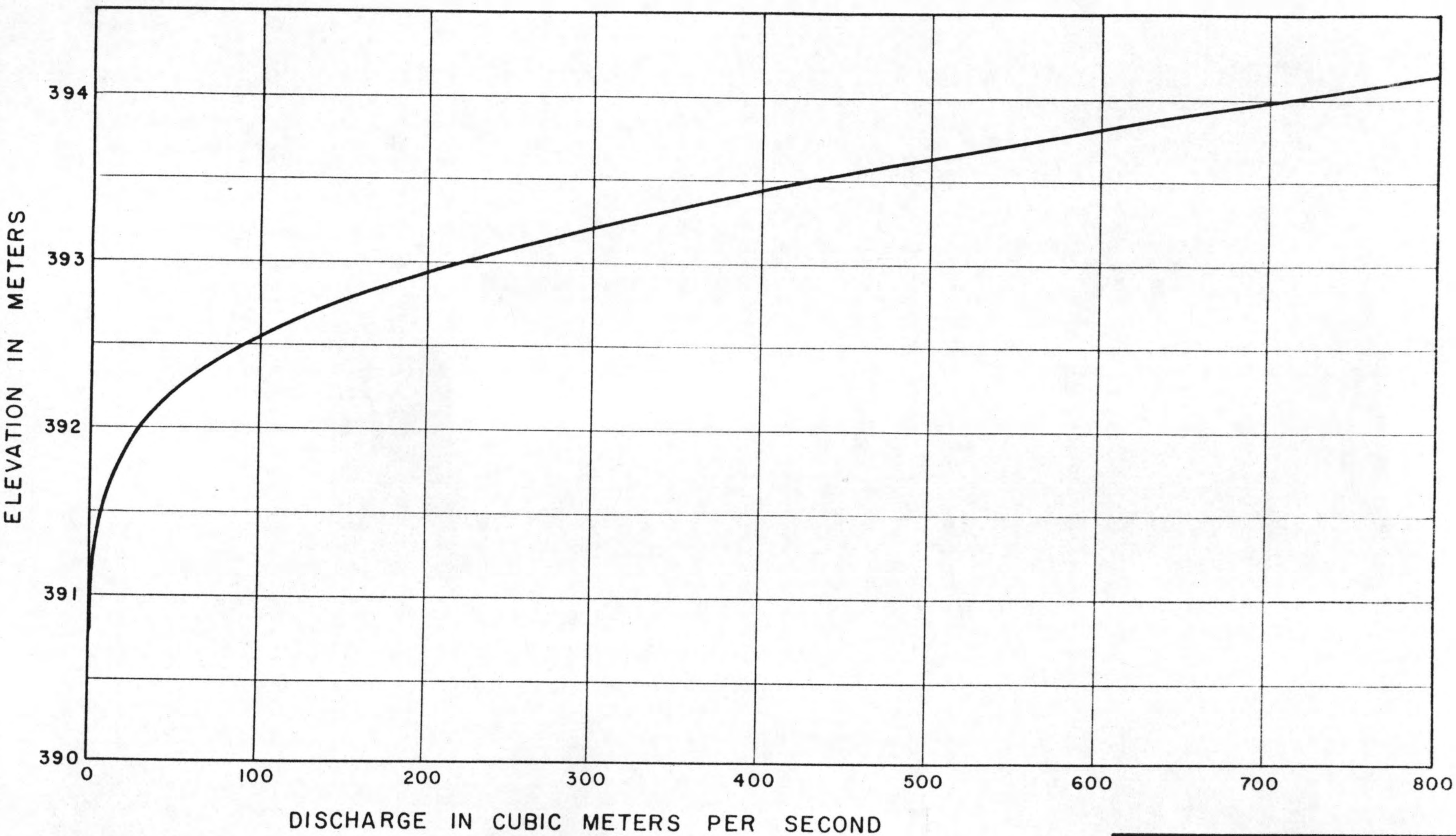
Inflow Spillway Design Flood. The hydrologic data available above the Bisri Dam site were too meager to permit the direct determination of an inflow spillway design flood. However, the use of a 3-hour unit hydrograph, determined by application of a 12.5-hour lag factor to the mean dimensionless graph for the general area, permitted the approximation of such a flood. This graph applied to the excess rainfall from the maximum expected storm resulted in the hydrograph shown on Plate XXIII-4 which has been assumed as the inflow spillway design flood for this site. This flood has a peak of 775 cubic meters per second and a 7-day volume of 96.3 million cubic meters. It may be expected to occur at any time between November and the end of March but is most likely to occur during January or February. The flood has been assumed to occur when the reservoir was at the level of the spillway crest, when routing it through the reservoir to determine the required size of the spillway.

Diversion During Construction. There were no suitable records available on the Bisri River for determining a frequency curve at the Bisri Dam site. However, the drainage area above this site is similar to that between the Karaoun and Khardale gages in the Litani Basin. Average coefficients of variation and skew were determined for the records at these gages and applied to the mean flood discharge from the area between them to obtain the frequency of floods at the Bisri Dam site. These frequencies were then used to determine the 5-, 10-, and 25-year hydrographs likely to occur at this site. These hydrographs are shown on Plate XXIII-5. They have the following significant values which should be considered in planning diversion requirements during the construction of Bisri Dam:

Frequency Years	Peak M^3/sec.	Volume $M^3 \times 10^6$ 1-day	3-day	5-day
5	65	4.2	7.2	7.8
10	75	4.8	8.2	9.0
25	88	5.6	9.8	10.7

Structural Design Data

Selection of Type of Dam. Local materials, economy, and the flexible foundation at the Bisri Dam site are the compelling factors which dictate the use of an earth-fill embankment for this structure. Accordingly, an homogeneous earth-fill section with protective rip-rap faces was chosen for the embankment.



Note: The tailwater curve shown was computed from valley cross-sections and channel characteristics determined from field survey data.

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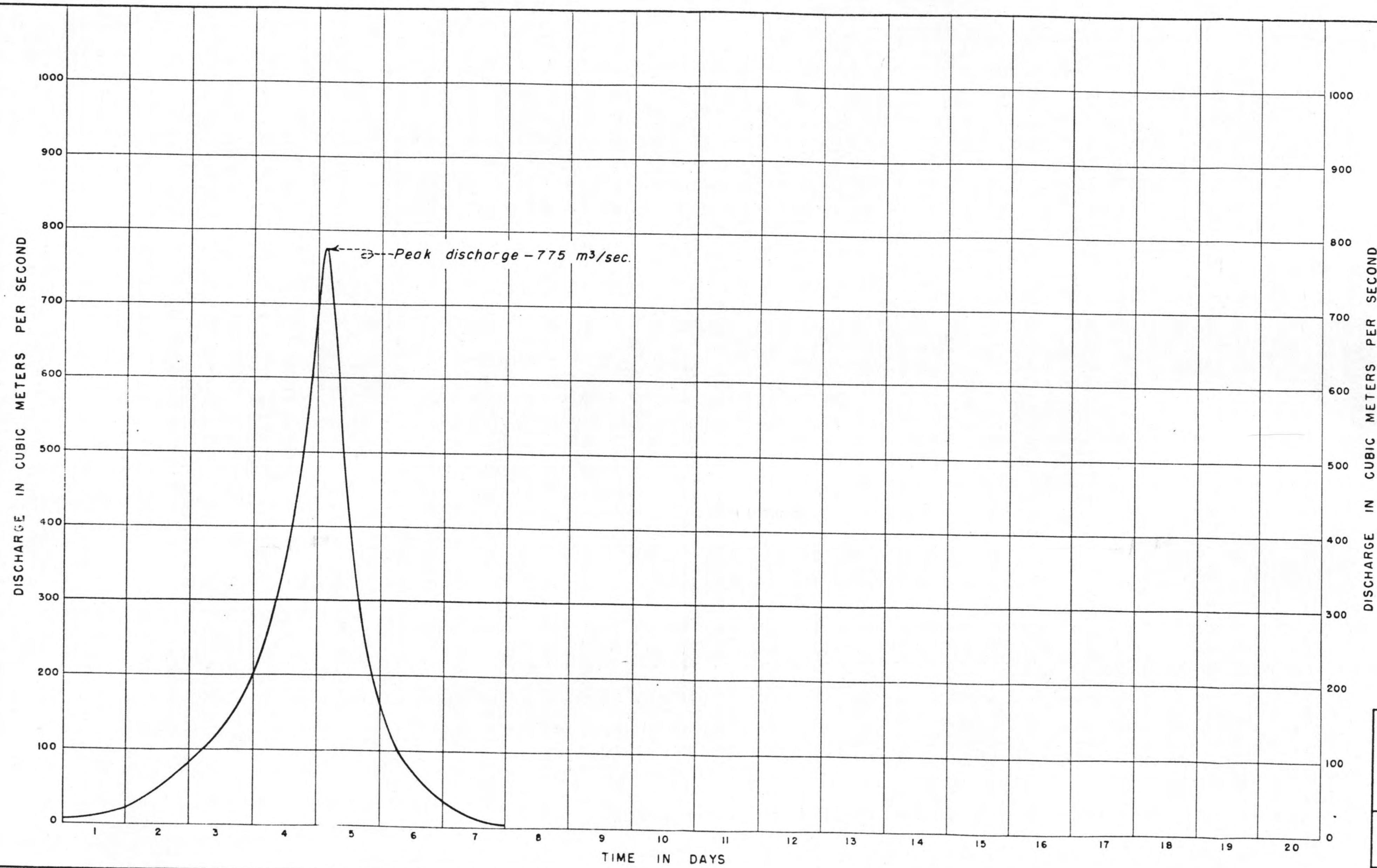
TAILWATER RATING CURVE
BISRI DAMSITE — AXIS B-B

[redacted]

BEIRUT, LEBANON 12-18-53 | OA-10-133

PLATE XXIII-3



NOTE

This inflow spillway design flood was determined by applying the Rational Method to the 222 square kilometers of area above the Bisri Damsite on the Bisri River. It has a 7-day volume of 96,340,000 cubic meters.

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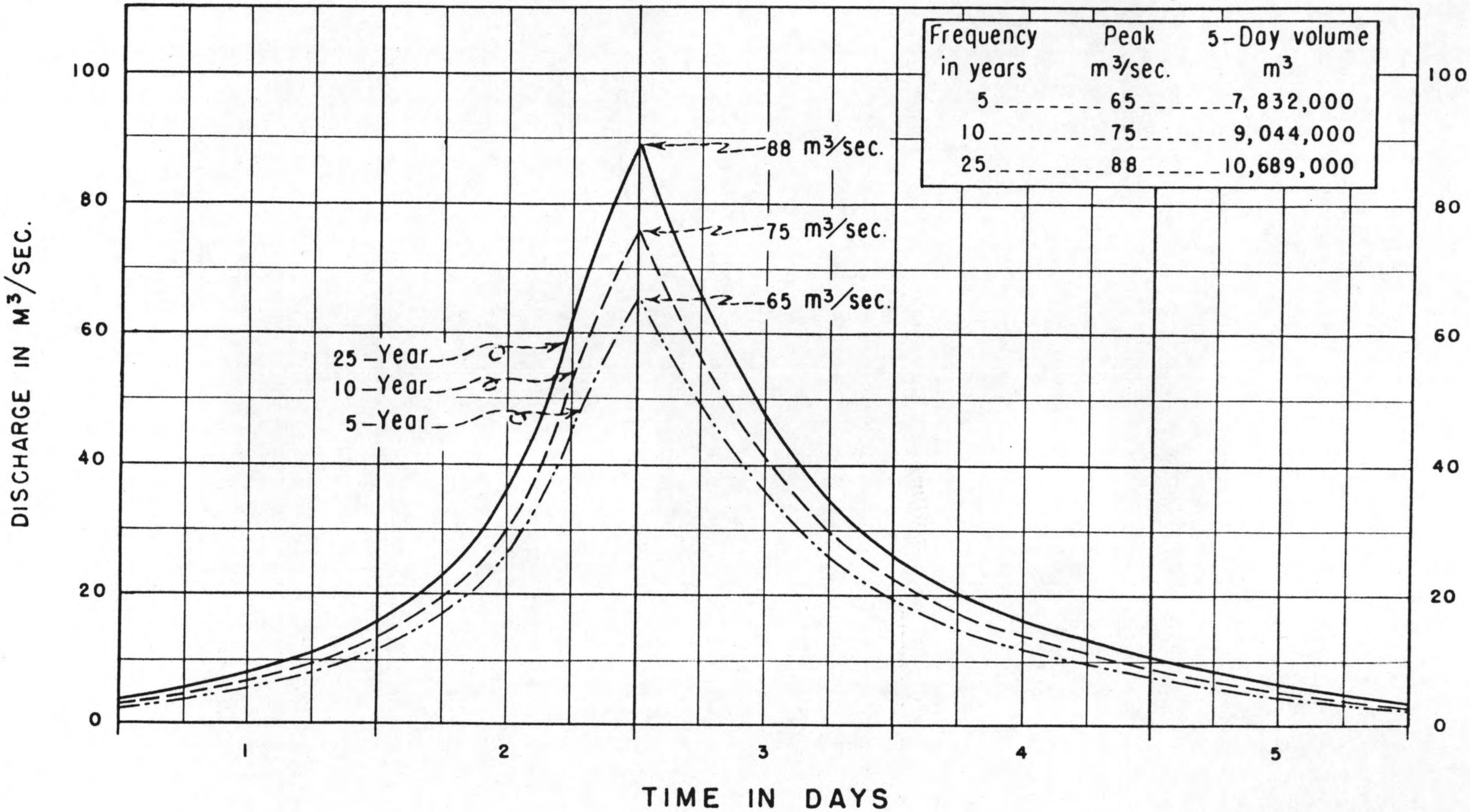
BISRI DAMSITE

INFLOW SPILLWAY DESIGN FLOOD

[redacted]

BEIRUT, LEBANON | SEP 21, 1953 | OA – 10 – 146

PLATE XXIII - 4



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LITANI RIVER PROJECT-LEBANON

BISRI DAMSITE

DIVERSION DURING CONSTRUCTION

[redacted]

BEIRUT, LEBANON 9-22-53 OA-10-140

filter and toe drain will prevent the downstream face of the abutment from becoming saturated.

Foundation Grouting and Reservoir Treatment. A curtain of cement-grout will be constructed below the concrete cut-off walls at the extreme ends of the embankment. For purposes of the cost estimate, grout holes were assumed at 1.50 meter centers to a depth of one-half the maximum hydrostatic head on the foundation or a minimum depth of 6 meters. No grout curtain is provided across the streambed.

The reservoir foundation is expected to be sufficiently water tight without any artificial treatment. Therefore, no cost for foundation treatment has been included in the estimate for Bisri Dam.

Grouting around the spillway structure and behind the diversion and spillway tunnel linings will be required. Spillway grouting has been assumed to consist of one line of grout holes to a depth of one-half the maximum hydrostatic head on the foundation.

All grout holes are assumed to require 0.10 cubic meters of cement-grout per lineal meter of depth. The rough preliminary cost estimate for Bisri Dam (Study B-10) includes a lump sum item for pressure grouting the dam foundation and tunnels to an amount of ɃL. 1,100,000. (Table XXXVII-2).

Sections of Dam. The embankment of the dam, with a maximum height of 39 meters above streambed, will have a 9 meter crest width at elevation 432 meters. The length of the crest will be about 440 meters. A concrete parapet to elevation 433 meters extends along the upstream edge of the crest. A service road will cross the top of the dam to provide access from Bisri village to the spillway structure and the Awali Tunnel intake portal at the right abutment.

The dam embankment will consist mainly of locally available impervious rolled fill. The upstream face of the dam will be protected by rip-rap and the downstream face will be protected by a layer of cobbles. At the downstream toe a mass of dumped rock-fill will serve as a drain and prevent percolation from transporting the earth-fill materials from the embankment.

The upstream face of the embankment will have a slope of 3-1/2:1 from the crest at elevation 432 meters to a 6 meter wide berm at elevation 410 meters. From this berm, the slope will be 5:1 down to the existing ground surface. The downstream face of the embankment will have a slope of 2-3/4:1 from the crest to elevation 410 meters. Below elevation 410 meters the slope will be 4:1 to the existing ground surface.

Proposed sections for Bisri Dam are shown on Plate XXIII-6. Quantities of materials required for construction of the embankment are shown on Table XXXVII-2.

Spillway and Spillway Bridge. The spillway structure, at the right abutment of the dam, will consist of an approach channel, a concrete crest, two-7.45 meter square-radial gates operated from a bridge above the crest, a vertical concrete lined shaft and elbow, a 7.50 meter diameter circular concrete lined tunnel, and a discharge channel to Bisri River. The proposed spillway arrangement is shown on Plate XXIII-6.

The spillway tunnel will utilize most of the diversion tunnel which will be plugged by concrete at the elbow of the spillway shaft. At the downstream end of the tunnel a concrete deflector will be constructed to dissipate most of the energy from high velocity spillway flows. A stilling basin is not provided at the spillway outlet since erosion of the streambed is this isolated location is not detrimental

The size of the spillway gates was determined by routing the reservoir inflow design flood of 775 cms over the spillway crest. The flood was assumed to start when the reservoir water surface was at the level of the spillway crest at elevation 422.55 meters. The computations show a maximum spillway discharge of 670 cms. The

A plentiful supply of suitable impervious material is available at the site. The required rock can be obtained from structural excavations and from adjacent quarry sites.

Design Stresses. For the design of appurtenant structures, such as the diversion and spillway tunnel and the spillway structure, it is recommended that the current Bureau of Reclamation values for allowable design stresses be used. The design stresses as now used in Lebanon are considered inadequate for work of such major importance as required for Bisri Dam.

Earthquake Design. Due to the magnitude of earthquake intensities in this region, all work for the design of Bisri Dam must include provisions for earthquake stresses. It is recommended that the applied design earthquake acceleration be 98-cm/sec^2 (0.10 gravity) with a period of one second.

Stability. Computations were made for the stability of the maximum embankment section by application of the principles discussed by Justin, Hinds, and Creager[1] for design of earth fill dams. Properties of the embankment and foundation materials assumed for the stability computations follow:

Weight of dry impervious embankment material	1,820 Kg/M^3
Weight of saturated impervious embankment material	2,130 Kg/M^3
Weight of submerged impervious embankment material . . .	1,130 Kg/M^3
Weight of moist impervious embankment material	2,090 Kg/M^3
Angle of internal friction for impervious embankment material	20°
Unconfined compressive strength of impervious embankment material	None
Weight of dry river fill foundation material	1,280 Kg/M^3
Weight of moist river fill foundation material	1,680 Kg/M^3
Angle of internal friction for river fill foundation material	0°
Unconfined compressive strength of river fill foundation material	1.27 Kg/cm^2

Preliminary computations for stability of the embankment indicate the selected maximum section is stable for all reasonable loading conditions including combined horizontal and vertical earthquake accelerations of 0.10 gravity. However, prior to construction of Bisri Dam, more complete investigations of stability should be made.

Drainage. It will be necessary to provide adequate filters in the base of the dam to control seepage through the dam. Cut-off trenches are recommended along the length of the dam. Where the embankment rests on firm rock, at the extreme abutments, concrete cut-off walls will be constructed. For the central portion of the dam, where the fill materials are placed on the foundation of stiff, moderately plastic clay, a cut-off trench will be excavated in the existing pervious streambed of sandy silt as shown on Plate XXIII-6. At the downstream toe of the dam a rock fill toe drain will be provided. At the maximum embankment section an inverted gravel filter will be constructed. The

1/ Justin, Hinds, and Creager - Engineering for Dams - John Wiley and Sons, New York, N.Y., 1944, Vol. III, pp 715-742.

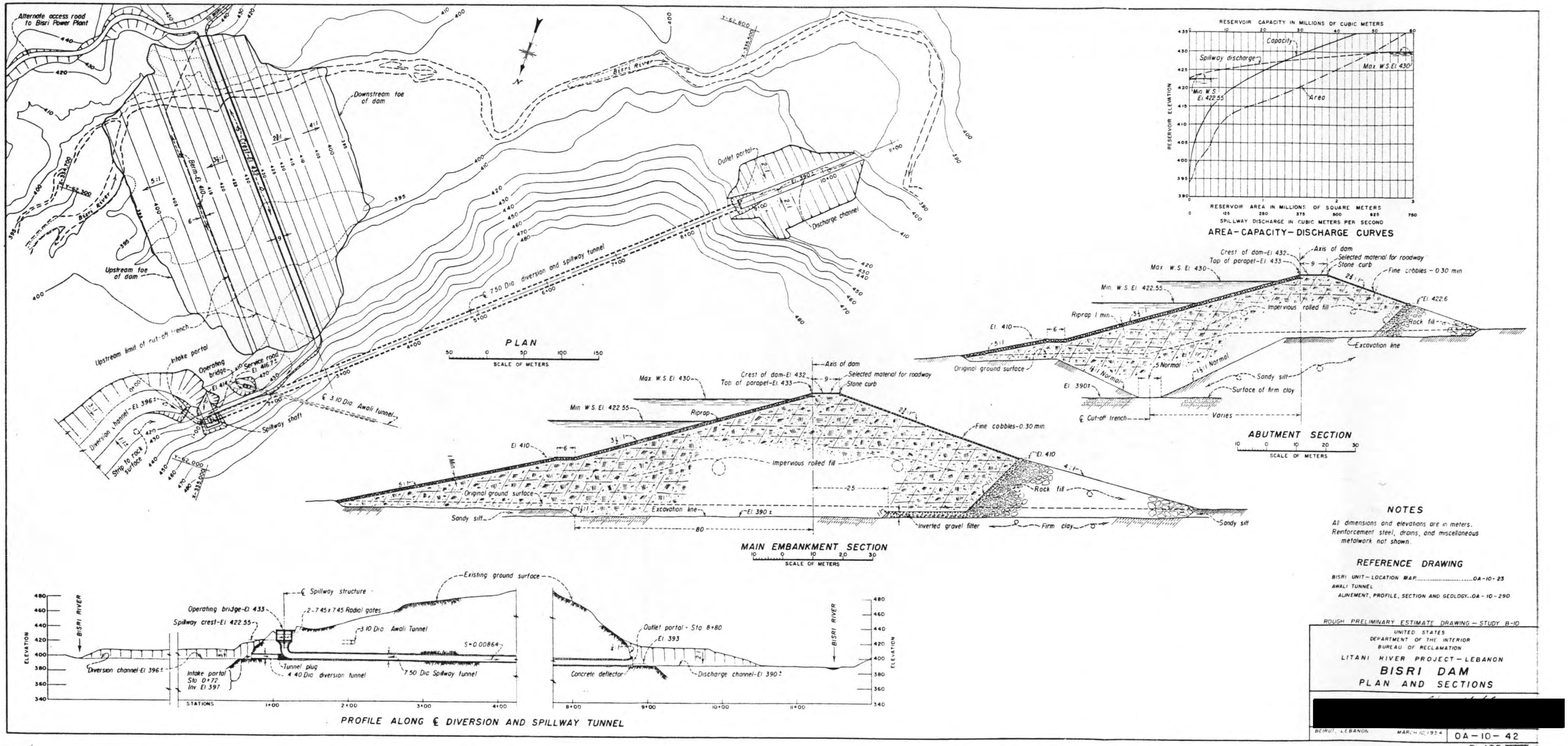
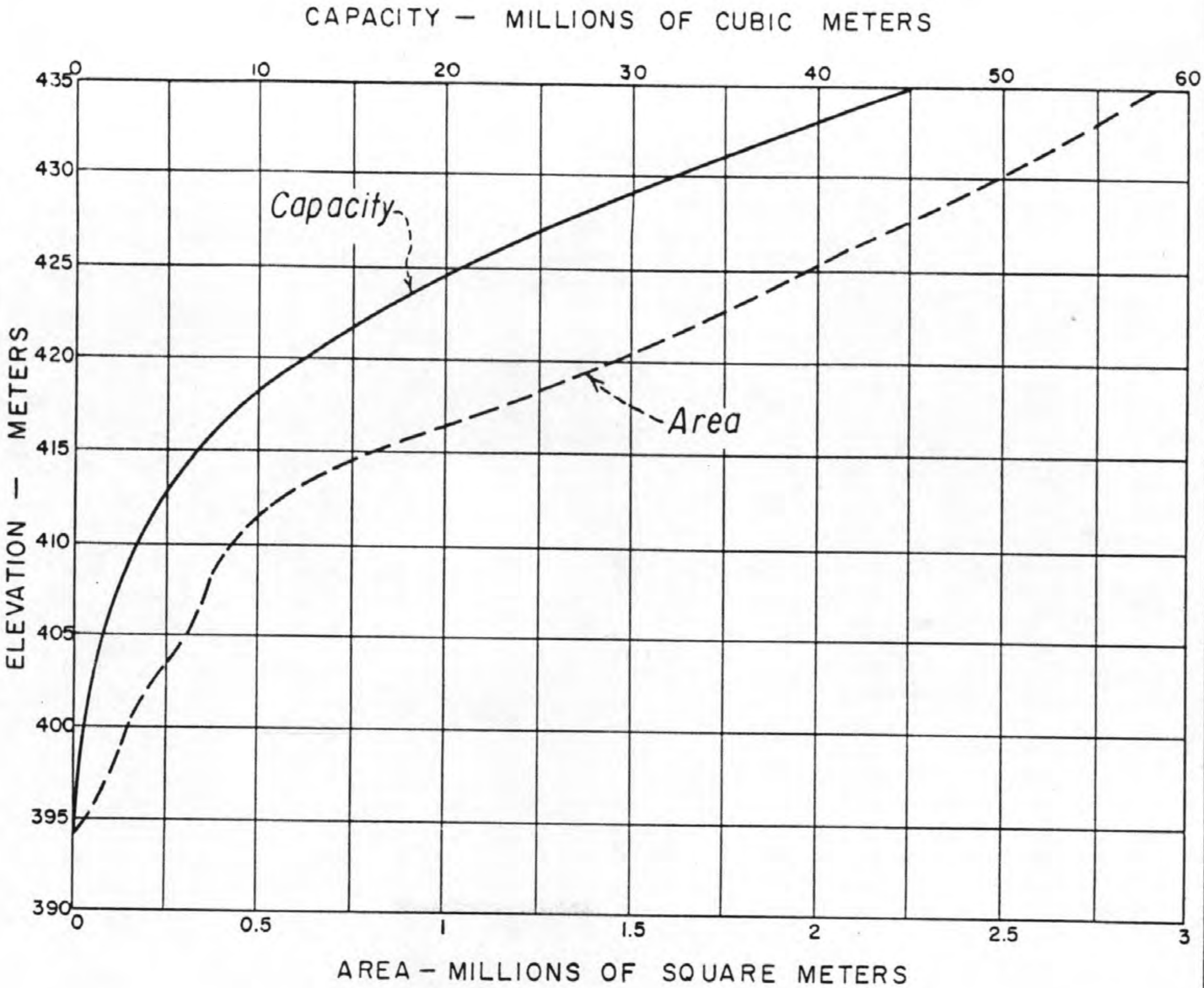

Alternate access road to Bisri Power Plant
Bisri River
Downstream toe of dam
Upstream toe of dam
Berm-El 410
Outlet portal
Discharge channel
℄ 7.50 Dia diversion and spillway tunnel
Upstream limit of cut-off trench
Intake portal
Operating bridge
Service road El 416.7±
El 414
℄ 3.10 Dia Awali tunnel
Diversion channel-El 396±
Spillway shaft
Strip to rock surface
PLAN
SCALE OF METERS
RESERVOIR CAPACITY IN MILLIONS OF CUBIC METERS
Capacity
Spillway discharge
Max W.S. El 430
Min. W.S. El 422.55
Area
RESERVOIR ELEVATION
RESERVOIR AREA IN MILLIONS OF SQUARE METERS
SPILLWAY DISCHARGE IN CUBIC METERS PER SECOND
AREA-CAPACITY-DISCHARGE CURVES
Crest of dam-El 432
Top of parapet-El 433
Axis of dam
Selected material for roadway
Stone curb
Fine cobbles - 0.30 min.
Max W.S. El 430
Min. W.S. El 422.55
Riprap 1 min.
Impervious rolled fill
Rock fill
Excavation line
Original ground surface
Sandy silt
Surface of firm clay
℄ Cut-off trench
Varies
ABUTMENT SECTION
SCALE OF METERS
Riprap
Fine cobbles-0.30 min.
El 410
Impervious rolled fill
Original ground surface
Rock fill
Sandy silt
Excavation line
El 390 ±
Inverted gravel filter
Firm clay
MAIN EMBANKMENT SECTION
SCALE OF METERS
NOTES
All dimensions and elevations are in meters.
Reinforcement steel, drains, and miscellaneous metalwork not shown.
REFERENCE DRAWING
BISRI UNIT-LOCATION MAP OA-10-23
AWALI TUNNEL
ALINEMENT, PROFILE, SECTION AND GEOLOGY ... OA-10-290
Existing ground surface
℄ Spillway structure
Operating bridge-El 433
Spillway crest-El 422.55
2-7.45 x 7.45 Radial gates
3.10 Dia Awali Tunnel
S=0.00864
Diversion channel-El 396±
Intake portal Sta 0+72 Inv El 397
Tunnel plug
4.40 Dia diversion tunnel
7.50 Dia Spillway tunnel
Outlet portal - Sta 8+80
El 393
Concrete deflector
Discharge channel-El 390±
ELEVATION
STATIONS
PROFILE ALONG ℄ DIVERSION AND SPILLWAY TUNNEL
ROUGH PRELIMINARY ESTIMATE DRAWING - STUDY B-10
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
LITANI RIVER PROJECT-LEBANON
BISRI DAM
PLAN AND SECTIONS
[redacted]
BEIRUT, LEBANON
MARCH 10, 1954
OA-10-42

PLATE XXII-6



ELEV. (METERS)	AREA (SQUARE METERS)	CAPACITY (CUBIC METERS)
394	0	0
395	38,000	19,000
400	141,000	469,000
405	310,000	1,596,000
410	428,000	3,441,000
415	832,000	6,592,000
420	1,468,000	12,340,000
425	1,972,000	20,950,000
430	2,492,000	32,110,000
435	2,915,000	45,630,000

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LITANI RIVER PROJECT— LEBANON

BISRI DAM—AXIS "B"

AREA—CAPACITY CURVES

[redacted]

BEIRUT, LEBANON 3-1-54 | OA-10-278

PLATE XXIII-7

Bisri Dam - Axis B - Looking Downstream

spillway crest elevation was chosen at the minimum reservoir operating level of 422.55 meters to avoid the necessity of a separate river outlet works structure. No provisions have been made for reservoir draw-downs lower than the elevation of the spillway crest, as such necessity has not been established. In the event of emergency need for additional draw-downs, the Awali Tunnel with invert elevation 417 meters, could be used. Also, the concrete plug in the diversion tunnel could be removed to permit total reservoir draw-down.

Construction Quantities

Principal construction quantities for Bisri Dam include 1,730,000 cubic meters of earth-fill, 320,000 cubic meters of rock-fill, and 17,800 cubic meters of reinforced concrete. Construction time is estimated as three years. The total estimated cost for the dam and reservoir, including interest, is LL. 24,500,000. The itemized rough preliminary estimate for this design (Study B-10) is shown on Table XXXVII-2. The estimate drawing showing the general plan and sections for Bisri Dam is shown on Plate XXIII-6.

Construction Materials

The impervious borrow material for the dam will be taken from benches beside the river. These rise 15- to 20-meters above the streambed and consist of silt and fine sand in typical loess-like structure. Excavation and placing this material will be a simple process. However, the excavation of this impervious material should be restricted to the areas above the dam site, since the area below the dam site is highly cultivated. The impervious material has a maximum compacted dry density of 1,820 kilograms per cubic meter at 15 percent moisture content.

Rock materials for the dam can be supplied from required excavation for the dam appurtenances and from unlimited potential quarries in the immediate vicinity.

SECTION XXIV

KHARDALE DAM AND RESERVOIR

Location and Description

Khardale Reservoir will be used as a multiple purpose reservoir for the storage of irrigation and power water and will be formed on the Litani River about 50 kilometers from its mouth, by the construction of Khardale Dam. It will have a usable water content of about 70 million cubic meters between normal water surface elevation 295 meters and minimum water surface elevation 257 meters. Khardale Dam will be an earth-fill embankment, about 61 meters high above streambed and will contain about 720, 000 cubic meters of earthfill; 428, 000 cubic meters of rock-fill; and 14, 000 cubic meters of concrete. It will require about 3 years to construct. The locations of this dam and reservoir are shown on Plates XX-1 and XXVIII-1.

Geology

The Khardale fault zone passes through the dam site area with a large displacement along the Litani River. This fault extends southward into Palestine where it joins the Yamoune fault and it extends northward for about 25 kilometers to the Bisri Reservoir area.

Step-faulting occurs on the left abutment with the older C2a and C2b Cretaceous rocks in contact with the younger C4 Cretaceous rocks along the river portion of the Khardale fault zone. Near the top of the dam site on the left abutment, the C2b series of rocks have been step-faulted to the surface again.

As a result of this intricate faulting the left abutment is badly broken and the large amount of talus covering the slopes makes it difficult from a surface study to visualize the structural relationships which exist. An anticline exists at this dam site with the axis of the anticline parallel to the river and with the rocks on the flanks of the anticline dipping 15° to 25° on each side of the river.

Two angle holes (DH7A and DH8A) were drilled under the river. Several zones of basalt and volcanic tuffs were found in the upper 27 meters of these holes which represent lava flows intruded into the Cretaceous sediments along this portion of the Khardale fault zone.

A steep cliff exists above the right abutment composed of the Cretaceous C4 limestones and dolomites. High above the dam site near the upper part of the cliff another branch of the Khardale fault zone exists.

Foundation explorations were made at axes located about 175 meters apart. Drill holes L3, R2 and R3 were drilled on the upper axis (B-axis) and disclosed badly broken and fractured limestones on the right abutment. The left abutment was also considerably broken and core losses were high.

Twenty-four holes were drilled at the lower or A-axis of this dam site. The location of these holes is shown on the topographic map of this area included in the folio of maps discussed in Section XXI. Thirteen of these holes are along the axis of the dam, five at the upper axis, two along the proposed spillway location, three along the diversion tunnel route and one near the proposed toe of the dam on the right abutment.

In the 13 holes along the A-axis, core losses and percolation losses were high. A deep cut-off wall and grout curtain will be necessary on the left abutment and for some distance beyond the abutment. In the river section, the rocks have been crushed and broken along the fault zone and the cut-off trench may have to be constructed 50 to 60 feet below river level with a heavy grout curtain below the cut-off wall. The right abutment is mainly in dolomite with marl bed. These rocks are weathered and fractured near the surface and will require considerable scaling before the embankment is placed. A deep grout curtain should also be placed in this abutment.

The shale and marl encountered at a depth of about 23 meters in drill hole 7 and 8, located on each side of the river, is impervious even though it is crushed, and may tightly seal the rocks below. However, sandstone was encountered in drill hole 7 at 32 meters which was impossible to core because it was fine grained and unconsolidated. This sandstone is dipping downstream and was encountered at a high elevation in drill hole R-2 located upstream from the toe of the dam. The sandstone outcrops in the reservoir a short distance above the dam. The area immediately upstream from the dam may require blanketing to control leakage through these porous sandstone.

The ground-water table on the left abutment rises above 30 meters between the river and drill hole 2. This fairly steep gradient occurs mainly in the C2a series of sandstone which contain many marly and shaly zones. Percolation losses were high in all of the holes drilled on the left abutment. Casing had to be carried to a depth of 169 feet in drill hole 2 in order to drill below this depth. The apparent discrepancy between the high percolation losses and fairly steep ground water gradient can be explained by the fact that considerable ground water is fed into this area from the mountains to the east which keeps the ground-water table high. Reservoir leakage through this abutment will be high unless a deep grout curtain is placed.

The diversion tunnel will be excavated on the left abutment. Drill holes DT1, DT2, 4, and DT3 were drilled near the proposed tunnel alinement. Since most of the rock is broken to some extent along this route, steel supports will be required in most of the tunnel.

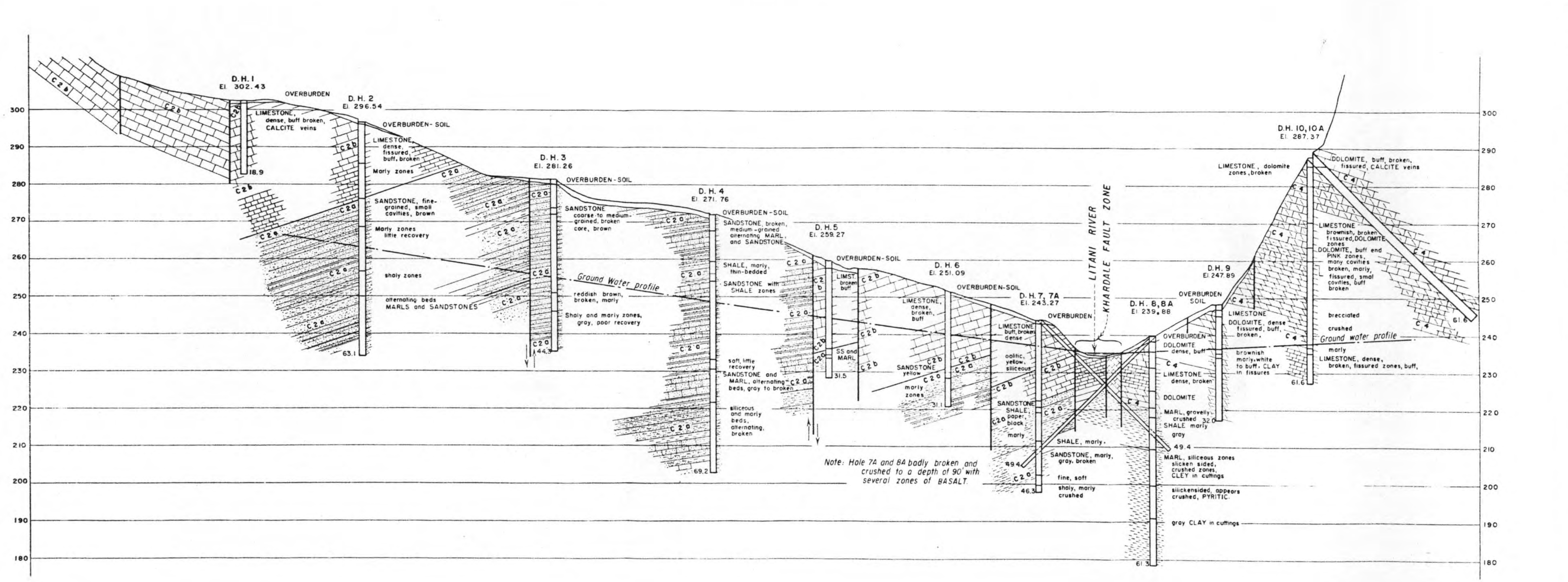
Drill holes 1, S2, and S3 were drilled along the proposed spillway route. Considerable overburden and talus occurs along this route and the excavation for the spillway footings should be carried into compact bedrock.

After drilling the lower axis (A-axis) it appeared there was little difference in the foundation at the two axes. The lower axis was selected because considerably less yardage would be required for a dam. The geologic section along the A-axis is shown on Plate XXIV-1. Talus over part of the area prevented the geologic structure from being located from the surface in some places. Many changes in the strike and amount of dip of the rocks indicate the presence of many faults and considerable folding in the area. The main faults above the dam site parallel the river and lead toward the dam site area. This condition should confine the reservoir leakage mainly to the dam site area. A deep cut-off wall and grout curtain, both beneath and beyond the abutments, with some blanketing upstream from the toe of the dam, should be the main measures taken to control reservoir leakage.

Hydrologic Data

Reservoir Levels. Sedimentation studies for Khardale Reservoir indicate that the maximum sediment deposition in this reservoir in a 50-year period will reach about elevation 242 meters. It is proposed to construct the Zaiye Tunnel for diversion from this dam to the Lower Nabatiye Irrigation Unit and the Zrariye Power Unit. This will be a free-flow tunnel about 2.2 meters in diameter and with an invert at elevation 250 meters at Khardale Dam. It has been assumed that 2-1/2 to 3 meters of water cover over this tunnel will be ample to permit its efficient operation. Therefore the bottom of usable storage has been established at elevation 257 meters. The maximum water surface at this dam has been established at 295 meters elevation. Plate XXIV-2 shows the usable storage determined as available after a 50-year period of operation and sedimentation. This curve has been used in operation studies for this reservoir.

Tailwater Curve. A tailwater curve has been determined from cross sections of the channel below the dam and assumed values of the roughness coefficient. This curve is shown on Plate XXIV-3 and indicates that tailwater will vary from elevation 233.5



AXIS - A

0 10 20 30 40 50
SCALE OF METERS

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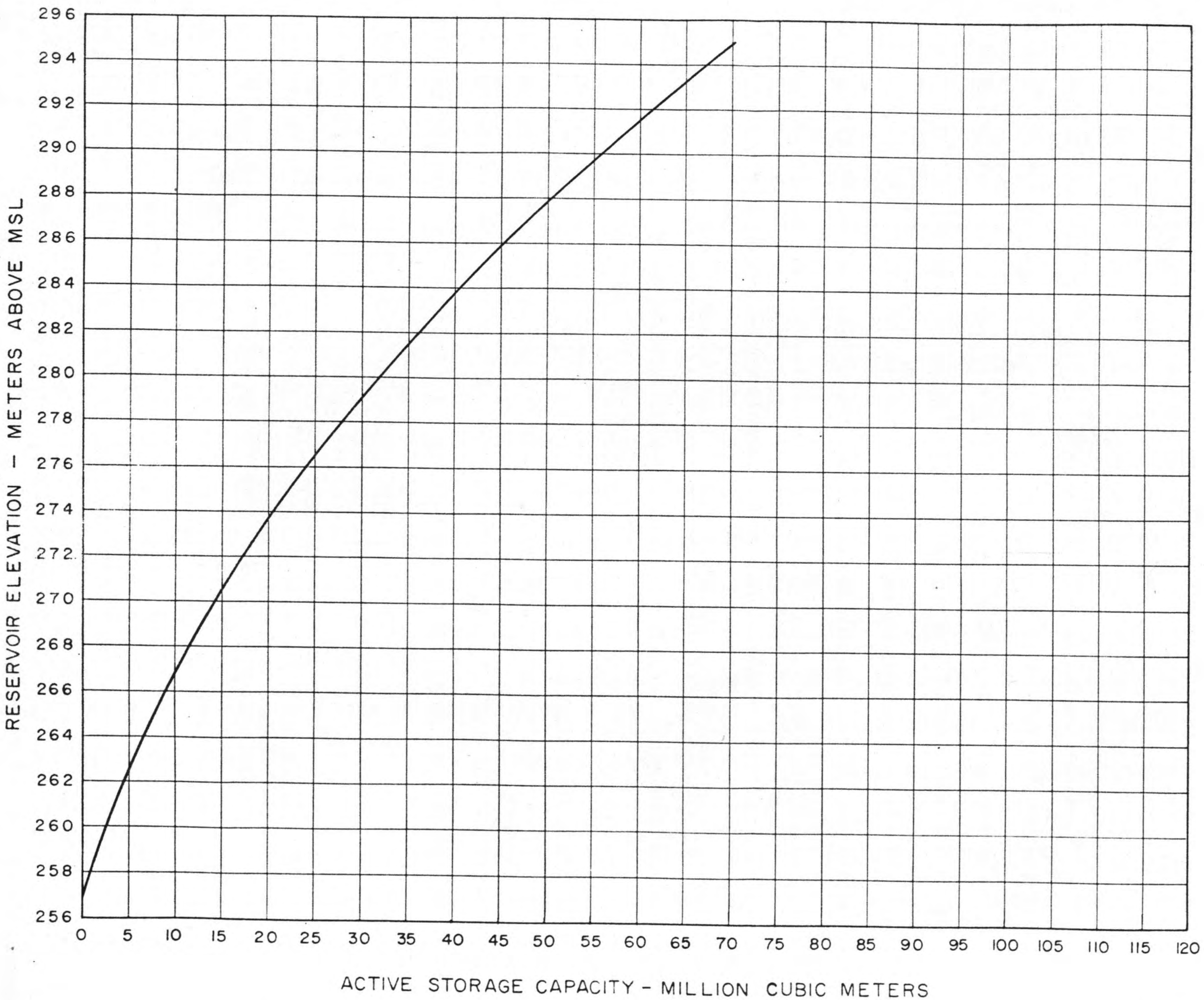
KHARDALE DAMSITE
GEOLOGIC EXPLORATION

[redacted]

BEIRUT, LEBANON	JAN 20, 1954	OA-10-351

PLATE XXIV -



NOTE

The curve shown indicates the useable storage estimated to be available at the end of a 50-year period, assuming a dam constructed at axis A-A and the estimated sediment accumulation over the period deducted.

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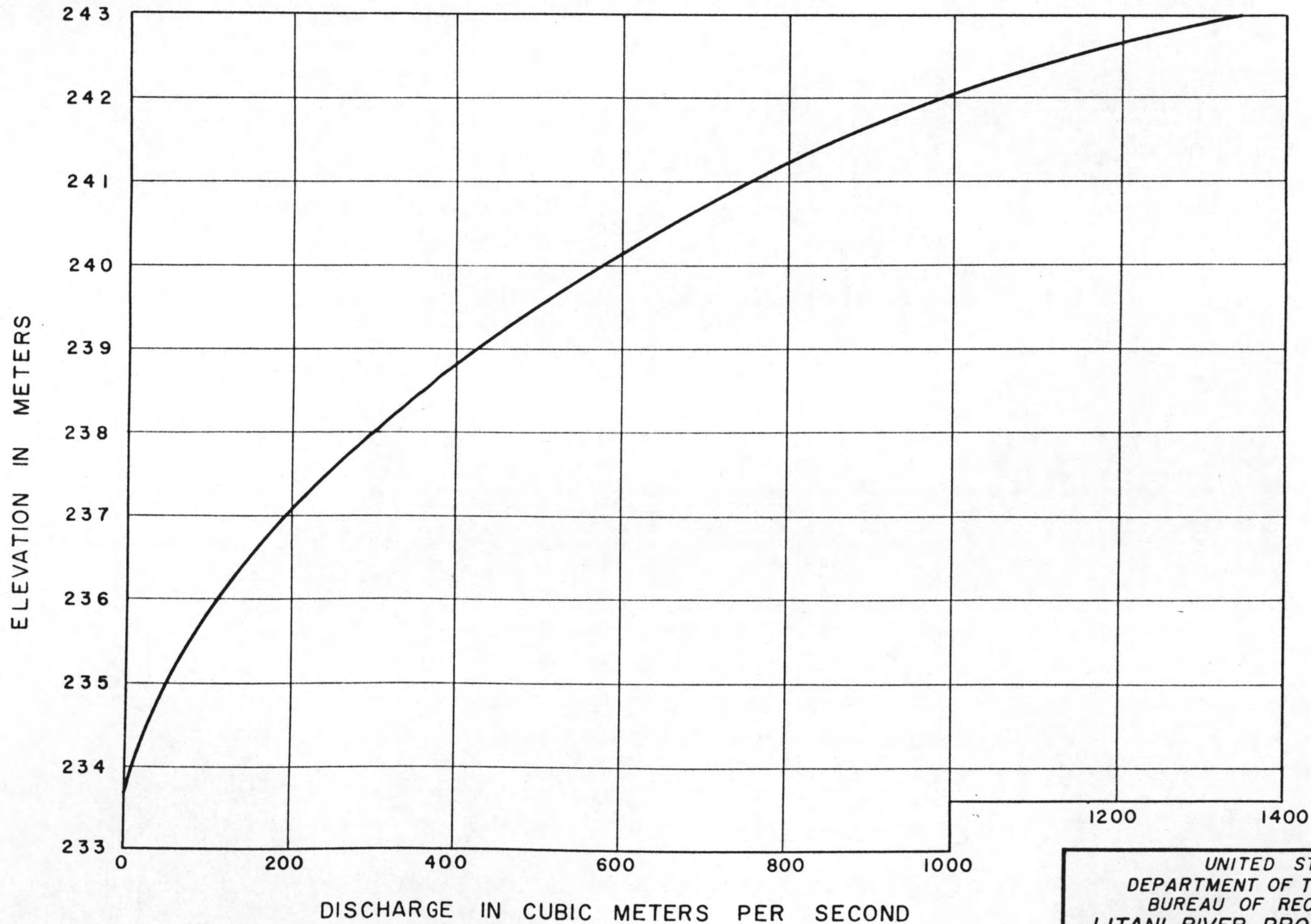
USEABLE STORAGE CAPACITY
KHARDALE RESERVOIR - LITANI RIVER

[redacted]

BEIRUT, LEBANON 12-8-53	OA-10-150

PLATE XXIV-2



Note: The tailwater curve shown was computed from valley cross-sections and channel characteristics determined from field survey data.

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TAILWATER RATING CURVE
KHARDALE DAMSITE—AXIS A-A

[redacted]

BEIRUT, LEBANON 12-18-53 | OA-10-134

PLATE XXIV-3

meters at low water to about 242.5 meters at the inflow spillway design flood. In the latter case, it was assumed that this flood would occur when the reservoir was full and that its peak discharge would remain the same as under natural conditions existing prior to construction of the dam.

Fetch and Freeboard. The maximum wind velocity expected over the Khardale Reservoir was assumed to be the maximum gust velocity of 47 meters per second from the west-southwest, recorded at Ksara Observatory on January 22, 1951. This wind direction when applied to the line perpendicular to the axis of Khardale Dam results in an angle greater than 90 degrees and the wind component parallel to the line acts in an upstream direction. Therefore, it has been assumed that no wind set-up will occur against the Khardale Dam, and that any minor depression that occurs at the dam will be insignificant.

Wave heights likely to occur at Khardale Dam have been determined by the Stevenson[1/] Formula as modified by Molitor[2/] which is as follows in English units for reservoirs whose fetch is less than 20 miles:

$$H = 0.17 \sqrt{VF} + 2.5 - \sqrt[4]{F}$$

Where

H = wave height in feet

V = wind velocity in miles per hour

F = wind fetch in nautical miles

The fetch on Khardale Reservoir is estimated at 4 kilometers (2.1 nautical miles), and the wind velocity assumed at 47 meters per second. These values converted to English units and applied to the above formula result in a maximum wave height of 1.17 meters that may be expected at Khardale Dam. This wave height has been considered in determining the freeboard for this dam.

Ice Conditions. Temperature records indicate that thin ice may form on the reservoir at times, but its formation will be infrequent and its occurrence of short duration. No special precautions need be taken to provide for operation of this reservoir or its appurtenances during icing conditions.

Sedimentation. It has been assumed that Karaoun Reservoir will be constructed prior to Khardale Reservoir, and that only the sediment from the intervening area between the two reservoirs will be deposited in Khardale Reservoir. This drainage area and its cover are similar to those above Bisri Reservoir and a rate of 908 metric tons per year per square kilometer of drainage area determined from measurements at the Bisri gage has been applied to determine deposition likely at Khardale. Assuming a density of 1.325 metric tons per cubic meter and a trap efficiency of 95 percent, the annual deposition in Khardale Reservoir would be about 164,500 cubic meters, and the deposition during a 50-year period would be about 8.3 million cubic meters. This deposition has been taken into account in determining the usable storage capacity of this reservoir at the end of a 50-year period as shown on Plate XXIV-2. Assuming deposition to continue at the same rate this reservoir would have a total life of about 450 to 500 years.

1/ Stevenson, Thomas, Design and Construction of Harbours: A Treatise on Maritime Engineering - Edinburgh 1874

2/ Molitor, D. A. - Wave Pressures on Sea Walls and Break Waters, Trans. Am. Soc. Civil Engrs. Vol. 100, 1935, p 984

Inflow Spillway Design Flood. The shape of the drainage basin and the variation in its characteristics, made it impracticable to determine an inflow spillway design flood directly for the Khardale Dam site. Therefore the spillway design flood determined for the Karaoun Dam site was transferred to the Khardale Dam site by flood-routing methods and the maximum expected inflow from the drainage area between the two sites determined and added to the transferred hydrograph. A 6-hour unit graph was determined for this intermediate area by applying a 22-hour lag factor to the mean dimensionless graph determined for the general area by analysis of several recorded flood occurrences at the Karaoun gage. This graph was applied to the excess rainfall for the maximum expected storm and the maximum inflow determined for the intermediate area. The summation of this hydro graph with the transferred hydrograph from Karaoun is shown on Plate XXIV-4 and constitutes the inflow spillway design flood for the Khardale Dam site. This flood has a peak discharge of 1,100 meters per second and a 20-day volume of 372.4 million cubic meters. It may be expected to occur at any time between November and March but is most likely during January and February. This flood was assumed to occur when the reservoir was at the level of the spillway crest, when routing it through the reservoir to determine the size of the spillway.

Diversion During Construction. Recorded and estimated maximum annual flood events were determined for the Khardale gage for each of the 31 years between 1921 and 1951. Frequency curves were determined for the peak values, and for the 1-day, 3-day, and 5-day volumes by application of the Hazen Annual Flood Method. Values taken from these curves were used to determine the 5-, 10- and 25-year hydrographs likely to occur at Khardale Dam site. These hydrographs are shown on Plate XXIV-5. They have the following significant values which should be considered in planning diversion requirements during the construction of Khardale Dam:

Frequency Years	Peak m^3/sec.	Volume $m^3 \times 10^6$		
		1-day	3-day	5-day
5	148	10.6	28.3	42.5
10	170	12.0	30.9	45.8
25	195	13.5	34.4	50.1

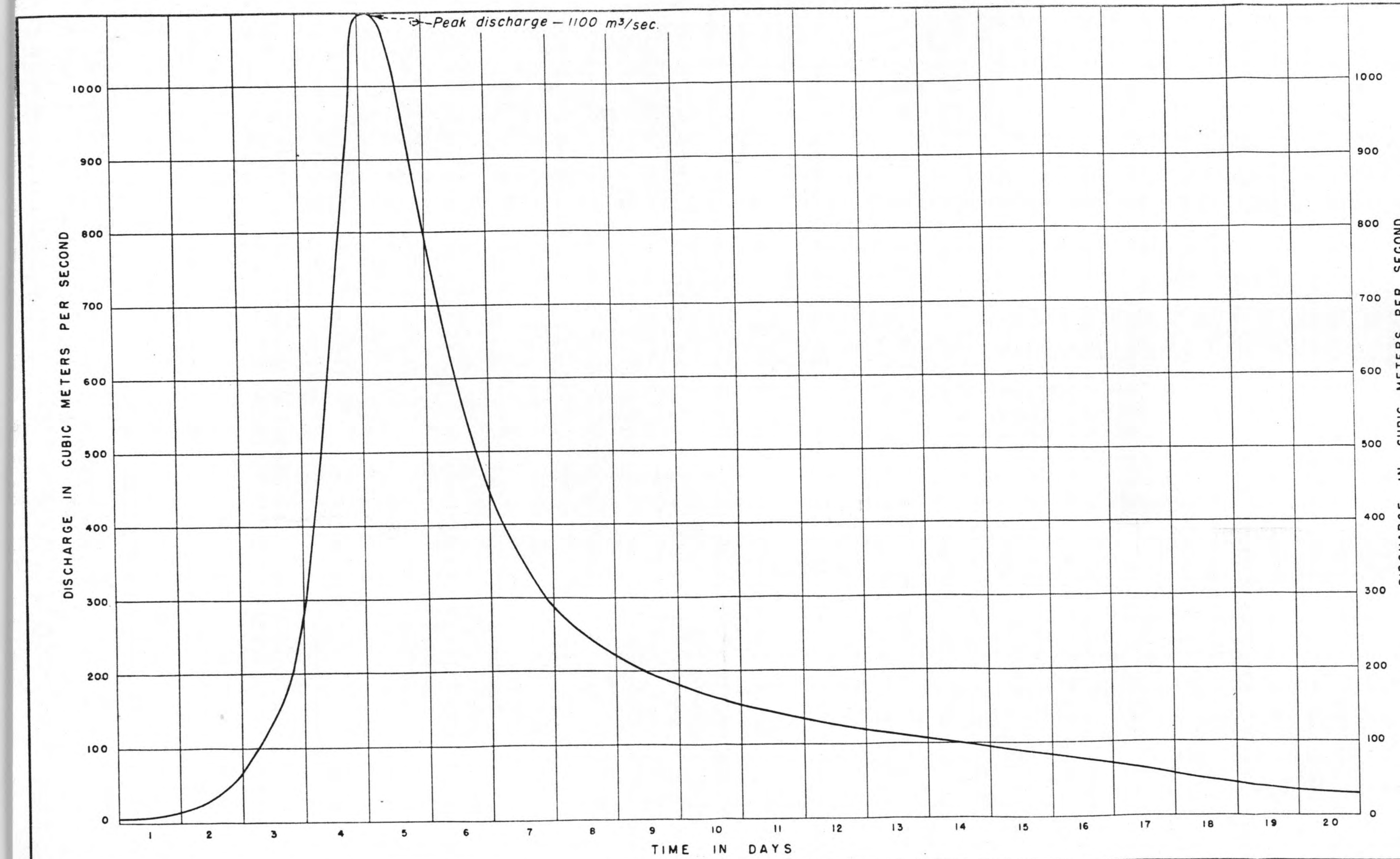
Structural Design Data

Selection of Type of Dam. Local materials, economy, and the existence of numerous faults at the dam site dictate the use of an earth-fill embankment for Khardale Dam. Accordingly, an earth-fill section with protective rock-fill zones at the face was selected for this site.

Sufficient borrow sources for the earth-fill materials are available in the reservoir area. In addition to rock excavation for the dam and its appurtenant structures, numerous rock sources are available in the immediate vicinity of the dam site.

Design Stresses. For the design of appurtenant structures, such as the diversion tunnel, river outlet works, and the spillway structure, it is recommended that the current Bureau of Reclamation values for allowable design stresses be used. The design stresses as currently used in Lebanon are considered inadequate for work of such major importance as required for Khardale Dam.

Earthquake Design. Due to the magnitude of earthquake intensities in this region, all work for the design of Khardale Dam must include provisions for earthquake stresses. It is recommended that the applied design earthquake acceleration be 98-cm/sec^2 (0.10 gravity) with a period of one second.



NOTE

This inflow spillway design flood was determined by applying the Rational Method to the 277 square kilometers of area between Karaoun and Khardale damsites on the Litani River and combining the resulting hydrograph with the inflow spillway design flood from the 1545 square kilometers of area ebove Karaoun Damsite after it had been transferred to Khardale Damsite by standard flood routing methods. It has a 20-day volume 372,390,000 cubic meters.

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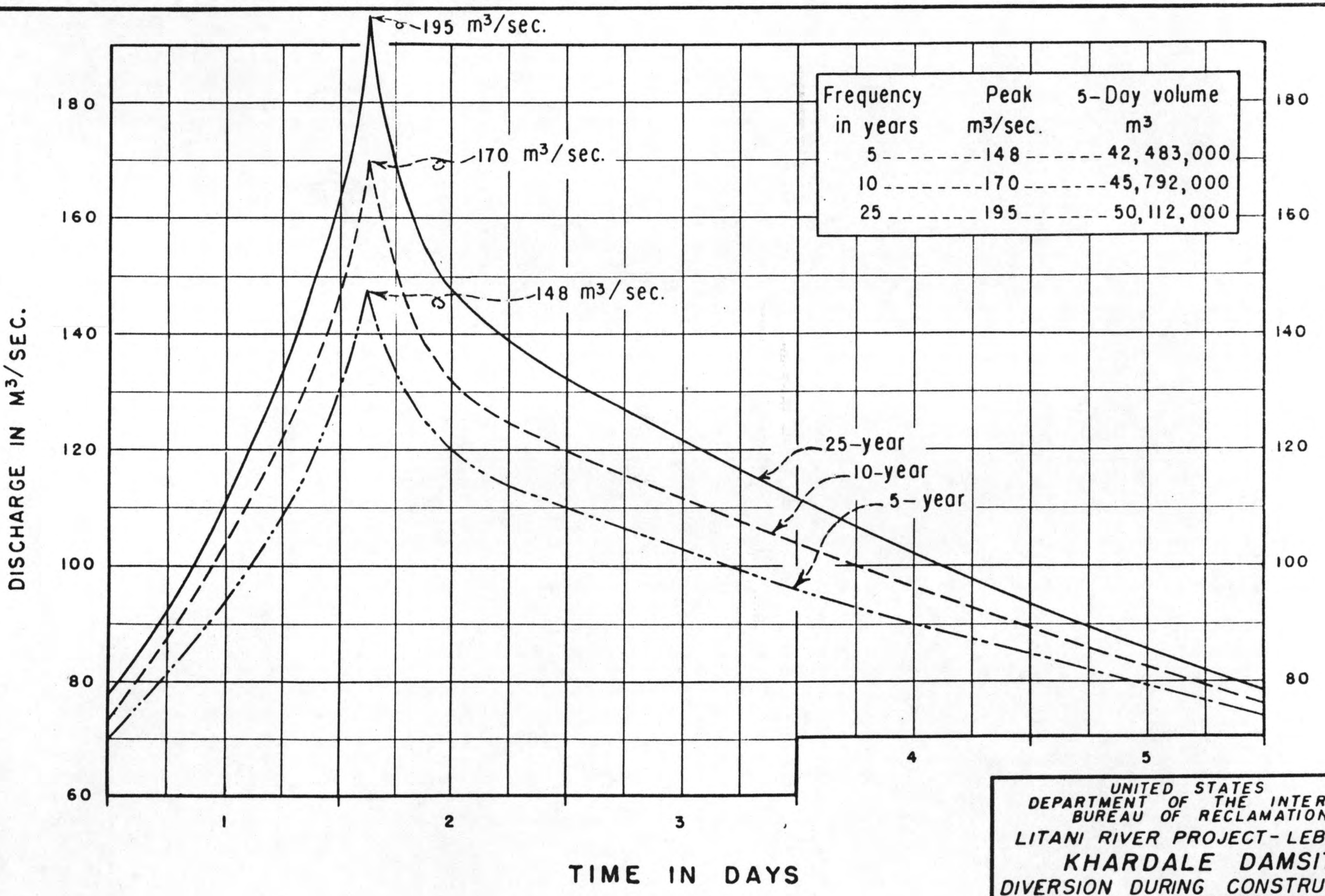
KHARDALE DAMSITE

INFLOW SPILLWAY DESIGN FLOOD

[redacted]

BEIRUT, LEBANON SEP 21, 1953 OA—10—145

PLATE XXIV-4



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LITANI RIVER PROJECT - LEBANON
KHARDALE DAMSITE
DIVERSION DURING CONSTRUCTION

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BEIRUT, LEBANON 9-22-53 | OA-10-139

PLATE XXIV-5

Stability. Computations were made for the stability of the maximum embankment section by application of the principles discussed by Justin, Hinds, and Creager[1/] for design of earth-fill dams. Properties of the embankment material assumed for the stability computation follow:

Weight of dry impervious embankment material.	1,520 kg/M^3
Weight of saturated impervious embankment material. . .	1,960 kg/M^3
Weight of submerged impervious embankment material . .	960 kg/M^3
Weight of moist impervious embankment material	1,580 kg/M^3
Angle of internal friction for impervious embankment material	17^{o}
Unconfined compressive strength of impervious embankment material	0.70 kg/cm^2

Preliminary computations for stability of the embankment indicate the selected maximum section is stable for all reasonable loading conditions including combined horizontal and vertical earth-quake accelerations of 0.10 gravity. However, prior to construction of Khardale Dam, a more complete investigation of stability should be made.

Drainage. The drainage of the Khardale Dam embankment is considered no problem since the material at the downstream face of the dam consists of a massive zone of graded rock-fill. In addition, toe drains are not necessary as the embankment materials are founded directly on bedrock. Adequate grading of the rockfill at the contacts with the impervious material will protect the core from being carried into the rockfill zones.

Behind structures, such as the spillway channel lining and training walls, drains should be provided to assist stability.

Foundation Grouting. To control seepage through the dam foundation, a cut-off trench will be excavated for the length of the dam as shown in Plate XXIV-6. At the bottom of the trench a concrete grout cap will be constructed to provide a base for pressure grouting the foundation.

For purposes of the cost estimate for pressure grouting the dam foundation, a single line of grout holes was assumed at 1.50 meter centers for the entire length of the cut-off trench. The depth of holes was assumed to be equal to the maximum hydrostatic head on the foundation, or a minimum depth of 6 meters.

Grouting around the spillway crest structure and behind the diversion and river outlet works tunnel linings will be required. Spillway grouting has been assumed to consist of one line of grout holes on 1.50 meters centers to a depth of 6 meters.

All grout holes are assumed to require 0.10 cubic meters of cement grout per linear meter of depth. The rough preliminary cost estimate for Khardale Dam (Study A-3) includes a lump sum item for grouting the dam foundation and tunnels to an amount of LL. 1,100,000 (Table XXXVII-3).

Sections of Dam. The embankment of the dam, with a maximum height of 61 meters above streambed, will have an 11 meter crest width at elevation 296.4 meters. The length of the crest will be about 290 meters. A concrete parapet to elevation 297.5

1/ Justin, Hinds, and Creager - Engineering for Dams - John Wiley and Sons, New York, N.Y., 1944, Vol. III, pp. 715-742.

meters extends along the upstream edge of the crest. The primary Nabatiye-Marjayoun highway will cross the top of the dam. A service road will lead from the top of the dam at the right abutment to the river outlet works control house at the base of the dam.

The dam embankment will consist of three zones of material--two rock-fill zones providing protection at the faces of the dam and an impervious rolled fill zone in the interior of the dam to provide protection against leakage.

The upstream face of the embankment will have a slope of 3:1; the downstream face of the embankment will have a slope of 2-1/2:1. The impervious rolled fill zone will have an upstream slope of 2:1 and a downstream slope of 1-1/2:1.

The proposed maximum embankment section is shown on Plate XXIV-6. Quantities of materials required for construction of the embankment are shown on Table XXXVII-3.

Spillway and Spillway Bridge. The spillway structure, at the left abutment of the dam, will consist of an approach channel, a concrete crest, three - 9 x 7 meter radial gates operated from a bridge over the crest, and a concrete lined spillway channel. The proposed spillway arrangement is shown on Plate XXIV-6.

The size of the spillway gates was determined by routing the reservoir inflow design flood of 1,100 cms over the spillway crest. The flood was assumed to start when the reservoir water surface was at the level of the crest at elevation 288 meters.

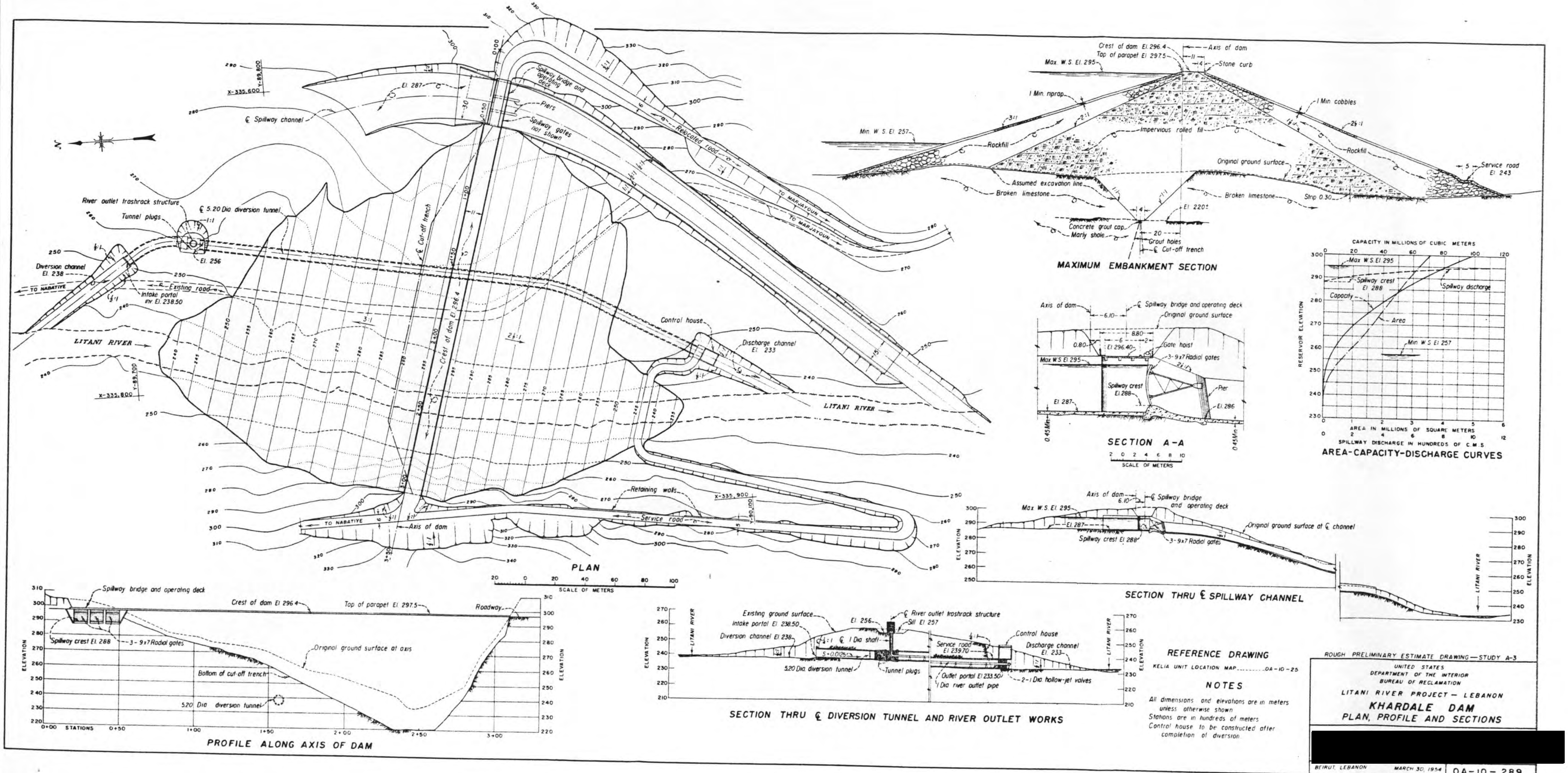
River Outlet Works. The river outlet works will consist of a trashrack structure, a bell-mouthed entrance into a vertical concrete lined shaft, an elbow and steel conduit through the downstream portion of the diversion tunnel, a control house with two hollow-jet valves, and an unlined discharge channel. The diversion tunnel will be plugged with mass concrete at the elbow of the outlet works shaft. The downstream portion of the diversion tunnel will be used for an access conduti for inspection and maintenance of the river outlet pipe and supports. Access to the control house will be provided by a service road from the right abutment of the dam.

Construction Quantities

Principal construction quantities for Khardale Dam include 720,000 cubic meters of earth-fill, 428,000 cubic meters of rock-fill, and 14,000 cubic meters of concrete. Construction time is estimated as three years. The total estimated cost for the dam and reservoir, including interest, is ŁL. 19,460,000. The itemized rough preliminary estimate for this design (Study A-3) is shown on Table XXXVII-3. The rough preliminary estimate drawing showing the general plan and sections for Khardale Dam is shown on Plate XXIV-6.

Construction Materials

Impervious earth material is available in shallow deposits in the reservoir area. Approximately 1,000,000 cubic meters of moderately plastic clay at an average depth of 1 meter is spread over the reservoir bottom at an average haul distance of 1.8 kilometers. This deposit bottoms out on large, angular rocks or bedrock. The surface material has up to 30 percent material larger than 5 mm with no appreciable quantity larger than 75 millimeters. Rockfill and riprap are available from required excavation and the surrounding limestone outcrops.


PLAN
SCALE OF METERS
MAXIMUM EMBANKMENT SECTION
SECTION A-A
SCALE OF METERS
AREA-CAPACITY-DISCHARGE CURVES
CAPACITY IN MILLIONS OF CUBIC METERS
AREA IN MILLIONS OF SQUARE METERS
SPILLWAY DISCHARGE IN HUNDREDS OF C.M.S.
RESERVOIR ELEVATION
SECTION THRU ℄ SPILLWAY CHANNEL
PROFILE ALONG AXIS OF DAM
SECTION THRU ℄ DIVERSION TUNNEL AND RIVER OUTLET WORKS
Crest of dam El. 296.4
Top of parapet El. 297.5
Max. W.S. El. 295
Min. W.S. El. 257
Axis of dam
Stone curb
1 Min. riprap
1 Min. cobbles
Rockfill
Impervious rolled fill
Original ground surface
Assumed excavation line
Broken limestone
Service road El. 243
Concrete grout cap
Marly shale
Grout holes
℄ Cut-off trench
℄ Spillway channel
Spillway bridge and operating deck
Piers
Spillway gates not shown
Relocated road
TO MARJAYOUN
TO NABATIYE
LITANI RIVER
River outlet trashrack structure
Tunnel plugs
℄ 5.20 Dia. diversion tunnel
Diversion channel El. 238
Existing road
Intake portal inv. El. 238.50
Control house
Discharge channel El. 233
Retaining walls
Service road
Spillway crest El. 288
3 - 9x7 Radial gates
Gate hoist
Pier
Capacity
Area
Spillway discharge
Original ground surface at axis
Bottom of cut-off trench
5.20 Dia. diversion tunnel
Roadway
STATIONS
ELEVATION
Existing ground surface
Sill El. 257
℄ 1 Dia. shaft
Outlet portal El. 233.50
1 Dia. river outlet pipe
2-1 Dia. hollow-jet valves
REFERENCE DRAWING
KELIA UNIT LOCATION MAP OA-10-25
NOTES
All dimensions and elevations are in meters unless otherwise shown
Stations are in hundreds of meters
Control house to be constructed after completion of diversion
ROUGH PRELIMINARY ESTIMATE DRAWING—STUDY A-3
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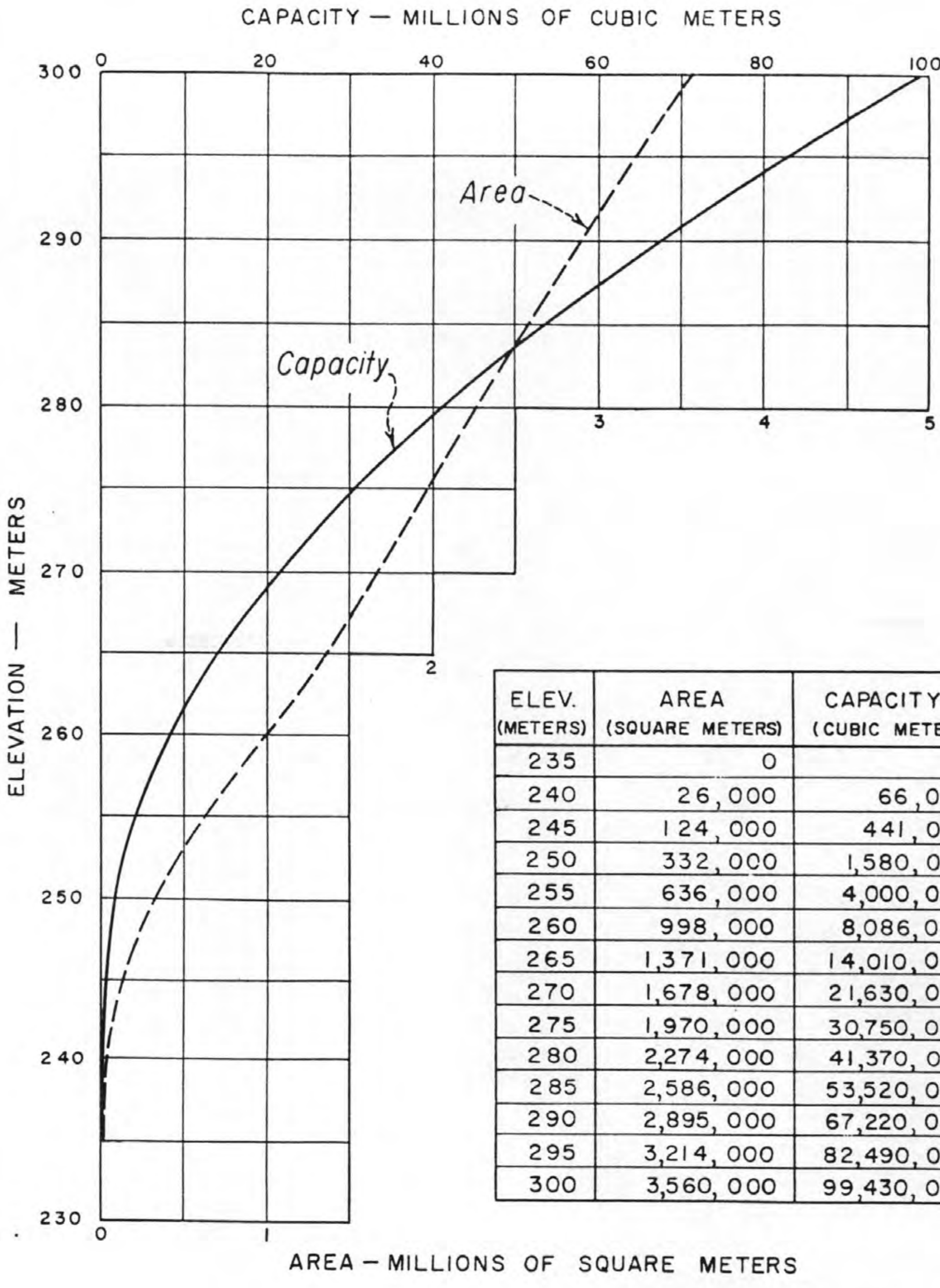
LITANI RIVER PROJECT — LEBANON
KHARDALE DAM
PLAN, PROFILE AND SECTIONS
[redacted]
BEIRUT, LEBANON
MARCH 30, 1954
OA-10-289

PLATE XXIV-6



ELEV. (METERS)	AREA (SQUARE METERS)	CAPACITY (CUBIC METERS)
235	0	0
240	26,000	66,000
245	124,000	441,000
250	332,000	1,580,000
255	636,000	4,000,000
260	998,000	8,086,000
265	1,371,000	14,010,000
270	1,678,000	21,630,000
275	1,970,000	30,750,000
280	2,274,000	41,370,000
285	2,586,000	53,520,000
290	2,895,000	67,220,000
295	3,214,000	82,490,000
300	3,560,000	99,430,000

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KHARDALE DAM—AXIS "A"
AREA—CAPACITY CURVES

[redacted]

BEIRUT, LEBANON 3-2-54 | OA-10-280

PLATE XXIV-7

Khardale Dam site - Looking Downstream

SECTION XXV

MARKABI DIVERSION DAM

Location and Description

The Markabi Diversion Dam will be constructed as a multiple purpose dam on the Litani River about 78 kilometers from its mouth and about 8 kilometers downstream from Karaoun Dam. It will divert water through the Markabi Tunnel and Canal for irrigation on the Upper Nabatiye Unit and for power production at Kelia Power Plant. The dam will have a concrete, ogee section, about 4 meters high and about 40 meters long on the crest, at elevation 660 meters. Sohmor Power Plant will discharge into the pool immediately above this dam. The location of this dam is shown on Plate XX-1 and XXVII-1.

Geology

Bedrock outcrops along the right abutment for this dam. A large fault parallels the river in the river channel. Many angular limestone boulders have rolled down into the river from either side and are embedded in clay and small talus rock in the bed of the river.

The test pits have been dug along the axis on the left abutment as shown on Plate XXV-1. Test pit 1 is located about 7 meters from the river bank. This pit revealed large limestone boulders embedded in compact clay to a depth of 2.5 meters. From 2.5 meters to the bottom of the hole at 7.8 meters, small angular limestone talus occurs embedded in hard compact clay.

Test pit 3, located 28 meters from the river along the axis was dug to 4.85 meters. Unconsolidated talus rock was found to a depth of 4.10 meters. From 4.1 to 4.85 meters compact clay with small angular limestone rocks was encountered.

Test pit 1 and test pit 2 (located 16 meters upstream from test pit 1) were dug in clay which was tight enough to keep ground water out of the pit to a depth of over 5 meters below river level. It is reasonable to expect that the clays in the river bed will also be impervious so that a shallow cut-off wall will be sufficient to control seepage of water beneath this low diversion dam. These clays are dense enough so that the walls of the cut-off trench should stand vertically until the trench is filled with concrete.

Hydrologic Design Data

Frequency studies indicate that the Markabi Diversion Dam should be designed for a 50-year flood having a peak estimate at about 180 cubic meters per second. It appears likely that the maximum flood to be expected would be equal to the maximum peak of 785 cubic meters per second determined for Karaoun Dam and likely to occur when Karaoun Reservoir was full.

This diversion dam is estimated to have a total height of about 4 meters and it will become almost completely submerged at high flows. There will be little or no pondage behind it and pool levels, sedimentation, and tailwater conditions do not need to be considered in its design or operation. Likewise the small amount of ice likely to form at irregular intervals will not seriously affect its operation.

Structural Design Data

The dam will divert water released from the Sohmor Power Plant combined with the water of the Litani River that has accumulated from springs along the river below the Karaoun Dam. Zarka Spring located just above the dam, is the largest, having an estimated average flow of 2 cubic meters per second. By locating the diversion dam just below the Sohmor Power Plant and Zarka Spring (the Sohmor Power Plant site is situated directly across the river from Zarka Spring) the height of the dam can be held to a minimum and a desirable slope obtained for the Markabi Tunnels and Canal between the dam and the Kelia forebay.

The dam will not be required to store any water released through the Sohmor Power Plant or collected from the Litani River, as the control gate for the Markabi Tunnel will normally remain open, thus the water will flow through the Markabi Tunnel and Canal and be stored in the Kelia forebay thence released through the Kelia Power Plant. If the forebay is filled, and the Kelia Power Plant is not using the incoming flow, the excess water will pass over the forebay spillway into a draw which flows into the Litani River two kilometers above the Kelia Power Plant. Water that passes over the diversion dam or over the Kelia forebay spillway may be: (1) stored in the Khardale Reservoir; (2) diverted through the Zaiye Tunnel for production of power at the Zrariye Power Plant or for irrigation of the Lower Nabatiye Irrigation Unit; (3) released into the Litani River through the Khardale Dam outlet works.

The diversion dam will consist of: (1) a simple ogee overflow section 27.5 meters long with a crest elevation 660 meters; (2) a right abutment consisting of upstream and downstream wingwalls with a top elevation of 663.50 meters, and (3) sluiceway, headworks and appurtenant retaining walls in the left abutment. The sluiceway will be controlled by a 3 x 3 meter radial gate with a sill elevation of 656.5 meters. The headworks will be controlled by a 5.0 x 2.6 meter radial gate that can be constructed to operate from the Kelia Power Plant or for automatic operation according to the water level desired at the Markabi Tunnel inlet. The headworks control gate will have a sill elevation of 657.50 meters. The required sand and silt traps, overflow spillway section and measuring devices will be between the control gate and the tunnel portal. The upstream left wingwall would be continued upstream to meet the retaining wall of the Sohmor Power Plant. If only a temporary bridge is used during the construction of the Sohmor Power Plant, a permanent bridge for the operation and maintenance road to the power plant can be constructed above the overflow section of the dam. The plan and sections of this dam are shown on Plate XXV-2.

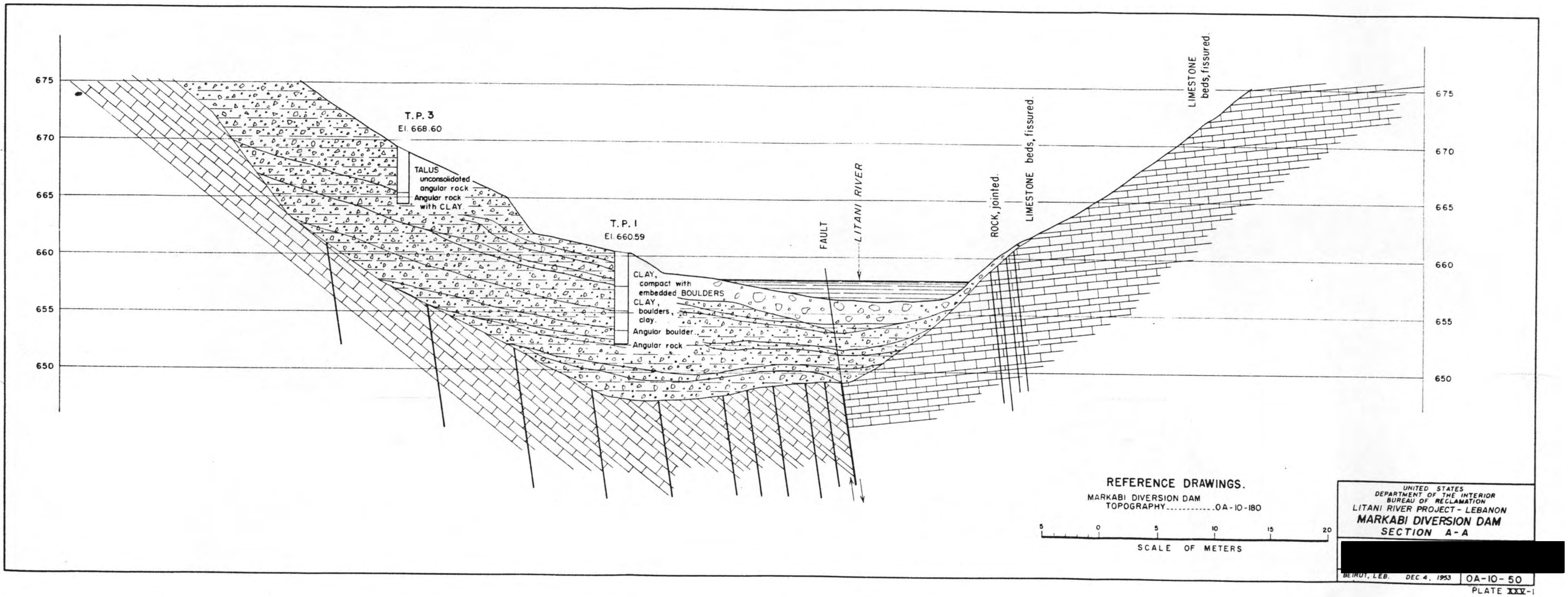
A cut-off wall approximately 2 meters deep is believed to be sufficient to control the seepage of water beneath this low diversion dam, as the test pits dug along the site were in ground tight enough so that no water flowed into them although they were 5 meters below the river level. The diversion dam, including the flume between the control gates and tunnel portal, would require 7,700 cubic meters of excavation, mostly loose rock mixed with clay; 910 cubic meters of concrete; 55,000 kilograms of steel reinforcement; 16,000 kilograms of structural steel for the control gates; and miscellaneous metal work.

It will be necessary to widen the stream bed and clear out the boulders and trees in the area between the Sohmor Power Plant and the diversion dam and from the area for a short distance below the dam.

Construction Operations

During construction of the dam the water can be diverted along the right bank of the river until the outlet works and sluiceway are completed, then diverted through the sluiceway while the overflow section and right wingwalls are constructed. Due to the large seasonal variation in the flow of the Litani River it will be necessary to construct features that require diversion of the river only during the stage of low flow, which is from April to November.

As the construction schedule, see Plate XX-3, for the Litani River Project calls for an early start of the Kelia Power Unit, about 18 months before completion of the Sohmor Power Plant, consideration should be given to the advisability of constructing the dam concurrently with the Sohmor Power Plant. This would eliminate the added cost for handling the water discharge from the Sohmor Power Plant during construction



675
670
665
660
655
650
T. P. 3
El. 668.60
TALUS
unconsolidated
angular rock
Angular rock
with CLAY
T. P. 1
El. 660.59
CLAY,
compact with
embedded BOULDERS
CLAY,
boulders,
clay.
Angular boulder
Angular rock
FAULT
LITANI RIVER
ROCK, jointed.
LIMESTONE beds, fissured.
LIMESTONE
beds, fissured.
675
670
665
660
655
650
REFERENCE DRAWINGS.
MARKABI DIVERSION DAM
TOPOGRAPHY............OA-10-180
5 0 5 10 15 20
SCALE OF METERS
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
LITANI RIVER PROJECT - LEBANON
MARKABI DIVERSION DAM
SECTION A-A
[redacted]
BEIRUT, LEB. DEC 4, 1953
OA-10-50
PLATE XXV-1

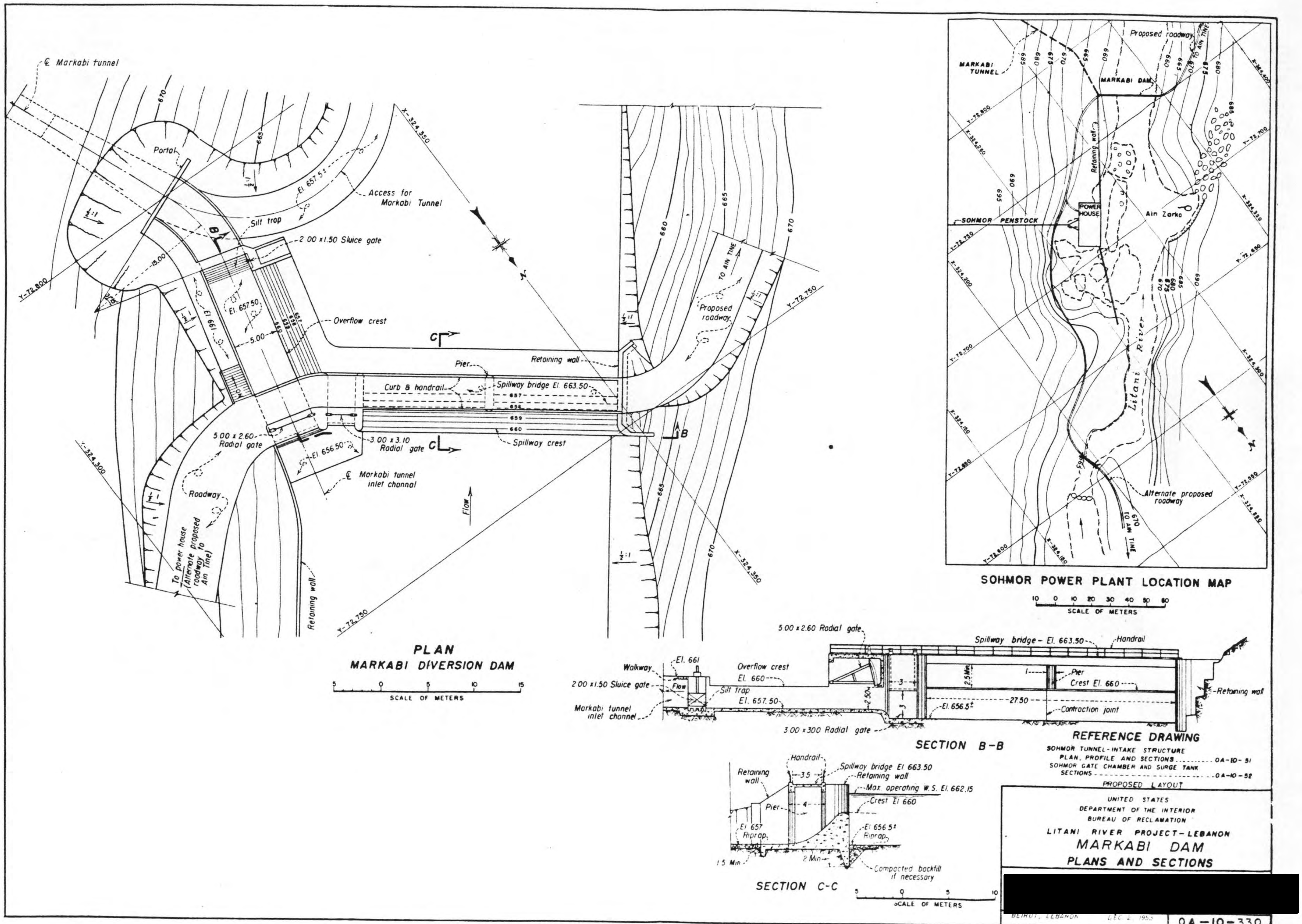
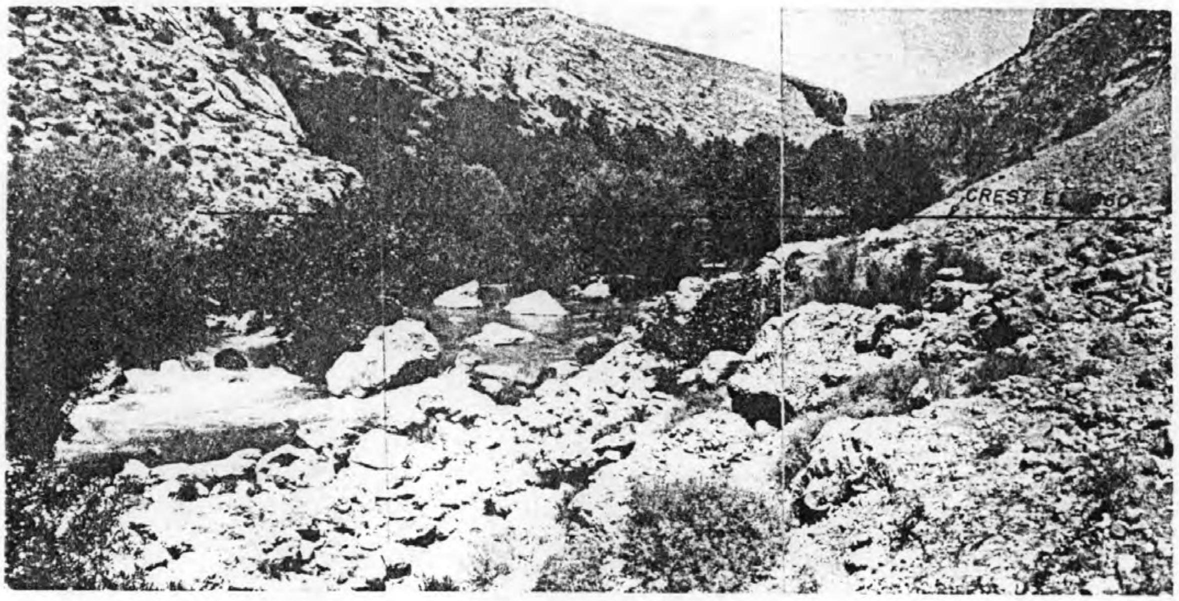

PLAN
MARKABI DIVERSION DAM
SCALE OF METERS
℄ Markabi tunnel
Portal
Access for Markabi Tunnel
Silt trap
2.00 x 1.50 Sluice gate
Overflow crest
Retaining wall
Pier
Curb & handrail
Spillway bridge El. 663.50
Spillway crest
5.00 x 2.60 Radial gate
3.00 x 3.10 Radial gate
El. 656.50
℄ Markabi tunnel inlet channel
Roadway
To power house (Alternate proposed roadway to Ain Tine)
Flow
TO AIN TINE
Proposed roadway
SOHMOR POWER PLANT LOCATION MAP
SCALE OF METERS
MARKABI TUNNEL
MARKABI DAM
Proposed roadway
SOHMOR PENSTOCK
POWER HOUSE
Ain Zarka
Litani River
Alternate proposed roadway
SECTION B-B
5.00 x 2.60 Radial gate
Spillway bridge - El. 663.50
Handrail
Walkway
El. 661
Overflow crest El. 660
2.00 x 1.50 Sluice gate
Silt trap El. 657.50
Markabi tunnel inlet channel
Pier
Crest El. 660
Retaining wall
El. 656.5±
Contraction joint
3.00 x 300 Radial gate
SECTION C-C
Handrail
Spillway bridge El 663.50
Retaining wall
Max. operating W.S. El. 662.15
Crest El 660
Pier
El 657 Riprap
El 656.5± Riprap
Compacted backfill if necessary
SCALE OF METERS
REFERENCE DRAWING
SOHMOR TUNNEL-INTAKE STRUCTURE PLAN, PROFILE AND SECTIONS OA-10-51
SOHMOR GATE CHAMBER AND SURGE TANK SECTIONS OA-10-52
PROPOSED LAYOUT
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
LITANI RIVER PROJECT - LEBANON
MARKABI DAM
PLANS AND SECTIONS
[redacted]
BEIRUT, LEBANON DEC. 2, 1953
OA-10-330

PLATE XXV-2



Markabi Diversion Dam Site - Looking Upstream of the dam and would facilitate operation of the Sohmor Power Plant by controlling the tailwater elevations. It would also make it feasible to combine the contracts for construction of the power-plant substructure and for the diversion dam. Due to the similarity of this work and the limited working space in the narrow canyon, the unit prices obtained with a single contract probably would be lower than prices obtained with separate contracts.

SECTION XXVI

JOUN DIVERSION DAM

Location and Description

The Joun Diversion Dam will be constructed as a multiple purpose dam on the Bisri River about 19 kilometers from the mouth of this river. It will divert water through the Joun Tunnel for the Saida-Beirut Irrigation Unit and for power production at the Joun Power Plant. The Awali Power Plant will discharge into the pool above this dam. The location of the dam is shown on Plates XX-1 and XXIX-1.

The dam has been designed to provide sufficient usable reservoir storage to permit power production at the Joun Plant during a daily four-hour peaking period. Reservoir storage is not required for the sole purpose of irrigation. Future studies may show that the peaking of the Awali Power Plant during the irrigation season for the Saida-Beirut Irrigation Unit is not necessary. Therefore, the Joun Diversion Dam could be redesigned as a simple diversion and tailwater regulating structure with the Joun Power Plant operating as a winter peaking plant in conjunction with the Awali Power Plant.

The proposed dam will be of the concrete, slab and buttress type about 18 meters high and 90 meters long on the crest, at elevation 220 meters. It will require about 7,000 cubic meters of concrete.

Two sites were considered for the location of the Joun Diversion Dam, the Awali Power Plant, and the inlet portal of the Joun Tunnel. The first site, where the streambed is at about elevation 205 meters, is about two kilometers below the Bisri slide area. The second site, where the streambed is at about elevation 170 meters, is in a narrower canyon and has a steeper stream gradient than the first site, and is about 1-1/2 kilometers below the first site.

Location of the structrues at the first (upper) site is recommended primarily because more irrigable land can be served by the Saida-Beirut Irrigation Unit by diverting water from the higher site. In addition, the upper site offers more room for the power plant, a shorter penstock with fewer grade changes, and a shorter access road than the lower site.

At the lower site, however, there are better foundation conditions and a narrower canyon for the location of the diversion dam. This site is suitable for the construction of a thin concrete arch dam at a lower cost than the slab and buttress dam proposed at the upper site. It is believed that the advantages of the upper site will offset the cheaper dam at the lower site. Further economic comparisons between the two locations should be made prior to construction of the Awali Unit, the Joun Unit, or the Saida-Beirut Irrigation Unit.

Geology

This site is located about 500 meters below the proposed Awali Power Plant site. At the dam location the valley narrows with limestone outcropping on both sides of the relatively flat river bottom.

The dam site is about 2 kilometers below the Bisri slide area and is located in an outwash region which was formed when the slide area partially washed out. When this occurred a heterogeneous mixture of limestone boulders of all sizes, as well as sand and silt, was carried downstream to the Joun Diversion Dam area and for some distance beyond. The deepest part of the river channel at the proposed dam site was filled to a depth of about 15 meters with this mass of material which still remains somewhat unconsolidated.

Five drill holes were drilled at this site, one on each abutment near the river bottom and three across the river bottom. Drill hole 1 was drilled on the left abutment to a depth of 10.7 meters in limestone. Below 4 meters the core recovery was very good, and the rock appeared to be reasonably tight. Drill hole 3, on the right abutment, was drilled to a depth of 11.6 meters with 3.3 meters of overburden and the balance in limestone. Below 3.3 meters the core recovery was good, but the rock appears to be somewhat broken and fissured and percolation losses were high.

Drill holes 2, 4, and 5 were drilled in the river bottom starting in the outwash materials from the landslide. The maximum depth to bed-rock was in drill hole 5, where 14.6 meters of overburden was found. It is possible that a slightly greater depth to bedrock might be found to the left of this hole. This overburden consisted of boulders of limestone and dolomite embedded in gravel and some silt. Percolation tests and the loss of drill water show this outwash material to be permeable in all three holes. The geology of this dam site is shown on Plate XXVI-1.

It appears from these drill holes that seepage losses will be excessive beneath a dam at this location unless a cut-off wall is carried to bedrock. This will involve stripping at least 15 meters of rock at the lowest point, at a cost that will be high since this material will be saturated with water. It will be impossible to drive piling in the river fill due to the large boulders scattered through its mass. Large amounts of water may be expected to seep into excavations for a cut-off wall. Blanketing of the reservoir area does not appear feasible since suitable blanketing materials are not available near the area.

The river gradient is steep through the landslide materials for about 2 kilometers above the dam site area and considerable erosion is taking place as the river cuts back into the old lake fill above the landslide.

Hydrologic Data

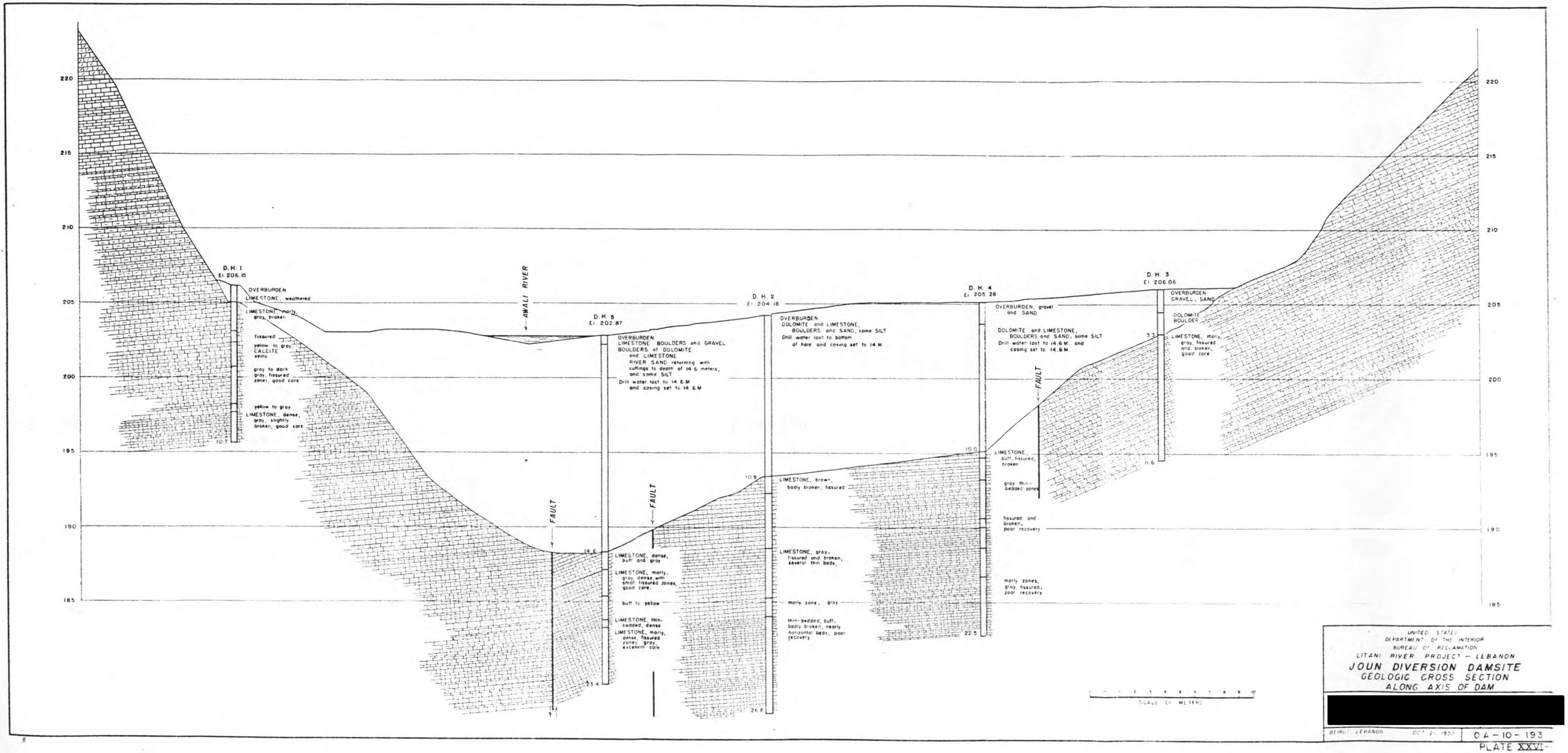
This diversion dam will divert water through the Joun Tunnel for the Saida-Beirut Irrigation Unit and for the Joun Power Unit. A storage capacity of 172,000 cubic meters is required behind this dam to provide peaking capacity for the Joun Power Plant.

Frequency studies indicate that this dam should be designed for a 100-year flood, having a peak discharge estimated at 115 cubic meters per second. The maximum flood to be expected has been assumed to be the same as that determined for Bisri Dam since there is very little contributing drainage area between the two sites and the maximum flood may occur when the Bisri Reservoir is completely filled. It will have a peak discharge of 775 cubic meters per second.

It is anticipated that the Bisri Dam and Reservoir will be constructed prior to the Joun Diversion Dam and will retain most of the sediment now moving in the Bisri River. Therefore, the amount of sedimentation to be expected in the pond above the Joun Diversion Dam is small and the dead storage of 80,000 cubic meters below elevation 212 meters will be sufficient to store such material for many years. Available topographic and hydrologic data are insufficient to determine a tailwater curve at this site. Therefore, the maximum tailwater has been assumed at elevation 203 meters.

Ice is unlikely to form on the pond behind this dam and any such formation will be too small to seriously affect the operation of this pond or to endanger the safety of the structure.

The invert elevation of the Joun Tunnel has been established at 208 meters at the Joun Diversion Dam. This elevation and the required water cover over the top of this tunnel fix the top of dead storage in the pond. The maximum water surface and the normal water surface of the pond have been established at 219 and 217 meters, respectively. An allowance of 1.2 meters required over the top of the 2.8 meter diameter Joun Tunnel establishes the minimum operating level of this pond at elevation 212 meters.


220
215
210
205
200
195
190
185
D. H. 1
El. 206.15
OVERBURDEN
LIMESTONE, weathered
LIMESTONE, marly, gray, broken
fissured
yellow to gray, CALCITE veins
gray to dark gray, fissured zones, good core
yellow to gray
LIMESTONE, dense, gray, slightly broken, good core
10.7
AWALI RIVER
D. H. 5
El. 202.87
OVERBURDEN
LIMESTONE BOULDERS and GRAVEL
BOULDERS of DOLOMITE and LIMESTONE
RIVER SAND returning with cuttings to depth of 14.6 meters, and some SILT
Drill water lost to 14.6 M and casing set to 14.6 M
14.6
LIMESTONE, dense, buff and gray
LIMESTONE, marly, gray dense, with small fissured zones, good core.
buff to yellow
LIMESTONE, thin-bedded, dense
LIMESTONE, marly, dense, fissured zones, gray, excellent core
23.4
FAULT
FAULT
D. H. 2
El. 204.18
OVERBURDEN
DOLOMITE and LIMESTONE, BOULDERS and SAND, some SILT
Drill water lost to bottom of hole and casing set to 14 M
10.9
LIMESTONE, brown, badly broken, fissured
LIMESTONE, gray, fissured and broken, several thin beds,
marly zone, gray
thin-bedded, buff, badly broken, nearly horizontal beds, poor recovery
26.8
D. H. 4
El. 205.28
OVERBURDEN, gravel and SAND
DOLOMITE and LIMESTONE, BOULDERS and SAND, some SILT
Drill water lost to 14.6 M and casing set to 14.6 M
10.0
LIMESTONE, buff, fissured, broken
gray thin-bedded zones
fissured and broken, poor recovery
marly zones, gray, fissured, poor recovery
22.5
FAULT
D. H. 3
El. 206.06
OVERBURDEN
GRAVEL, SAND
DOLOMITE BOULDER
3.3
LIMESTONE, marly, gray, fissured and broken, good core
11.6
SCALE OF METERS
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
LITANI RIVER PROJECT – LEBANON
JOUN DIVERSION DAMSITE
GEOLOGIC CROSS SECTION
ALONG AXIS OF DAM
[redacted]
BEIRUT, LEBANON
04-10-193
PLATE XXVI

Structural Design Data

Selection of Type of Dam. A concrete dam was selected for this site because of the scarcity of locally available impervious material for an earth-fill structure and the expense of constructing an embankment of rock fill. A slab and buttress dam was chosen in lieu of other types of concrete dams because of the existing foundation conditions in the streambed. It is believed that a stable, slab and buttress dam can be constructed without the necessity for excavating all of the river fill for the dam foundation. The proposed design suggests excavation of the river fill to bedrock. The excavation work may be difficult, but cribbing of the side slopes and pumping the accumulated ground water in trench should permit the successful construction of a concrete cut-off wall along the upstream toe of the dam. The trench will then be backfilled with compacted material. Buttress footings will be constructed on the ground surface to support the superstructure of the dam.

Design Stresses. For the design of Joun Diversion Dam and its appurtenant structures, such as the spillway structure and the river outlet works, it is recommended that the current Bureau of Reclamation values for allowable design stresses be used. The design stresses as now used in Lebanon are considered inadequate for work of such major importance as required for Joun Diversion Dam.

Earthquake Design. Due to the magnitude of earthquake intensities in this region, all work for the design of Joun Diversion Dam must include provisions for earthquake stresses. It is recommended that the applied design earthquake acceleration be 98-cm/sec^2 (0.10 gravity) with a period of one second.

Stability. Due to the complexity of slab and buttress type dams and the preliminary nature of the present investigation for Joun Diversion Dam, no computations for stability of the structure have been made. The proportions of the structural members of the dam have been selected by comparison of other dams of this type which have been constructed in the United States Prior to construction of the Joun Diversion Dam it will be necessary to perform complete designs for the structure and to investigate the stability of the perfected designs.

Drainage. Drainage of the dam foundation may present some problems in order to control percolation under the dam. Drains may be necessary behind the foundation cut-off wall to reduce uplift on the dam. The concrete floors of the spillway and river outlet works discharge aprons will require adequate drains.

Foundation Grouting. A curtain of cement grout will be constructed below the concrete cut-off wall along the upstream toe of the dam. A single line of grout holes was assumed at 1.50 meter centers to a depth equal to the hydrostatic head on the foundation, or a minimum depth of 6 meters. The grout holes were assumed to require 0.10 cubic meters of cement grout per linear meter of depth. The rough preliminary cost estimate for Joun Diversion Dam (Study J-2) includes a lump-sum item for pressure grouting the dam foundation to an amount of LL. 125,000. (Table XXXVII-5)

Sections of the Dam. The maximum overflow and nonoverflow sections and the river outlet works section, as proposed for Joun Diversion Dam, are shown on Plate XXVI-2. The upstream face of the dam will consist of reinforced concrete slabs supported by the buttress corbels and tongues. Buttresses are spaced on 4.60 meter centers as determined by approximate studies of economical buttress spacings for this dam.

The top of the dam, at the nonoverflow sections will consist of a two-meter wide walkway at elevation 220 meters. A parapet will be constructed to elevation 221 meters to provide additional freeboard to the dam.

At the upstream toe of the dam, the cut-off wall and the dam foundation are designed so that the cut-off will will not carry any load from the dam. In addition to reducing the tendency for cracks to form in the cut-off wall, this design allows the dam to settle on its foundation independently of the cut-off wall. A bituminous joint filler will be placed in the joint between the cut-off wall and the toe of the dam to prevent stress concentrations in these members.

A walkway is provided through the dam to allow access for operation of the river outlet works, inspection, and maintenance. Horizontal beams, or struts, are provided between buttresses to stiffen the structure and reduce lateral deformation of the buttresses.

Spillway. The spillway structure will consist of an ogee section through the dam and a spillway chute for the formation of a hydraulic jump in the tailwater. The spillway will occupy four bays of the dam. The length of the spillway was determined from the standard formula for rectangular weirs assuming a design inflow flood of 115 cms with a maximum rise in the reservoir of 2 meters above the uncontrolled spillway crest at elevation 217 meters.

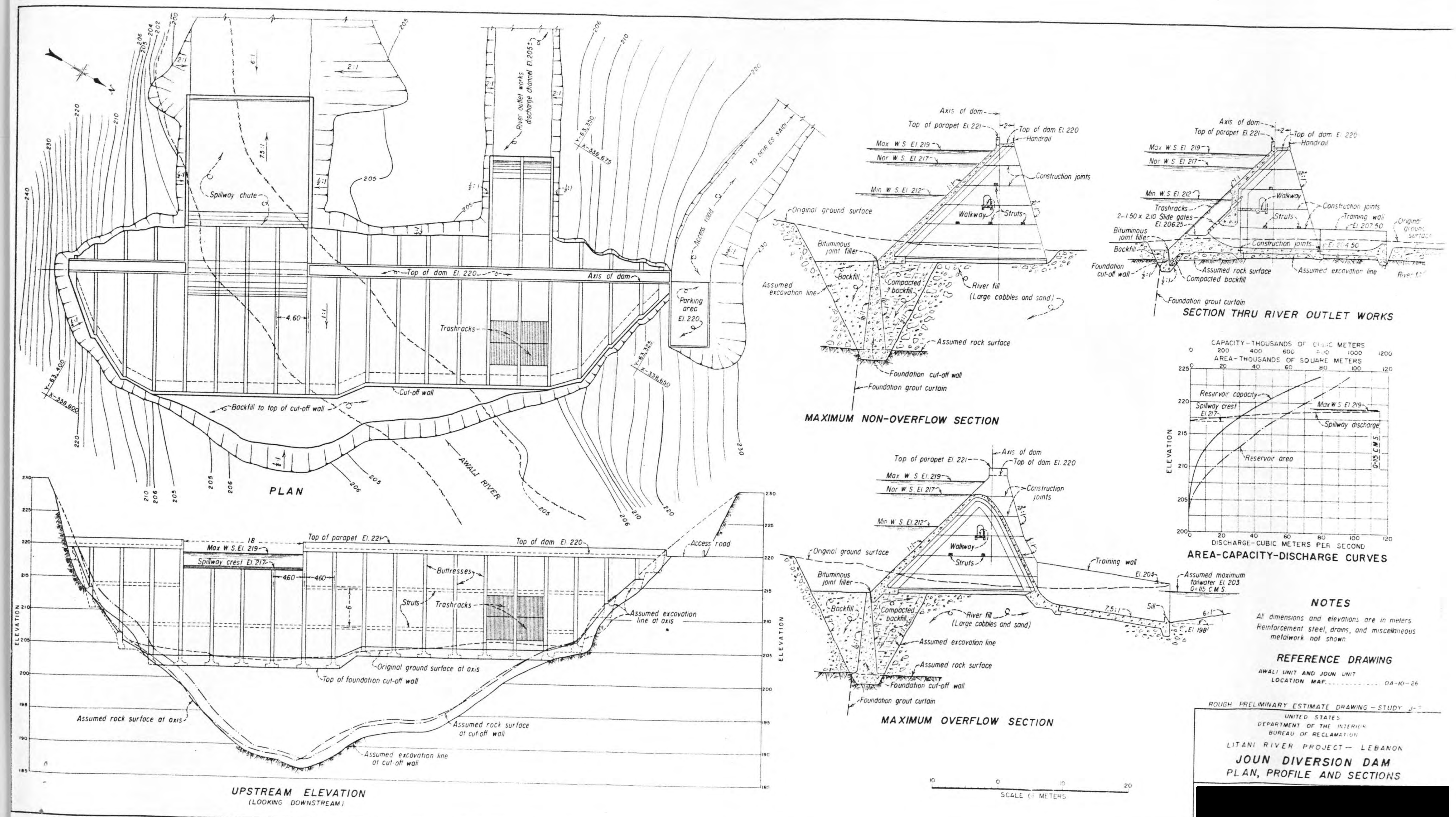
The 100-year flood of 115 cms was chosen for the design spillway discharge due to the high cost required for a greater spillway capacity and due to the relatively minor effect of floods on the lower reaches of the Awali River.

Construction Quantities

Principal construction quantities for Joun Diversion Dam include 18,000 cubic meters of excavation, 7,000 cubic meters of concrete, and 250,000 kilograms of reinforcement steel. Construction time is estimated as three years. The total estimated cost for the dam and reservoir, including interest, is ŁL. 2,790,000. The itemized rough preliminary estimate for this design (Study J-2) is shown on Table XXXVII-5. The rough preliminary estimate drawing showing the general plan, profile, and sections of Joun Diversion Dam is shown on Plate XXVI-2.

Construction Materials

Concrete aggregates will be manufactured at the site by quarrying, crushing, and grading the dolomitic limestones available in the immediate vicinity. Cement is available from the local mill, but will require hauling by truck from railheads on the sea coast. Reinforcement steel, from foreign sources, is locally available.



NOTES

All dimensions and elevations are in meters.
Reinforcement steel, drains, and miscellaneous metalwork not shown.

REFERENCE DRAWING

AWALI UNIT AND JOUN UNIT LOCATION MAP OA-10-26

ROUGH PRELIMINARY ESTIMATE DRAWING – STUDY J-3

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

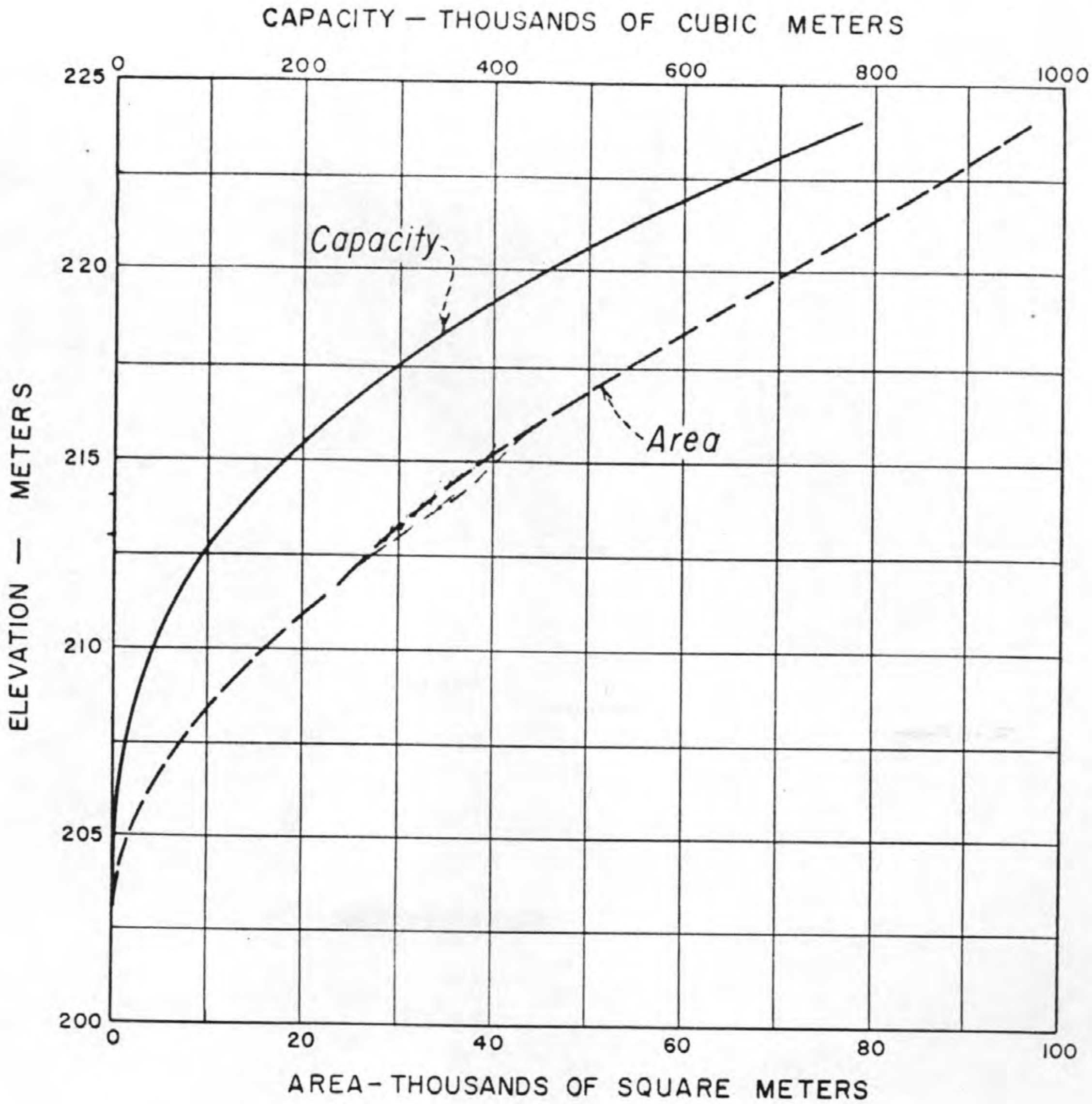
LITANI RIVER PROJECT — LEBANON

JOUN DIVERSION DAM
PLAN, PROFILE AND SECTIONS

[redacted]

BEIRUT, LEBANON APRIL 6, 1954 OA-10-275

PLATE XXVI



ELEV. (METERS)	AREA (SQUARE METERS)	CAPACITY (CUBIC METERS)
203	0	0
204	250	130
206	3, 440	3, 820
208	8, 640	15, 900
210	15, 500	40, 100
212	24, 900	80, 500
214	36, 000	141, 300
216	44, 500	221, 800
218	57, 800	324, 100
220	70, 200	452, 000
222	84, 000	606, 200
224	96, 400	786, 700

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
LITANI RIVER PROJECT-LEBANON
JOUN DIVERSION DAM-AXIS "A"
AREA-CAPACITY CURVES

[redacted]

BEIRUT, LEBANON 3-2-54 | OA-10-277

SECTION XXVII

KELIA POWER UNIT

Location and Description

This is a multiple purpose power unit to be constructed on the Litani River about 58 kilometers above the mouth. An installed capacity of 48, 000 kilowatts, consisting of four 12, 000 kilowatt units direct-connected to impulse-type turbines is proposed. Water will be supplied by diversion from the Litani River at Markabi Diversion Dam, through the free-flow Markabi Tunnels and Canal to the Kelia forebay, then through the Kelia pressure tunnel and penstock to the power plant. The Markabi Tunnel consists of three tunnel sections that will be 3 meters in diameter and which have been designated Tunnels Nos. 1, 2, and 3. The total length of these tunnel sections is about 7400 meters. Tunnels Nos. 1 and 2 will be connected with a concrete conduit about 335 meters long and Tunnels Nos. 2 and 3 by the Markabi Canal which has a length of about 9560 meters. These structures will convey power water for Kelia Power Plant and irrigation water for the Upper Nabatiye Irrigation Unit. The Kelia pressure tunnel is 2.9 meters in diameter and about 1010 meters long. The penstocks will consist of two steel tubes with an average diameter of 1.85 meters and about 1780 meters long. Kelia forebay will be provided by constructing an earth and rock-fill dam across a minor tributary of the Litani River near Dibbine. The dam will be about 13 meters high above streambed and contain about 41, 000 cubic meters of earth and rock-fill. It will impound about 236, 000 cubic meters of water to provide the pondage needed to operate Kelia Power Plant as a peaking unit. The plant will discharge into the upper end of the Khardale Reservoir. Water for the Upper Nabatiye Irrigation Unit will be diverted from the end of the Kelia Tunnel into the main canal of this unit during the irrigation season. The location and principal features of this unit are shown on Plate XXVII-1.

Geology

Markabi Tunnels No. 1 and No. 2. The geologic section along this tunnel route is shown on Plate XXVII-2. The inlet portal of Tunnel No. 1 is located in limestone talus. This talus is unconsolidated to a depth of about 8 meters so that some support and a short section of open cut will be necessary. At 25 meters distance, the tunnel will enter the Eocene (N2) marly and cherty limestones, which are faulted, jointed, and somewhat broken.

From Station 1+00 to 5+50 (stationing in hundreds of meters) the tunnel will pass thru the Eocene marly and cherty limestones which are dipping about 38° towards the Litani River and about 17° in the direction of the tunnel. Several short sections will probably require light steel support or roof bolts due to the blocky nature of the rock as the different beds are reached.

Near Station 5+50 the upper part of the Cretaceous (C6) marly limestones will be reached. These rocks show some surface faulting near Station 7+00 then flatten out and remain nearly flat to the outlet portal of Tunnel No. 1 at Station 19+66. Some faulting and jointing has occurred near Station 13+00 and 14+50. The west end of the Sohmor anticline disappears near Station 10+00 so that a small amount of folding exists in this area. From 25 to 30 percent of this tunnel will probably require some type of light tunnel support or roof bolts.

Tunnel No. 2 will be entirely in Eocene beds which are mainly marly and cherty beds of the "E2a" series to Station 43+00 where the lower part of the "E2b" series of dense, fine-grained, limestones will be found. These beds lie nearly flat in the direction of the tunnel with the true dip being about 11° to the southwest. This tunnel closely parallels the Litani River but is above the ground-water table. Numerous small faults and jointed areas are crossed along the tunnel route which will carry small amounts of water during the rainy seasons. Some support will be needed near these faulted and jointed areas and it is estimated that 25 percent of Tunnel No. 2 will require light steel support.

Markabi Tunnel No. 3. The geology for this tunnel is shown on Plate XXVII-3. This tunnel is located in the upper part of the upper Cretaceous (C6) marly limestones except near the outlet portal where it will pass through the lower part of the Eocene (E2a) limestones and marly limestones. Three faults are crossed by this tunnel and some crushed and brecciated zones will be found on each side of these faults.

Due to the proximity of the Yamoune fault at the outlet portal, the rocks are broken and some support will be needed at this end of the tunnel. A small amount of water and short sections of clay gouge will probably be encountered in the three fault areas. It is anticipated that support will be required for about one-third of the length of this tunnel. Some blocky ground will be encountered due to the dip of the limestones and roof bolts may be required in these areas.

Markabi Canal. This canal is located in the Eocene (E2b) series of limestones. See Plates XXVII-4 and 5. These limestones are both massive and thin-bedded along the canal route. They contain some chert nodules and some alternating layers of thin-bedded marly limestones. From Station 0+00 to 12+00 (stationing in hundreds of meters) the canal passes through short sections where a small amount of overburden covers the limestones. This overburden is mainly residual clay weathered from the limestones and often contains fragments of limestone and small chert nodules. The limestones are deeply weathered, porous, and the massive limestones weather out into irregular limestone nodules with cavity-filled clay pockets interspersed between the nodules. These nodules and cavities range from very small to several meters across and may be excavated to canal grade in many places by hand labor.

The geologic structure from Station 12+00 to 35+00 consists of limestone, badly weathered and broken with cavity-filled pockets of clay. Little overburden exists along this stretch of canal. Many sections of this canal are suitable for excavation by hand labor.

The canal from Station 35+00 to 37+00 represents a cut thru dense and sometimes fissured limestones which are slightly weathered and broken at invert grade.

From Station 37+00 to 78+00 the canal passes through dense to fissured limestones that are weathered and broken and contain zones of residual clay. Portions of this section of canal may be excavated by hand.

From Station 78+00 to 95+00 the canal passes thru short sections of limestone which is weathered and broken. Between these sections from zero to three meters of overburden, consisting of clay (both residual and alluvial) and limestone fragments, covers the weathered limestones.

The entire canal will require lining due to the porous nature of the limestones.

Kelia Forebay. This small reservoir is located in the large Yamoune fault zone and as a result considerable brecciation and crushing of the limestones has occurred in the reservoir area. The reservoir area is well blanketed with clay and should not present a leakage problem, using a low dam, however, some grouting should be done below the dam and for a short distance beyond the abutments.

Several small springs enter the reservoir area along the east side of the reservoir. These springs are located along a branch of the Yamoune fault which leads ground-water into this area. The presence of these springs shows that the ground-water table is higher than the reservoir area along this side and that ground-water is not escaping to lower elevations along the fault.

Kelia Tunnel. This tunnel is located in the lower part of the Jurassic (J7) limestones. The tunnel is in the main Yamoune fault zone and as a result many small faults will be intersected in the tunnel and the limestones will have many crushed and broken zones. The ground-water table is below tunnel level and very little water or

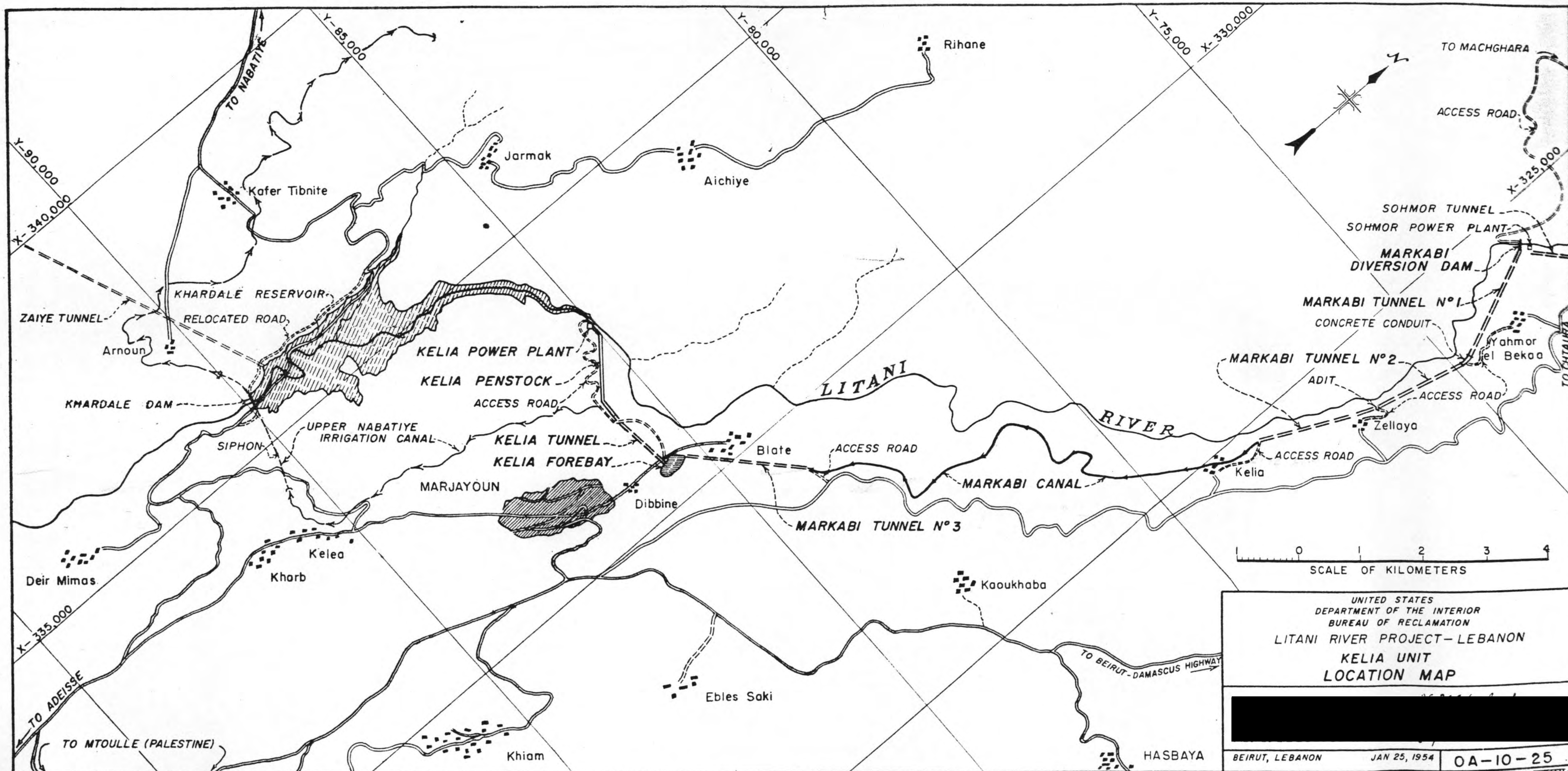
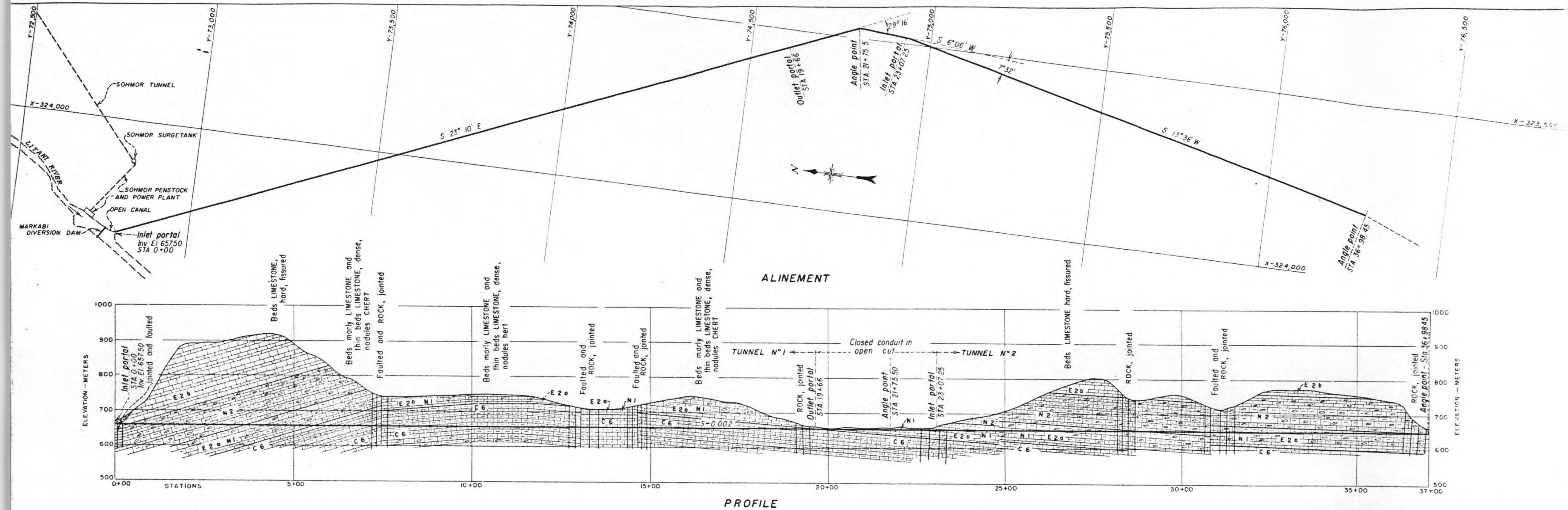
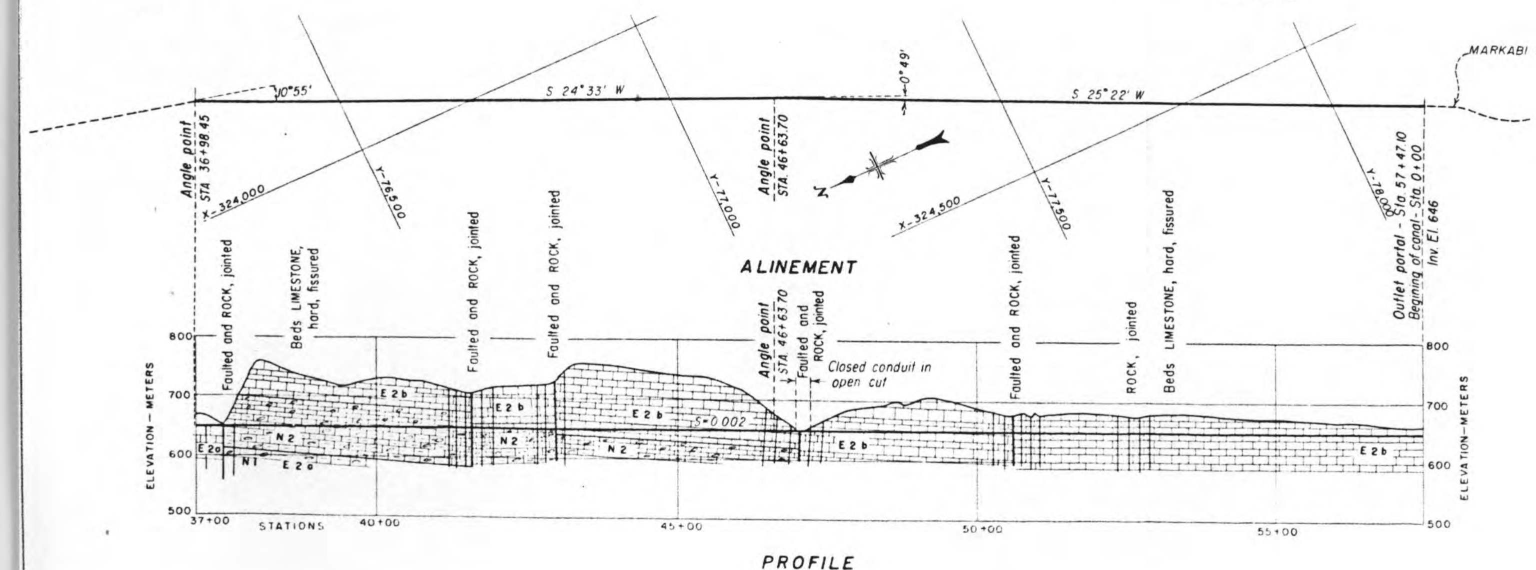
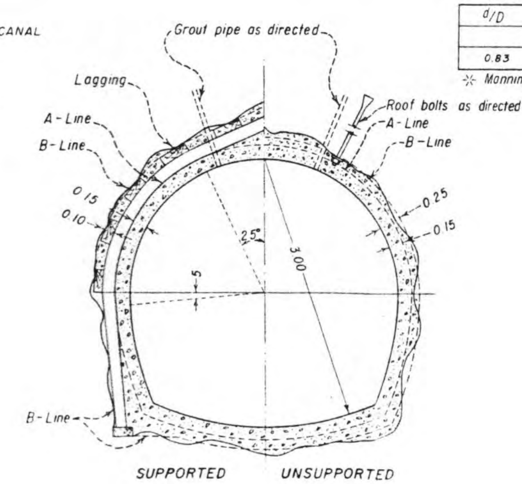


PLATE XXVII-1







TYPICAL SECTION

SCALE OF METERS

HORSESHOE TYPE TUNNEL HYDRAULIC PROPERTIES *

d/D	D		A		V		Q		r	n	s
	m	ft	m²	ft²	m/s	ft/s	m³/s	cfs	for ft	for ft	
0.83	3.0	9.84	6.67	71.76	3.1	10.3	20.9	738	3.015	0.014	0.002

* Mannings Formula

NOTES

All dimensions in meters
Stations in hundreds of meters.
Dimensions and Hydraulic Properties for closed conduit same as tunnel

REFERENCE DRAWINGS

MARKABI CANAL STA 0+00 TO STA 40+00
ALINEMENT, PROFILE, SECTIONS AND GEOLOGY.....OA-10-295
KELIA UNIT
LOCATION MAP.....OA-10-25

ROUGH PRELIMINARY DRAWING

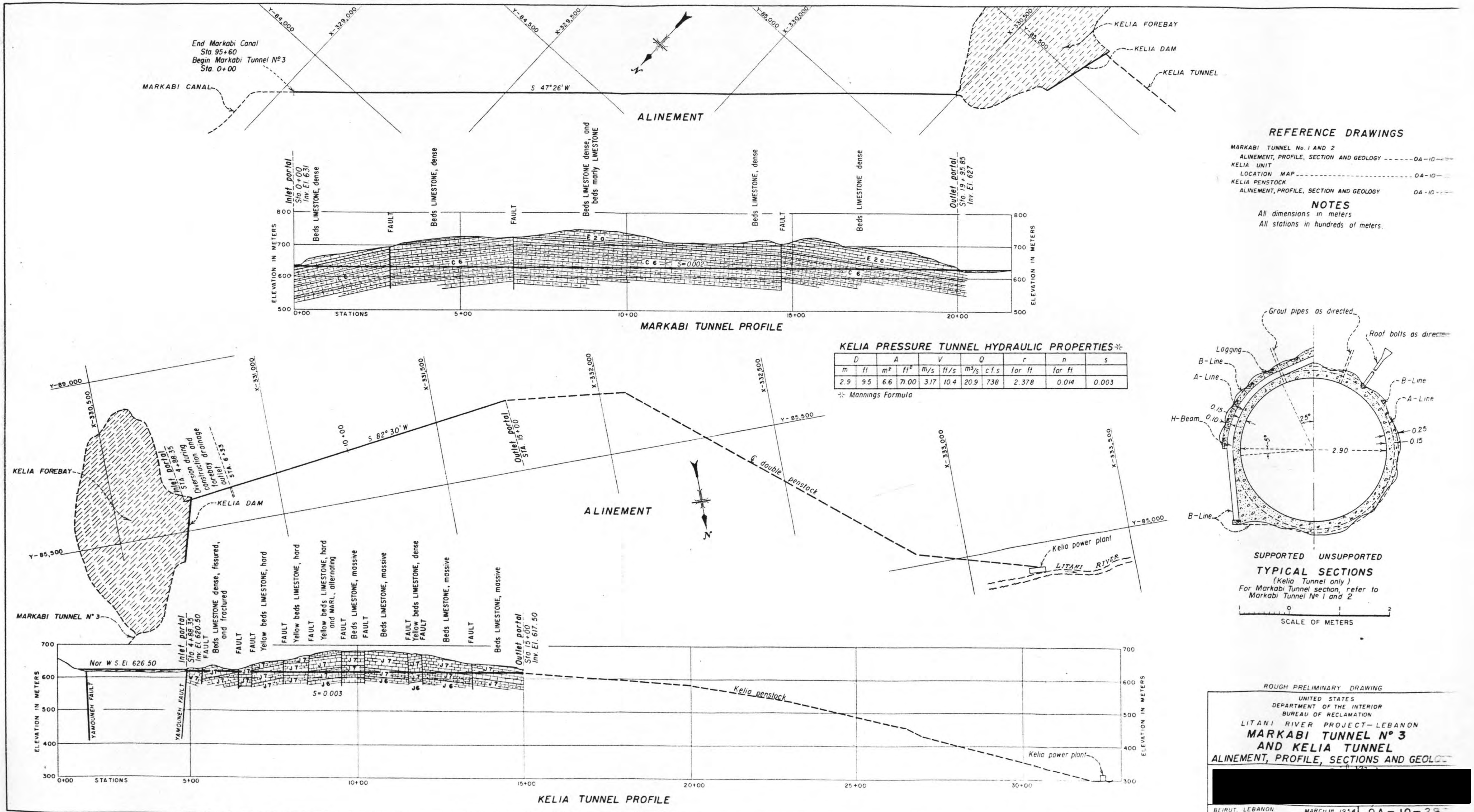
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
LITANI RIVER PROJECT — LEBANON

MARKABI TUNNEL N°1 AND 2
ALINEMENT, PROFILE, SECTIONS AND GEOLOGY

[redacted]

BEIRUT, LEBANON | MARCH 23 1954 | OA-10-294

PLATE ~~XXVII~~



KELIA PRESSURE TUNNEL HYDRAULIC PROPERTIES *

D		A		V		Q		r	n	s
m	ft	m²	ft²	m/s	ft/s	m³/s	c.f.s	for ft	for ft	
2.9	9.5	6.6	71.00	3.17	10.4	20.9	738	2.378	0.014	0.003

* Mannings Formula

REFERENCE DRAWINGS

MARKABI TUNNEL No. 1 AND 2
ALINEMENT, PROFILE, SECTION AND GEOLOGY OA-10-
KELIA UNIT
LOCATION MAP OA-10-
KELIA PENSTOCK
ALINEMENT, PROFILE, SECTION AND GEOLOGY OA-10-

NOTES

All dimensions in meters
All stations in hundreds of meters.

ROUGH PRELIMINARY DRAWING

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LITANI RIVER PROJECT—LEBANON

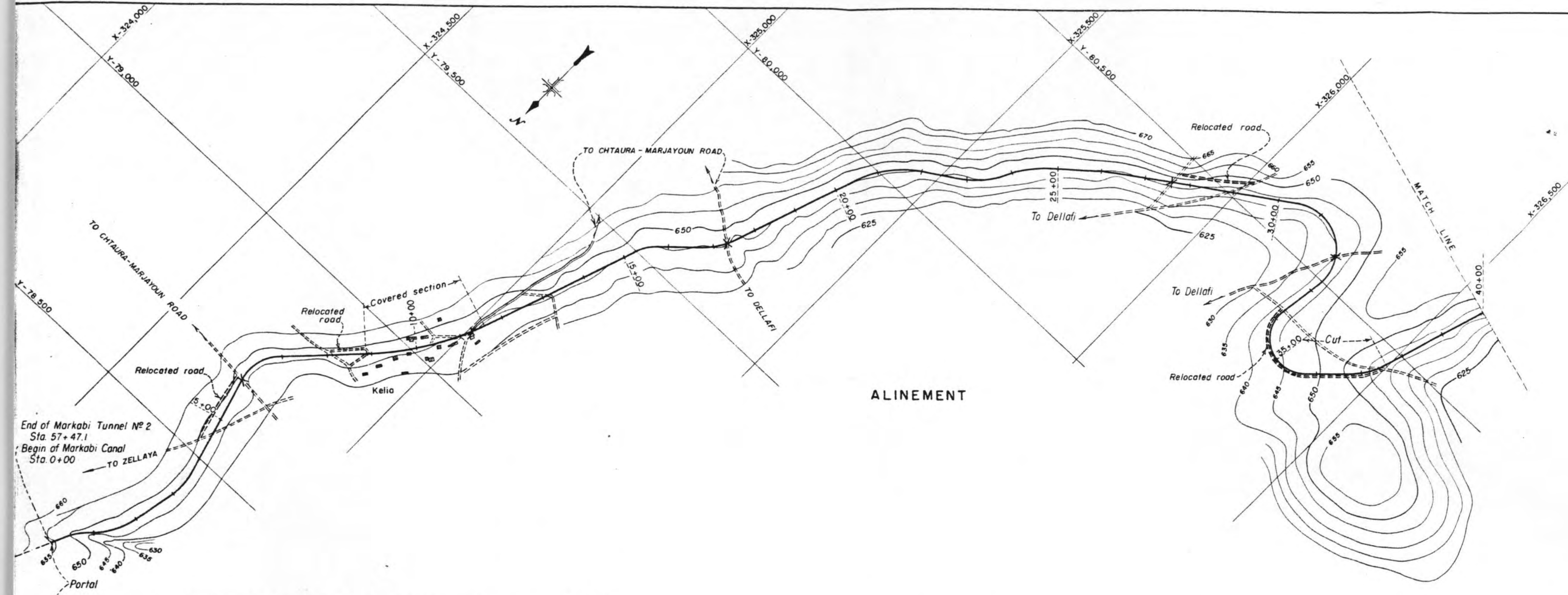
MARKABI TUNNEL Nº 3 AND KELIA TUNNEL

ALINEMENT, PROFILE, SECTIONS AND GEOLOGY

[redacted]

BEIRUT, LEBANON — MARCH 18, 1954 — OA-10-29

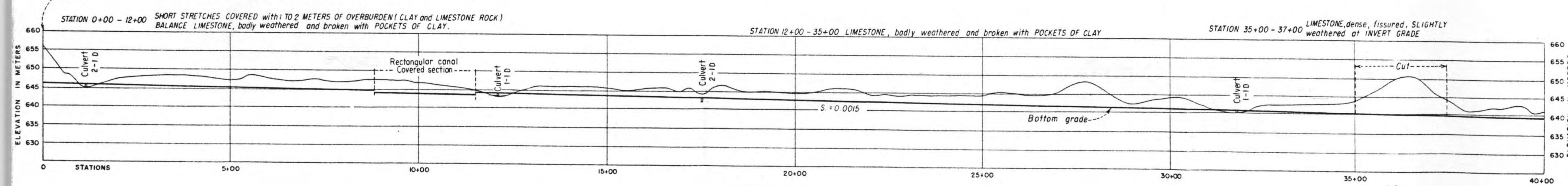
PLATE XX



ALINEMENT

NOTES

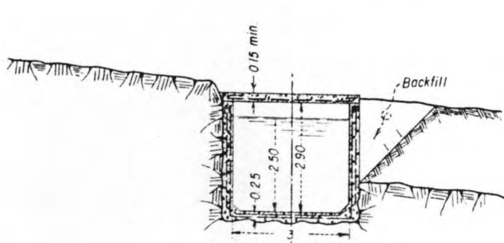
All dimensions in meters.
Stations in hundreds of meters.



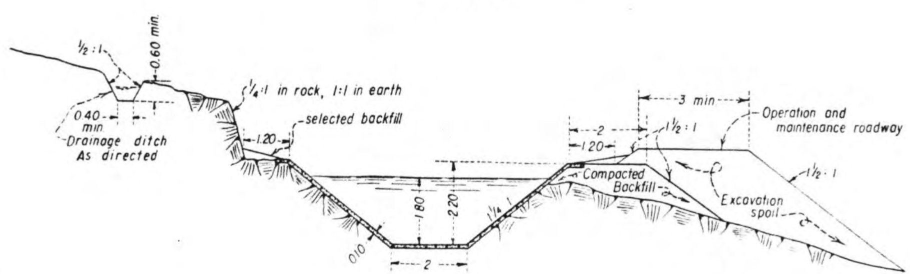
PROFILE

HYDRAULIC PROPERTIES (MANNING'S FORMULA)

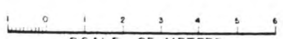
Section	Q		A		V		r	n	S
	m^3/s	cfs	m^2	ft^2	m/s	ft/s	ft	for ft	
Trapezoid	20.9	738	7.65	82.3	2.73	8.98	3.234	0.014	0.0015
Rectangle	20.9	738	7.78	83.8	2.70	8.81	3.138	0.014	0.0015



IN ROCK | IN EARTH
COVERED SECTION



TYPICAL SECTION



SCALE OF METERS

REFERENCE DRAWINGS

MARKABI TUNNELS Nº 1 AND 2
ALINEMENT, PROFILE, SECTIONS AND GEOLOGY OA-10-294
MARKABI CANAL – STA 40+00 TO STA 95+60
ALINEMENT, PROFILE, SECTIONS AND GEOLOGIE OA-10-296
KELIA UNIT
LOCATION MAP OA-10-25

ROUGH PRELIMINARY DRAWING

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LITANI RIVER PROJECT – LEBANON

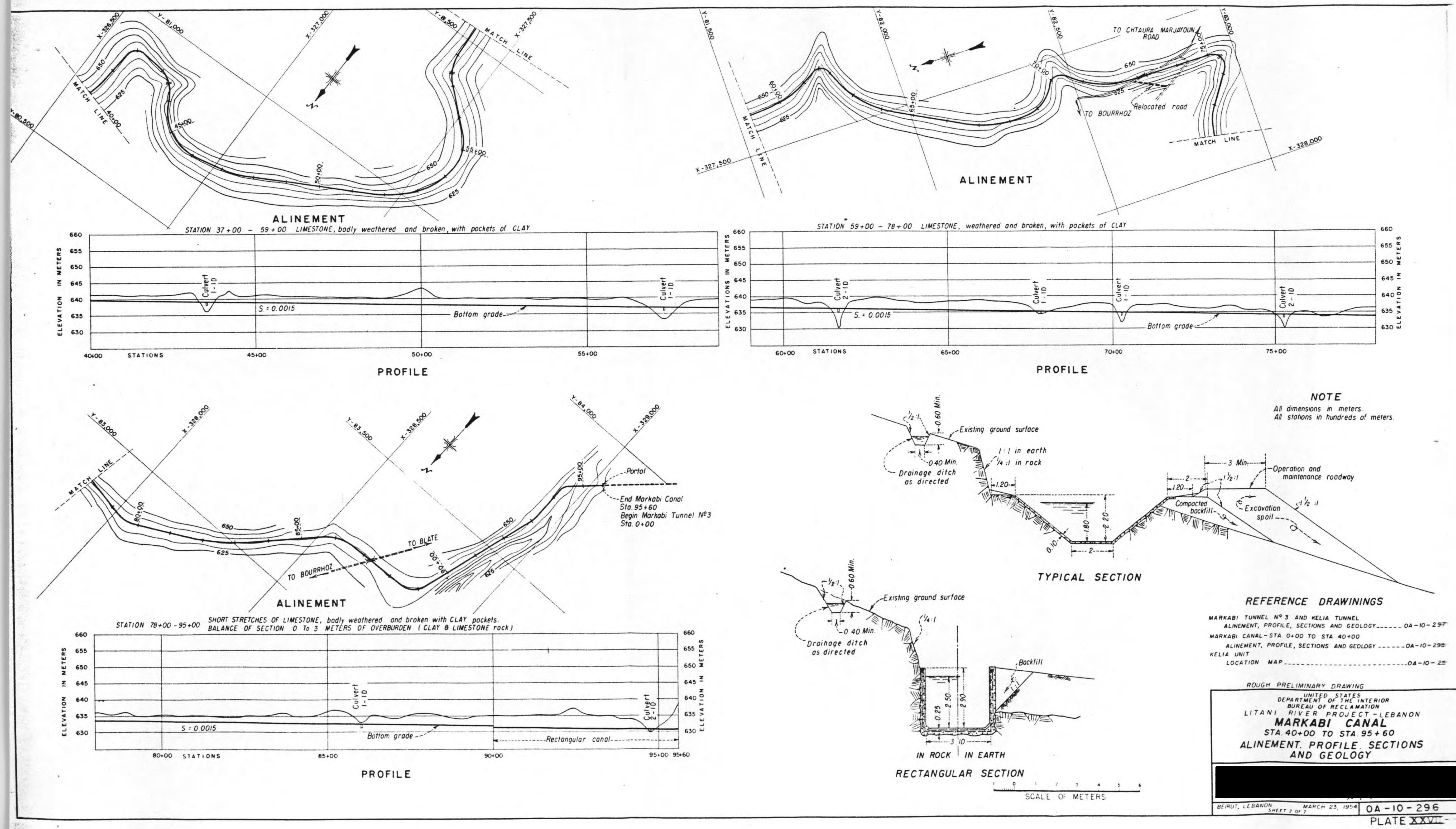
MARKABI CANAL

STA. 0+00 TO STA. 40+00
ALINEMENT, PROFILE, SECTIONS AND GEOLOGY

[redacted]

BEIRUT, LEBANON SHEET 1 OF 2 MARCH 23, 1954 | OA-10-295

PLATE ~~XXVII~~ -4


ALINEMENT
STATION 37+00 - 59+00 LIMESTONE, badly weathered and broken, with pockets of CLAY
ELEVATION IN METERS
Culvert 1-1D
S = 0.0015
Bottom grade
40+00 STATIONS 45+00 50+00 55+00
PROFILE
ALINEMENT
TO CHTAURA MARJAYOUN ROAD
Relocated road
TO BOURRHOZ
MATCH LINE
STATION 59+00 - 78+00 LIMESTONE, weathered and broken, with pockets of CLAY
Culvert 2-1D
Culvert 1-1D
S. = 0.0015
Bottom grade
60+00 STATIONS 65+00 70+00 75+00
PROFILE
NOTE
All dimensions in meters.
All stations in hundreds of meters.
ALINEMENT
Portal
End Markabi Canal
Sta. 95+60
Begin Markabi Tunnel Nº3
Sta. 0+00
TO BLATE
TO BOURRHOZ
STATION 78+00 - 95+00 SHORT STRETCHES OF LIMESTONE, badly weathered and broken with CLAY pockets.
BALANCE OF SECTION 0 To 3 METERS OF OVERBURDEN (CLAY & LIMESTONE rock)
S = 0.0015
Bottom grade
Rectangular canal
80+00 STATIONS 85+00 90+00 95+00 95+60
PROFILE
Existing ground surface
1:1 in earth
1/4:1 in rock
0.60 Min.
0.40 Min.
Drainage ditch as directed
Operation and maintenance roadway
3 Min.
Compacted backfill
Excavation spoil
TYPICAL SECTION
Existing ground surface
Drainage ditch as directed
Backfill
IN ROCK IN EARTH
RECTANGULAR SECTION
SCALE OF METERS
REFERENCE DRAWININGS
MARKABI TUNNEL Nº 3 AND KELIA TUNNEL ALINEMENT, PROFILE, SECTIONS AND GEOLOGY OA-10-297
MARKABI CANAL - STA. 0+00 TO STA 40+00 ALINEMENT, PROFILE, SECTIONS AND GEOLOGY OA-10-295
KELIA UNIT LOCATION MAP OA-10-25
ROUGH PRELIMINARY DRAWING
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
LITANI RIVER PROJECT - LEBANON
MARKABI CANAL
STA. 40+00 TO STA. 95+60
ALINEMENT, PROFILE, SECTIONS AND GEOLOGY
[redacted]
BEIRUT, LEBANON MARCH 23, 1954
SHEET 2 OF 2
OA-10-296
PLATE XXVII

wet gouge zones should be encountered. Short sections of this tunnel will require support due to the many broken areas. Some blocky ground will be encountered due to the dip of the limestones and roof bolts may be required. The geology for this tunnel is shown on Plate XXVII-3.

Kelia Penstock. The entire length of this penstock route is along the slope of lower part of the upper Cretaceous limestones. From the outlet portal of the Kelia Tunnel at Station 0+00 to Station 7+35 these limestones are gray and massive-bedded with little overburden. From Station 7+35 to the power house area the limestones are yellow with alternating marl beds. Some overburden occurs near the lower end of the penstock and in the powerhouse area it is 3-1/2 to 5-1/2 meters deep.

The penstock slope is relatively flat for its entire length and follows the bedding of the limestones. Little weathering has occurred below the surface in these limestones and they are sufficiently competent to carry the penstock anchors with little excavation. The geology of the penstock is shown on Plates XXVII-6, 7 and 8.

Kelia Power Plant. Five test pits were dug at the powerhouse location. The locations of these holes are shown on Plate XXVII-8. The overburden consists of angular blocks of limestone talus embedded in clay. This limestone came from the limestone cliffs west of the powerhouse area. Considerable faulting has occurred along these cliffs which has broken the limestones into large blocks that break away from the cliffs and cover the slopes above this powerhouse location.

Bedrock was encountered in these holes at depths ranging from 3.5 to 5.5 meters. This rock is composed of thin-bedded marl and limestone which is either flat or is dipping from 3 to 5 degrees to the west in the direction of the faulted area along the cliffs. This bedrock is weathered at the surface and the zone of weathering penetrates about 1-1/2 meters into the rocks. At this depth the rocks appear compact with little weathering.

In order to gain as much head as possible the river channel below the powerhouse should be cleared of boulders and other debris in order to utilize the fall in the river immediately below the powerhouse. If this is done the footings for this structure can be excavated several meters into bedrock.

Hydrologic Data

Conveyance structures for the Kelia Power Unit consist of the three free-flow, tunnel sections of the Markabi Tunnel, the first and second sections connected by a concrete conduit and the second and third connected by the Markabi Canal; the Kelia forebay; the Kelia pressure tunnel and the Kelia penstock. All but the Kelia penstock will be used jointly to convey water for the Kelia Power Plant and for the Upper Nabatiye Irrigation Unit. The Kelia penstock will be used for conveying water from the Kelia Tunnel to the power plant. The discharge capacity of all of these tunnels and canals was selected at 21 cubic meters per second, or that required to operate the four 12,000 kilowatt units, to be installed in Kelia Power Plant, at maximum peaking capacity. It was assumed that during the irrigation season there would be insufficient water available to supply the full requirement of 1.8 cubic meters per second for the Upper Nabatiye Unit and to operate the Kelia Power Plant at full load for more than a few hours each day. The Kelia forebay was designed with 236,000 cubic meters of usable capacity to provide the additional water necessary for peaking the Kelia Power Plant for those few hours each day during the irrigation season, when it would be required.

The normal water surface at Markabi Dam has been selected at elevation 660 and will be maintained as much of the time as practicable. The Markabi Tunnel has been designed as a free-flow tunnel in order to utilize a canal section between two of its sections. The tunnel invert elevation has been selected as 657.5 meters at Markabi Division Dam and elevation 627 meters where it discharges into Kelia forebay. The diameter of this tunnel has been so selected as to permit it to discharge a maximum of 21 cubic meters per second when flowing partially full. The canal section connecting the two tunnel sections has been disigned with a trapezoidal section capable of carrying the same discharge.

Kelia forebay has been designed to operate between a normal water surface at elevation 626.5 meters and a minimum water surface at elevation 624.5 meters, while its spillway was designed to limit its maximum water surface to 627.4 meters. The local drainage area, contributing to this forebay, was estimated at 2.5 square kilometers and use of the Burkli-Zeigler Formula 1/ indicates it is likely to produce a maximum peak inflow of 14.5 cubic meters per second. The spillway was designed to discharge such flows as would be required if this inflow should occur when the forebay was full to its normal water surface, elevation of 626.5 meters.

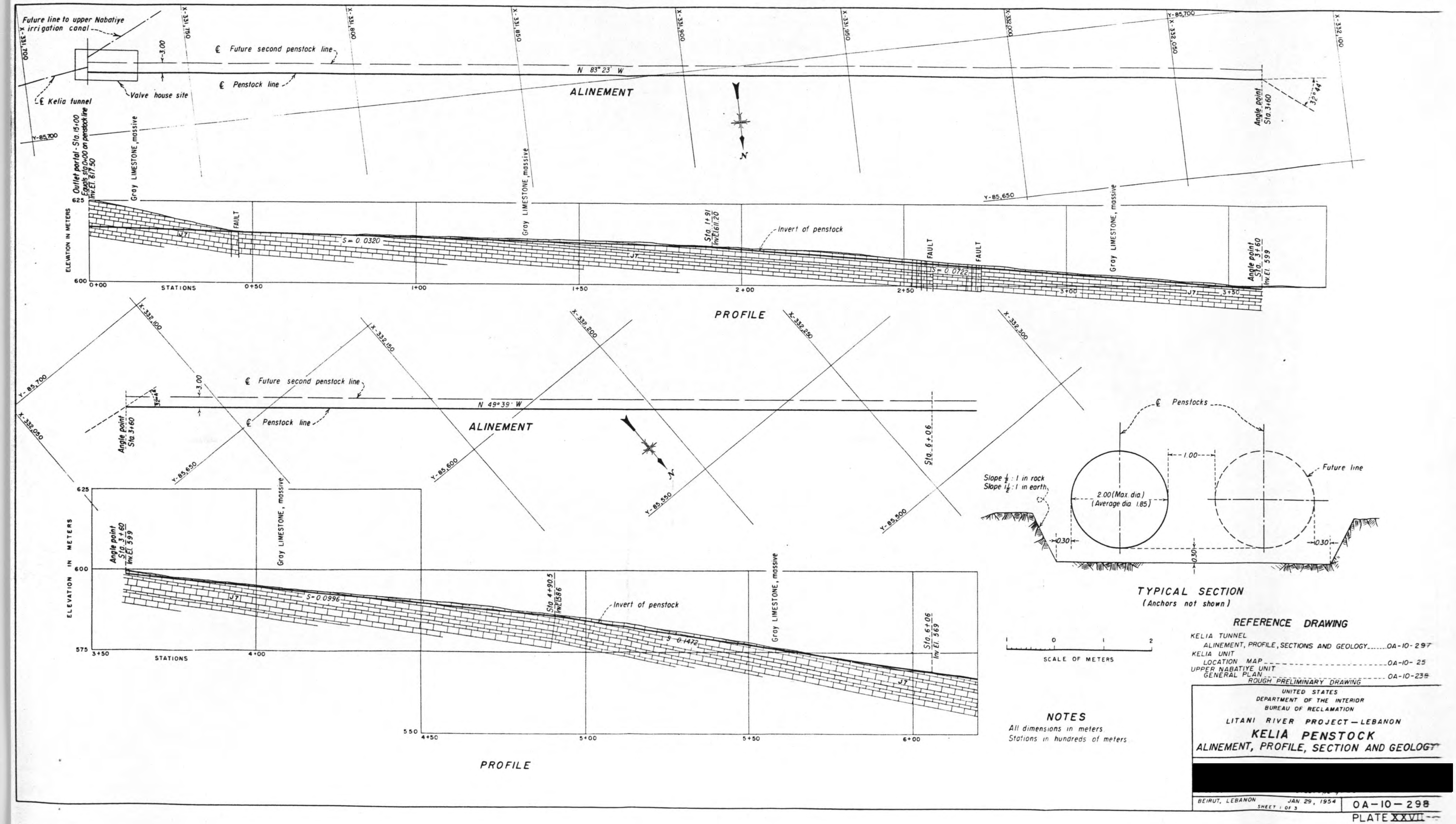
The Kelia Tunnel was designed as a pressure tunnel 2.9 meters in diameter to discharge a maximum of 21 cubic meters per second with the Kelia forebay at minimum elevation of 624.5 meters. The invert at Kelia forebay was fixed at elevation 620.5 so as to provide a minimum of 1.1 meters of cover over the entrance of this tunnel when the Kelia forebay is at minimum water surface elevation of 624.5 meters. It is expected that this forebay will generally be operated at the normal water surface elevation of 626.5 meters which will provide a cover of 3.1 meters over the entrance to this tunnel. The average loss of head in this tunnel has been estimated at 3 meters.

Tail water at the Kelia plant is not a factor since it is proposed to install impulse-type turbines in this plant. The average elevation of the nozzle for these units was assumed at 300 meters. Head on this plant was assumed to vary from elevation 624.5 to 626.5 meters as controlled by elevations in the Kelia forebay.

Project Features

Markabi Tunnels. These consist of three tunnel sections. Tunnel No. 1 will be 1970 meters long between the portal just below the Markabi Diversion Dam and the outlet portal which will be connected to a steel reinforced concrete conduit 335 meters long that will join the outlet of Tunnel No. 1 to the inlet of Tunnel No. 2. Between the head gate of the Markabi Diversion Dam and the portal of Tunnel No. 1 will be a short (about 30 meters) flume which will contain the necessary measuring and desilting devices, by-pass gates, and an overflow section adjacent to the tunnel portal. The head gate will have a sill elevation of 657.5 meters and the outlet portal of Tunnel No. 1 will have an invert elevation of 653.57 meters. Tunnel No. 2 will be 3440 meters long and will supply water to the Markabi Canal at an invert elevation of 646 meters. Tunnel No. 3 will be 1995 meters long and will deliver water from the Markabi Canal to the Kelia forebay at an invert elevation of 627 meters. The tunnel will be concrete lined, free-flow, horseshoe-shaped with an inside diameter of 3.00 meters. The conduit between Tunnels No. 1 and No. 2 will be constructed of reinforced concrete and will have the same internal diameter and hydraulic properties as the tunnels. Light to moderate supports will be required for approximately 1/3 the length of Tunnels Nos. 1, 2, and 3. As the overburden is not excessive and no external hydrostatic pressure is anticipated, normal thickness of concrete lining will be used. Weep holes through the concrete lining will be placed where required. Roof bolts would be used wherever the geologic formation is suitable. All of the tunnels will be free-flow type and will not be subjected to internal pressure, therefore, no reinforcement will be required except in areas where the ground may be highly crushed or subject to swelling or squeezing. The alinement, profiles, and sections of these tunnels are shown on Plates XXVII-2 and 3.

1/ Transactions. American Society of Civil Engineers, New York, N.Y., Volume 20, pp. 1, 18.


Future line to upper Nabatiye irrigation canal
€ Future second penstock line
€ Penstock line
€ Kelia tunnel
Valve house site
3.00
N 83° 23' W
ALINEMENT
N
Angle point Sta. 3+60
32° 44'
X-331,700
X-331,750
X-331,800
X-331,850
X-331,900
X-331,950
X-332,000
X-332,050
X-332,100
Y-85,700
Y-85,650
Outlet portal - Sta. 15+00 Equals sta 0+00 on penstock line Inv. El. 617.50
Gray LIMESTONE, massive
ELEVATION IN METERS
625
600
FAULT
J7
S = 0.0320
Sta. 1+91 Inv. El. 611.20
Invert of penstock
S = 0.0722
Angle point Sta. 3+60 Inv. El. 599
0+00
STATIONS
0+50
1+00
1+50
2+00
2+50
3+00
3+50
PROFILE
X-332,100
X-332,150
X-332,200
X-332,250
X-332,300
Y-85,700
X-332,050
Y-85,650
Y-85,600
Y-85,550
Y-85,500
€ Future second penstock line
3.00
N 49° 39' W
€ Penstock line
ALINEMENT
Angle point Sta. 3+60
Sta. 6+06
ELEVATION IN METERS
625
600
575
550
Angle point Sta. 3+60 Inv. El. 599
Gray LIMESTONE, massive
J7
S = 0.0996
Sta. 4+90.5 Inv. El. 586
Invert of penstock
S = 0.1472
Sta. 6+06 Inv. El. 569
3+50
STATIONS
4+00
4+50
5+00
5+50
6+00
PROFILE
€ Pestocks
1.00
Future line
Slope ½ : 1 in rock
Slope 1½ : 1 in earth
2.00 (Max. dia.)
(Average dia. 1.85)
0.30
0.30
0.30
TYPICAL SECTION
(Anchors not shown)
1
0
1
2
SCALE OF METERS
NOTES
All dimensions in meters
Stations in hundreds of meters
REFERENCE DRAWING
KELIA TUNNEL
ALINEMENT, PROFILE, SECTIONS AND GEOLOGY 0A-10-297
KELIA UNIT
LOCATION MAP 0A-10-25
UPPER NABATIYE UNIT
GENERAL PLAN 0A-10-239
ROUGH PRELIMINARY DRAWING
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
LITANI RIVER PROJECT — LEBANON
KELIA PENSTOCK
ALINEMENT, PROFILE, SECTION AND GEOLOGY
[redacted]
BEIRUT, LEBANON
JAN 29, 1954
SHEET 1 OF 3
0A-10-298
PLATE XXVII

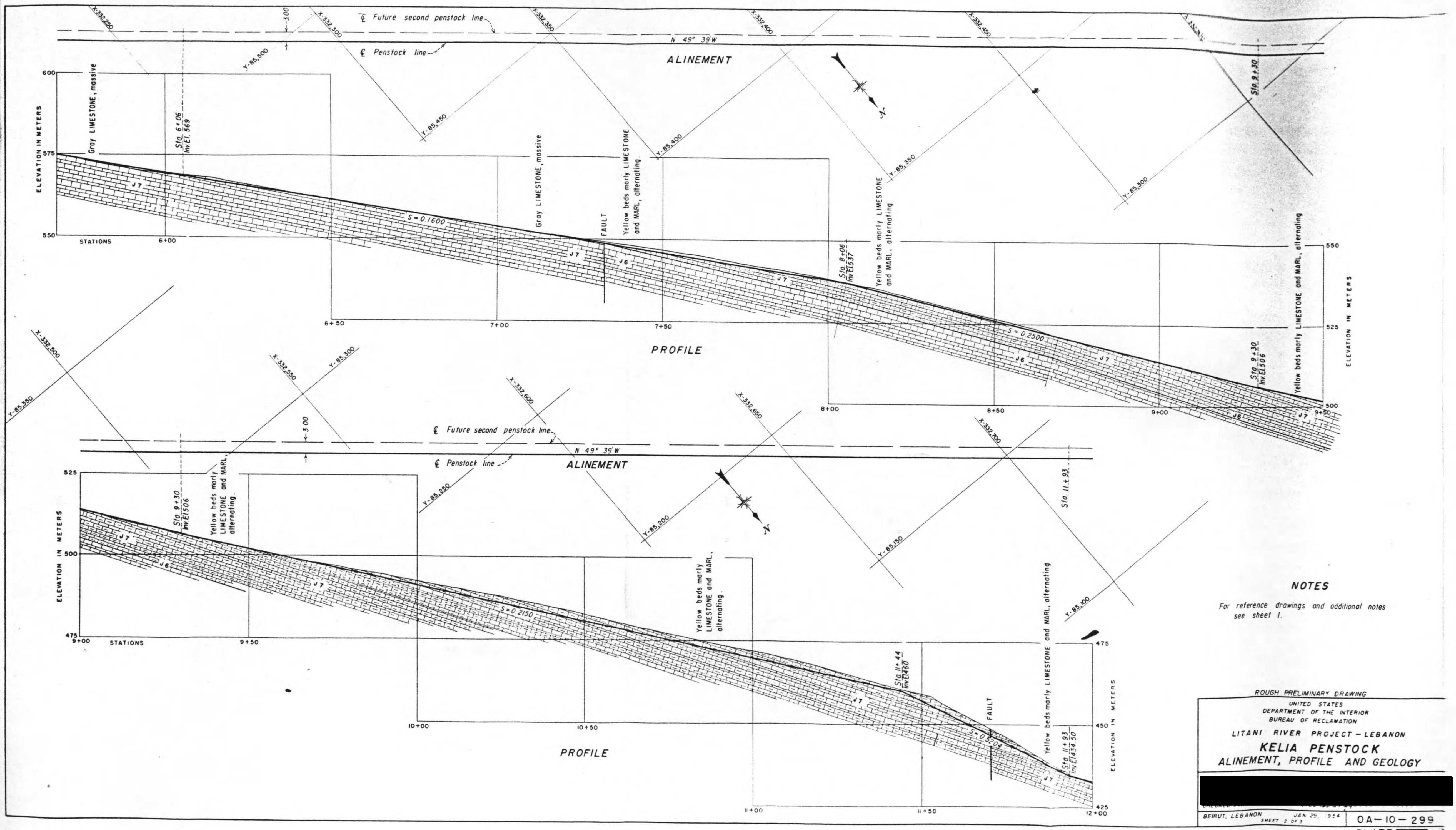

ALINEMENT
℄ Future second penstock line
℄ Penstock line
N 49° 39' W
3.00
X-332,250
X-332,300
X-332,350
X-332,400
X-332,450
Y-85,500
Y-85,450
Y-85,400
Y-85,350
Y-85,300
Sta. 9+30
N
PROFILE
ELEVATION IN METERS
STATIONS
600
575
550
525
500
6+00
6+50
7+00
7+50
8+00
8+50
9+00
9+50
Gray LIMESTONE, massive
Sta. 6+06
Inv. El. 569
S=0.1600
FAULT
Yellow beds marly LIMESTONE and MARL, alternating.
Sta. 8+06
Inv. El. 537
Yellow beds marly LIMESTONE and MARL, alternating
S=0.2500
Sta. 9+30
Inv. El. 506
Yellow beds marly LIMESTONE and MARL, alternating
J7
J6
ALINEMENT
℄ Future second penstock line
℄ Penstock line
N 49° 39' W
3.00
X-332,500
X-332,550
X-332,600
X-332,650
X-332,700
Y-85,350
Y-85,300
Y-85,250
Y-85,200
Y-85,150
Y-85,100
Sta. 11+93
N
PROFILE
ELEVATION IN METERS
STATIONS
525
500
475
450
425
9+00
9+50
10+00
10+50
11+00
11+50
12+00
Sta. 9+30
Inv. El. 506
Yellow beds marly LIMESTONE and MARL, alternating.
S=0.2150
Yellow beds marly LIMESTONE and MARL, alternating.
Sta. 11+44
Inv. El. 460
FAULT
S=0.5204
Yellow beds marly LIMESTONE and MARL, alternating
Sta. 11+93
Inv. El. 434.50
NOTES
For reference drawings and additional notes see sheet 1.
ROUGH PRELIMINARY DRAWING
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
LITANI RIVER PROJECT – LEBANON
KELIA PENSTOCK
ALINEMENT, PROFILE AND GEOLOGY
[redacted]
BEIRUT, LEBANON
JAN 29, 1954
SHEET 2 OF 3
OA–10–299

PLATE XXVII

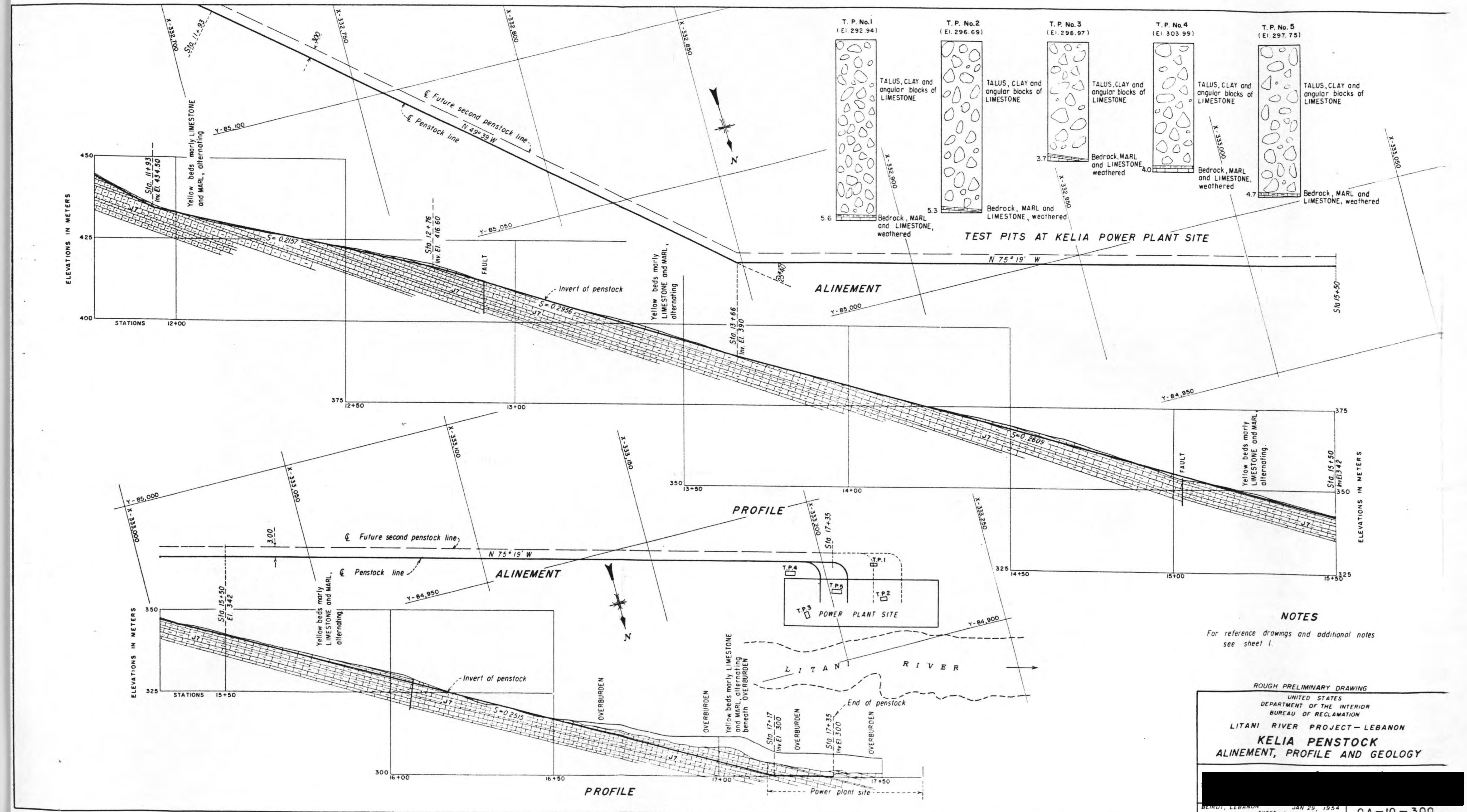


PLATE XXVII-3

Markabi Canal. The Markabi Canal will be 9560 meters long, between Tunnel No. 2 and Tunnel No. 3. The canal will be concrete-lined and have a bottom width of 2.0 meters, a water depth of 1.8 meters, side slopes of 1-1/4 to 1, and a designed velocity of 2.73 meters per second for a capacity of 20.9 cubic meters per second. A cut and cover section 250 meters long is proposed through the edge of the village of Kelia, and a bench flume 500 meters long adjacent to the Markabi Tunnel No. 3 inlet. No other major structures are required. The minor structures required are; a two-lane bridge, 6 single lane bridges, 12 small culverts for cross drainage and 2 overflow spillways. Several small turnouts probably will be required to supply irrigation water for small flat areas of land along the canal. The lining will be steel-reinforced only in sections over cross drains, fills and where required by unsuitable foundations. The alinement, profile and sections of this canal are shown on Plates XXVII-4 and 5.

The principal items of work are 185,000 cubic meters of canal excavation (mostly shattered limestone), 10,880 cubic meters of concrete for canal lining and bench flume, and 215,000 kilograms of steel reinforcement bars. These items do not include the material and work required for the minor structures.

Kelia Forebay. This forebay site is 1-1/2-kilometers north of Marjayoun at the village of Dibbine. The forebay will provide regulation storage for the Kelia Power Plant by impounding water from the Markabi Canal and Tunnels and by discharging water into the Kelia pressure tunnel. The forebay will have an active storage capacity of 236,000 cubic meters, between normal water surface elevation of 626.5 meters and minimum water surface elevation of 624.5 meters. This is equivalent to 150 percent of the water volume in the Sohmor Tunnel and penstock and the Markabi Canal and Tunnels flowing at peak capacity.

The forebay dam will be an earth-rock fill embankment about 177 meters long. The crest of the dam at elevation 629 meters will be 6-meters wide to accommodate the Marjayoun-Blate highway. The upstream and downstream slopes of the dam will be 2-1/2:1 and 2:1, respectively. A thin, impervious, earth core is proposed in order to utilize most of the rock materials available from the tunnel and spillway excavations. Sufficient impervious earth material is available in the reservoir area in the form of moderately plastic silts and clays for construction of the core. The embankment will have a maximum height of 13 meters above streambed and will contain about 41,000 cubic meters of placed material, consisting of 11,000 cubic meters of earth fill, 30,000 cubic meters of rock fill. The forebay will also require 13,300 cubic meters of excavation (9,400 common, 3,900 rock) for the dam and spillway, 720 cubic meters of concrete, and 58,000 kilograms of reinforcing steel.

An open channel spillway will be constructed at the right abutment of the dam to limit the maximum reservoir level to elevation 627.40 with a design discharge of 35.4 cubic meters per second. The design discharge includes 20.9 cubic meters per second from Markabi Tunnel and 14.5 cubic meters per second from the forebay drainage area. The spillway crest will be an uncontrolled section with an effective length of 22.5 meters. A reinforced concrete bridge will be provided to carry the roadway over the spillway section. A concrete-lined trapezoidal channel about 70 meters long will be used to discharge excess flows into the natural waterway below the dam.

The water from the drainage area will be diverted, during construction of the forebay dam, through a short section of the Kelia Tunnel as discussed under the description of the Kelia Tunnel. Only a small cofferdam and channel will be required to divert the small intermittent flow of water to the tunnel inlet. Because the dam will be constructed across a small intermittent stream that only flows during the rainy season and for a short time thereafter, the outlet connected to the Kelia Tunnel and about 100 meters from the dam axis, will be sufficient to provide for drainage of the forebay in event of failure of the Kelia Tunnel, gate house, or penstock. The intake structure for the Kelia Tunnel will be located at the left abutment of the dam. The proposed arrangement of the forebay is shown on Plate XXVII-9.

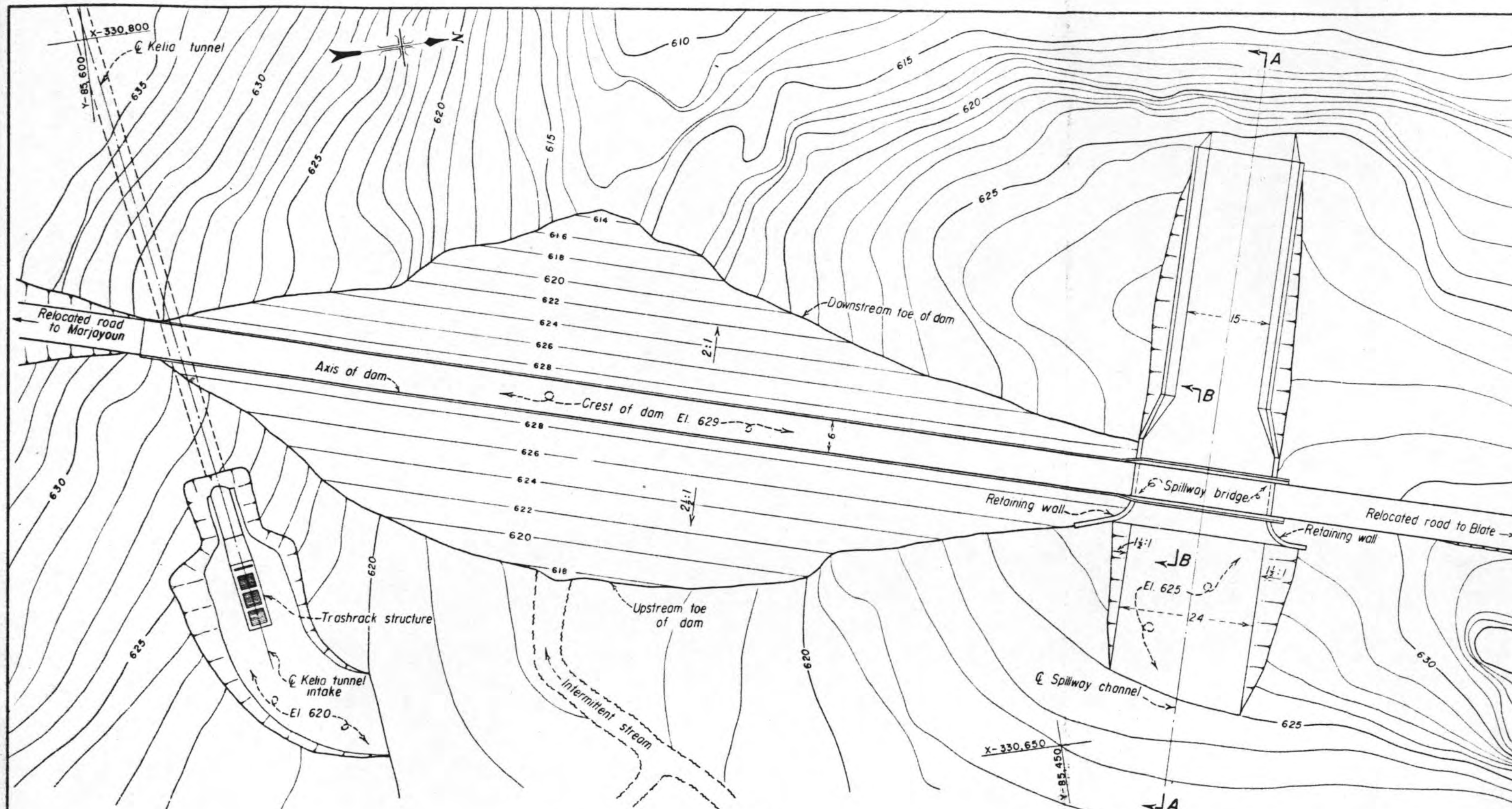
Kelia Tunnel. This tunnel will be 1012 meters long and will supply water from the Kelia forebay at an invert elevation of 620.5 meters to a gate house at an invert elevation of 617.5. The gate house will contain emergency gates for the two Kelia power penstocks and a control gate for the Upper Nabatiye Irrigation Unit. The tunnel will be a circular, concrete lined, pressure-type with a finished diameter of 2.9 meters. It will be steel reinforced only near the portals where the overburden is insufficient to withstand the internal pressure. The geologic formation for the remainder of the tunnel is believed to be sufficiently solid to withstand the small internal pressure without the use of any steel reinforcement. The first section of the tunnel, 150 meters between the inlet portal and a small draw where the tunnel daylights, will be used as a diversion tunnel during the construction of the Kelia forebay dam. Since no outlet is planned through the forebay dam a drainage system will be constructed where the tunnel crosses the small draw, so that the forebay may be emptied without discharging the water through the Kelia Tunnel and Power Plant. The intake structure will consist only of a simple box type trashrack structure with stop log slots. No control gates will be installed at the Kelia Tunnel inlet since the flow of water into the forebay will be controlled by the Markabi Diversion Dam and the Kelia Tunnel is very short and terminates at a gate house. The alinement, profile and sections for Kelia Tunnel are shown on Plate XXVII-3.

Construction of the Markabi Tunnels and the Kelia Tunnel will require 92,000 cubic meters of tunnel excavation, 27,210 cubic meters of concrete for lining, 200,000 kilograms of reinforcement steel, and 745,000 kilograms of steel rib supports.

Kelia Penstock. This penstock will convey water from the outlet of the Kelia Tunnel to the Kelia Power Plant. It is proposed to use two steel pipe lines for the full length of the penstock but to initially construct only one line to serve the first two Kelia Units. The other line will be constructed whenever the second two Kelia Units are installed. The penstock will have a slope length of 1780 meters and a difference in elevation of 318 meters. The center section of the pipe line will have a diameter of approximately 1.85 meters. To facilitate transportation of the sections and to reduce the overall cost, the upper section will be slightly larger and the lower section slightly smaller than this diameter. A gate house will be constructed at the beginning of the penstock to control the flow of water to the penstock lines and to an irrigation canal for the Upper Nabatiye Irrigation Unit. The penstocks will require; 13,500 cubic meters of excavation, mostly rock; 5,000,000 kilograms of steel pipe; 1,220 cubic meters of concrete; 195,000 kilograms of steel reinforcement bars; and material and equipment for the control house. The alinement, profile and sections of Kelia penstock are shown on Plates XXVII-6, 7, and 8.

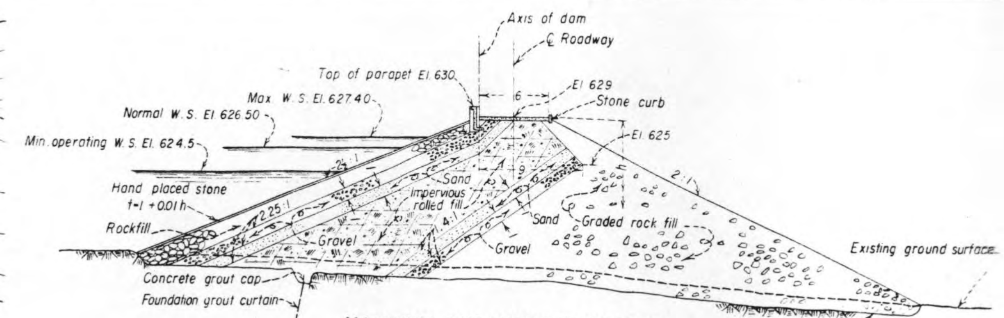
Consideration should be given to the possibility of reducing the overall cost by constructing a steel-reinforced concrete pipe for the first section of the penstock due to the flat slope of this section. Such a concrete section should be large enough to serve the four Kelia Power Units and the Upper Nabatiye Irrigation Unit. A gate house should be constructed at the end of the concrete line to regulate the flow to the power penstocks and to the irrigation line. A small regulating reservoir would be required in the irrigation line to even out any fluctuations caused by surges in the penstock lines and to prevent air from entering the penstocks during the beginning of a heavy demand at the turbines.

Kelia Power Plant. This power plant site is located on the south bank of the Litani River in a narrow canyon just above the maximum high water elevation of the Khardale Reservoir. See Plate XXVII-8. The proposed power plant will have an operating head from about 318 to 321 meters which is in the same range as the proposed Bisri plant, therefore it is believed that identical turbines and generators should be installed at these two plants. Power market studies indicate that only two units would be initially required and that the two additional units would be required approximately 10 years later.



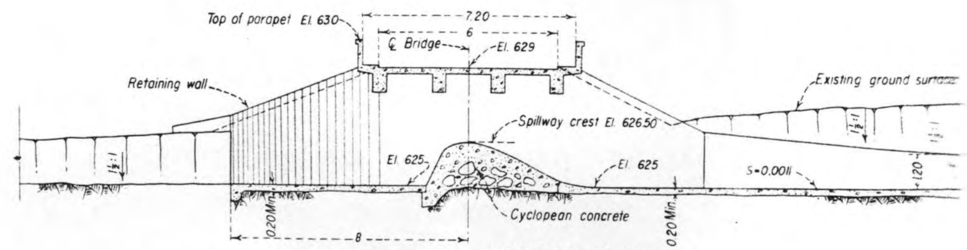
PLAN

SCALE OF METERS



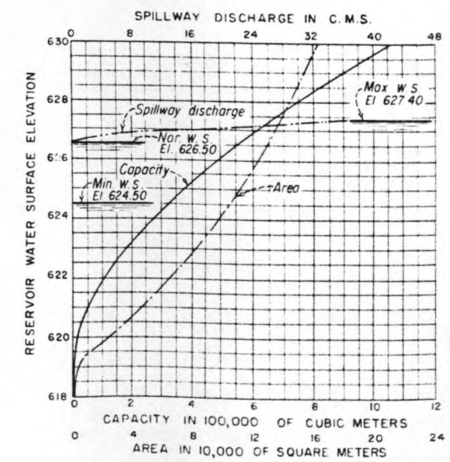
MAXIMUM EMBANKMENT SECTION

SCALE OF METERS

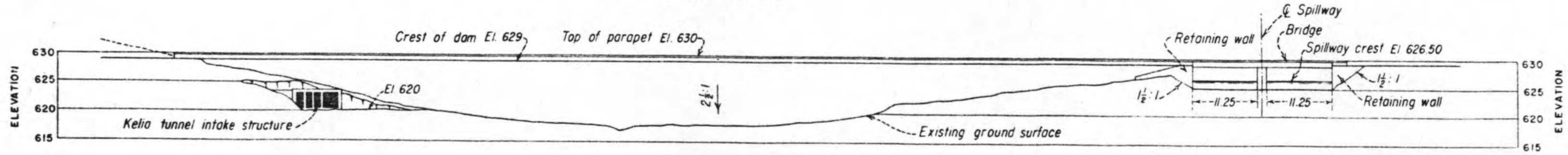


SECTION B-B

SCALE IN METERS

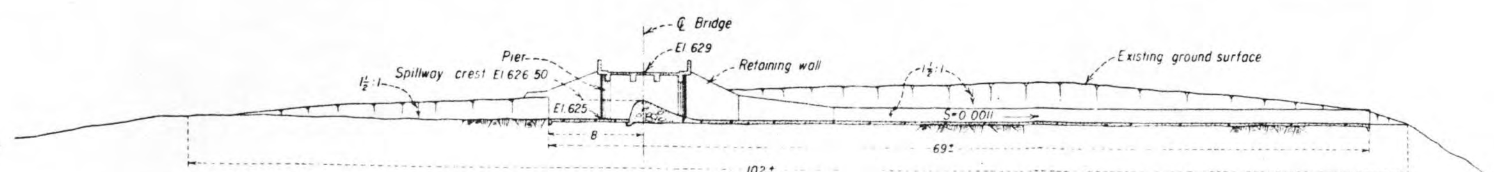


AREA-CAPACITY-DISCHARGE CURVES



UPSTREAM ELEVATION

SCALE OF METERS



SECTION A-A

SCALE IN METERS

NOTES

All dimensions and elevations are in meters.
Anchor bars, reinforcement steel, drains and miscellaneous metalwork not shown.

REFERENCE DRAWINGS

KELIA UNIT - LOCATION MAP 0A-10-25
MARKABI TUNNEL Nº 3 AND KELIA TUNNEL (ALINEMENTS, PROFILES, SECTIONS AND GEOLOGY) ... 0A-10-195

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
LITANI RIVER PROJECT-LEBANON

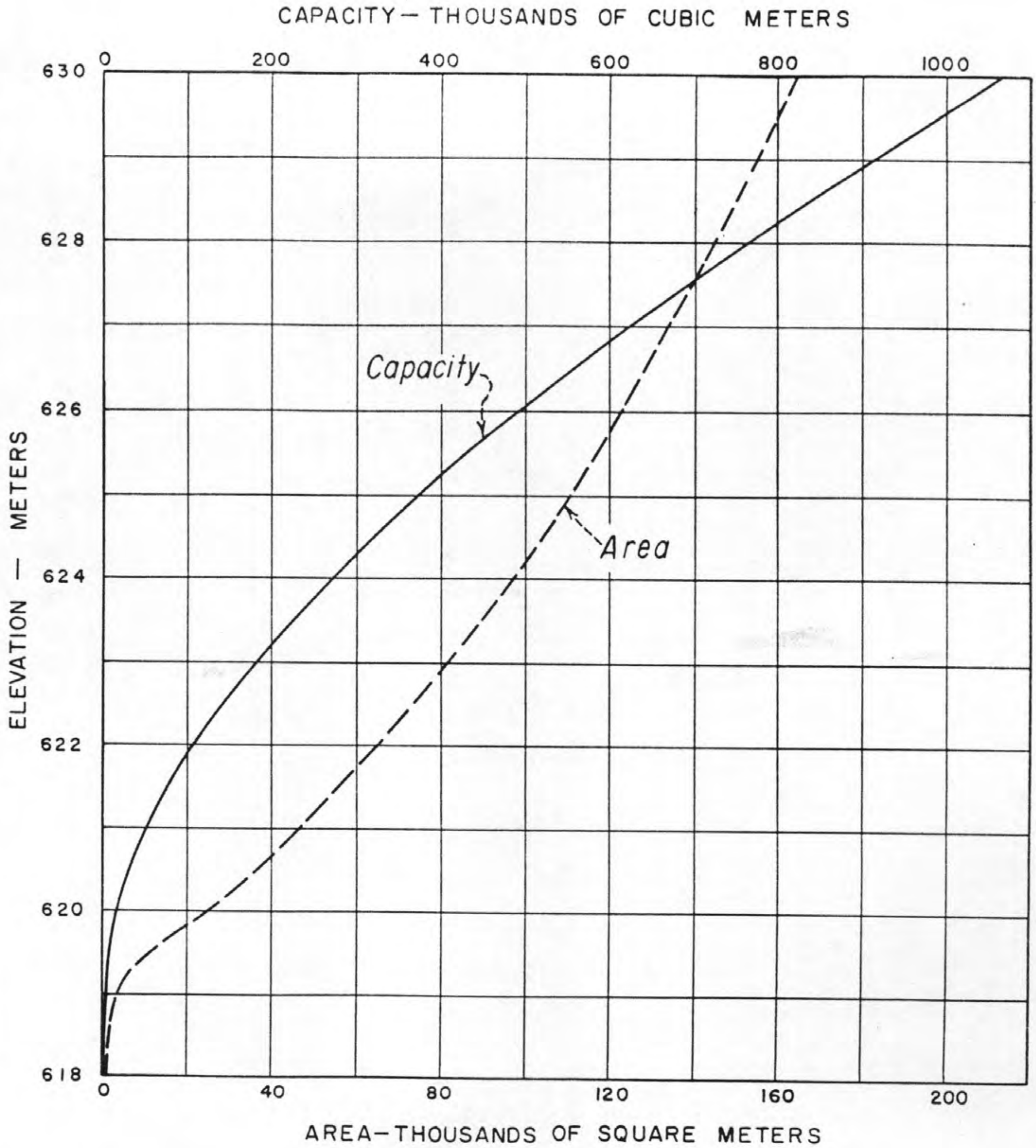
KELIA FOREBAY

PLAN, ELEVATION AND SECTIONS
ROUGH PRELIMINARY ESTIMATE DRAWING

[redacted]

BEIRUT, LEBANON | FEB. 5, 1954 | 0A-10-194

PLATE XXVI



ELEV. (METERS)	AREA (SQUARE METERS)	CAPACITY (CUBIC METERS)
618	850	0
620	25,200	16,500
622	64,600	108,000
624	96,600	271,000
626	123,000	492,000
628	145,000	761,000
630	165,000	1,070,000

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
LITANI RIVER PROJECT—LEBANON

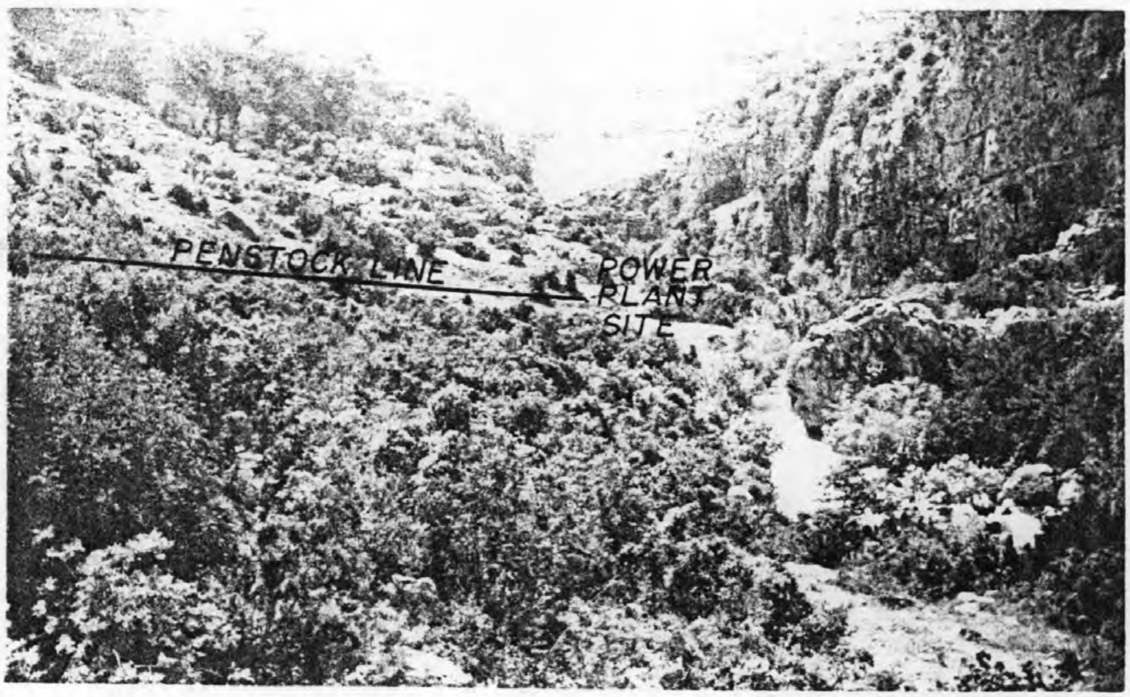
KELIA FOREBAY
AREA—CAPACITY CURVES

[redacted]

BEIRUT, LEBANON 3-1-54 | OA—10—279

Kelia Forebay Dam Site - Looking Downstream

Narrow Litani River Canyon above Kelia Power Plant Site
Looking Downstream



Kelia Power Plant Site - Looking Downstream

It is believed that horizontal impulse type turbines are the most suitable type for both the Kelia and Bisri plants. Either the single overhung-type, horizontal impulse turbine with double jets or the double overhung-type with either one or two jets per turbine appears to be the most suitable for these plants. It is believed that the operating head is somewhat too high to warrent consideration of reaction type turbines.

No special or unusual problems are anticipated and no special protection required. The Litani River channel adjacent to and below the power plant will require some improvement as the present channel is narrow and filled with large boulders. Consideration should be given to the possibility of gaining additional head since the water surface of the river drops approximately 9 meters in 150 meters below the plant. This might be done by extensive clearing, widening, and deepening the channel from the power plant site to the high water elevation of the Khardale Reservoir.

Kelia Switchyard. The switchyard will be located downstream adjacent to the powerhouse. It will be necessary to construct the switchyard at two or more levels due to the steep slope. The switchyard will be of the double bus-bar, single breaker type with one bus-tie breaker and will require; four 11/69 kv. power transformer bay, one 69/35 kv. power transformer bay, two 69 kv. line breaker bays, one 69 kv. bus-tie, two 35 kv. line breaker bays, a station service bay, and a possible future 69/35 kv. power transformer bay. The switchyard will be connected with 69 kv. lines to the Sohmor and Zrariye switchyards and 35 kv. lines to the Hasbaya and Tibnine districts. Carrier current equipment will be required on the two 69 kv. circuits.

Construction Operations

Only one adit, about the mid-point of Markabi Tunnel No. 2, should be considered to shorten the haul distance from the headings. Although other adits and shafts might be used to facilitate the construction of the tunnels they are not believed economically justifiable due to the short length between headings. The excavated material from the outlet of the Markabi Tunnel No. 3 and from the inlet of the Kelia Tunnel will be used in the rock section of the Kelia forebay dam. Materials removed from the other portals will be dumped near the tunnel entrances or may be used for concrete aggregate if properly screened.

Access Roads. Short access roads of from 1/4 to 1/2 kilometer long will have to be built from the Marjayoun-Chtaura Highway to reach the tunnel adits and portals except the inlet of the Markabi Tunnel No. 1, which can be reached from the operation and maintenance road for the Sohmor Power Plant, and to reach the inlet of the Kelia Tunnel which is located along the Marjayoun-Blate road. The access roads to the adit for Tunnel No. 2 and to the junction of Tunnels Nos. 1 and 2 will be over steep rocky slopes and will be difficult to construct. The other access roads will be over flatter slopes and more easily constructed.

The construction of an access, operation, and maintenance road to the powerhouse is proposed along the penstock route from which short construction roads could be built to facilitate the installation of the penstocks. This road should be constructed from the Marjayoun-Blate road, and should follow the general alinement of the Kelia Tunnel and penstock. No major bridges or structures will be required, but due to the steep grade of the terrain a series of switch-backs will be required. The road will have a maximum grade of 10 percent, curves with minimum radii of 30 meters and bridges and culverts adequate to carry the large, heavy equipment for the power plant. The Beirut to Blate roads and allied structures are well constructed and adequate for transportation of the power plant equipment.

SECTION XXVIII

ZRARIYE POWER UNIT

Location and Description

The Zrariye power unit will be a multiple purpose unit constructed on the Litani River about 17 kilometers from the mouth. It will have an installed capacity of 12,000 kilowatts, consisting of two 6000 kilowatt units, direct-connected to Francis-type reaction turbines. Water will be furnished this unit from Khardale Reservoir through the free-flow Zaiye Tunnel and Canal, and the Zrariye Tunnel, Canal and penstock. The Zaiye Tunnel will be 2.2 meters in diameter and about 7,700 meters long. The Zaiye Canal will be about 12,600 meters long and have a maximum capacity of about 9.0 cubic meters per second. The Zrariye Tunnel will be 2.2 meters in diameter and about 2330 meters long. It will discharge into the Zrariye Canal, which has an enlarged section to act as a forebay. A steel penstock 1.85 meters in diameter and about 335 meters long will convey water from the Zrariye Canal to the Zrariye Power Plant. Irrigation water will be diverted from the Zrariye Canal into the main canal of the Lower Nabatiye Irrigation Unit during the irrigation season. The Zrariye Power Plant will discharge into the Litani River just above the diversion dam for the Kasmie Project and this discharge will be available for irrigation use by that project. The location and principal features of this unit are shown on Plate XX-1 and XXVIII-1.

Geology

Zaiye Tunnel. The inlet portal for this tunnel is located about 500 meters above the right abutment of the Khardale Dam. The outlet portal leads into the Zaiye Canal. A geologic section along the tunnel route is shown on Plate XXVIII-2.

The inlet portal is located near the base of a steep ravine in the Cretaceous (C4) limestones and dolomites. The western limits of the Khardale fault zone extends a short distance west of the inlet portal of this tunnel. As a result the rocks are crushed, brecciated, and fractured for about 30 meters at this end of the tunnel.

The C4 limestones extend from the inlet portal to about Station 18+00 (Stationing computed in hundreds of meters). At tunnel level the C4 dolomites will probably be encountered throughout this section. Some faulting occurs near Station 4+60 and 8+15. Near Station 18+00 a large fault occurs with the upthrown side to the northeast. A similar fault occurs near Station 20+00. Between these faults the upper Cretaceous (C6) and lower Eocene (E2a) beds outcrop, with the upper part of the Cretaceous (C4) limestones at tunnel level.

Southwest of these two faults the Eocene (E2b) beds are at the surface extending to Station 47+40. Based on measured sections in this area the lower part of the Eocene (E2a) and the upper part of the Cretaceous (C6) beds should be at tunnel level throughout this section. Several minor faults occur.

Southwest of Station 47+40 to the outlet portal, the Eocene (E2a) marly limestones are at the surface with the upper C6 series at tunnel grade. Several small faults and jointed areas occur in this section.

Since the tunnel is crossed by several faults and jointed areas, some support will be necessary at these zones. The rocks along this tunnel route are dipping from 2 degrees to a maximum of 24 degrees near the Khardale fault. As a result some blocky ground will be encountered which will require the use of roof bolts in some sections. Support will be necessary at both portals. It is estimated that approximately 20 percent of the tunnel will require light steel support. Very little water should be encountered.

Zaiye Canal. The geology for this canal is shown on Plates XXVIII-3 and 4. The type of rock and overburden to be found along this tunnel route are shown along the profile according to stations.

This canal is all located in the Cretaceous (C6) series of marly limestones, with a few thin-bedded limestones. Short sections of the canal will pass through overburden which is mostly clay derived from the marly limestones beneath some sections. In clay, overburden will be found at the surface which grades into compact bedrock at or near canal grade.

The marly nature of much of the limestone along this canal route makes it somewhat softer than pure limestone. As a result portions of the weathered surface, residual clays can be easily stripped along the canal route, and along short sections of the canal this weathering extends below canal grade.

Zrariye Tunnel. The geologic section along this tunnel route is shown on Plate XXVIII-5. The inlet portal for this tunnel is located in the Cretaceous (C4) dolomitic limestones. Near Station 5+00 massive limestones should occur at tunnel grade. These massive limestones grade into thin-bedded dense limestones near Station 10+00 which are jointed and faulted at the surface in the vicinity of Station 15+00.

The Cretaceous (C5) marly limestones and limestones occur beyond the faulted area near Station 15+00, and continue to about Station 20+50 where the lower part of the Cretaceous (C6) marly limestones will be encountered at tunnel grade.

These limestones are dipping from 6° to 11° in the direction of the slope of the tunnel and as a result some blocky areas will be found which will need light support or roof bolts. Some support will be needed at the two portals and at the two faulted areas near Stations 15+00 and 20+00. It is estimated that 15 percent of the tunnel will require support.

Zrariye Canal. The geology along this canal route is shown on Plate XXVIII-5. This canal is located in the Cretaceous (C6) marly limestones and in overburden mainly derived from weathering of these rocks. From Station 0+00 to approximately Station 7+50 the limestones are marly, fairly soft, and appear to be weathered from 1 to 2 meters in depth. Portions of this section could be excavated to canal grade by a bull dozer and much of the balance could be pried out with hand tools and the use of a small amount of dynamite.

From Station 7+50 to the Zrariye penstock, the canal passes along an area containing considerable residual clay which is fairly soft when moist and which can be easily stripped along the canal route. Short sections of the canal will be in marly limestones which are weathered from one to two meters in depth.

Zrariye Penstock and Powerhouse. The geologic section along this penstock is shown on Plate XXVIII-6. This penstock route is along a uniform 37° slope except at the upper part where the slope flattens toward the canal. The limestones are dipping from 12° to 14° to the west. They outcrop along the entire slope with the exception of the partially terraced slope at the top of the penstock.

The upper portion of the penstock will rest on the Cretaceous (C5) limestones and the lower part will be on Cretaceous (C4) limestones and dolomites. No faulting or jointing was found along this slope. The limestones and dolomites are not deeply weathered and are competent to carry the penstock anchors with little excavation.

The foundation area for the power plant was not investigated. Limestone occurs to the bottom of the slope and it appears from the surface that bedrock is not deeply buried by overburden near the toe of the slope.

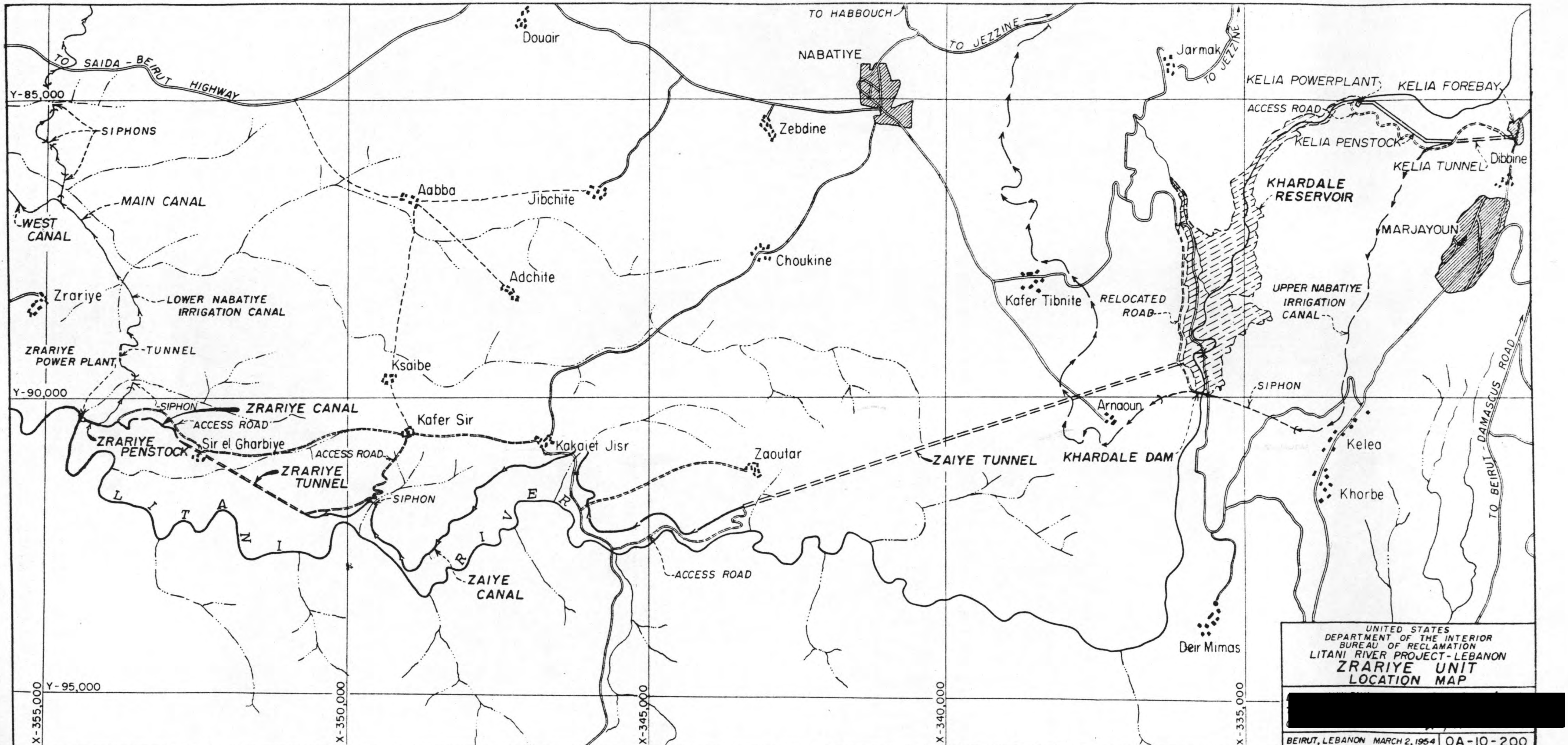
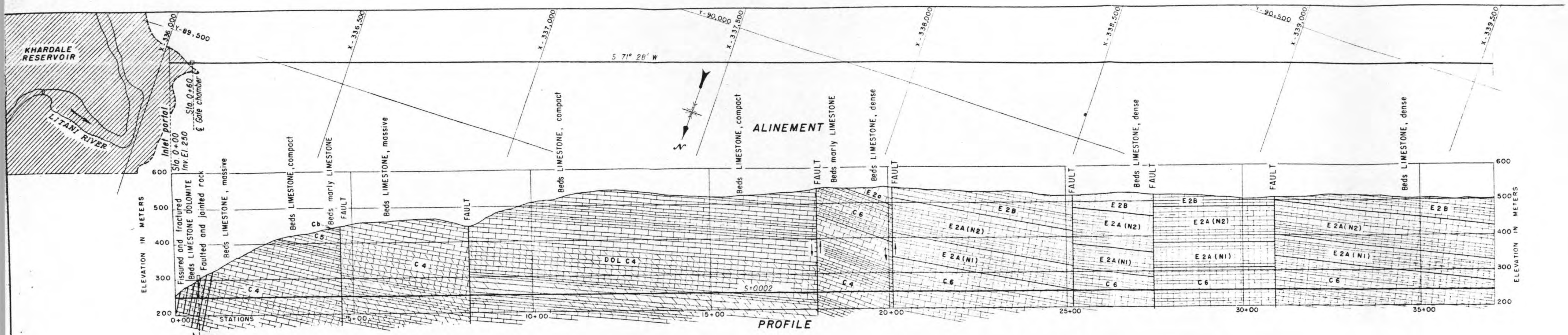
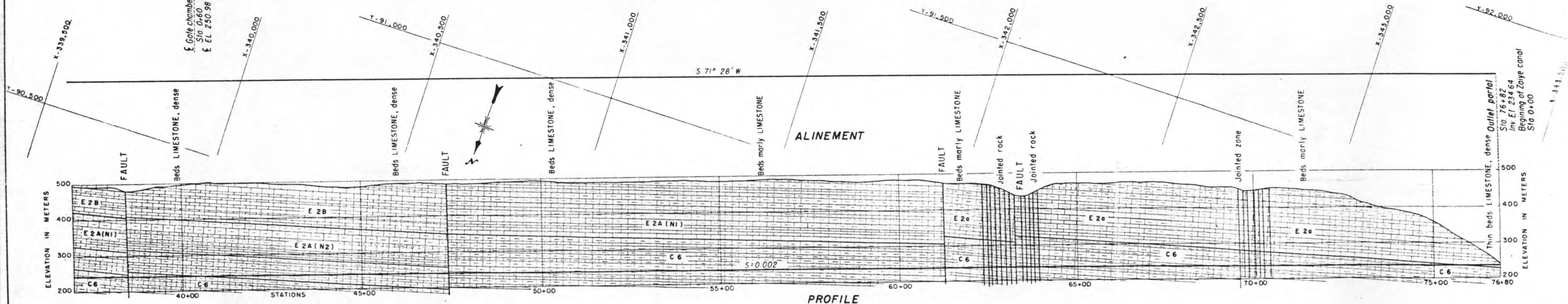
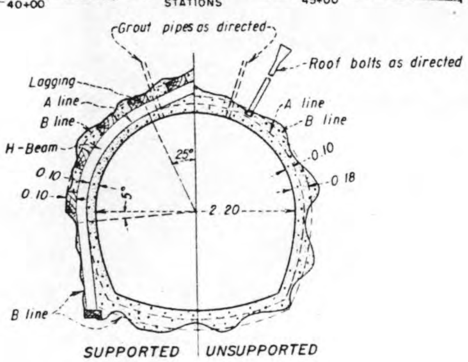


PLATE XXVIII-1







TYPICAL SECTIONS

SCALE OF METERS

NOTES

All dimensions in meters

Stations in hundreds of meters

HORSESHOE TYPE TUNNEL HYDRAULIC PROPERTIES *

d/D	D		A		V		Q		r	n	s
	m	ft	m^2	ft^2	m/s	ft/s	m^3/s	cfs	for ft	for ft	
0.85	2.2	7.22	3.66	39.35	2.46	8.07	9.0	318	2.205	0.014	0.002

* Mannings formula

REFERENCE DRAWINGS

ZRARIYE UNIT
LOCATION MAP 0A-10-200
ZAIYE CANAL - STA 0+00 TO STA 0+65
ALINEMENT, PROFILE, SECTIONS & GEOLOGY 0A-10-361

ROUGH PRELIMINARY DRAWING

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

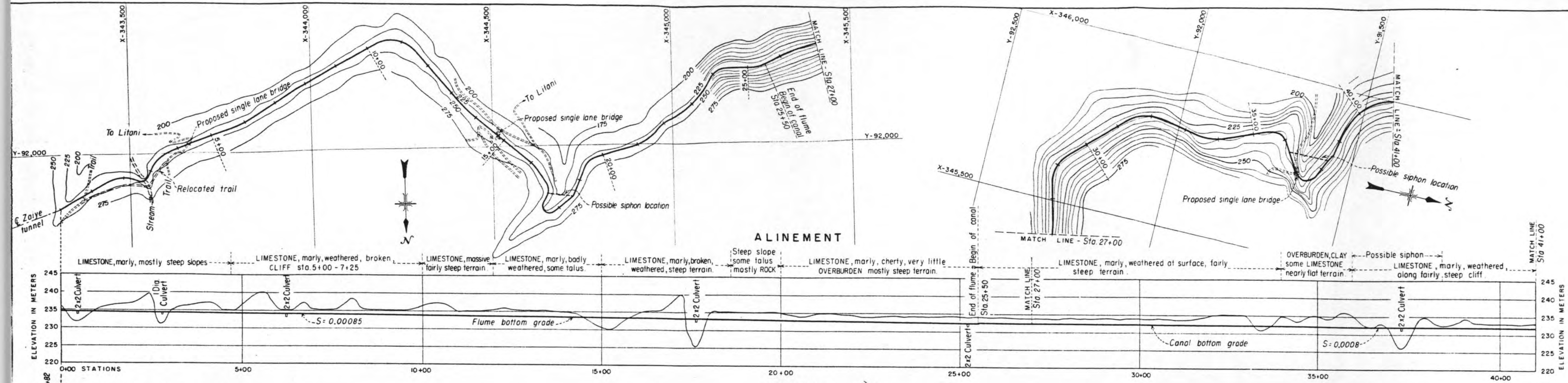
LITANI RIVER PROJECT - LEBANON

ZAIYE TUNNEL
ALINEMENT, PROFILE SECTION AND GEOLOGY

[redacted]

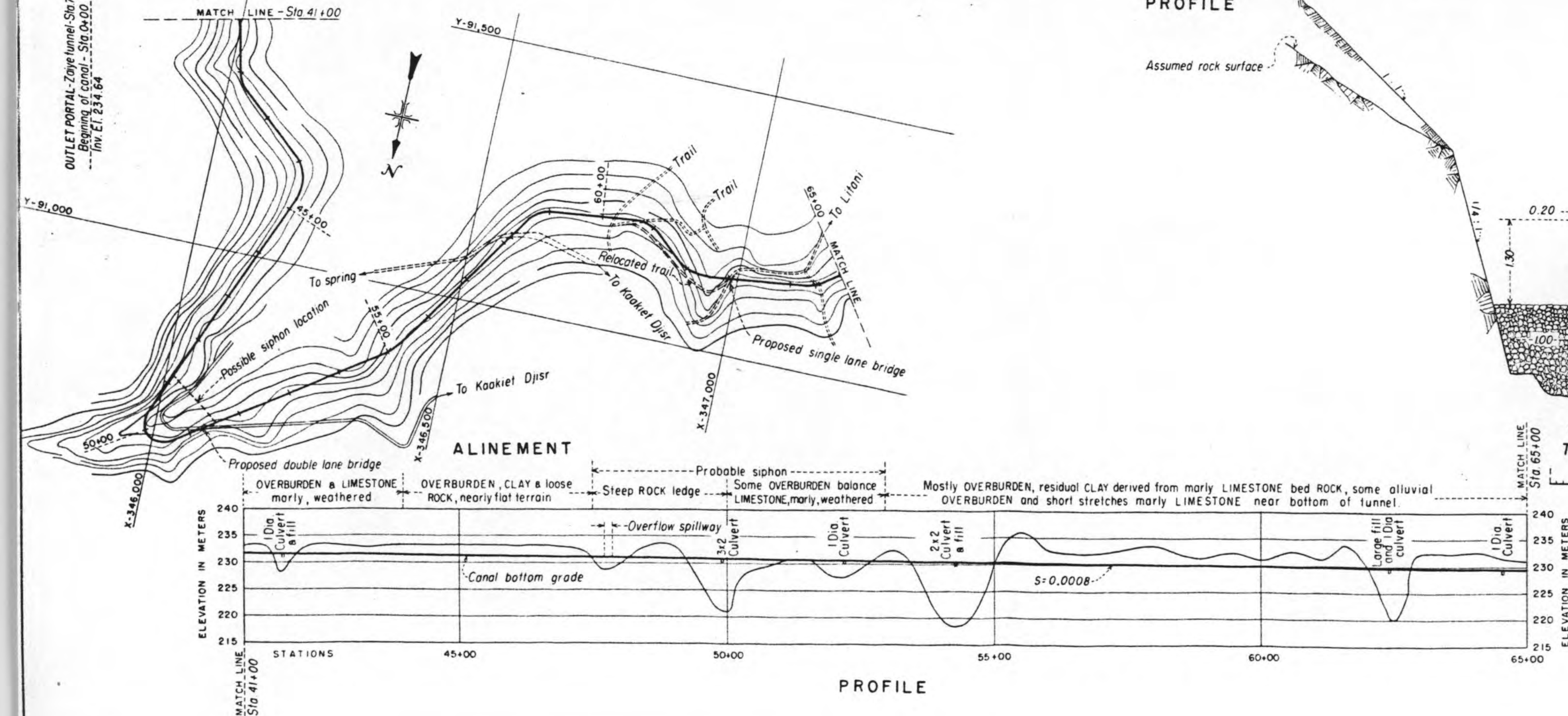
BEIRUT, LEBANON | MARCH 26, 1954 | 0A-10-360

PLATE XXVIII



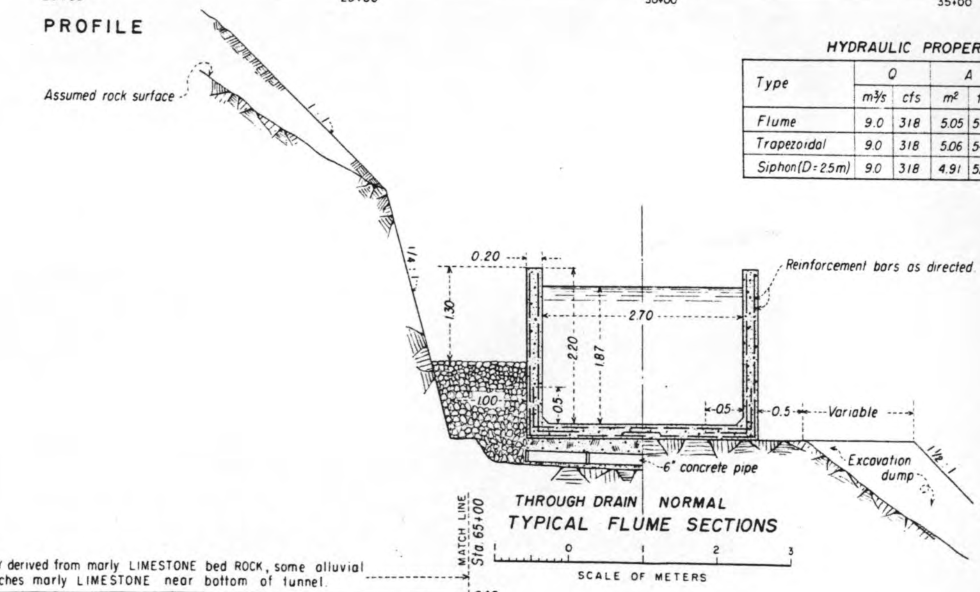
ALINEMENT

PROFILE



ALINEMENT

PROFILE



THROUGH DRAIN NORMAL

TYPICAL FLUME SECTIONS

HYDRAULIC PROPERTIES BY MANNING'S FORMULA

Type	Q		A		V		r	n	S
	m³/s	cfs	m²	ft²	m/s	ft/s	for feet	for feet	
Flume	9.0	318	5.05	54.35	1.79	5.86	2.57	0.014	0.00085
Trapezoidal	9.0	318	5.06	54.49	1.78	5.83	2.64	0.014	0.00080
Siphon (D = 2.5m)	9.0	318	4.91	52.84	1.83	6.01	2.05	0.014	0.000125

NOTES

All dimensions in meters.
Stations in hundreds of meters.

REFERENCE DRAWINGS

ZRARIYE UNIT
LOCATION MAP OA-10-200
ZAIYE TUNNEL
ALINEMENT, PROFILE, SECTION & GEOLOGY OA-10-360
ZAIYE CANAL - STA.65+00 TO STA 125+40
ALINEMENT, PROFILE, SECTIONS & GEOLOGY OA-10-362

ROUGH PRELIMINARY DRAWING

UNITED STATES
DEPARTMENT OF THE INTERIOR
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LITANI RIVER PROJECT - LEBANON

ZAIYE CANAL

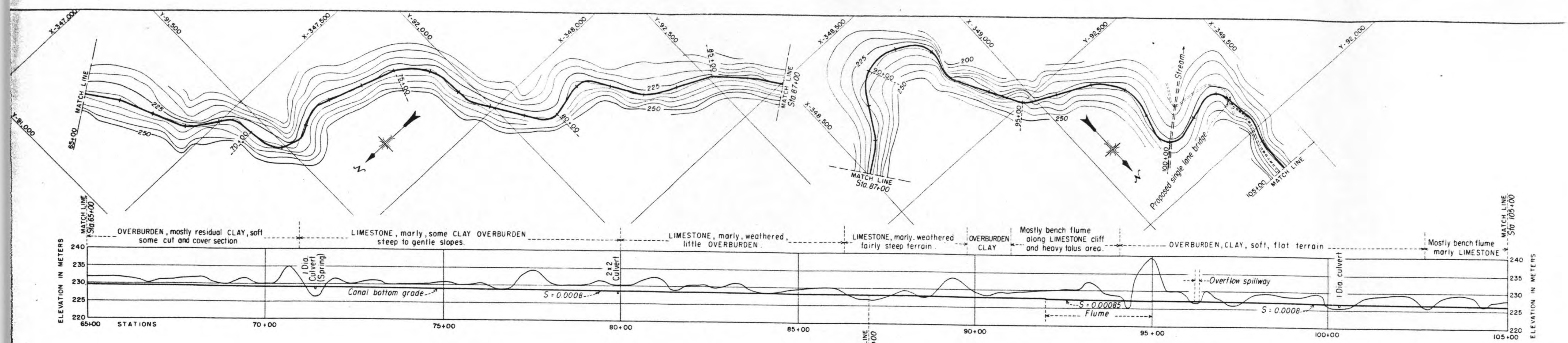
STA. 0+00 TO STA. 65+00
ALINEMENT, PROFILE, SECTIONS AND GEOLOGY

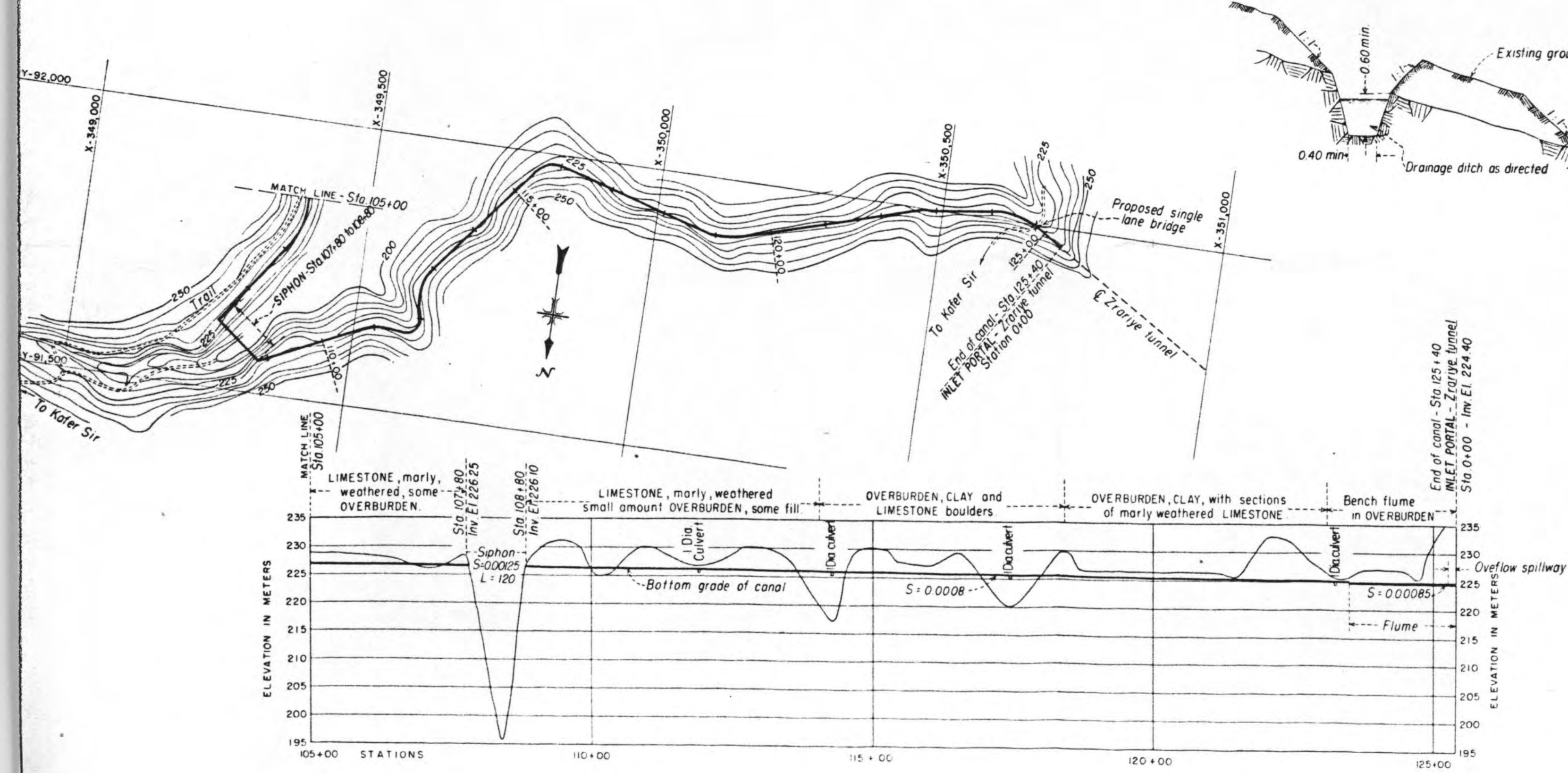
[redacted]

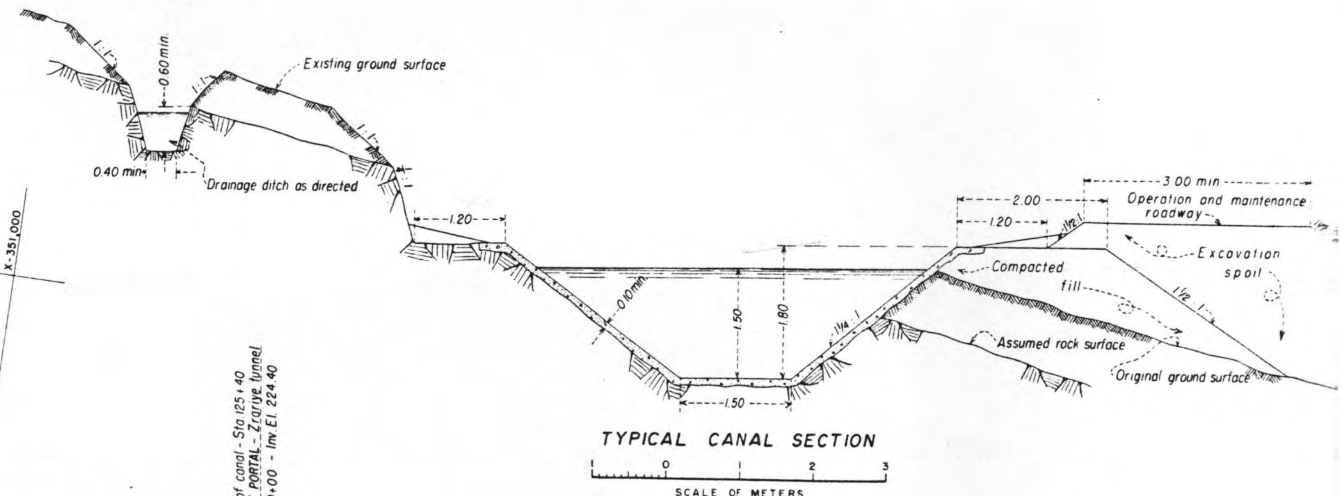
BEIRUT, LEBANON MARCH 10, 1954
SHEET 1 OF 2

OA - 10 - 361

PLATE XXVIII - 3







NOTES

All dimensions in meters.
All stations in hundreds of meters.
For hydraulic properties see sheet 1.

REFERENCE DRAWINGS

ZAIYE CANAL - STA. 0+00 TO STA. 65+00 ALINEMENT, PROFILE, SECTIONS & GEOLOGY	OA-10-361
ZRARIYE TUNNEL & CANAL ALINEMENT, PROFILE, SECTIONS & GEOLOGY	OA-10-363
ZRARIYE UNIT LOCATION MAP	OA-10-200

ROUGH PRELIMINARY DRAWING

UNITED STATES
DEPARTMENT OF THE INTERIOR
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LITANI RIVER PROJECT - LEBANON

ZAIYE CANAL

STA. 65+00 TO STA. 125+40

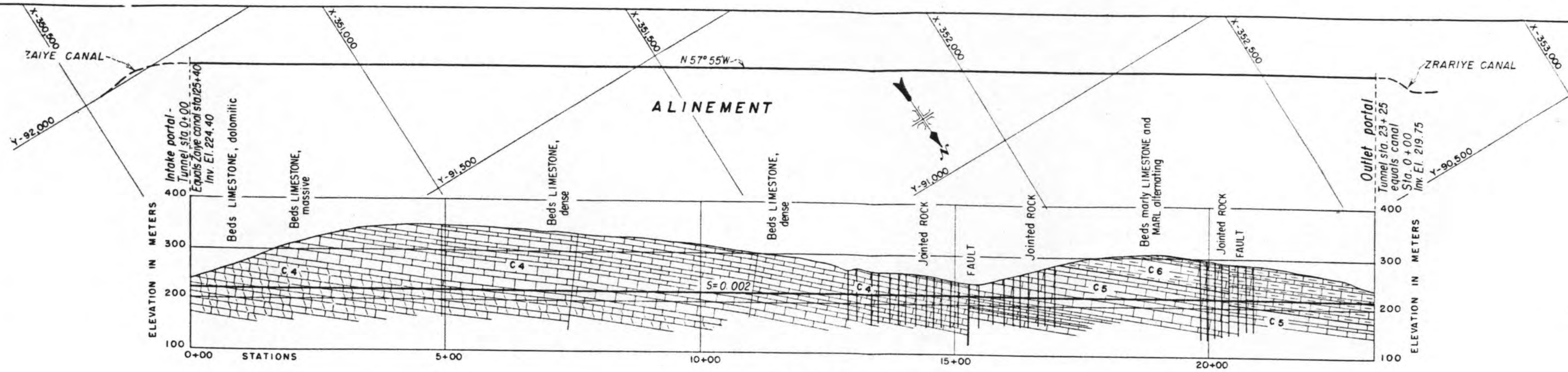
ALINEMENT, PROFILE, SECTIONS AND GEOLOGY

[redacted]

BEIRUT, LEBANON MARCH 10, 1954 SHEET 2 OF 2

OA - 10 - 362

PLATE XXVIII



PROFILE

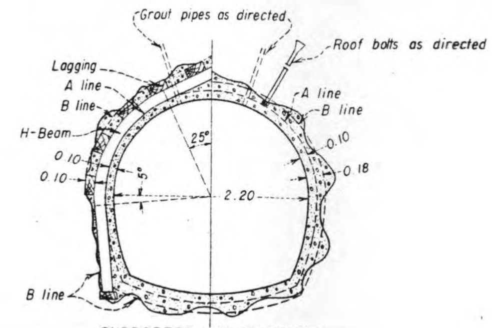
ZRARIYE TUNNEL

SCALE OF METERS

HORSESHOE TYPE TUNNEL HYDRAULIC PROPERTIES *

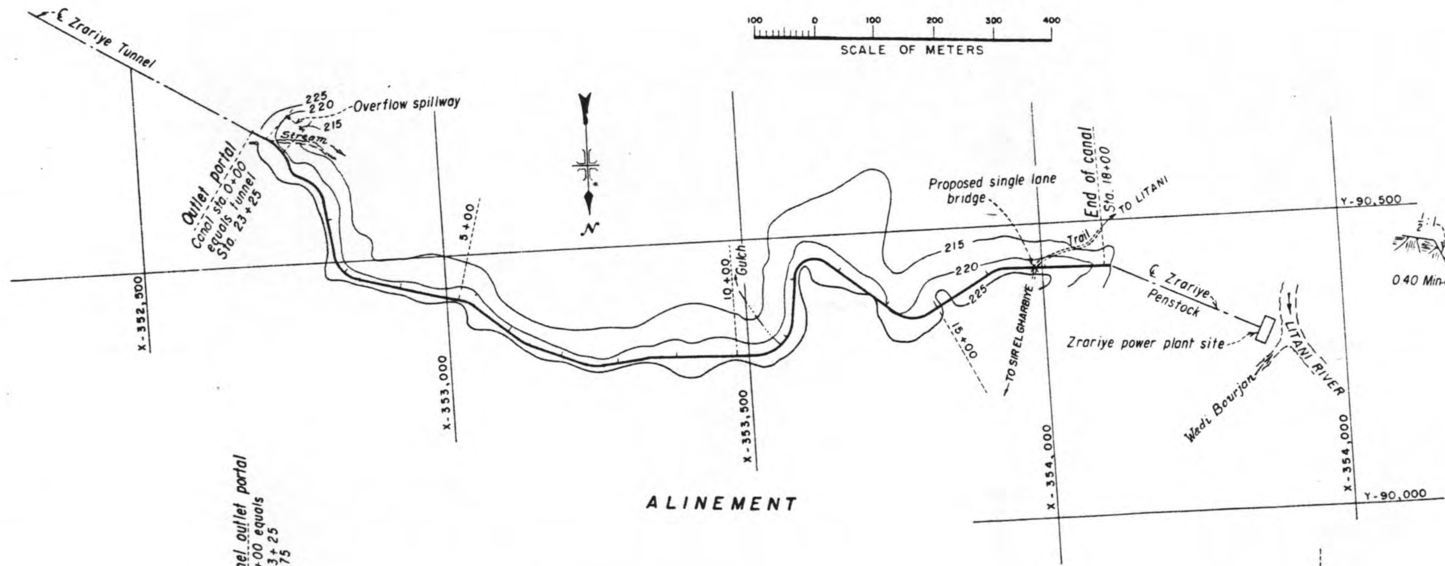
d/D	D		A		V		Q		r	n	s
	m	ft	m^2	ft^2	m/s	ft/s	m^3/s	cfs	for ft	for ft	
0.85	2.2	7.22	3.66	39.35	2.46	8.07	9.0	3.18	2.205	0.014	0.002

* Manning's formula

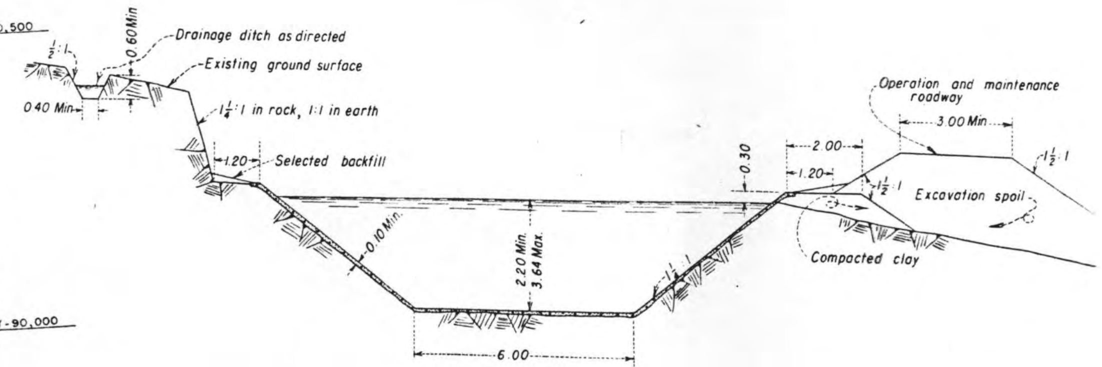


TYPICAL SECTIONS

SCALE OF METERS

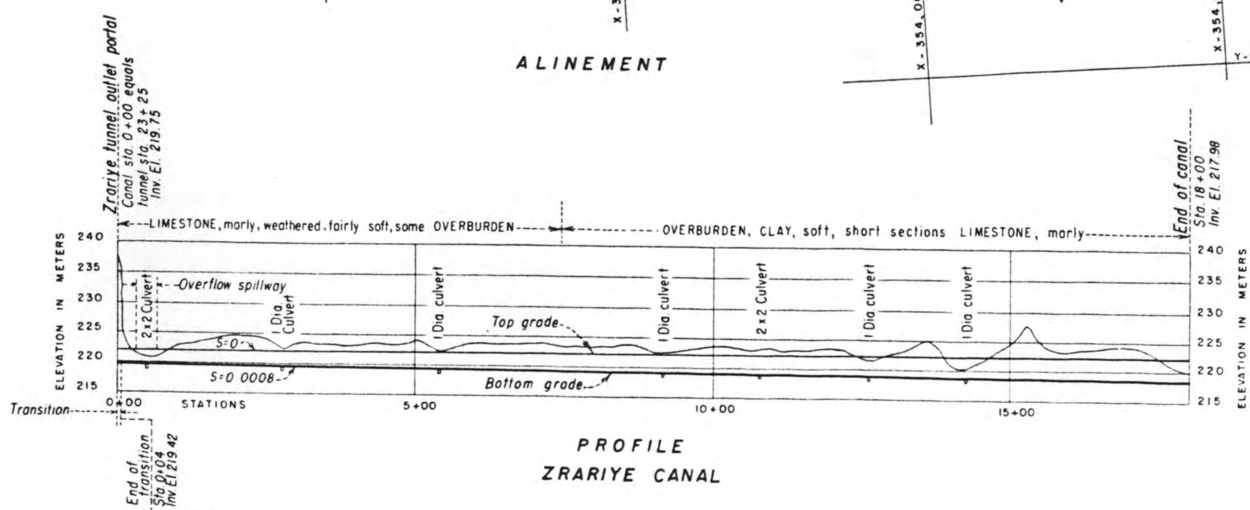


ALINEMENT



TYPICAL SECTION

SCALE OF METERS



PROFILE

ZRARIYE CANAL

NOTES

All dimensions in meters.
Stations in hundreds of meters.
Dimensions and hydraulic properties of canal are variable.

REFERENCE DRAWINGS

ZRARIYE UNIT
LOCATION MAP.......... OA-10-200
ZAIYE CANAL - STA.65+00 TO STA.125+40
ALINEMENT, PROFILE, SECTIONS & GEOLOGY..... OA-10-362
ZRARIYE PENSTOCK
ALINEMENT, PROFILE, SECTION & GEOLOGY..... OA-10-364

ROUGH PRELIMINARY DRAWING

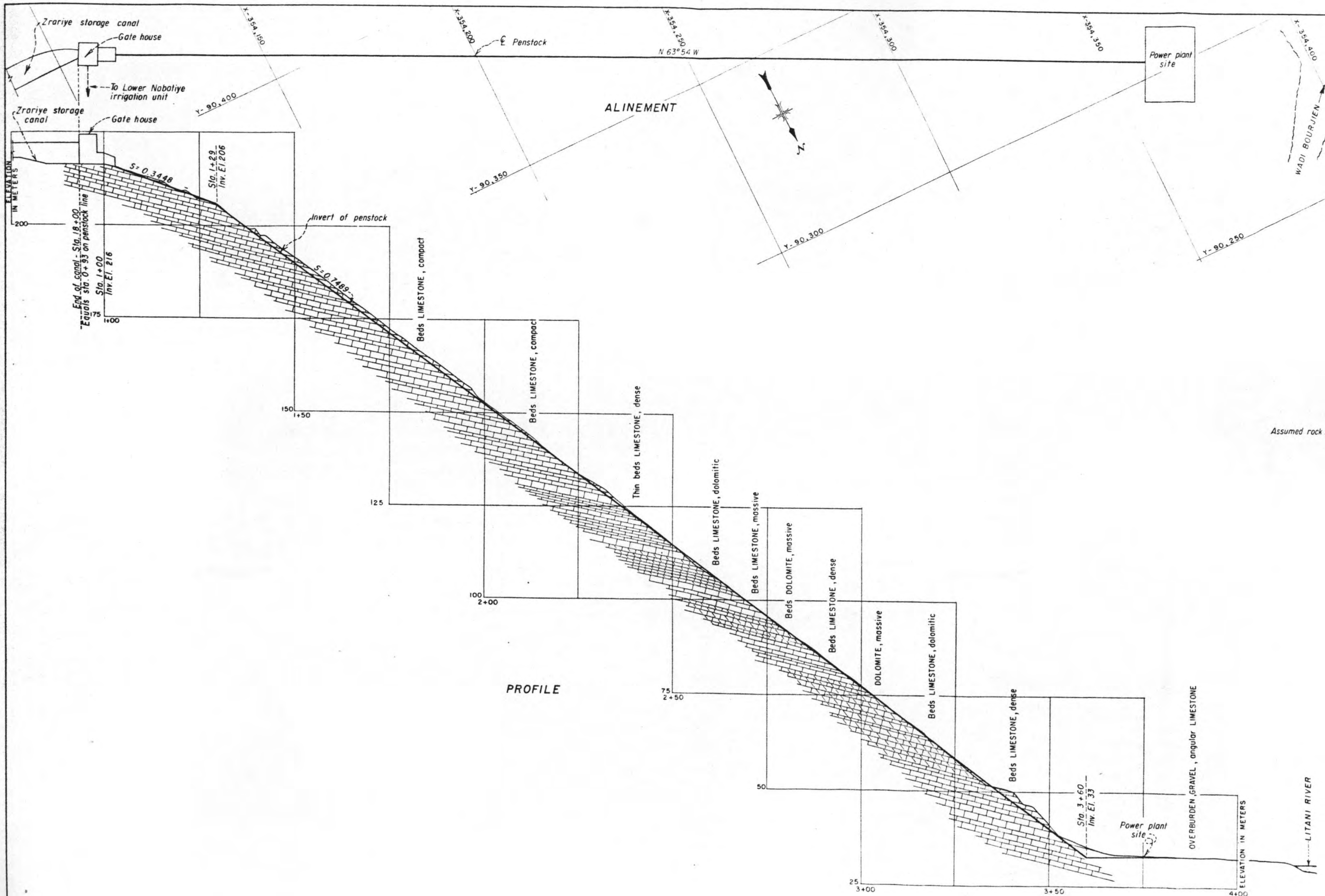
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
LITANI RIVER PROJECT - LEBANON

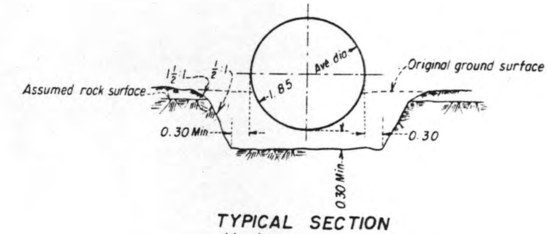
ZRARIYE TUNNEL AND CANAL
ALINEMENT, PROFILE SECTION AND GEOLOGY

[redacted]

BEIRUT, LEBANON | MARCH 25, 1954 | OA-10-363

PLATE XXVIII





TYPICAL SECTION
(Anchors not shown)

SCALE OF METERS 0 1 2 3

NOTES

All dimensions in meters
Stations in hundreds of meters

REFERENCE DRAWINGS

ZRARIYE TUNNEL AND CANAL
ALINEMENT, PROFILE, SECTION AND GEOLOGY......OA-10-363
ZRARIYE UNIT
LOCATION MAP......OA-10-200
LOWER NABATIYE UNIT
GENERAL PLAN......OA-10-239

ROUGH PRELIMINARY DRAWNG

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
LITANI RIVER PROJECT - LEBANON

ZRARIYE PENSTOCK
ALINEMENT, PROFILE
SECTION AND GEOLOGY

[redacted]

BEIRUT, LEBANON | MARCH 20, 1954 | OA-10-364

PLATE XXVII

Hydrologic Data

Conveyance structures for the Zrariye power unit will be used jointly to convey water for the Zrariye Power Plant and for the Lower Nabatiye Irrigation unit. During the irrigation season most of the water required for the Kasmie Irrigation Project will be passed through these structures and used to produce power in the Zrariye Power Plant, prior to being diverted below the plant for irrigation use.

The discharge capacity selected for the tunnels and canals is 9.0 cubic meters per second and is that required to operate the two 6000 kilowatt units selected for the Zrariye Power Plant at full capacity. It was assumed that during the irrigation season there would be insufficient water available to supply the 2.1 cubic meters per second required for the Lower Nabatiye Irrigation Unit and at the same time to operate the Zrariye Power Plant at full load for more than a few hours each day. Therefore the 1800-meter length of the Zrariye Canal was designed with enlarged capacity to provide about 48,000 cubic meters of storage water for use in peaking the Zrariye Power Plant during the irrigation season. The requirement of the Kasmie Irrigation Project is estimated at 5.0 cubic meters per second and since most of this water can be passed through the Zrariye Power Plant during the irrigation season it will be sufficient to operate one of the 6000 kilowatt units at full load most of the time. The other unit would be available for peaking capacity throughout most of each irrigation season.

Consideration of topography and sediment conditions in Khardale Reservoir resulted in the establishment of the invert of the Zaiye Tunnel at elevation 250 meters at this point. A study of the geology and cost of construction resulted in the design of this tunnel as a free-flow tunnel with a length of 7700 meters, and the Zaiye Canal with a length of 12,540 meters. Therefore the Zrariye Tunnel was also designed as a free-flow tunnel with a length of 2330 meters. The canal sections for the Zaiye and the Zrariye Canals have been designed as trapezoidal sections with a capacity of 9.0 cubic meters per second in Zaiye Canal and an additional storage of about 50,000 cubic meters in the Zrariye Canal. The use of this enlarged canal section immediately above the upper end of the Zrariye penstock eliminates the need for a surge tank for the Zrariye Power Plant and also makes it possible to divert the irrigation water for the Lower Nabatiye Irrigation Unit directly from this canal section. This design is much less expensive than the construction of a surge tank for this plant.

Tail water at the Zrariye Power Plant has been assumed to be at elevation 33 meters and the head on this plant used in the operation studied was assumed to vary from elevation 183 to 186 meters.

Project Features

Zaiye Tunnel. This tunnel will divert water from the Khardale Reservoir, with an invert elevation of 250 meters, to the Zaiye Canal, with an invert elevation of 235 meters. Between the inlet and outlet portals of the tunnel the Litani River flows in a very deep narrow canyon. Near the inlet portal the river flows in a southerly direction while at the outlet portal it flows in a westerly direction, thus the tunnel roughly forms the hypotenuse of a 45 degree triangle, with the Litani River forming the other two legs. It will be a concrete-lined, horseshoe-shaped, free-flow tunnel, 7700 meters long, with a finished diameter of 2.2 meters. The tunnel will not be subjected to any internal hydrostatic pressure and no excessive external pressures are anticipated. Normal concrete lining will be used and no steel reinforcement will be required, except for a short distance at the portals. It is estimated that no more than 20 percent of the tunnel will require light steel supports and that the remainder of the tunnel will be unsupported except through blocky sections where roof bolts will be sufficient. Since the tunnel will be excavated through marly limestone very little water is expected to be encountered, and permanent drains will not be required. The small flows of water that enter the tunnel during construction can be handled in temporary drains, constructed along the sides of the tunnel. The water encountered at the fault zones should be permitted to enter the tunnel by placing weep holes through the lining, to eliminate hydrostatic pressures behind the lining.

No shafts or adits can be used to facilitate construction since the center section of the tunnel has a minimum overburden of 250 meters and does not pass any valleys or deep draws. The hard limestone anticipated in the first 2000 meters of the tunnel is suitable for use in the rock-fill sections of the Khardale Dam or for concrete aggregate, but the marly limestone encountered in the rest of the tunnel is unsuitable for either, and should be dumped near the portal.

A simple trashrack structure will be used since the trash burden of the Khardale Reservoir will be very light. Emergency and regulating gates, to be controlled from the Zrariye Power Plant, will be installed in a gate chamber located approximately 60 meters from the tunnel portal. Access to the chamber will be by means of a vertical shaft. A stilling basin will be required in front of the regulating gate. Electric measuring devices will be installed, so that the amount of water entering the tunnel may be known at the Zrariye Power Plant at all times. The alinement, profile and section of the Zaiye Tunnel is shown on Plate XXVIII-2.

The principal items of work are estimated as 43, 200 cubic meters of tunnel excavation; 12, 380 cubic meters of concrete for tunnel lining; 22, 500 kilograms of steel reinforcement; 240, 000 kilograms of steel supports; and 22, 250 meters of roof bolts.

Zaiye Canal. This canal will be 12, 600 meters long (including a 100 meter siphon) and will form a connecting link in the conduit from the Khardale Reservoir to the Zrariye Power Plant. It will be located along the right bank of the Litani River and will have a section of reinforced concrete bench flume at each end where the side slopes are very steep and a short section on the top of a cliff. The flume will have a total length of 3, 050 meters, a width of 2. 7 meters and a water depth of 1. 9 meters. A steel reinforced concrete siphon about 100 meters long will be required for a canyon crossing. Construction of several additional siphons across deep draws should be considered before final plans are prepared. The remaining 9, 400 meters of the canal will have a concrete lined trapizoidal section, with a bottom width of 1. 5 meters, a water depth of 1. 5 meters and side slopes of 1-1/4 to 1. The canal will have a designed capacity of 9. 0 cubic meters per second with a velocity of about 1. 8 meters per second through both the flume and the trapezoidal section. The alinement, profile and sections of the Zaiye Canal are shown on Plates XXVIII-3 and 4.

The canal is estimated to require 250, 000 cubic meters of excavation, mostly marly limestone; 4, 580 cubic meters of reinforced concrete for the bench flume; 457,500 kilograms of steel reinforcement for the bench flume; and 7, 520 cubic meters of concrete for canal lining. The following structures will be required; one siphon 100 meters long; one double lane highway bridge; 7 single lane bridges; 8 reinforced concrete box culverts, averaging about 2 x 2 meters; 10 circular concrete culverts (about 1 meter diameter); 3 overflow spillways; and probably several small turnouts to supply irrigation water for some small, flat areas of land along the canal.

Zrariye Tunnel. The Zrariye Tunnel will be required due to the very steep side slopes and cliffs that lie between the Zaiye Canal and the Zrariye Canal. The tunnel will be 2330 meters long and will have an inlet invert elevation of 224 meters and an outlet invert elevation of 220 meters. The tunnel will be the same size and type as the Zaiye Tunnel and will have the same hydraulic properties and general design features. The alinement, profile and sections of the Zrariye Tunnel and Canal are shown on Plate XXVIII-5.

It is estimated that not over 15 percent of the tunnel will require light steel rib supports and that the geologic formation for the rest of the tunnel will not require any supports, except some roof bolts where the rock encountered is blocky. The tunnel will require 13, 000 cubic meters of excavation; 3, 620 cubic meters of concrete for lining; 52, 000 kilograms of steel supports; and 17, 300 kilograms of reinforcement steel. No adits are planned due to the short length of the tunnel, but if desired a short adit may be placed in a deep draw at about the mid-point of the tunnel. The dense limestone excavated from the inlet portal would be suitable for concrete aggregate if properly screened, but the marly limestone, that will constitute the bulk of the material to be excavated from the outlet portal, is not suitable for concrete aggregate and should be dumped near the outlet portal.

Zrariye Canal. This canal will be 1800 meters long and will convey the water from the Zrariye Tunnel to the Zrariye penstock and to the Lower Nabatiye Irrigation Canal. The canal will be constructed to form a regulating reservoir for the irrigation canal and a forebay for the power plant by constructing the lower end of it several times larger than required. Studies were made for an alternate plan which would utilize a small reservoir site near the top of a very deep draw about 2500 meters from the power plant site. The principle objections to this alternative were; (a) the drainage area of 160 hectares, above the reservoir, is very erosive and the reservoir would soon silt up unless a large canal was constructed around it to by-pass flood water; (b) about 400 meters of pressure conduit over a steep rocky slope would be required between the reservoir and the power plant penstock; (c) the penstock would be 100 meters longer and would be over a less uniform slope; and (d) a tunnel 1300 meters long would be required for diversion to the Lower Nabatiye Irrigation Canal. The advantages were that; (a) the Zrariye Tunnel would be 300 meters shorter; and (b) a small canal, (capacity of 2.1 cubic meters per second) 1000 meters long, a 350 meters siphon and a 200 meters tunnel would replace the 1300 meters of tunnel for diversion to the Lower Nabatiye Irrigation Canal.

Estimates have been based on an enlarged canal with a storage capacity of approximately 50,000 cubic meters. This capacity would be sufficient to operate one unit at full capacity for a period of time equal to the time required for water released from the Khardale Reservoir to reach the penstock. During the irrigation season one unit will operate at full capacity continuously to supply the irrigation demands of the Kasmie Project and during the rainy non-irrigation season both units will operate as base units with emergency peaking only. It is believed in view of these facts that the 50,000 cubic meters storage capacity is adequate.

Consideration should be given to the possibilities of using a bituminous or some other non-concrete lining, or eliminating the lining altogether, from a large part of the canal, since it is located almost entirely through a marly limestone formation. The principal items of work are estimated to be 100,000 cubic meters of excavation, mostly marly limestone; 2880 cubic meters of concrete lining; and the following structures; bifurcation structure with appurtenant control gates; 2 large box culverts; 5 small circular culverts; and one single-lane bridge.

Zrariye Penstock. This penstock will supply water to the Zrariye Power Plant from the bifurcation structure located at the end of the Zrariye Canal and at the beginning of the Lower Nabatiye Irrigation Canal. It is proposed to supply the two turbines from one steel penstock with a diameter of about 1.85 meters. The overall cost might possibly be reduced, and transportation of the steel penstock facilitated, by enlarging the upper section and reducing the lower sections of the line. The penstock will have an invert elevation of about 216 meters at the bifurcation works and about 33 meters at the power plant. It will have a slope length of 335 meters. Preliminary designs and estimates were based on a capacity of 9.0 cubic meters per second and a velocity of 3.35 meters per second, with a pipe diameter of 1.85 meters. The alinement, profile and sections of the Zrariye penstock are shown on Plate XXVIII-6.

Installation of the steel penstock will require 500 cubic meters of rock excavation; the furnishing and placing of 247,000 kilograms of steel pipe; and of 155 cubic meters of reinforced concrete. The steel pipe will have to be installed by skidding the sections along the excavated trench to their final location due to the steep, rocky slope, which averages about 36 degrees with the horizontal. Construction of access roads along the penstock alinement to facilitate the placing of the sections is believed to be too costly to warrent further consideration at this time.

Zrariye Power Plant. This power plant will be located on the right bank of a sharp curve on the Litani River, approximately 17 kilometers above the mouth of the river. The power plant will have an installed capacity of 12, 000 kilowatts and will consist of two 6000 kilowatt units, operating at a head of about 183 meters. Vertical reaction turbines, of the Francis-type, with vertical generators are believed to be the most suitable.

Zrariye Switchyard. The switchyard will require two power transformer bays; two 69 kilovolt bays; and one 35 kilovolt bay. The switchyard will be located on a fairly flat piece of ground at the top of the penstock due to lack of space in the narrow canyon adjacent to the power plant. A narrow, steep, switch-back road may be constructed for jeep travel from the power plant to the switchyard, to enable the power plant operators to reach the switchyard. An alternate location is available for the switchyard. It is below the power plant and across an intermittent stream that has formed an alluvial fan at its junction with the Litani River. This site would require a wall to protect the switchyard from a possible shift of the stream in the alluvial fan. This location would also be more difficult to reach with the transmission lines than the site located at the top of the penstock.

Although the mountain side behind the power plant is very steep, no special protection is believed necessary for this structure provided all loose rocks and boulders are removed from the mountainside before starting construction of the power plant. No unusual conditions or special problems are anticipated. Future studies may indicate the need for an afterbay to regulate the releases of irrigation water for the Kasmie Project, if so a low dam could be constructed just above the present Kasmie Diversion Dam. The operation and maintenance road, powerhouse, and appurtenant structures therefore, should be so constructed that no major changes or modifications will be necessary if such a forebay is required.

Construction Operations

No unusual conditions are anticipated during construction of the Zaiye Tunnel. Local contractors are believed to be capable and qualified to construct the tunnel provided sufficient time for completion is allowed. This time is estimated at 4 years. If modern methods are used the tunnel could be completed in 2-1/2 to 3 years.

Access Roads. The inlet portal of the Zaiye Tunnel can be reached by constructing a short (400 meters) access road from the Nabatiye-Marjayoun Road. To reach the outlet portal an access road 3 km. long, from the Nabatiye-Ghandouriye road will be required. This road may be constructed along a trail which follows the Litani River for 2.5 kilometers, thence 1/2 kilometer of switchbacks to the portal, or the road may be built along the Zaiye Canal line which would eliminate the steep switchbacks to the portal, but would interfer with construction of first section of the canal.

Access to the beginning and end of the Zaiye Canal will be from the access roads constructed for the outlet portal of the Zaiye Tunnel and for the inlet portal of the Zrariye Tunnel. Access will also be available from existing roads and trails that cross the canal alinement. The estimated cost of the access roads has been pro-rated among the several features using these facilities.

Access to the inlet portal of the Zrariye Tunnel will require improving 2 kilometers of narrow roadway between Kakiet Jisr and Kafr Sir and 2 kilometers beyond Kafr Sir and constructing 2 kilometers of new roadway between Kafr Sir and the inlet portal. Access to the outlet portal will require improving 3 kilometers of roadway between Kafr Sir and Sir el Gharbiye and constructing 0.5 kilometer of new roadway from Sir el Gharbiye to the outlet portal. These access roads would also be used to facilitate construction of the adjoining canals.

Access to the beginning of the Zrariye Canal will be over the same route as described for the outlet of the Zrariye Tunnel. The bifurcation structure and the lower

Zrariye Penstock and Power Plant Site on Litani River
Looking Upstream

end of the canal can be reached by constructing a one kilometer road from the access road built to the Zrariye Tunnel outlet portal from Sir el Gharbiye. These roads will also be used for operation and maintenance after construction. The estimated cost of these roads has been pro-rated among the other adjacent features.

The top of the Zrariye penstock can be reached from the road to be constructed to the bifurcation structure and the bottom of the penstock by the road to be constructed to the power plant.

Access to the power plant will be from the Tyr-Saida Highway along a 15-kilometer road that roughly parallels the Litani River. The first 10 kilometers of road, which is built along the Kasmie Canal will require extensive improvements, the remaining 5 kilometers will have to be constructed along narrow strips of land between the Litani River and the Kasmie Canal or along the steep canyon walls. This road should be constructed so that it will be adequate for future use as an operation and maintenance road to the power house. Several bridges and culverts will be required across small draws.

SECTION XXIX

AWALI POWER UNIT

Location and Description

The Awali Power Unit will be a single purpose unit constructed on the Awali (Bisri) River about 4 kilometers below the Bisri Dam and about 19 kilometers from the mouth of this river. It will have an installed capacity of 36,000 kilowatts, and will discharge into the Bisri River just above the Joun Diversion Dam. This installation will consist of three 12,000 kilowatt units direct-connected to Francis-type reaction turbines. Water from Bisri Reservoir will be conveyed to this power plant by a pressure tunnel 3.1 meters in diameter and about 3300 meters long, and by a steel penstock 2.9 meters in diameter and about 360 meters long. This pressure tunnel will carry both power water and water for the irrigation of the Saida-Beirut Irrigation Unit. Hydro power will be produced by the irrigation water during the season before it is diverted for irrigation use. The location and principal features of this unit are shown on Plates XX-1 and XXIX-1.

Geology

Awali Tunnel - The tunnel will be excavated in the Cretaceous (C4) marly limestones for its entire length. A few small faults cross the tunnel as well as several jointed areas. Near the inlet portal the rocks are dipping about 12° in the direction of the tunnel. They flatten out to about 7° at Station 4+00 and are nearly flat for the rest of the distance to the outlet portal. Short sections may require support at the fault zones and at the jointed areas. Since the bedding is relatively flat it is not anticipated that much of the tunnel will be in blocky ground, but, where encountered, roof bolts can be used. The geologic section along this tunnel route is shown on Plate XXIX-2.

Awali Penstock - This penstock passes over the Cretaceous C4 and C3 series of marly limestones and dolomites. These rocks are dipping gently into the mountain above elevation 260 meters. Below 260 meters, the bedding is parallel to the slope of the mountain to the powerhouse area.

Very little overburden is found along the penstock route except near the lower end. Weathering extends about one meter into the C3 limestones and marls located below elevation 310 meters and below this weathered zone the rocks soon become compact and sufficiently strong to carry the penstock anchors. Above elevation 310 meters the C4 series of marly limestones and dolomites show little weathering and are competent to carry the penstock anchors with little excavation.

Test pits 1 and 2 were dug near the lower end of the penstock as shown on the penstock profile. These holes are near the final profile selected and are shown on Plate XXIX-3 as projected holes.

Awali Power Plant - Six diamond drill holes were drilled at the Awali Power Plant site. Drill holes 1 to 5 were drilled in the power plant area. Drill hole 6 was drilled about 33 meters south of the power plant area, towards the Awali River, in order to determine that the limestone ledge continued beyond the power plant area.

The foundation for the Awali Power Plant is located in the Cretaceous (C3) marly limestones. From 6 to 8 meters of overburden was found in drill holes 1, 2, and 3 and consisted mainly of dolomite boulders in sand, gravel and clay. This overburden was carried downstream from the large landslide located a short distance upstream. Bedrock consisted of marly limestone with thin-bedded zones. The core recovery was very good and these limestones were very compact and dense, with some broken zones.

Drill holes 4 and 5 were drilled about 16 meters north of drill holes 1, 2 and 3 just above the toe of the slope. From 2 to 3 meters of overburden was found in these two holes. Surface examination shows this overburden to be slowly creeping down-dip from the slopes above during the winter months when it is saturated with water. Bedrock beneath the overburden consisted of marly limestones which were compact and dense.

In order to gain head at this power plant a tailrace about 10 meters deep leading out from the power plant is proposed. At a depth of 10 meters the footings should be in bedrock sufficiently strong to carry this structure.

Hydrologic Data

The conveyance structures for the Awali Power Plant consist of the Awali Tunnel; the Awali surge tank; and the Awali penstock. All of these are required exclusively for the Awali Power Unit although any water diverted from the Bisri Reservoir for use in the Awali Power Plant may also be diverted by the Joun Diversion Dam, immediately downstream from this plant, for irrigation use on the Saida-Beirut Irrigation Unit.

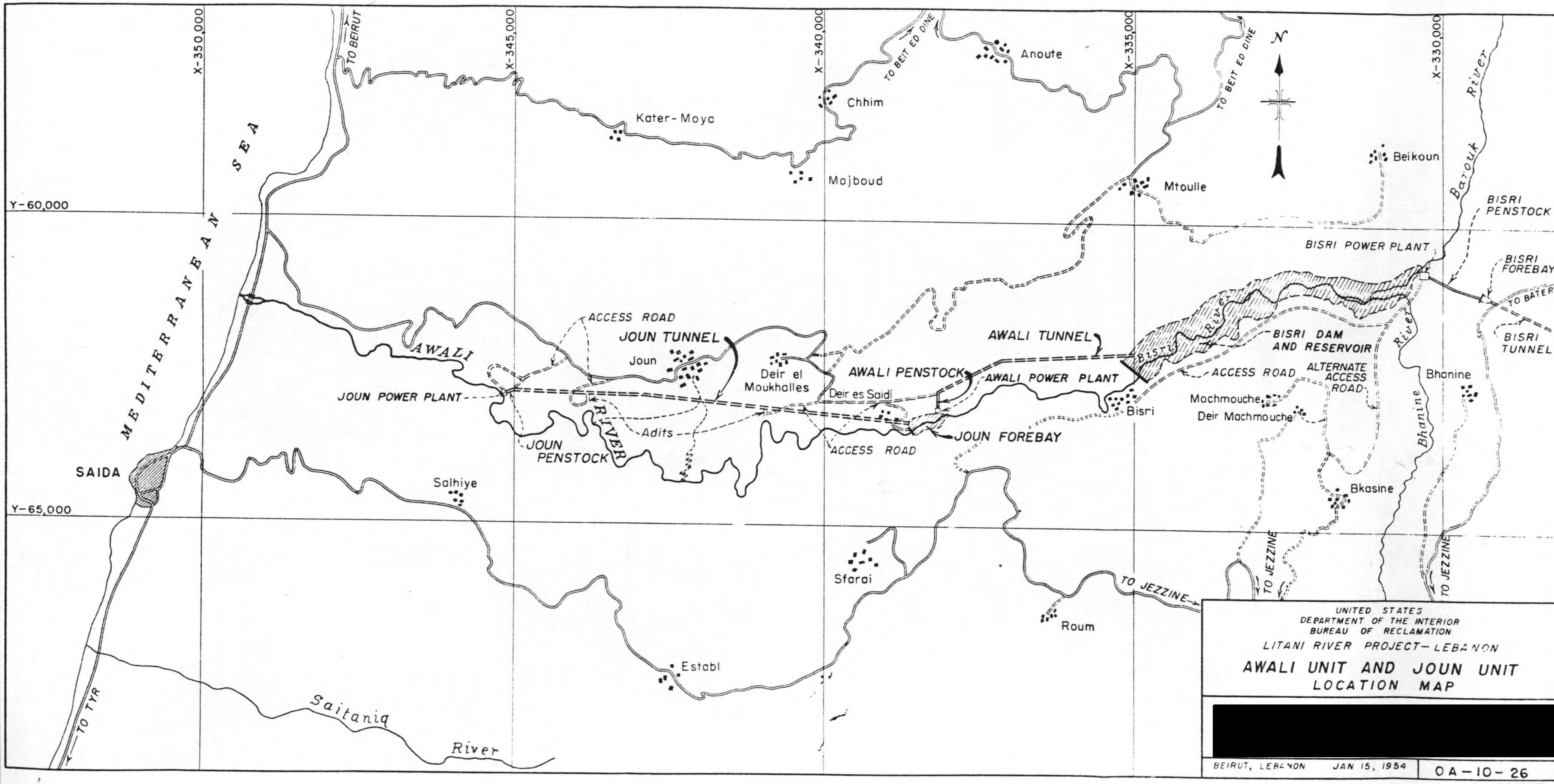
The Awali Tunnel has been designed as a pressure tunnel in order to use the additional head provided by storage in the Bisri Reservoir. It has a diameter of 3.1 meters and a length of about 3300 meters. It is designed to supply the full requirements for the three 12,000-kilowatt units to be installed in the Awali Power Plant. These units will operate with a tail-water elevation that is expected to vary from elevation 212 meters to 217 meters and their headwater elevation, as controlled by the Bisri Reservoir, will vary from elevation 422.5 meters to 430 meters. It is proposed that Francis-type reaction turbines, direct-connected to 12,000-kilowatt generators, be installed in this power plant.

Project Features

Awali Tunnel - This tunnel will supply water from the Bisri Reservoir with an invert elevation of 417 meters to the Awali penstock with an invert elevation of 395 meters. The tunnel will be 3294 meters long with a 24°05' angle at a point 2149 meters from the inlet portal. It will be a concrete lined, circular, pressure tunnel with a finished diameter of 3.1 meters. As no swelling, or squeezing ground, or excessive external hydrostatic pressure is anticipated, normal thickness of concrete lining will be used. It is estimated that 20% of the tunnel length will require light steel rib supports and that the geologic formation for the remainder of the tunnel is suitable for installation of roof bolt supports. As the tunnel has sufficient overburden to withstand the internal hydrostatic pressure and adequate grouting behind the lining is planned, no steel reinforcement will be used except in the sections adjacent to the portals and in sections through highly faulted or crushed zones. It is estimated that these sections will comprise 25% of the tunnel length. The alinement, profile and sections of the tunnel are shown on Plate XXIX-2.

Adits and shafts could be used to facilitate the construction of the tunnel but they are not believed to be economically justified due to the short length between portals. The excavated material from the inlet portal will be suitable for use in the riprap zones of the Bisri Dam which is only a short distance from the portal. The excavated material from the outlet portal should be dumped near the tunnel entrance. The excavated material will be suitable for use as concrete aggregate if properly screened.

A simple trashrack structure will be sufficient since the trash burden in the Bisri Reservoir will be light. An emergency gate which can be operated from the Awali Power Plant will be connected to this structure or installed in a short shaft. A surge tank will be constructed in a shaft 65 meters back of the outlet portal to reduce the water hammer in the penstock and to facilitate the over-all operation of the power plant. The tank will be steel reinforced, concrete-lined, with a finished diameter of approximately 10 meters and a height of 47 meters. Since only a single penstock and a relatively short tunnel are planned for Awali Power Plant, it is believed that the emergency gate at the inlet of the tunnel will be sufficient to control the flow to the powerhouse in event of an emergency; therefore, no control gate will be needed at the outlet portal.



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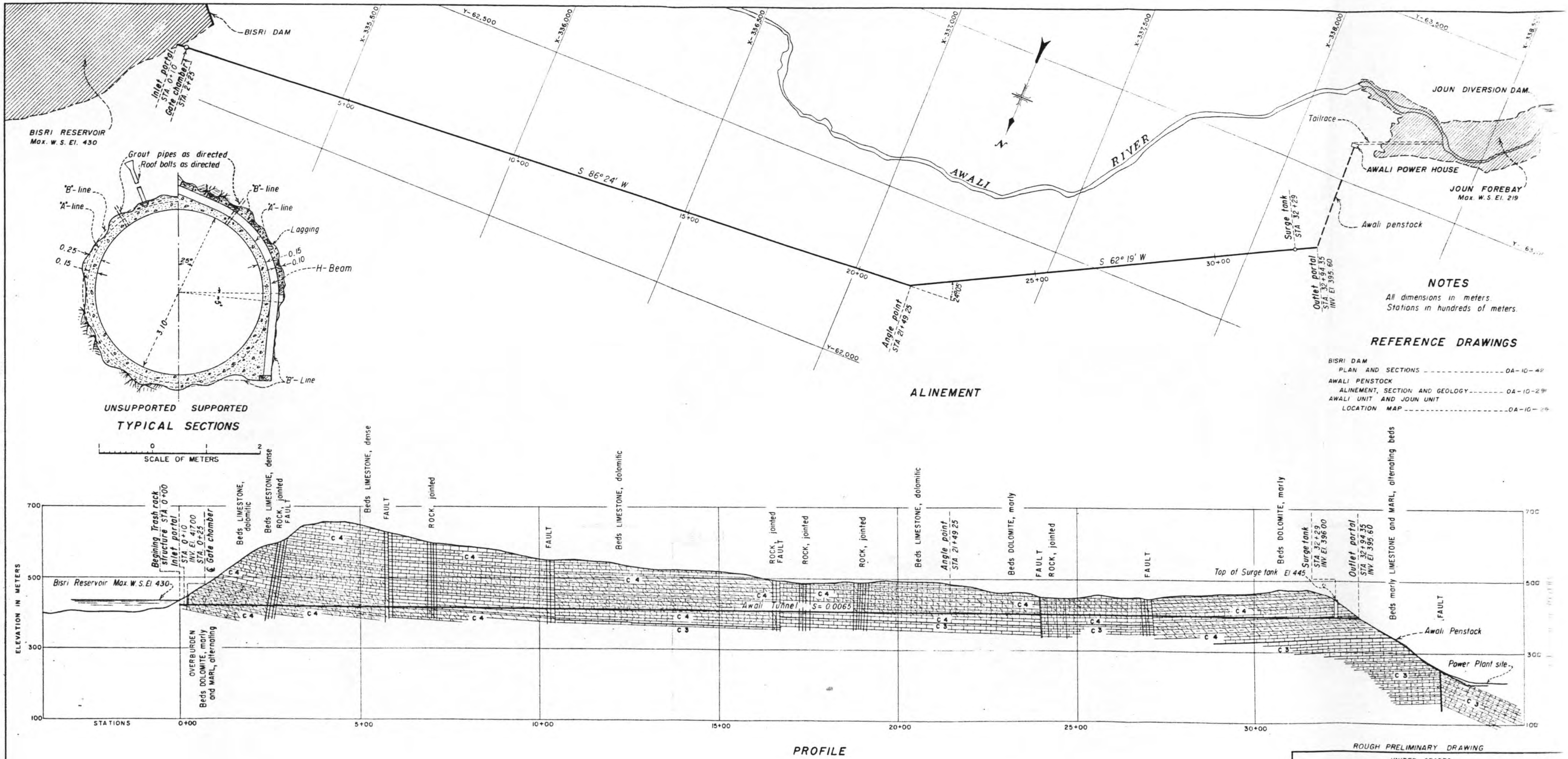
AWALI UNIT AND JOUN UNIT
LOCATION MAP

[redacted]

BEIRUT, LEBANON JAN 15, 1954 OA-10-26

PLATE XXIX-1



PRESSURE TUNNEL HYDRAULIC PROPERTIES *

D		A		V		Q		r	n	s
m	ft	m^2	ft^2	m/s	ft/s	m^3/s	cfs	for ft	for ft	
3.10	10.17	7.55	81.24	3.07	10.06	23.16	817.6	2.542	0.014	0.0026

* Mannings Formula

ROUGH PRELIMINARY DRAWING

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LITANI RIVER PROJECT – LEBANON

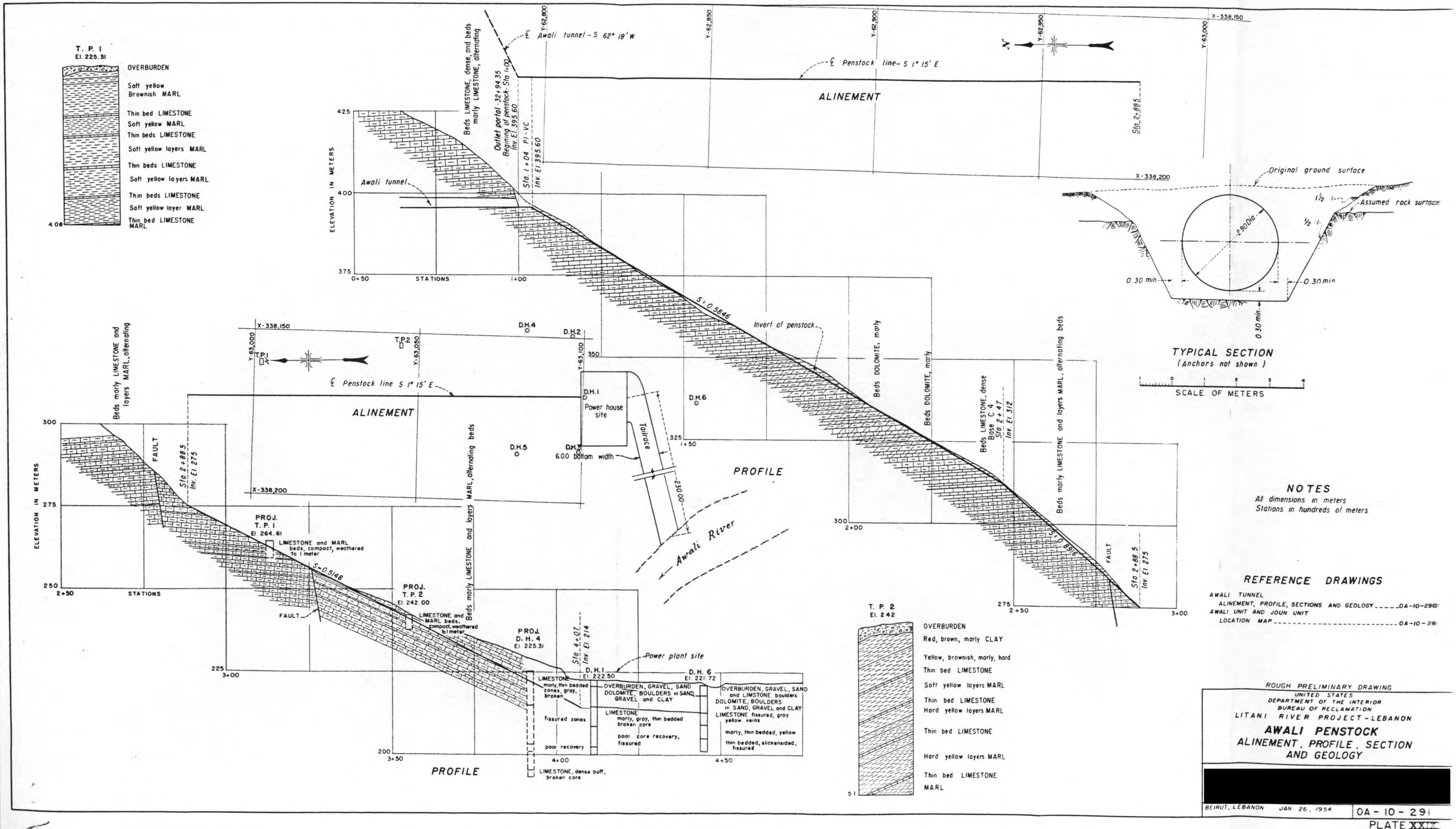
AWALI TUNNEL

ALINEMENT, PROFILE, SECTIONS, AND GEOLOGY

[redacted]

BEIRUT, LEBANON JAN 26, 1954 OA-10-29

PLATE XXIX


T. P. 1
El. 225.31
OVERBURDEN
Soft yellow Brownish MARL
Thin bed LIMESTONE
Soft yellow MARL
Thin beds LIMESTONE
Soft yellow layers MARL
Thin beds LIMESTONE
Soft yellow layers MARL
Thin beds LIMESTONE
Soft yellow layer MARL
Thin bed LIMESTONE MARL
4.08
℄ Awali tunnel - S 62° 19' W
℄ Penstock line - S 1° 15' E
ALINEMENT
Beds LIMESTONE, dense, and beds marly LIMESTONE, alternating
Outlet portal - 32+94.35 Begining of penstock - Sta 1+00 Inv. El. 395.60
Sta. 1+04 PI-VC Inv. El 395.60
Sta. 2+88.5
X-338,150
X-338,200
Y-62,800
Y-62,850
Y-62,900
Y-62,950
Y-63,000
Awali tunnel
ELEVATION IN METERS
STATIONS
S=0.5846
Invert of penstock
Beds DOLOMITE, marly
Beds DOLOMITE, marly
Beds LIMESTONE, dense Base C 4 Sta. 2+47 Inv. El. 312
Beds marly LIMESTONE and layers MARL, alternating beds
FAULT
Sta. 2+88.5 Inv. El. 275
PROFILE
Original ground surface
Assumed rock surface
2.90 Dia.
0.30 min
0.30 min
TYPICAL SECTION
(Anchors not shown)
SCALE OF METERS
NOTES
All dimensions in meters
Stations in hundreds of meters
REFERENCE DRAWINGS
AWALI TUNNEL
ALINEMENT, PROFILE, SECTIONS AND GEOLOGY OA-10-290
AWALI UNIT AND JOUN UNIT
LOCATION MAP OA-10-25
Beds marly LIMESTONE and layers MARL, alternating
T.P.1
T.P.2
D.H.1
D.H.2
D.H.4
D.H.5
D.H.6
Power house site
Tailrace
6.00 bottom width
230.00
Awali River
℄ Penstock line S 1° 15' E
ALINEMENT
Beds marly LIMESTONE and layers MARL, alternating beds
FAULT
Sta. 2+88.5 Inv. El. 275
PROJ. T. P. 1 El. 264.61
LIMESTONE and MARL beds, compact, weathered to 1 meter
S=0.5148
FAULT
PROJ. T. P. 2 El. 242.00
LIMESTONE and MARL beds, compact, weathered to 1 meter
PROJ. D. H. 4 El. 225.31
Sta. 4+07 Inv. El. 214
Power plant site
D. H. 1 El. 222.50
D. H. 6 El. 221.72
LIMESTONE marly, thin bedded zones, gray, broken
fissured zones
poor recovery
LIMESTONE, dense buff, broken core
OVERBURDEN, GRAVEL, SAND DOLOMITE, BOULDERS in SAND, GRAVEL and CLAY
LIMESTONE marly, gray, thin bedded broken core
poor core recovery, fissured
OVERBURDEN, GRAVEL, SAND and LIMSTONE boulders DOLOMITE, BOULDERS in SAND, GRAVEL and CLAY
LIMESTONE fissured, gray yellow veins
marly, thin bedded, yellow
thin bedded, slickensided, fissured
PROFILE
T. P. 2
El. 242
OVERBURDEN
Red, brown, marly CLAY
Yellow, brownish, marly, hard
Thin bed LIMESTONE
Soft yellow layers MARL
Thin bed LIMESTONE
Hard yellow layers MARL
Thin bed LIMESTONE
Hard yellow layers MARL
Thin bed LIMESTONE
MARL
ROUGH PRELIMINARY DRAWING
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LITANI RIVER PROJECT - LEBANON
AWALI PENSTOCK
ALINEMENT, PROFILE, SECTION AND GEOLOGY
[redacted]
BEIRUT, LEBANON JAN 26, 1954
OA-10-291
PLATE XXIX

Awali Penstock - This penstock will convey water from the Awali Tunnel at elevation 396 meters to the Awali Power Plant, situated on the right bank of the Awali River above the Joun forebay at elevation 214 meters. Whenever it is necessary to shut down the power plant for maintenance or repairs the water from the Bisri Reservoir can be released directly into the river. This would be done by raising the radial gates over the Bisri Dam spillway, whose crest elevation is 422.5, the minimum water surface elevation of the reservoir. Since this will in no way affect the operation of other power and irrigation units only one penstock is proposed to serve the three turbines. The steel penstock would have a slope length of 360 meters and a diameter of 2.9 meters, which would require a velocity of 3.5 meters per second for a capacity of 23.2 cubic meter per second. The overall cost could possibly be reduced and transportation of the penstock sections facilitated by using a slightly larger diameter for the upper sections and a smaller diameter for the lower. The alinement, profile and sections of the penstock are shown on Plate XXIX-3.

The penstock could be installed by skidding the sections from the top of the penstock line down the excavated trench to their final location, since the penstock line is located on a rather uniform, steep, rocky slope averaging about 30 degrees with the horizontal. Short construction roads could be built from the tunnel and power plant access road to facilitate the placing of the sections. Installation of the penstock will require about 2000 cubic meters of excavation, (mostly rock) the placing of 911,000 kilograms of steel pipe, 50,000 kilograms of reinforcement steel bars, and 310 cubic meters of concrete.

Awali Power Plant. The power plant will be located on the right bank of the Awali River 500 meters upstream from the proposed Joun Diversion Dam. It would have an installed capacity of 36,000 kilowatts and would operate on a gross head varying from 218 to 205.5 meters and will consist of three 12,000-kilowatt units. Vertical reaction turbines of the Francis-type with vertical generators appear to be the most suitable.

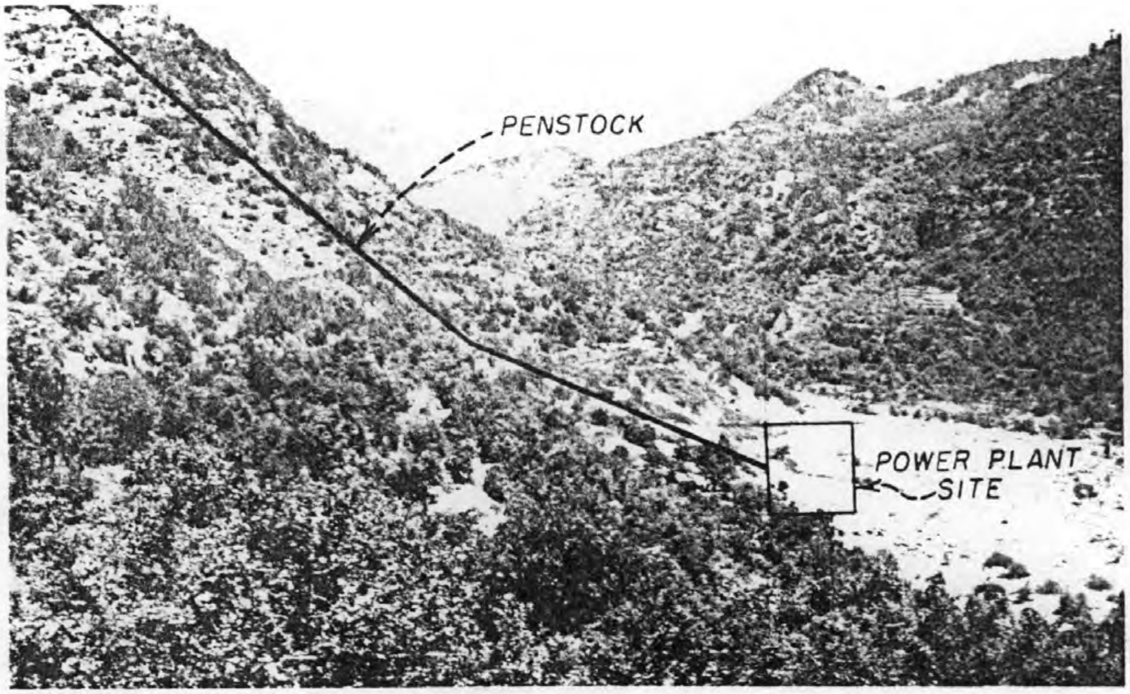
No special protection from rolling or sliding rocks will be required since the hillside behind the power plant has already been terraced. The power plant will be situated about 10 meters above the minimum water surface elevation of the proposed Joun Diversion Dam, therefore, a fairly long, deep tailrace will be required in order to utilize the maximum head of the power plant prior to the construction of Joun Diversion Dam and when this forebay is operated at the minimum water surface elevation. No other special problems are anticipated. Consideration should be given to the possibility of eliminating the long tailrace by moving the power plant downstream closer to the proposed location of Joun Diversion Dam, if future studies indicate that it would be more economical to construct a regulating reservoir at the outlet end of the Joun Tunnel instead of the costly Joun Diversion Dam.

Awali Switchyard. This switchyard will be located adjacent to the power plant on either the up or the downstream side, the final location to be selected after further study and prior to final design. It will be a double bus-bar, single breaker system with one tie breaker, and will consist of 3-11/69 kv. power transformer bays; 1 station service bay; 3-69 kv-line, or bus-tie breaker bays; and 2-69 kv. future line breaker bays. The 69 kv. lines will connect with the Bisri and Joun Power Plants and the two future lines with the Beirut area. Carrier current equipment and lightning arresters will be provided on the 69 kv. lines.

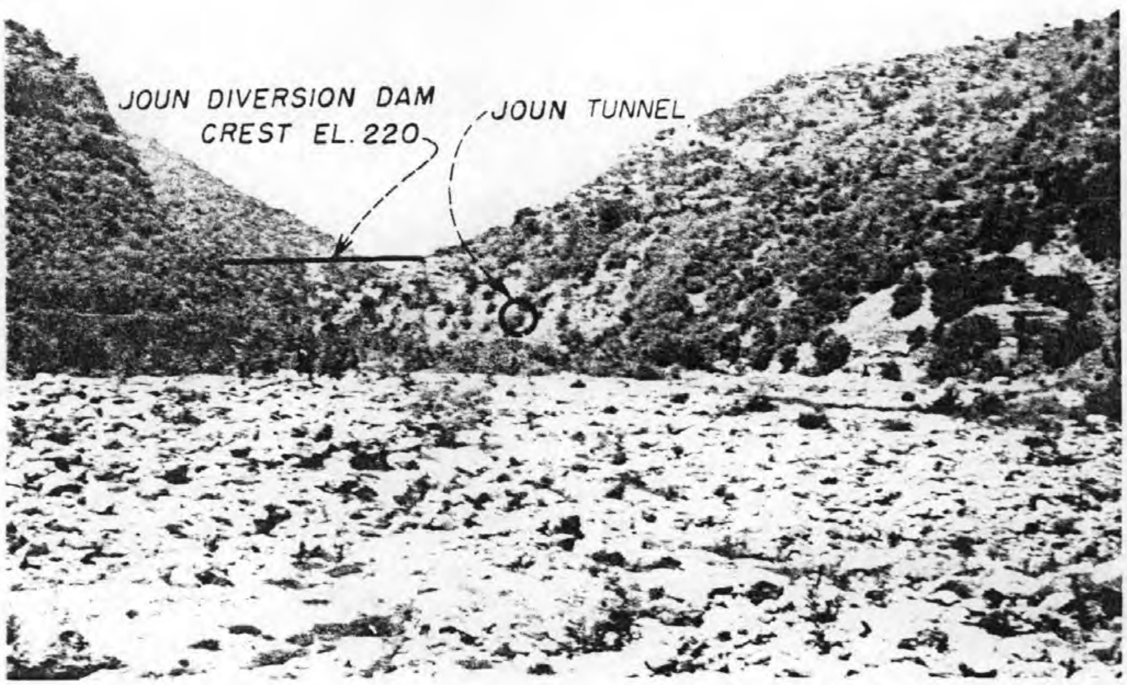
Construction Operations

No unusual conditions are anticipated during construction of the Awali Tunnel. The tunnel is expected to be fairly dry from May to December but during the rainy winter season considerable water may infiltrate the tunnel, especially through the faulted zones. This water should be handled in temporary drains constructed along the sides of the tunnel. The section of the tunnel excavated from the inlet portal will require pumping to remove the water from the heading. No permanent drains will be required. The principal items of work are, 34,000 cubic meters of tunnel excavation, 10,000 cubic meters of concrete for tunnel lining, 160,000 kilograms of steel reinforcement, and 143,000 kilograms of steel supports.

Access Roads. A short access road, with a bridge over the Awali River, will be constructed from the Bisri village road to the inlet portal of Awali Tunnel. The outlet portal will require an access road from Deir el Moukhalles. An access road, which will also be used for operation and maintenance of the Awali Power Plant and for construction of the Joun Diversion Dam can be constructed from the access road to be built to the outlet portal of the Awali Tunnel. The road will be located over steep, rocky terrain and will require several switchbacks in order to keep the maximum grade below 10 percent. No major bridges or structures will be required. The roads from Beirut to the beginning of the access road are adequate for transporting the heavy power plant equipment.



Awali Penstock and Power Plant Site
Looking Upstream from Joun Diversion Dam Site



Joun Diversion Dam Looking Downstream from
Awali Power Plant Site

SECTION XXX

JOUN POWER UNIT

Location and Description

The Joun Power Unit will be a multiple purpose unit constructed on the Awali (Bisri) River about 6 kilometers from its mouth. It will have an installed capacity of 24,000 kilowatts, consisting of two 12,000 kilowatt units direct-connected to Francis-type reaction turbines. Water for this unit and for the Saida-Beirut Irrigation Unit will be diverted from the Bisri River by the Joun Diversion Dam, located immediately below the Awali Power Plant. The water will be carried through a tunnel 2.8 meters in diameter and about 6400 meters long to the head of the Joun penstock. There the water for the Saida-Beirut Irrigation Unit will be released into the main canal of that unit and the power water for Joun plant released into a steel penstock, for conveyance to this plant. This penstock is about 2.6 meters in diameter and about 220 meters long. Water discharged from this power unit will flow into the Awali River and be discharged into the Mediterranean Sea near Saida. The locations of this unit and its principal features are shown on Plates XX-1 and XXIX-1.

Geology

Joun Tunnel - The geologic profile for this tunnel is shown on Plate XXX-1. The inlet portal is located in the Cretaceous (C3) marly limestones. These limestones continue at tunnel grade to about Station 6+00. Near Station 2+50 the dip of the beds increases from 3 to about 12 degrees with the dip of the beds along the tunnel being very close to the direction of the tunnel.

From Station 6+00 to Station 49+00 the tunnel will pass through the Cretaceous (C4) beds which are mainly dolomitic at the beginning of this section with alternating beds of dolomite and limestone near the end of this section. Some jointing and minor faulting is found along this section. The dip of the beds varies from about 14^{o} through the first part of the section to about 7^{o} near the end of the section.

From Station 49+00 to the outlet portal the Cretaceous (C5) limestones occur at tunnel level. Several small faults and changes of dip occur in this section with the dip of the rocks varying from 5^{o} to 11^{o}.

Since the rocks are dipping, some blocky ground will be encountered along bedding planes and roof bolts may be necessary at some of these points. Some light steel support will be required near jointed areas and at small faults. It is estimated that 15 percent of the tunnel will require support.

Joun Penstock and Power Plant - The geology for this penstock is shown on Plate XXX-2. The slope for this penstock is fairly steep. Very little overburden exists along the entire route. The upper portion of the penstock will rest on the Cretaceous (C5) limestones above elevation 100 and on the Cretaceous (C4) dolomitic limestones below elevation 100. These rocks are dipping from 5^{o} to 7^{o} towards the river. No faulting or jointing was found along this slope. The limestones are not deeply weathered and are competent to carry the penstock anchors with little excavation.

The river flows along the toe of the penstock slope in the powerhouse area. The river appears to be flowing on bedrock near the toe of the slope and this area was not explored by test pits or drill holes.

Hydrologic Data

Conveyance structures for the Joun Power Plant consist of the Joun Tunnel; the Joun surge tank; and the Joun penstock. The Joun Tunnel is used to convey water from the Joun Diversion Dam to the Main Canal of the Saida-Beirut Irrigation Unit in addition to its conveyance of water for the Joun Power Plant and should be considered a multiple-purpose structure.

The Joun Tunnel has been designed as a pressure tunnel with a diameter of 2.8 meters and a length of 6400 meters. It is designed to carry a maximum flow of 18.8 cubic meters per second that is required to operate the two 12,000-kilowatt units to be installed in the Joun Power Plant, at full peaking capacity. The assumption has been made that during the irrigation season there will be insufficient water to supply the irrigation requirements of the Saida-Beirut Irrigation Unit and to peak this plant at full capacity. Therefore, this tunnel has been designed to furnish complete supply for full power operation outside the irrigation season and to provide water for only partial power operation during this season. A usable storage capacity of about 172,000 cubic meters has been provided in Joun diversion pond behind the Joun Diversion Dam for short-period peaking of this plant during the irrigation season.

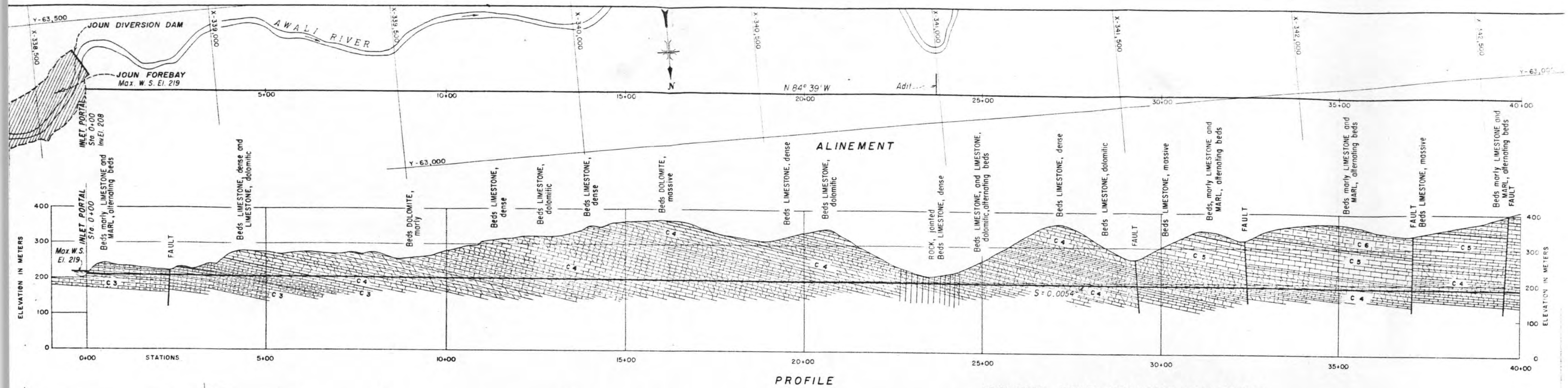
The average tail water at Joun Power Plant has been assumed at elevation 33 meters, and the Joun diversion pond has been assumed to fluctuate from a normal water surface at elevation 217 meters to a minimum water surface at elevation 212 meters. These fluctuations result in a change in gross head from 184 meters to 179 meters during peaking operations. It is proposed that two Francis-type reaction turbines, direct-connected to 12,000-kilowatt generators be installed in this plant.

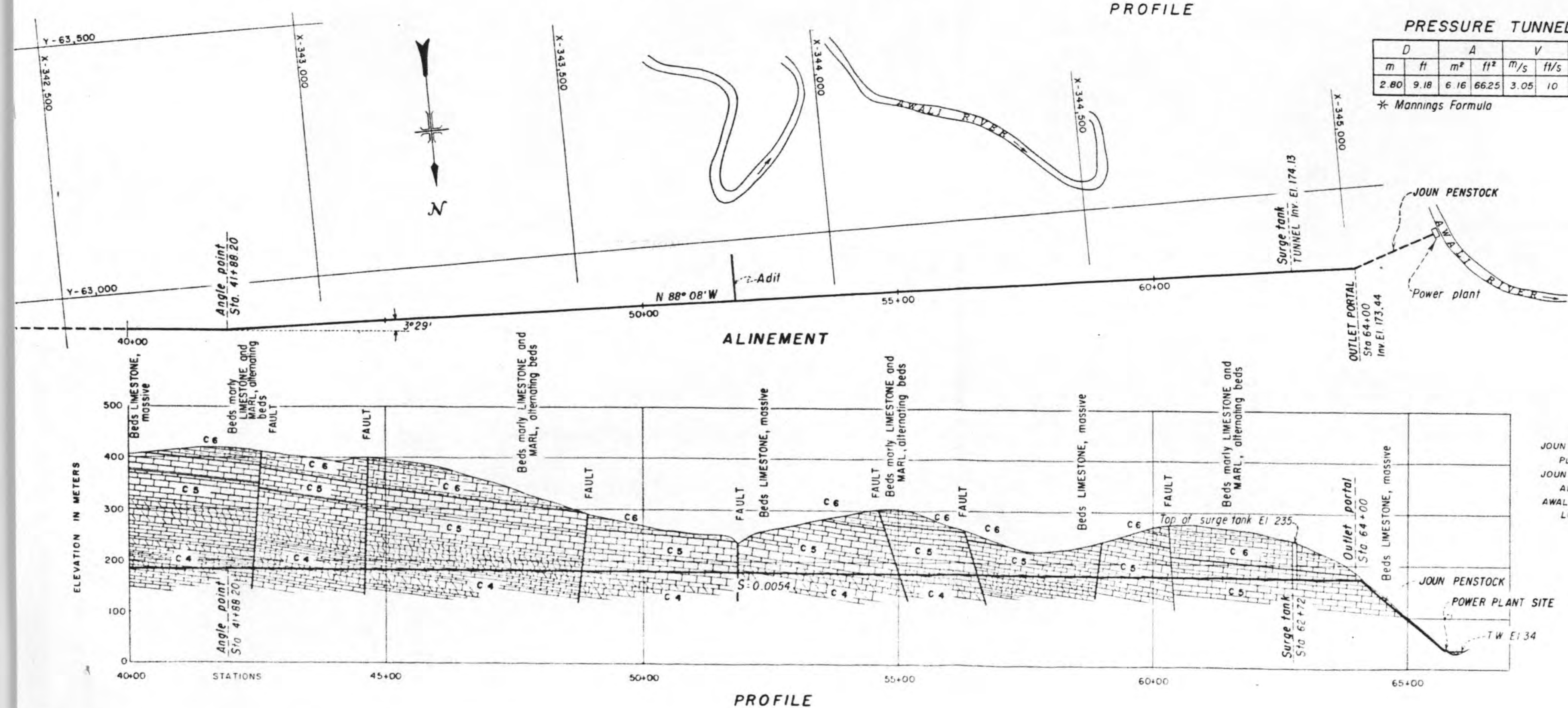
The maximum diversion required for the Saida-Beirut Irrigation Unit would be 2.2 cubic meters per second which would be carried through the Joun Tunnel and diverted at the Joun surge tank to the main canal of the irrigation unit.

<u>Project Features</u>

<u>Joun Tunnel</u> - This tunnel will have an invert elevation of 208 meters at the inlet portal at the Joun Diversion Dam and an outlet invert elevation of 173 meters at the bifurcation structure. The tunnel will be located on the right side of the Awali River and will roughly parallel the river. It will have two adits located about one-third the tunnel length from the outlet and inlet portals to facilitate construction. It will be a concrete lined, circular, pressure tunnel with a finished diameter of 2.8 meters. It is estimated that 15 percent of the tunnel will require light steel rib supports and that the geologic formation for the remainder of the tunnel is suitable for installation of roof bolt supports. Normal thickness of concrete lining will be used as no swelling or squeezing ground or excessive external hydrostatic pressure is anticipated. No steel reinforcement will be required except; in sections where the overburden is insufficient to withstand the internal hydrostatic pressure, in sections adjacent to the portals, and in sections through highly faulted, or crushed zones. The tunnel alinement, profile and sections are shown on Plate XXX-1.

A simple box-type trashrack structure will be used since the trash will be extremely light. This structure will have sufficient openings so as to allow for a very slow entrance velocity and to avoid the forming of a vortex, so that the water in the forebay may be drawn down to less than a meter above the top of the structure. The emergency gate (usually placed at the inlet of a pressure tunnel for a power plant) will be omitted from the inlet of the Joun Tunnel, as it is planned to install an emergency gate at the beginning of the penstock at the bifurcation structure, which can be operated from the power plant in event of a failure in the penstock or power plant equipment. In the event of a tunnel failure water can be prevented from entering the tunnel by discharging the water in the Awali River through outlet works of the Joun Diversion Dam and by placing stop logs in the trashrack structure for the tunnel. A surge tank will be constructed in a shaft located 128 meters from the outlet portal. The shaft will be lined with reinforced concrete and will have a finished diameter of 10 meters and a height of approximately 57 meters.





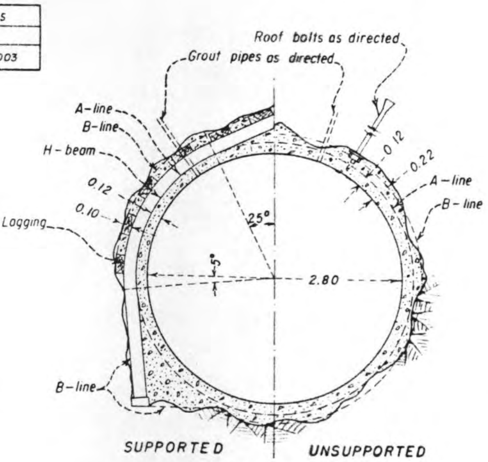
PRESSURE TUNNEL HYDRAULIC PROPERTIES*

D		A		V		Q		r	n	s
m	ft	m²	ft²	m/s	ft/s	m³/s	cfs	for ft	for ft	
2.80	9.18	6.16	66.25	3.05	10	18.75	662	2.30	0.014	0.003

* Mannings Formula

NOTES

All dimensions in meters
Stations in hundreds of meters.



TYPICAL SECTION

SCALE OF METERS

REFERENCE DRAWINGS

JOUN DIVERSION DAM
PLAN, PROFILE AND SECTIONS OA-10-275
JOUN PENSTOCK
ALINEMENT, PROFILE, SECTION AND GEOLOGY OA-10-293
AWALI UNIT AND JOUN UNIT
LOCATION MAP OA-10-26

ROUGH PRELIMINARY DRAWING

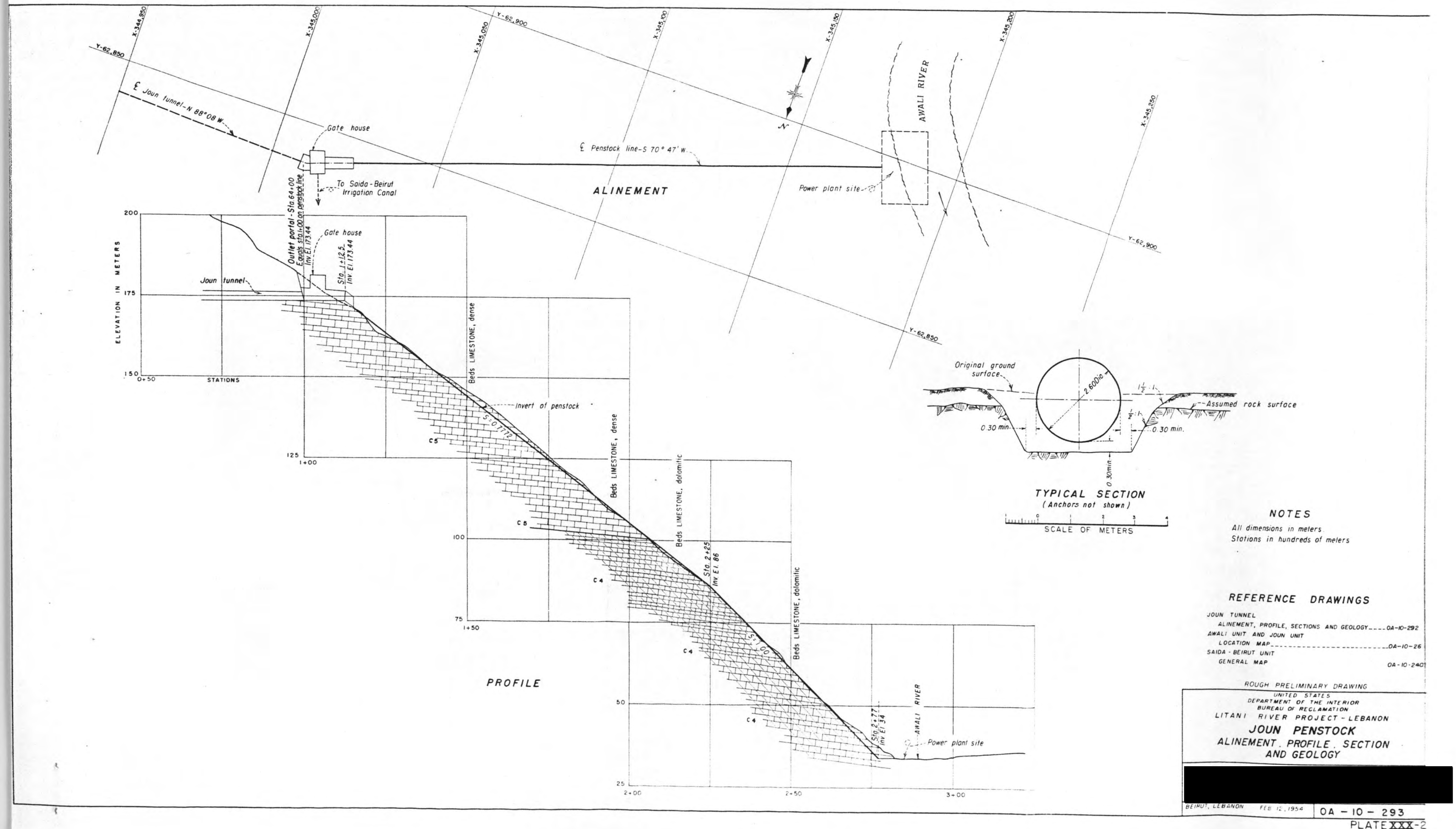
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
LITANI RIVER PROJECT — LEBANON

JOUN TUNNEL
ALINEMENT, PROFILE, SECTIONS AND GEOLOGY

[redacted]

BEIRUT, LEBANON | FEB 12, 1954 | OA-10-292

PLATE ~~XXX~~-



ALINEMENT
Y-62,850
Y-62,900
X-344,950
X-345,000
X-345,050
X-345,100
X-345,150
X-345,200
X-345,250
℄ Joun tunnel-N 88° 08' W.
Gate house
℄ Penstock line-S 70° 47' W.
To Saida-Beirut Irrigation Canal
N
AWALI RIVER
Power plant site
PROFILE
ELEVATION IN METERS
STATIONS
Joun tunnel
Outlet portal-Sta 64+00 Equals sta 1+00 on penstock line Inv. El. 173.44
Gate house
Sta. 1+12.5 Inv. El. 173.44
Beds LIMESTONE, dense
Invert of penstock
S=0.7772
C5
Beds LIMESTONE, dense
Beds LIMESTONE, dolomitic
Sta. 2+25 Inv El. 86
Beds LIMESTONE, dolomitic
S=1.00
C4
Sta. 2+77 Inv El. 34
AWALI RIVER
Power plant site
200
175
150
125
100
75
50
25
0+50
1+00
1+50
2+00
2+50
3+00
TYPICAL SECTION
(Anchors not shown)
Original ground surface
2.600 dia.
1½:1
½:1
Assumed rock surface
0.30 min.
0.30 min.
0.30 min.
SCALE OF METERS
NOTES
All dimensions in meters.
Stations in hundreds of meters
REFERENCE DRAWINGS
JOUN TUNNEL
ALINEMENT, PROFILE, SECTIONS AND GEOLOGY OA-10-292
AWALI UNIT AND JOUN UNIT
LOCATION MAP OA-10-26
SAIDA - BEIRUT UNIT
GENERAL MAP OA-10-240
ROUGH PRELIMINARY DRAWING
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
LITANI RIVER PROJECT - LEBANON
JOUN PENSTOCK
ALINEMENT, PROFILE, SECTION AND GEOLOGY
[redacted]
BEIRUT, LEBANON
FEB. 12, 1954
OA-10-293
PLATE XXX-2

The tunnel is expected to be relatively dry during most of the year with some of the faulted zones flowing fairly heavy during the rainy winter months. No permanent drains are proposed but the water which filters into the tunnel during construction should be handled in temporary drains, placed along the sides of the tunnel. The sections of the tunnel which slope down toward the heading will require pumping to remove the water from the headings. No unusual conditions or problems are anticipated during construction. The principal items of work are estimated to be; 55, 000 cubic meters of tunnel excavation, 15, 000 cubic meters of concrete for tunnel lining, 200, 000 kilograms of steel reinforcement, and 200, 000 kilograms of steel tunnel supports. Excavated material from the tunnel may be used for concrete lining aggregate if properly screened. Excavated material not required for this purpose may be dumped adjacent to the tunnel portals and adits.

Joun Penstock. This penstock will supply water to the Joun Power Plant from a bifurcation structure located at the outlet end of Joun Tunnel. The bifurcation structure will contain the necessary gates and valves to control the water delivered to the penstock and to the Saida-Beirut Irrigation Canal. Only one penstock is planned to supply the two turbines at the Joun Power Plant. The penstock will have an invert elevation at the bifurcation structure of approximately 173 meters and at the power plant of 34 meters. It will have a slope length of 220 meters and a diameter about 2. 6 meters, which will require a velocity of 3. 53 meters per second to provide a capacity of 18. 75 cubic meters per second. The upper section of the penstock should be constructed slightly larger than the lower section to facilitate transportation and to effect a possible reduction in the overall cost. A pressure pipe line will be required between the bifurcation structure and the beginning of the Saida-Beirut Irrigation Canal at about elevation 188 meters.

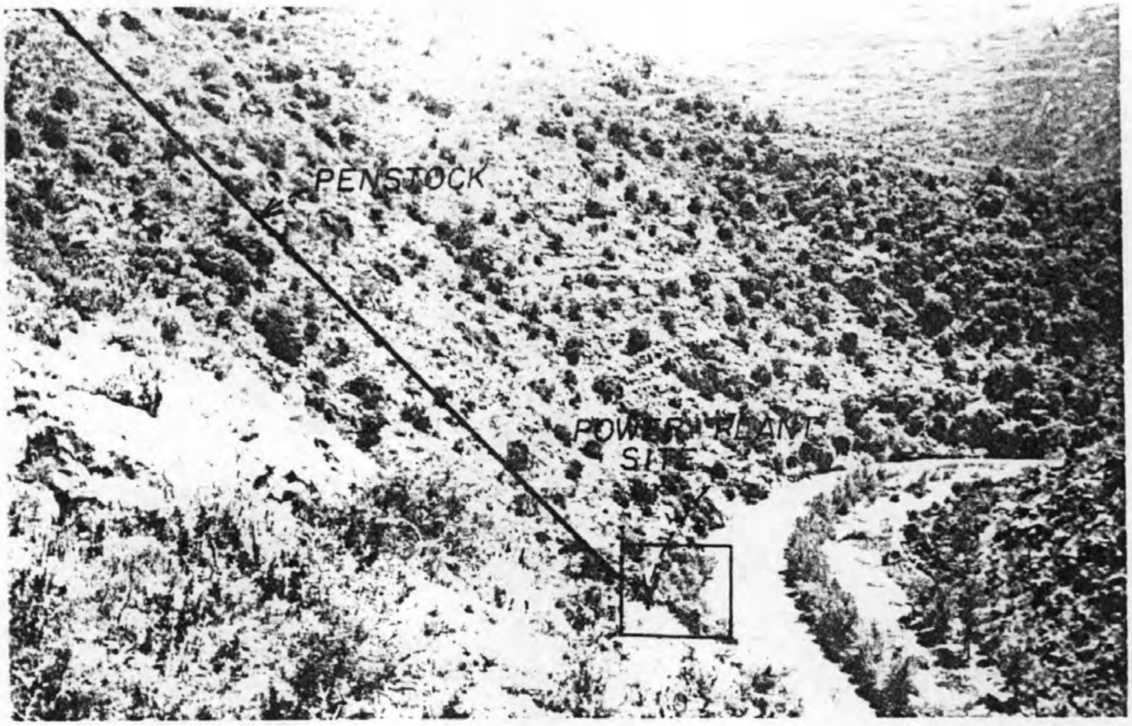
Installation of the steel penstock will require the excavation of 600 cubic meters of rock, the placing of 40, 000 kilograms of steel reinforcement bars and 250 cubic meters of concrete. Short construction roads may be built from the access road for the power plant and tunnel portal to facilitate the placing of the steel pipe, or the pipe sections may be skidded down the excavated trench which will have a steep rocky slope, averaging 39 degrees with the horizontal. The penstock alinement, profile and sections are shown on Plate XXX-2.

Joun Power Plant - This power plant will be located on the right bank of the Awali River just below a sharp curve in the river, approximately 6 kilometers above the mouth. Because the river at this point is very narrow and is flowing along the right bank it will be necessary to notch out a bench on which to place the powerhouse. An alternative would be to direct the river through the low flat land adjacent to the left bank and construct the power plant partly over the present river channel. The power plant will have an installed capacity of 24, 000 kilowatts and will consist of two 12, 000 kilowatt units operating at a gross head varying from 184 to 179 meters. Vertical reaction turbines of the Francis-type with vertical generators are believed to be the most suitable. A heavy retaining wall should be constructed on the steep slope back of the powerhouse to deflect any rolling rocks.

Joun Switchyard - The narrow, steep canyon both up and downstream from the power plant site makes it uneconomical to locate the switchyard, which will ultimately be quite large, adjacent to the power plant. Therefore, it is proposed to locate the power plant transformer and appurtenant equipment on top of the powerhouse and to locate the switchyard on a flat knoll about one kilometer southwest of the power plant. The switchyard would be of the double-bus, single breaker system with one bus-tie breaker, and will consist of two 11/69 kv. power transformer bays; five line or bus-tie breaker bays, one service station bay and three future possible line breaker bays. The 69 kv. lines would connect with the Awali and Zrariye Power Plants and the Les Pins (Beirut) substation. Two future 69 kv. lines would be connected with a substation at Beirut and one at Saida. Carrier current equipment and lightning arresters will be installed in the 69 kv. line circuits.

Construction Operations

Access Roads - Access to the inlet portal of Joun Tunnel will be over the same roads as described for the Awali Power Plant. Access roads several kilometers long will have to be built over rocky terrain to the entrances of the adits and outlet portal as shown on Plate XXIX-1. The road to the outlet portal may also be used for access to the Joun powerhouse and switchyard. The road to the power plant will be located over steep, rocky terrain and will require several switchbacks in order to keep the grade under 10 percent. No major bridges or structures are required. The present roads from Beirut to the village of Joun are adequate for transporting the heavy power plant and switchyard equipment.



Joun Penstock and Power Plant Site

SECTION XXXI

BEKAA PUMPING UNIT

Description of the Unit

The Bekaa Pumping Unit will irrigate 4,700 hectares of land of the South Bekaa as shown on Plate XXXI-1. Water stored in Karaoun Reservoir will be pumped to elevation 909 meters to reach 2,600 hectares of land lying above and to the east of the Bekaa Gravity Unit, and 2,100 hectares on the west side of the Litani River and south of the Ammik swamp.

A pumping plant installed immediately below Karaoun Dam on the left bank of the Litani River will raise a maximum of 2,600 liters (2.6 cubic meters) of water per second to elevation 909 meters. The maximum lift will be 89 meters and the length of the pump discharge line will be 860 meters. From the pump discharge line, water will flow northerly in the concrete lined Main Canal along the toe of the Anti-Lebanon mountains for a distance of 14.1 kilometers. At this point, a bifurcation works will divide the flow of the Main Canal with 1,400 liters per second continuing along the edge of the mountains in the East Canal, and 1,200 liters per second crossing the Litani River through a long siphon and flowing northerly in the West Canal.

The East Canal will flow northerly from the bifurcation works along the edge of the mountains and will terminate at the Damascus highway. It will be concrete lined and will gradually decrease in size from its initial 1,400 liters per second capacity, as it served laterals along its route, to a final capacity of 80 liters per second at its last turnout. It will have a total length of 24.0 kilometers.

The Litani River siphon, with its inlet at the bifurcation works will cross the Litani River Valley and will terminate on the lower slopes of the Lebanon mountains on the west side of the Bekaa. It will have a capacity of 1,200 liters per second, a total length of 2.2 kilometers and will be constructed of precast reinforced pipe, 80 centimeters in diameter. The minimum static head on the siphon will be about 50 meters.

The West Canal will begin at the outlet of the Litani River siphon. It will flow northerly along the 885-882 meter contour for a distance of 14.7 kilometers. With an initial capacity of 1,200 liters per second, it will be gradually reduced in size as it serves laterals along its route. The capacity will be 150 liters per second at the last turnout. The canal will be lined with reinforced concrete throughout its length.

Other than those already mentioned, no siphon or other major structures will be required for the unit.

A distribution system, based on village ownerships as far as practicable, will be provided. The topography of the unit is favorable to a distribution system constructed as village units, and no special problems of distribution are anticipated. About eight turnouts to serve village laterals will be required on the East Canal and about five will be required on the West Canal. No land will be served directly from the Main Canal.

Land Classification

A detailed description of land classification of the South Bekaa, including the Bekaa Gravity Irrigation Unit and the Bekaa Pumping Unit is contained in Volume II - Section XVII. This discussion includes a description of the soil and land classes, and of land use problems of soil fertility and drainage.

The areas in hectares of the various soils mapped in the Bekaa Pumping Unit include:

Class 1	Class 2	Class 3	Total Arable
3,495	1,149	54	4,698

Water Supply

Litani River water, stored in Karaoun Reservoir will be used as the water supply for the Bekaa Pumping Unit. This water will be largely that of presently unused winter run-off so that there is no problem of water rights. Sufficient releases from the reservoir will be made through power plants and otherwise to satisfy all downstream rights and prior uses during the summer time.

The amount of water used from the reservoir for irrigation represents only a small fraction of the total storage, and irrigation needs will receive preference over power production in case of any water shortages. A full water supply for irrigation each year is thus assured.

Litani River water is considered to be of excellent quality for irrigation. It contains no salts or dissolved minerals in sufficient quantities to have any harmful effects on the land or crops receiving the water. It is anticipated that any silt carried by the river will be almost entirely settled out in the reservoir and not carried into the pumping plant in sufficient quantity to have any noticeable effect on the pumps, canals, or land.

Water Requirements

The water requirements for the Bekaa Pumping Unit are the same as those shown for the Bekaa Gravity Unit in Volume II. The lands of each unit are adjacent, the same types of crops will be raised on each, and the climatic conditions are identical.

In normal years, full field moisture capacity is reached during the winter rains thus making available for crop use a larger amount of soil moisture. This is estimated to be 20 centimeters, and has been assumed to be distributed throughout the entire summer. This distribution is believed to be realistic, and provides for the most efficient use of the pumping plant. Transmission losses have been assumed as being 15 percent of the quantity pumped, and farm losses as 25 percent of farm or village deliveries. The consumptive use of water has been computed by the Blaney-Criddle method using a consumptive use coefficient of 0.85. The consumptive use is shown in Table XXXI-1, and the irrigation requirements in Table XXXI-2.

TABLE XXXI-1

CONSUMPTIVE USE OF WATER
BEKAA PUMPING UNIT

Month	Daylight Hours %	Mean Temperature °F	Consumptive Use Factor	Consumptive Use In	Consumptive Use Cm.	Precipitation Cm.	Net Consumptive Use Cm.
April	8.80	58	5.1	4.3	10.9	3.7	7
May	9.72	66	6.4	5.4	13.7	1.5	12
June	9.70	72	7.0	5.9	15.0	0.1	15
July	9.88	75	7.4	6.3	16.0	0	16
August	9.33	75	7.0	5.9	15.0	0.1	15
Sept.	8.36	71	5.9	5.0	12.7	0.1	13
Oct.	7.90	64	5.1	4.3	10.9	1.7	9

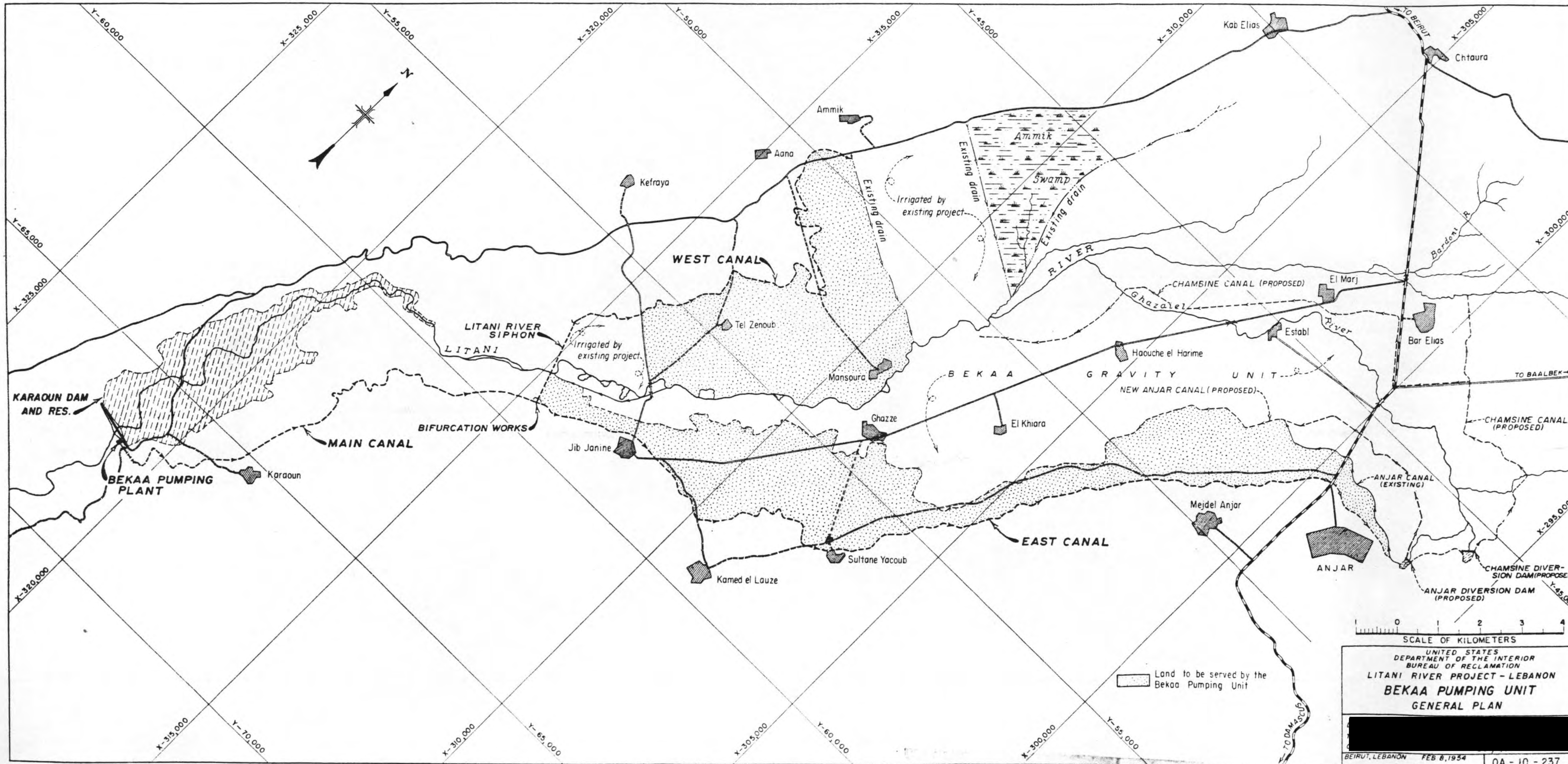

N
Kab Elias
TO BEIRUT
Chtaura
Ammik
Ammik Swamp
Aana
Existing drain
Irrigated by existing project
Kefraya
WEST CANAL
RIVER
Bardoni R.
El Marj
CHAMBINE CANAL (PROPOSED)
Ghazalel River
LITANI RIVER SIPHON
LITANI
Irrigated by existing project.
Tel Zenoub
Mansoura
Establ
Bar Elias
Haouche el Harime
TO BAALBEK
BEKAA GRAVITY UNIT
NEW ANJAR CANAL (PROPOSED)
KARAOUN DAM AND RES.
BIFURCATION WORKS
MAIN CANAL
Ghazze
El Khiara
CHAMSINE CANAL (PROPOSED)
Jib Janine
BEKAA PUMPING PLANT
Karaoun
ANJAR CANAL (EXISTING)
Mejdel Anjar
EAST CANAL
Sultane Yacoub
Kamed el Lauze
ANJAR
CHAMSINE DIVERSION DAM (PROPOSED)
ANJAR DIVERSION DAM (PROPOSED)
TO DAMASCUS
SCALE OF KILOMETERS
0 1 2 3 4
Land to be served by the Bekaa Pumping Unit
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
LITANI RIVER PROJECT - LEBANON
BEKAA PUMPING UNIT
GENERAL PLAN
[redacted]
BEIRUT, LEBANON FEB 8, 1954
OA - 10 - 237
Y-60,000
X-325,000
Y-55,000
X-320,000
Y-50,000
X-315,000
Y-45,000
X-310,000
X-305,000
Y-65,000
X-300,000
X-295,000
X-320,000
X-315,000
Y-70,000
X-310,000
Y-65,000
X-305,000
Y-60,000
X-300,000
Y-55,000
Y-45,000

PLATE XXXI-I

TABLE XXXI-2

WATER REQUIREMENTS
BEKAA PUMPING UNIT

Month	Net Consumptive Use Cm.	Soil Moisture Use Cm.	Irrigation Requirements Net Cm.	Farm Delivery Cm.	Cm.	Diversion $m^3/sec.$	$m^3 \times 10^6$
April	7	0	7	10	11	2.0	5.2
May	12	2	10	13	15	2.6	7.0
June	15	5	10	13	15	2.6	6.8
July	16	6	10	13	15	2.6	7.0
August	15	5	10	13	15	2.6	7.0
Sept.	13	3	10	13	15	2.6	6.8
Oct.	9	0	9	12	14	2.5	6.6

Description of Features

Karaoun Pumping Plant. The Karaoun Pumping Plant which will supply the unit will be located on the left bank of the Litani River about 100 meters below the toe of Karaoun Dam. The general location of the pumping plant in reference to the dam is shown on Plate XXII-1.

At the Sohmor gate chamber, a 42 inch (or 110 cm) steel, pump intake line will branch from the Sohmor Tunnel supply line as shown on Plate XV-4, Volume II.

The pump intake line will be controlled by a 42 inch gate valve installed in the gate chamber and will run through an adit which will have been used for the construction of Sohmor Tunnel. The length of the intake line will be approximately 165 meters.

The pump and pumphouse will be located near the portal of the adit about 40 meters from the river. The single centifugal pump will have a capacity of 2.6 cubic meters (91 cubic feet) per second and a maximum total discharge load of 94 meters (308 feet). The pump will be driven by an electric motor of 4350 horsepower connected directly to the pump. Electric power will be obtained from the Sohmor Power Plant.

A 42 inch (or 110 cm) steel pipe discharge line will run from the pump to the beginning of the Main Canal on the hillside east of the dam and pumping plant. The line will pass under the reservoir spillway channels, one road, and a small natural drainage channel so it will be completely buried for the greater portion of its 860 meter length. The line will terminate at elevation 909 meters in a stilling basin and transition to the Main Canal.

The pumping requirements are very uniform throughout the irrigation season as shown on Table XXXI-2, and the required pump capacity varies only from 2.0 to 2.6 cubic meters per second. For this reason no savings are likely to be obtained by using two or more pump units instead of the single unit planned.

While the maximum discharge head at minimum reservoir elevation of 820 meters is 94 meters, the average lift will be considerably less. Reservoir operation studies show that the average total discharge head will be 60.5 meters. Figured on this basis, the average annual power consumption for the plant will be 10,300,000 kwh.

Main Canal. Beginning at the end of the transition from the Karaoun Pumping Plant discharge line, the Main Canal will flow northerly and then easterly to the bifurcation works, 14.1 kilometers from its beginning. The canal will supply no laterals and will have a constant capacity of 2.6 cubic meters (2,600 liters) per second, and a slope of .00025, throughout its length. The canal will be trapezoidal in cross section with a bottom width of 150 centimeters, a water depth of 111 centimeters, and side slopes of 5:4. The entire canal will be lined with plastered, unreinforced concrete, 8 centimeters in thickness. A bank of two meters top width will be placed on the downhill side of the canal and a berm or bank one meter wide will be provided on the uphill side. The entire length of canal will be located on a side hill with cross slopes varying from 5% to about 35%, with the average being about 15%. Considerable rock will be encountered in excavation.

No major structures will be required on the Main Canal. Minor structures include three small bridges, twelve foot bridges, five drain culverts under the canal, and one livestock watering ramp. Typical designs of minor structures are shown on Plate XXI-9 through 16.

Bifurcation Works. The flow of the Main Canal will be split at kilometer 14.1 at the bifurcation works. This structure will be quite simple and constructed of masonry. Simple controls for the flows for the two branching canals, as well as for entirely stopping the flow into either branch, will be provided by stop logs. After once setting, there should be little reason for adjusting the flows to either of the branch canals as both will usually receive uniform amounts of water. Of the 2,600 liters per second carried by the Main Canal, 1,400 liters per second will be supplied to the East Canal, and 1,200 liters per second to the West Canal.

East Canal. Beginning at the bifurcation works, the East Canal, with a length of 24 kilometers, will flow in a northeasterly direction along the eastern edge of the Bekaa valley. It will roughly parallel the New Anjar Canal of the proposed Bekaa Gravity Unit, but at a higher elevation. Near its beginning, the East Canal will be about 35 meters above the New Anjar Canal and it will serve 2,600 hectares of land lying between the two canals.

The canal will have an initial capacity of 1,400 liters per second and a trapezoidal section identical with that of the Main Canal except that the bottom width will be 120 centimeters, and the water depth will be 86 centimeters. It will be lined with unreinforced concrete throughout its length. The route of the canal is on a side hill and cross slopes vary from 5% to about 50%, the average being about 15%. Through two reaches of the steeper side hill slopes where the excavation is likely to be mostly in rock, it is planned to use a rectangular, masonry bench flume section in place of the usual trapezoidal section. About two kilometers of such flume will be required.

Seven turnouts will be served by the canal and its size and capacity will be gradually reduced to the final capacity of 80 liters per second at the last turnout.

Structures on the canal will include two bridges for primary roads, eight small village road bridges, seven turnouts varying in size from 30 to 370 liters per second, five checks, seventeen footbridges, six culverts under roads, eight drain culverts under the canal, and six livestock watering ramps. Typical designs for minor canal structures are shown on Plates XXI-9 through 16.

Litani River Siphon. The water supplying the West Canal will be carried from the bifurcation works under the Litani River to the west side of the Bekaa through the Litani River siphon. A short transition will carry the water from the bifurcation structure to the siphon inlet. The siphon will have a capacity of 1,200 liters per second and a total length of 2.2 kilometers. The siphon barrel will be constructed of precast reinforced concrete pipe 80 centimeters in diameter. Precast concrete pipe suitable for the maximum static head of 50 meters that will be required for the siphon is not manufactured locally at present. It is believed, however, that pipe having such requirements could be made using existing plant facilities when the demand for it occurs. If not, a monolithic section will be used through the lower valley and under the river itself. The siphon will terminate at about elevation 885 meters and will discharge into the West Canal.

West Canal. Beginning at the outlet of the Litani River siphon, the West Canal will flow north and west along the west slope of the Bekaa Valley for 14.7 kilometers to a point south of the Ammik swamp. The first two kilometers of the canal route will be through land which is presently irrigated by water from springs and no turnouts will be needed through this section. From the edge of the existing project to the end of the canal, 2,100 hectares of land lying between the canal and the Litani River will be served.

The canal will have an initial capacity of 1,200 liters per second, a cross section similar to the Main Canal and East Canal, and will be lined with unreinforced concrete. At its beginning, the canal will have a bottom width of 120 centimeters, a water depth of 80 centimeters and a slope of .00025. The size will be gradually reduced as deliveries are made to laterals along its route until reaching the final capacity of 150 liters per second. The canal will end at a drain which forms the boundary of an area which is presently irrigated by water from the springs of Ammik.

Structures which will be required for the canal include two bridges on primary roads, and two on secondary roads, six turnouts varying in size from 70 to 350 liters per second, five checks, eleven foot bridges, six culverts under roads, eight drain culverts under the canal, and five livestock watering ramps. Typical designs for minor structures are shown on Plate XXI-9 through 16.

Distribution System

The distribution system for the unit will be similar in principle to the one proposed for the Bekaa Gravity Unit. Generally, one turnout from the Main Canal will be provided for the land of each village and a distribution system, complete in itself, will be laid out for each village. The reasons for this are more fully discussed in Volumes I and II. While no rationing of water will be necessary for the unit, administration of the project can be made more simple and efficient by organizing the distribution of water on a village unit basis.

The land of the unit lies well topographically with gentle and fairly uniform slopes over most of the area. Generally speaking, turnouts on the main canals will serve a lateral running down the slope which, in turn, will serve smaller laterals running along the contours and paralleling the Main Canal. All principal laterals will be lined with concrete.

Drainage

Portions of the unit lying adjacent to the Litani River on the west side are subject to flooding by the river during the winter and early spring. Elsewhere, only small localized areas will be inundated by heavy rains. The completion of the projected flood control work on the Litani will greatly ameliorate, if not completely eliminate, the flooding of the lower valley lands. Even so, a system of surface drains will be necessary throughout most of the unit to carry off excess rain water. Where such drains have already been constructed, they can easily be incorporated into the new system. Examination of irrigated land in the Bekaa Valley where high ground water tables exist for a long period each year shows no sign of alkali damage. Because of the calcareous nature of most of the soil and the excellent quality of irrigation water, trouble from alkali is not anticipated.

Cost Estimate

The estimate of costs for the Karaoun Pumping Plant was made by using United States costs for similar installations, modified slightly to fit local conditions. The estimate for the remainder of the unit was based on local prices for similar work. Unit prices shown for the Bekaa Gravity Unit cost estimate in Volume II were used for the unit. A summary of the cost estimate in Lebanese pounds is shown below:

Karaoun Pumping Plant	2,340,000
Main Canal	1,032,000
Bifurcation Works	12,000
East Canal	896,000
Litani River Siphon	307,000
West Canal	478,000
Distribution System	1,621,000
Drainage System	132,000
Subtotal	6,818,000
Engineering and Overhead	682,000
Grand Total	7,500,000 L.L.

SECTION XXXII

UPPER NABATIYE IRRIGATION UNIT

Description of the Unit

The Upper Nabatiye Unit lies on a plateau-like area north of the Litani River and west of the Jarmaq River as shown on Plate XXXII-1. Portions of the unit lie in valleys running west from the main body of the project. Although the area is known as the Nabatiye Plateau, it is not as flat or uniform as the name might imply. The surface of the land is quite irregular and is spotted with numerous knolls or higher areas which cannot be economically reached by a gravity system.

The unit will be served with Karaoun Reservoir water which will be taken from the outlet of Kelia Tunnel near the village of Marjayoun. A canal will carry the water from the tunnel southward for about 7 kilometers to a point near the village of El Qlaiaa above the Khardale Dam site. From this point, the water will be carried across the deep valley of the Litani River in a siphon and up to the plateau on the west side of the river. The siphon barrel will be 90 centimeters in diameter and 2,650 meters long. It will be constructed of steel since the maximum static head on the siphon will be 375 meters. From the siphon, water will flow by gravity in a lined canal northward along the edge of the plateau for 13 kilometers to a point near the village of Kafer-Roummane. Seven laterals will be served by the Main Canal along its route.

The distribution system for the unit will be laid out on a topographic basis because of the irregularity of the land surface. In many instances it will be possible to serve only the land of an individual village as a single unit, but in others a single lateral may serve more than one village. In the southern part of the unit the distribution system will be fairly simple, but in the remainder of the project many small but long laterals will be required to reach the small, irregular, irrigable areas.

Land Classification

Because of the similarity and proximity of the Upper and Lower Nabatiye Irrigation Units regarding land classification, general characteristics of the two will be discussed together in the following paragraphs.

The Nabatiye area is essentially a dissected plateau lying between the coastal plain and the Lebanon mountains. The lower portion of the plateau is separated from the coastal plain by an escarpment varying in height from 20 to 100 meters above sea level. There is a gradual rise in elevation from the lower to the upper plateau, with elevations reaching 500 meters on some of the more level lands in the latter sector. The area is dissected by numerous small drainage ways, resulting in a landscape which is hilly and rough, with many isolated hills and ridges. Erosion has been especially severe in the soft marls of the lower and middle sectors.

Based on geographic location and future irrigability, the plateau was divided into four units - the upper, middle, lower, and Zahrani - Awali Units. A preliminary investigation of the area revealed that the upper and lower units offered the best potential with regard to irrigability, especially in view of the limited water supply available. This subsequent survey and report, therefore, is confined to a more thorough appraisal of these two units.

Soils and Land Classes. The soils of the Upper and Lower Nabatiye Plateau may be divided into two categories - those developed from underlying limestone, marl, or marly limestone, and those formed from alluvium washed from the uplands. The majority of the soils are in the former category. In the upper plateau they consist largely of reddish brown to red clay loams developed from Turonian and Cenomanian limestone. These soils could be included in the Terra Rossa soil group common to the Mediterranean region.

The lower plateau is made up largely of soils formed from Senonian marl or soft marly limestone. On smooth, level sites they are dark grayish brown clays, possibly belonging to the Rendzina group, which includes dark, heavy textured soils developed from soft calcareous material. On steeper topography the soils are gray to white, shallow silt loams. A more or less definite line of demarcation exists between the red limestone soils of the upper plateau and the marly soils of the middle and lower units. Roughly this approximates a line extending from Innsar (250 meters elevation) to slightly east of Merouniye (300 meters). However, small areas of marly soils exist in the upper plateau and small tracts of reddish soils are found in the lower unit.

Alluvial soils occupy the relatively flat wadi bottoms and the portion of the coastal plain included in the Lower Nabatiye Unit. They are usually deep and of variable character depending on the mode of origin.

Six land classes were mapped in the upper and lower plateaus - Classes 1, 2, 3 under the general standards; Class 1 and 2 terrace lands; and Class 6 nonarable. Reference should be made to the section on land classification - Volume I, for a description of these classes. The land classification standards used in the Nabatiye Plateau are presented in Table XXXII-1. A description of the soils and land classes for the Upper Nabatiye Plateau follows:

Uplands. The majority of the soils of the upper plateau are formed from underlying limestone, or derived from alluvium of limestone origin. The soils of the gently sloping to hilly uplands are extremely variable in depth, grading from relatively deep to shallow, stony soils. The depth to bedrock usually varies with topographic position - the shallower soils being found on the steeper, eroded hillsides. However, some areas have very shallow soils even on level sites. Soils formed from underlying limestone usually consist of a reddish brown, granular, loam surface underlaid by uniform reddish brown loam or clay loam subsoils. The parent limestone, which may be reached at depths of a few centimeters to a meter or more, is usually well weathered. Limestone fragments are found in both surface and subsoils. Internal drainage of these soils appears good due to their friable nature, and the fragmented and porous condition of the underlying limestone. External drainage is excessive on areas of steep topography. As a result, sheet erosion has been particularly active on these sites.

The pH range of these soils is from 6.5 to 7.7, being quite uniform in individual profiles. The soils are slightly to moderately calcareous, and have very low contents of soluble salts, ranging from a trace to .09%.

Only small areas of soils derived from marl are found in the upper plateau, largely around Habbouch and Deir Ez Zahrani. They are gray to pale brown silty clay loams of variable depth to marl, depending on topographic characteristics.

The deeper soils found on the flat to gently sloping uplands have been given a Class 1 irrigability classification. They are relatively small, widely separated tracts. They appear to be productive for a variety of dryland crops, and would undoubtedly be very productive for any climatically adapted crops under irrigation and good management. Upland areas designated as Class 2 were due to soil and topographic limitations. These include relatively shallow soils, deep but stony soils, and moderately sloping or gently undulating topography. Most frequently a combination of these deficiencies was responsible for placing the lands in Class 2. The soils are measurable lower in productive capacity than Class 1 areas, the topographic limitations will require greater expense in leveling, or special care in irrigating. They are probably well suited for general farm crops, vegetables and grapes under irrigation. Class 2 lands are found throughout the upper plateau, comprising about 30% of the irrigable land. Class 3 lands are closely associated with Class 2 areas. They are of distinctly restricted suitability for irrigation because of more extreme deficiencies in soil and topographic characteristics. They consist of smooth, gently sloping lands having shallow soils over limestone, or extremely rocky, but relatively deep soils. In a large part of the upper plateau

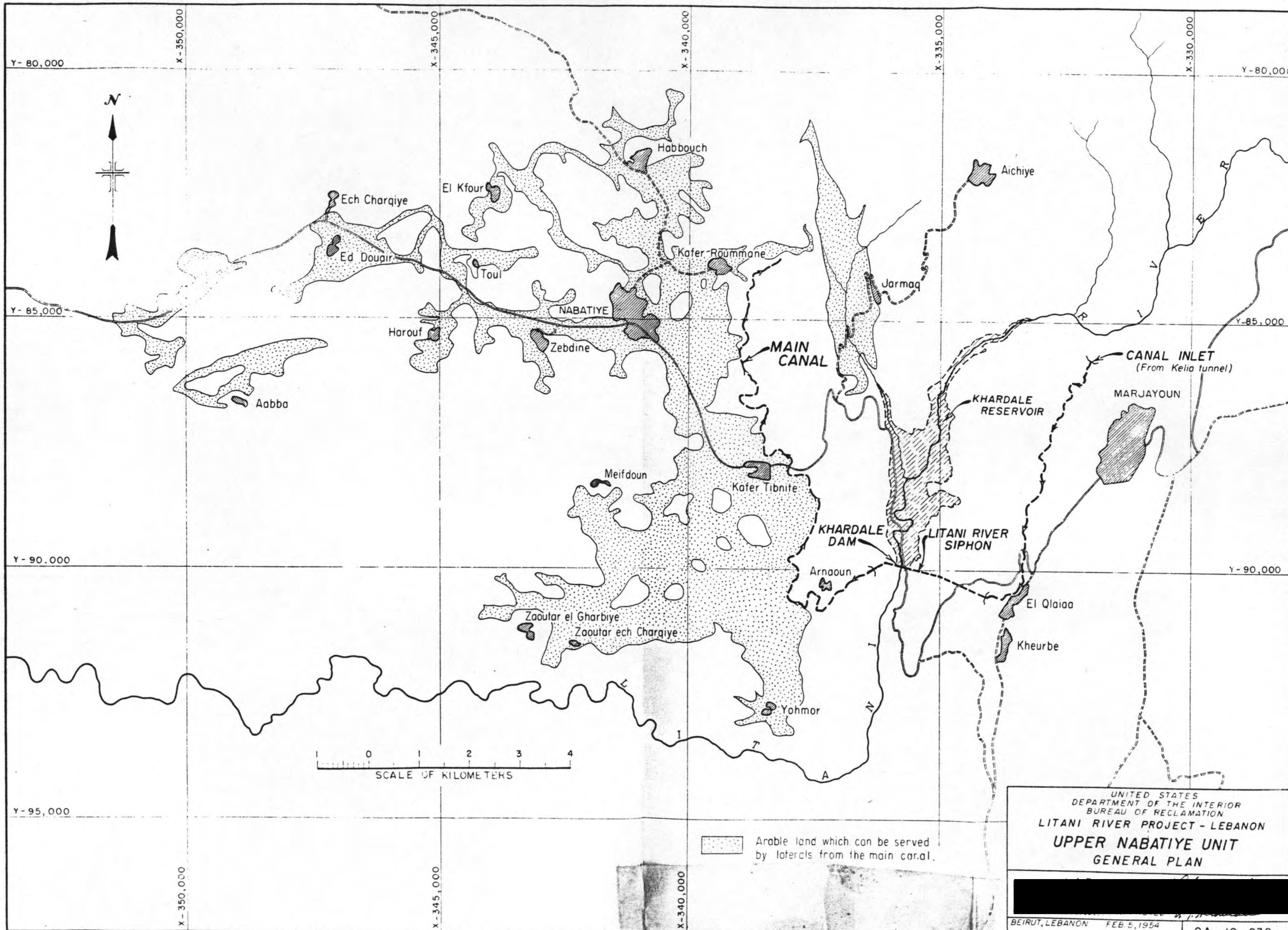


PLATE XXXII-1

TABLE XXXII-1

RECONNAISSANCE LAND CLASSIFICATION STANDARDS
NABATIYE PLATEAU

Land Characteristics	Class 1 - Arable	Class 2 - Arable	Class 3 - Arable
Soils			
Texture	Sandy loams to slowly permeable clays	Loamy sands to very slowly permeable clays	Loamy sands to very slowly permeable clays
Depth (Centimeters) to limestone bedrock or marly limestone	120 cm. plus	90 cm. plus	60 cm. plus
To firm marl (penetrable)	90 cm. plus	60 cm. plus	30 cm. plus
To very soft penetrable marl or raw marly soil	60 cm.	30 cm.	-
To clean sand, gravel or cobble1/	60 cm. plus	45 cm. plus	25 cm. plus
Alkalinity (pH)2/	8.6 or less	9.0 or less	9.0 or less
Salinity (total salts)2/	0.2% or less	0.2 - 0.4%	0.4 - 0.6%
Topography			
Slopes	Uniform slopes to 4%	4 - 8% Rougher slopes to 4%	8 - 12% Rougher slopes to 8%
Surface	Even enough to require only a small amount of leveling	Moderate grading required	Heavy grading required
Cover (Rocks)	Insufficient to modify productivity or cultural practices-Clearing cost small	Sufficient to reduce productivity and interfere with cultural practices - Moderate clearing	Present in sufficient amounts to require expensive but feasible clearing.
Drainage	No drainage problem anticipated	Drainage may be required but at moderate cost	Expensive but feasible drainage may be required.

Class 6 - Nonarable -
Includes lands considered permanently nonarable for irrigation.

1/ At the minimum depth the overlying soil should be a sandy loam or heavier.

2/ May be slightly higher in open permeable soils and under good drainage conditions.

TABLE XXXII-1 (Continued)

TERRACE STANDARDS

Land Characteristics	Class 1 - Arable	Class 2 - Arable
Soils		
Texture	Sandy loams to slowly permeable clays	Loamy sands to very slowly permeable clays
Depth to limestone bedrock 1/ or marly limestone	90 cm. plus	45 cm. plus
To firm marl (penetrable) 1/	60 cm. plus	30 cm. plus
To very soft penetrable marl or raw marly soil 1/	45 cm. plus	-
To clean sand, gravel or cobble 1/ 2/	60 cm. plus	30 cm. plus
Alkalinity 3/	pH 8.6 or less	9.0 or less
Salinity 3/	Total salts not to exceed 0.2%	Total salts not to exceed 0.6%
Topography		
Slopes 4/	Uniform slopes to 20% 5/	Uniform slopes over 20%
Cover	Loose rock may be present; should not exceed amount required to build terrace walls	Loose rock present in excess of amount to build terrace walls. Moderate to expensive clearing costs.
Drainage	No drainage problems anticipated.	

1/ Average depth of unterraced land or minimum depth of terraced land

2/ At the minimum depth the overlying soil should be a sandy loam or heavier

3/ May be slightly higher in open permeable soils and under good drainage conditions

4/ Soil depth will normally determine the upper slope limit

5/ Classes 2 and 3 (General classification) susceptible of terracing could revert Class 1 (Terrace). Almost all Class 1 (Terrace) lands in the Nabatiye Plateau are presently terraced.

a combination of moderately sloping or undulating topography, together with shallow, stony soils has been responsible for placing most of the lands in this category. Class 3 lands should be utilized, wherever possible, only after water has been provided for the better lands.

Class 1 terrace lands are of only minor extent in the upper plateau. They are characterized by gently to moderately steep slopes with relatively deep, medium textured soils. All of the areas in this class are terraced, with olives, figs, grapes and field crops being grown. These lands should be very productive for any of the climatically adapted crops with the addition of irrigation water.

In the upper plateau, Class 2 terrace lands are made up of steep, hilly topography, and stony, shallow soils. Most of the area consist of good sized, contiguous units and include about 36% of the irrigable land. Approximately 60% of these areas are terraced to some extent, with most of the terraces being of a makeshift or temporary nature. Considerable expense will be involved in preparing these lands for irrigation, and they will be more difficult to irrigate and manage than the better land classes. However, they should be capable of fair to good production for a variety of field crops, vegetables, grapes, etc. under irrigation and with proper management. Unless these lands are terraced they should be considered as being nonarable in so far as irrigation is concerned.

Approximately 58% of the land area in the upper plateau, lying solely in the uplands, has been included in Class 6 - nonarable. These areas consist essentially of very steep, rough topography, with extremely shallow, stony soils or rock outcrop. Lands in this category are considered permanently unfit for irrigated agriculture, and many of these areas are not suited for dry land cultivation. The poorer lands afford a limited amount of grazing for goats. However, their carrying capacity could be increased through the introduction of more palatable plant species and with proper range management techniques.

Wadis. The alluvial soils of the wadis are variable in characteristics. In the wadi near Jarmaq they are deep silty clay loams to silty clays of dark brown or reddish brown color. The wadis near Nabatiye and Kafer Roumman vary from dark brown to dark red loams or clay loams of considerable depth. Small limestone fragments are usually found on the surface and in the subsoils of the alluvial soils. pH's vary from 7.0 to 8.0, being uniform in individual profiles. The soils are slightly to highly calcareous, and are very low in soluble salts. The larger wadis normally have smooth, very gently sloping bottom lands with gently sloping to steep sides. The smaller wadis are usually narrow, and bordered by steep lands. Drainage is apparently good in most wadis. The wadi soils appear to be more productive than most upland areas due to a better moisture and fertility status. They should be very productive for a variety of climatically adapted crops under irrigation, including citrus, vegetables and general field crops. Aside from the wadi near Jarmaq, which is partially irrigated, most of the wadi lands in the upper plateau are dry farmed. The majority were placed in Class 1 due to a favorable combination of soil, topographic, and drainage characteristics. Some areas were delineated as Class 2 due to stony soils or to deficiencies in topography.

A tabulation of the area of the various land classes in the upper unit is contained in Table XXXII-2.

The reconnaissance land classification is shown on Plate XXXII-2.

TABLE XXXII-2

LAND CLASSES BY AREA (HECTARES)

Area	General Land Classes 1	2	3	Terrace Classes 1	2	6 High	Total Arable
Upper Nabatiye							
Total Area Classified	542	1807	880	254	1532	597	5612
Total Irrigable[1/]	491	1274	633	195	1205	0	3798

1/ Gross area which can be reached from proposed canals. Net area in the Upper Nabatiye is about 3500 hectares.

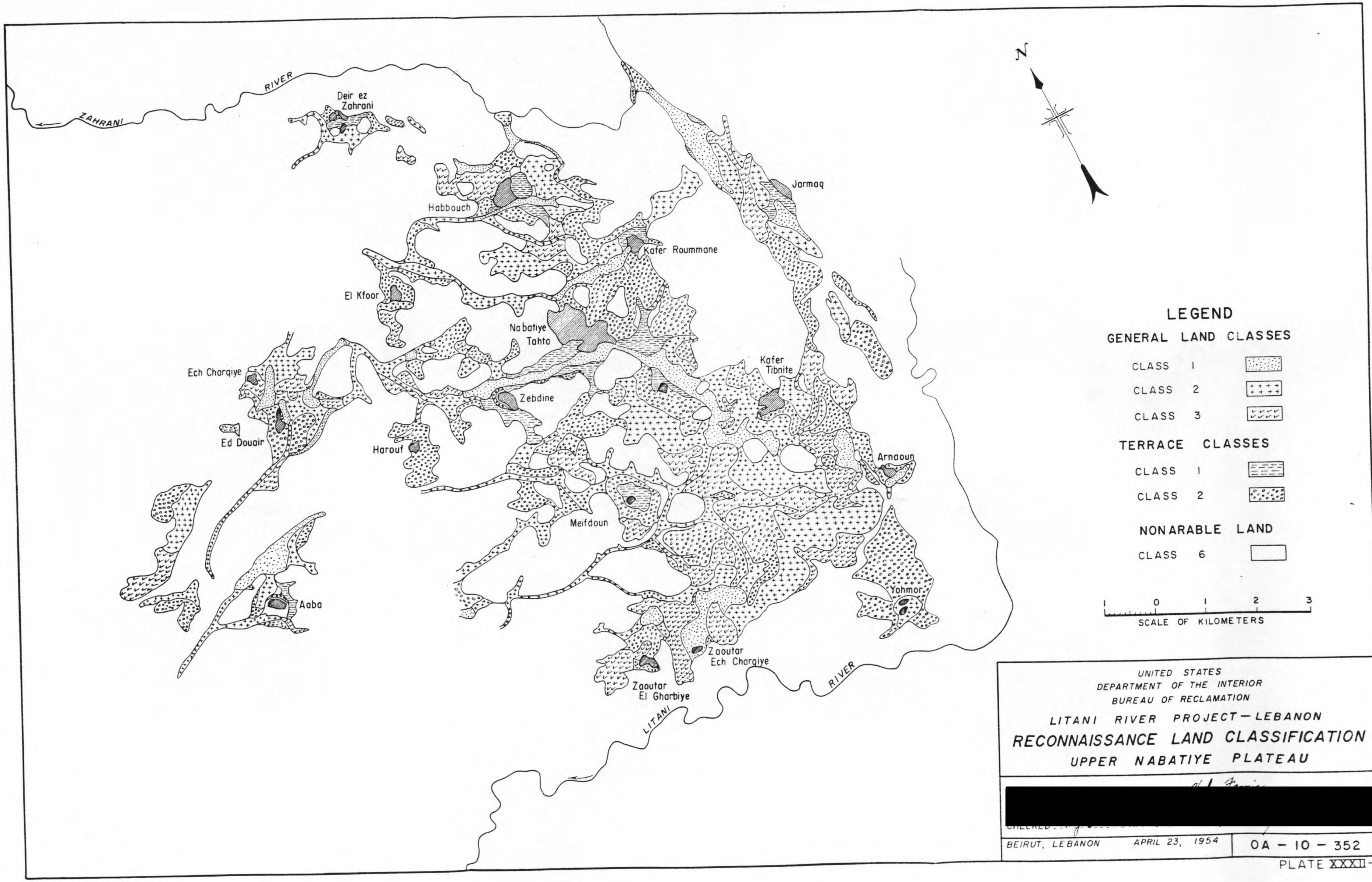
Land Use Problems

The rough, dissected nature of the Nabatiye Plateau, with many discontinuous areas of arable land, will undoubtedly require an expensive irrigation system. This is especially true in the lower plateau, where the terrain is more dissected, resulting in numerous isolated or noncontiguous arable areas. The upper plateau in this respect, offers the best potentialities in regard to irrigability, having several sizeable, reasonably compact bodies of good land.

The sloping or hilly nature of much of the topography will require considerable care in irrigating to avoid soil erosion. The use of contour furrows on the more moderate slopes, wherever possible, and terraces on the steeper lands would assist in preventing erosion and facilitate water distribution. It would appear that the use of sprinkler irrigation would be one method that might be tried on the Nabatiye Plateau. This method is especially useful for areas of irregular topography, steep slopes, and shallow soils, involving less cost for land preparation, and a saving in water through controlled use. It does have the disadvantage of high initial investment, depreciation costs, and power costs for operation.

Although both upper and lower plateaus consist dominantly of hilly topography, terracing has not been carried out as extensively or in such detail as in other parts of Lebanon. In the upper unit rather inexpensive, mediocre, rock wall terraces have been constructed on many areas having steep slopes. Better quality rock wall terraces of small extent are found on the more gently sloping sites having deeper soils. Many steep, hilly lands in the lower plateau have been terraced to some extent, with native vegetation being utilized as retaining walls, since large rocks are not ordinarily available for the building of walls. In this respect, the construction of good, durable terraces is much more difficult on the marly soils of the lower unit than on the limestone soils of the upper plateau. The proposed use of irrigation water on the better quality steep lands will necessitate the construction of good terraces on both upper and lower units.

A large proportion of the land area in both units is too rough or rocky to be used agriculturally. Most of these lands have a rather sparse covering of spiny, drought resistant shrubs, and are utilized to some extent for the grazing of goats. The carrying capacity of these areas is very low. In view of the large land area involved, it would seem feasible that a study be made relative to their rehabilitation as good grazing lands. The introduction of more palatable grasses and shrubs, the use of water and soil conserving practices such as contour furrows, inexpensive terraces and water spreading devices, together with controlled grazing, might be a few measures that would increase the potential of these lands.



UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

LITANI RIVER PROJECT—LEBANON

RECONNAISSANCE LAND CLASSIFICATION

UPPER NABATIYE PLATEAU

[redacted]

BEIRUT, LEBANON APRIL 23, 1954 | OA – 10 – 352

PLATE XXXII-2

Soil and agronomic research especially pertaining to fertility and crop adaptability of the light colored marly soils of the lower and middle plateaus, would greatly assist in solving problems associated with the cultivation and future irrigation of these areas. This might include the type and amount of fertilizer needed, the adaptability of various crops (including citrus and bananas). To this soil type, soil permeability studies, soil conservation, and others.

Water Supply

The water for the unit will come from Karaoun Reservoir, and will first pass through the Sohmor Tunnel and Sohmor Power Plant. Diverted at Markabi Diversion Dam, it will pass through the Markabi Tunnel and Canal and the Kelia Tunnel. Most of the flow of the tunnel will go to serve the Kelia Power Plant, but 1.8 cubic meters (1,800 liters) per second will be diverted near the tunnel outlet into the Main Canal of the Upper Nabatiye Unit.

The water represents presently unused winter flow of the Litani River which will be stored in Karaoun Reservoir, and no problem of water rights or prior use exists. Irrigation uses will have priority over power uses so that there will always be a dependable supply. A full irrigation supply with no shortages can be expected each year.

The quality of water is excellent and it contains no salts or other dissolved or suspended matter in sufficient quantity to cause any damage to lands or crops.

Water Requirements

Water requirements for the unit have been computed by the Blaney-Criddle method of estimating consumptive use. A consumptive use coefficient of 0.66 was used. This is a weighted average coefficient which was computed as follows:

Crop	Area %	Consumptive Use Coefficient: Individual Crop	Consumptive Use Coefficient: Weighted
Deciduous Fruits	15	0.60	0.09
Grain (2 crops)	25	0.72	0.18
Truck	40	0.60	0.24
Forage	20	0.75	0.15
Weighted Average			0.66

The land of the unit will normally be moistened to full field capacity at the beginning of the irrigation season. This available soil moisture has been distributed throughout the season in such a manner as to cut down the peak demands of the irrigation supply and to make possible a more economical irrigation system. The consumptive use of water for the unit is shown in Table XXXII-3 and the water requirements are shown in Table XXXII-4.

TABLE XXXII-3

CONSUMPTIVE USE OF WATER
UPPER NABATIYE UNIT

Month	Daylight Hours %	Mean Temper-ature-°F	Consumptive Use Factor	Consumptive Use In.	Consumptive Use Cm.	Precipi-tation Cm.	Net Consumptive Use - Cm.
April	8.77	64	5.6	3.7	9.4	4.4	5
May	9.68	71	6.9	4.6	11.7	1.4	10
June	9.65	74	7.2	4.8	12.2	0.2	12
July	9.84	78	7.7	5.1	13.0	0	13
Aug.	9.30	80	7.4	4.9	12.4	0	12
Sept.	8.35	78	6.5	4.3	10.9	0.4	11
Oct.	7.92	74	5.9	3.9	9.9	2.4	8

TABLE XXXII-4

WATER REQUIREMENTS
UPPER NABATIYE UNIT

Month	Net Consump-tive Use - Cm.	Soil Moisture Use Cm.	Irrigation Requirements Net Cm.	Farm Delivery Cm.	Diversion Cm.	Diversion m^3/sec.	Diversion $m^3 \times 10^6$
April	5	0	5	7	8	1.1	2.8
May	10	1	9	12	14	1.8	4.9
June	12	3	9	12	14	1.8	4.7
July	13	4	9	12	14	1.8	4.9
August	12	3	9	12	14	1.8	4.9
Sept.	11	3	8	11	13	1.7	4.5
Oct.	8	1	7	9	11	1.4	3.8

Description of Features

Main Canal - Km. 0 to Km. 6.7. Water for the unit will be diverted from the Kelia Tunnel into the Main Canal at the valve house between the tunnel and the Kelia Power Plant penstock. Beginning at elevation 617 meters, the canal will flow in a southerly direction for 6.7 kilometers to the inlet of the Litani River siphon. It will have an initial capacity of 1,800 liters (1.8 meters) per second. The canal prism will be trapezoidal with a bottom width of 90 centimeters, a water depth of 96 centimeters, and side slopes of 5:4. The gradient of the canal will be .0004. Banks of two meters top width will be provided along the canal. In one-bank sections, a berm of one meter width will be provided on the uphill side. No deliveries will be made from the canal in this section.

The structures on this reach of the Main Canal will include one bridge on a primary road, eight footbridges, one drop of about 7.5 meters, ten drain culverts under the canal, and two livestock watering ramps. Typical designs of the minor structures are shown on Plates XXI-9 through 16.

Litani River Siphon. The water of the Main Canal will be carried across the Litani River Valley through a long steel siphon. A plan and profile of the siphon crossing are shown on Plate XXXII-3. The siphon inlet will be at elevation 605 meters, and the outlet at 573 meters. The outlet is at the highest elevation at which the Main Canal could be located, and the inlet was kept as high as practicable in order to keep the diameter of the siphon as small as possible. With 32 meters of head available for siphon losses, the diameter of the pipe has been set at 90 centimeters. This diameter will result in a water velocity in the pipe of about 2.7 meters per second at the maximum capacity of 1,800 liters per second. The actual slope length of the pipe will be about 2,650 meters, and the maximum static head on the siphon will be 375 meters.

The pipe will run down the relatively gentle slope on the left side of the river, crossing two roads, a small intermittant stream, and the buried pipe line of the Trans Arabian Pipe Line Company (tapline) before reaching the river. Relief valves will be required at two high points along the pipe. The river crossing itself will be made immediately downstream from the Khardale Dam site. If the construction of the dam is accomplished before the installation of the siphon, the siphon will be carried across the river on the crest of the dam, thus simplifying the river crossing proper. The right side of the river is an almost vertical rock cliff for a considerable distance above the river, gradually decreasing in slope towards the top. It will be a difficult location for most of the distance, but the rock should afford good foundations for necessary pipe supports. No roads or other man-made obstacles will be encountered on the right side of the river. The siphon will terminate at the crest of the valley side at about elevation 573 meters.

Main Canal - Km. 9.1 to Km. 22.1. Continuing from the outlet of the Litani River siphon, the Main Canal will run west and north along the eastern edge of the Nabatiye Plateau for 13 kilometers. About seven turnouts will be required along the canal to serve laterals which will carry water to 3,500 hectares of land lying below the canal. The canal will terminate near the village of Kafer Roummane. This reach of canal will initially have the same section as the first portion, but will gradually be reduced in size from its initial capacity of 1,800 liters per second to 150 liters per second at the last turnout.

Structures on this reach of the Main Canal will include two bridges on village roads, seven turnouts varying in size from about 100 to about 450 liters per second, five checks, four drops each having a drop of about five meters, five footbridges, two culverts under roads, two drainage culverts under the canal, and three livestock watering ramps. Typical designs of minor structures are shown on Plates XXI-9 through 16.

Distribution System

The layout of a distribution system for the unit will be quite difficult. The noncontiguous nature of the land will require a rather complex system of laterals. The location of main laterals will be fairly easy, but sublaterals will be numerous and, in many cases, quite long. The greater portion of the arable area lies adjacent to the Main Canal but portions of the unit are as much as fourteen kilometers distant from the Main Canal. The distribution system will have to be laid out on a topographic basis with the village unit being a secondary consideration. It will be possible to economically provide individual distribution units from some villages, but in other cases, land or more than one village must be served by the same main lateral. The small fingers of arable land shown on the General Plan, Plate XXXII-1, are generally lower portions of narrow valleys which extend westward from the main body of the unit. These will require a small lateral running down each side of the valley. Fortunately, the longest laterals will also have the smallest capacities so that their cost will not be too great. Practically all laterals will be lined in order to hold down transmission losses and maintenance costs.

Drainage System

Drainage on the unit will not be a particular problem if water is used properly. Nearly all the land has considerable slope and adequate natural drainage for surface runoff. Some small localized areas on the floors of valleys may need some treatment but a comprehensive drainage system for the entire unit will not be required. For the areas which will need treatment, only shallow surface drains will be necessary.

Cost Estimate

The cost estimate for the unit is shown on Table XXXVII-20. This estimate was based on United States prices for the steel siphon, and on local costs for work of a similar nature for the remainder of the unit. A summary of the estimated cost follows:

Main Canal.	1,000,000 L.L.
Litani River Siphon	2,180,000
Distribution System	1,750,000
Drainage System	70,000
Subtotal	5,000,000
Engineering and Overhead (10%)	500,000
Total	5,500,000

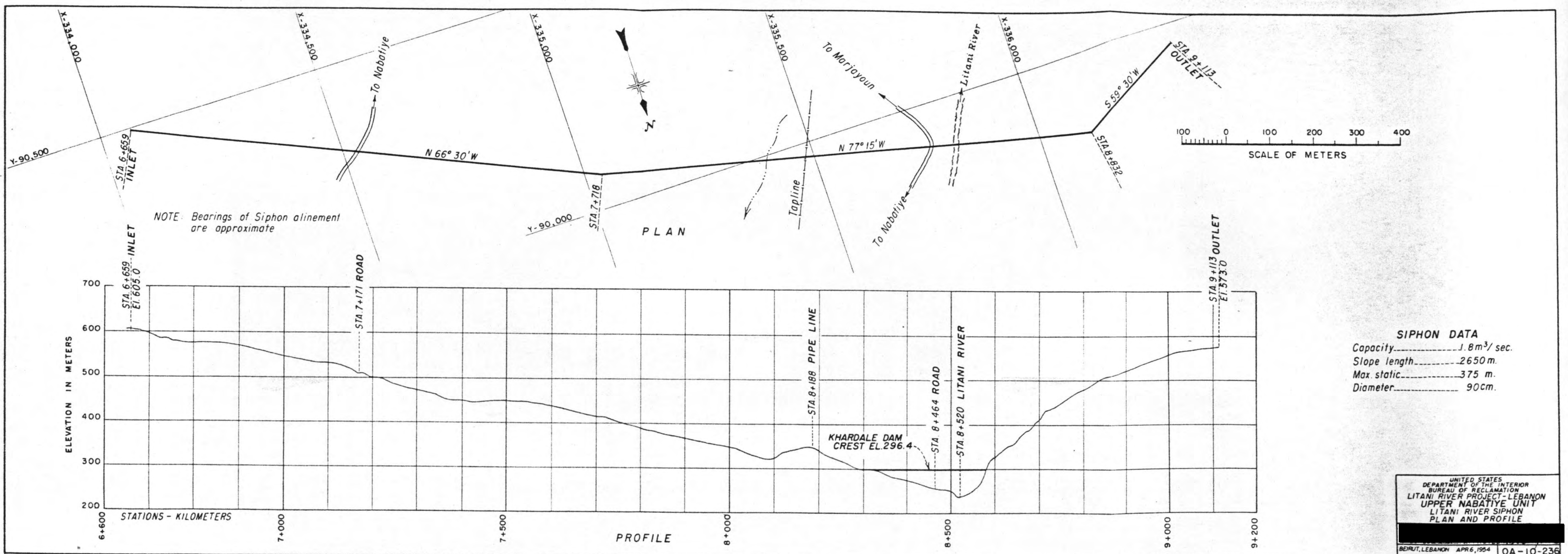
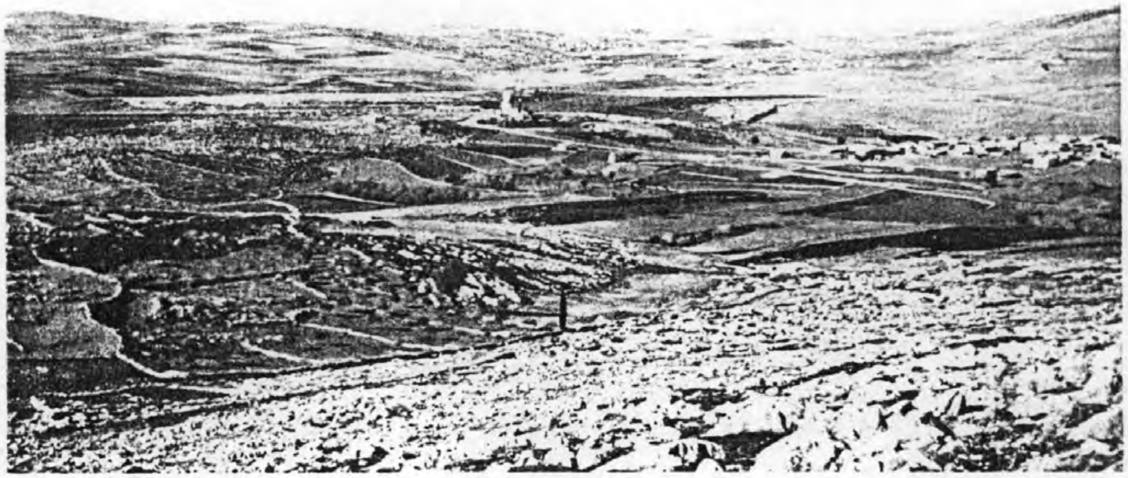
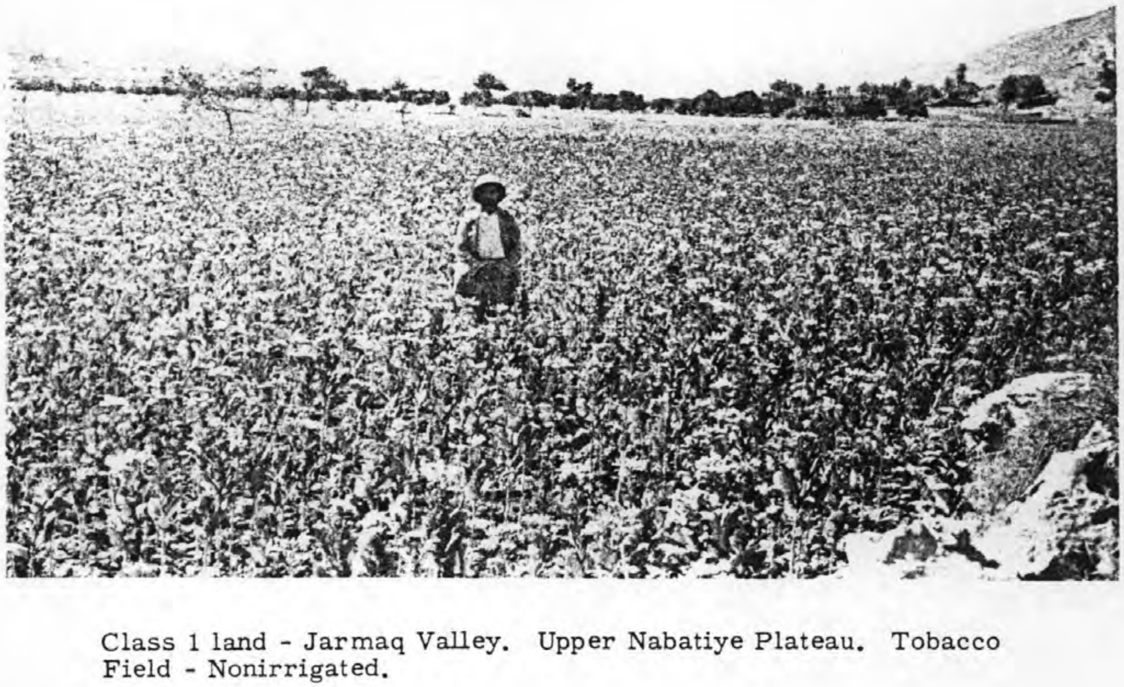


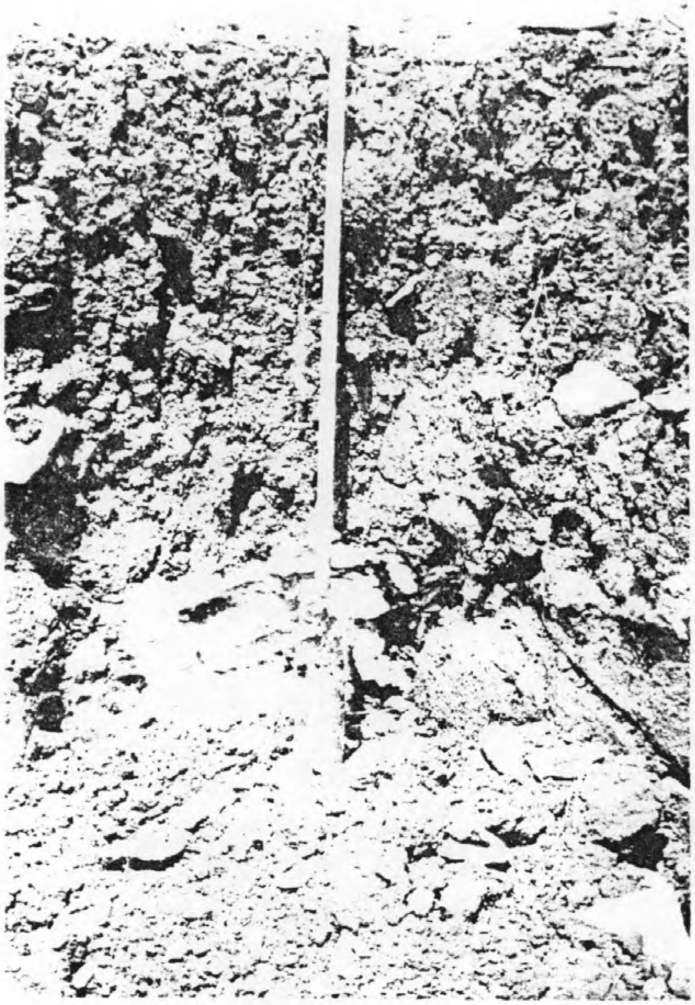
PLATE XXXII-3



General view - Upper Nabatiye Plateau with the village of Nabatiye in the middle background. Note rocky lands in the foreground and extending through the left of the picture.



Class 1 land - Jarmaq Valley. Upper Nabatiye Plateau. Tobacco Field - Nonirrigated.



Upland Soil - Upper Nabatiye Plateau - Reddish Brown Clay Loam with Limestone bedrock at 75 centimeters.

SECTION XXXIII

LOWER NABATIYE UNIT

Description of the Unit

The Lower Nabatiye Unit lies on the coastal hills between the Litani River and the Zahrani River; and between about 200 meters elevation and the existing Kasmie Canal, as shown on Plate XXXIII-1. Arable lands of the unit lie in non-contiguous patches in valleys, and on steeply sloping hillsides. Very few level areas of appreciable size occur any place in the Unit. The total area of arable land which will be served is 3700 hectares.

Water for the Unit will be taken from the Zrariye Canal near the Litani River and conveyed northward in the Main Canal 39.6 kilometers to the Zahrani River. The small West Branch Canal will take off from the Main Canal at about kilometer 5, and will run west and south to serve an area just north of the Litani River. The route of the canal crosses a number of deep valleys which will require the use of siphons. One short tunnel will be required on the main canal.

Because of the irregularity of the topography of the Unit, it will be difficult to design a distribution system. Much of the arable land is on steep hillsides and will require terracing for cultivation. Other portions are in narrow valleys having steep sides. Many small, long laterals will be required to serve the separated and irregular shaped parcels of arable land.

Land Classification

A general discussion of the lands, the land use problems and land classification of the Nabatiye Plateau, which includes both the Upper and Lower Nabatiye Irrigation Units is included in Section XXXII. A discussion of the soils in the Lower Nabatiye Unit is included in the following paragraphs.

Uplands. Upland soils formed from marl or soft marly limestone are dominant in the lower plateau. On areas of level to very gently sloping topography, which make up a minority of the lower plateau, the surface soils are dark grayish-brown, granular clays having deep wide cracks when dry. Subsoils consist of dark grayish-brown clays of massive structure, extending to depths of 30 to 45 centimeters. Below this lies a very pale brown silt loam or silty clay loam mixed with marl. Soft white marl or soft marly limestone is usually encountered at depths of 60 to 90 centimeters or more. Marl or marly limestone fragments are usually present in both surface and subsoils. Occasionally small land snails (Helix Vestalis) are quite prominent, especially in the more marly soils. In some areas the dark grayish-brown clay horizon lies directly over the marl without an intervening transition zone. In other sectors a considerable amount of marl is mixed with the clay. This soil type has a range in characteristics which includes a variable soil depth over marl, varying degrees of hardness of the underlying material, and a slight variability in soil color and texture. The pH of the dark clay horizon is in the range of 7.8 to 8.0 and it is moderately calcareous. The pale brown silt loam layer has the same pH range as the latter, but is highly calcareous. The underlying marl has pH's from 8.0 to 8.4 and is highly calcareous. Soluble salts are low in all horizons ranging from a trace to 0.12%. Internal drainage of the clay horizon is probably slow due to its heavy texture, although initial infiltration rates should be good because of the granular topsoil and deep cracks. The softer marls appear to be very slowly permeable to water, with the harder marls and marly limestone being impermeable. The limited amount of water available for irrigation, together with good management practices, should preclude any drainage problems on the deeper soils. Adverse permeability conditions on the shallower soils would necessitate the judicious use of irrigation water. Areas having relatively deep soils over marl were placed in a Class 1 irrigability classification and should be well suited for any of the crops climatically adapted to the area. They are quite small, widely separated tracts. The shallower soils were classified as

2 or 3 depending on the depth to marl. They have a more restricted crop adaptability, and probably are better suited to common field crops and vegetables. The planting of citrus on these areas would not seem advisable until sufficient experimental work is accomplished to determine their suitability. The very calcareous nature of the shallow underlying marl might induce chlorosis (yellowing of the green portions of plants) in citrus. Also, the heavy texture of these soils is not as desirable for citrus as those of lighter texture.

On the more gentle slopes the soils are of two types. One consists of light yellowish-brown silt loam or silty clay loam grading into a marly pale yellow silty clay loam at a depth of about 60 centimeters. The underlying marl may be reached at varying depths, usually below 90 centimeters. The other type, which covers the largest land area, consists of rather uniform light gray silt loam, with marl being found at depths of 30 to 90 centimeters. Drainage is good on both of these types because of land slope. These areas were classified as 2 or 3 depending on the degree of slope and soil depth. They should be quite productive for a variety of field crops, and vegetables under irrigation. Some of these lands having relatively deep soils and moderate slopes have been terraced and have been placed in a Class 1 terrace classification. They are found adjacent to some of the wadis or near the coastal plain - plateau escarpment. They appear productive under dry land conditions, and under irrigation, should be well suited for any of the crops ground in the area.

On the steep uplands, which comprise the major portion of the lower plateau, the soils are a light gray to white silt loam, with a shallow depth to marl. Soil depth and degree of slope are usually closely correlated - the steeper the slope the shallower the soil. Drainage is often excessive. Many steep, hilly areas have been completely denuded of soil by erosion, leaving nothing but bare marl. An exception to this occurs on steep slopes that have been terraced, or where runoff has been retarded by native vegetative cover. On these areas the soils are quite deep. The light colored marly soils have pH's of 7.8 to 8.2, are highly calcareous, and have very low concentrations of soluble salts. They are undoubtedly very low in nitrogen. Usually in highly calcareous, light colored soils such as these, most of the available phosphorus, as well as some of the minor elements necessary for good crop production (manganese, boron, etc.) are combined chemically as calcium compounds, and are unavailable for plant growth. Whether this fertility problem exists on the Nabatiye plateau is not known. Data on field crops would indicate that the steep, marly soils are not very productive in their present state. However, some areas have been terraced and appear to be producing satisfactory crops of olives, figs and grapes under dry land condition. Areas having a reasonable soil depth, and which appear susceptible of terracing have been delineated as Class 2 - terrace land, and could be utilized for general field crops and grapes under irrigation. It should be emphasized that most of these areas are strictly marginal for crop production in their present state and considerable work and expense will be required before they are suitable for irrigation. They should be utilized, for the most part, only after water has been provided for the better lands, and should be considered as being nonarable from an irrigation standpoint unless terraced.

Steep, rough lands having very shallow soils have been included in a Class 6 - nonarable classification. Approximately 65% of the lower plateau has been included in this category. As in the upper plateau, these lands are permanently unfit for irrigated agriculture, with most of the areas not suited for dry land cultivation. They do afford a limited amount of grazing and could be improved considerably for that purpose.

Wadis. Alluvial soils in the lower plateau are found principally in the larger wadis terminating in the coastal plain; in numerous small drainage ways which dissect the plateau, and in the relatively large wadi below Ez Zrariye. Also included in this category is that portion of the coastal plain lying between the Kasmie Canal and the lower plateau escarpment. Topography is usually level to gently sloping, with drainage being apparently good, except in some of the smaller wadis during periods of heavy rainfall. The coastal wadis, and the small wadis draining the lower plateau are composed of light gray, pale brown or brown loams, silt loams and clay loams of considerable depth, with

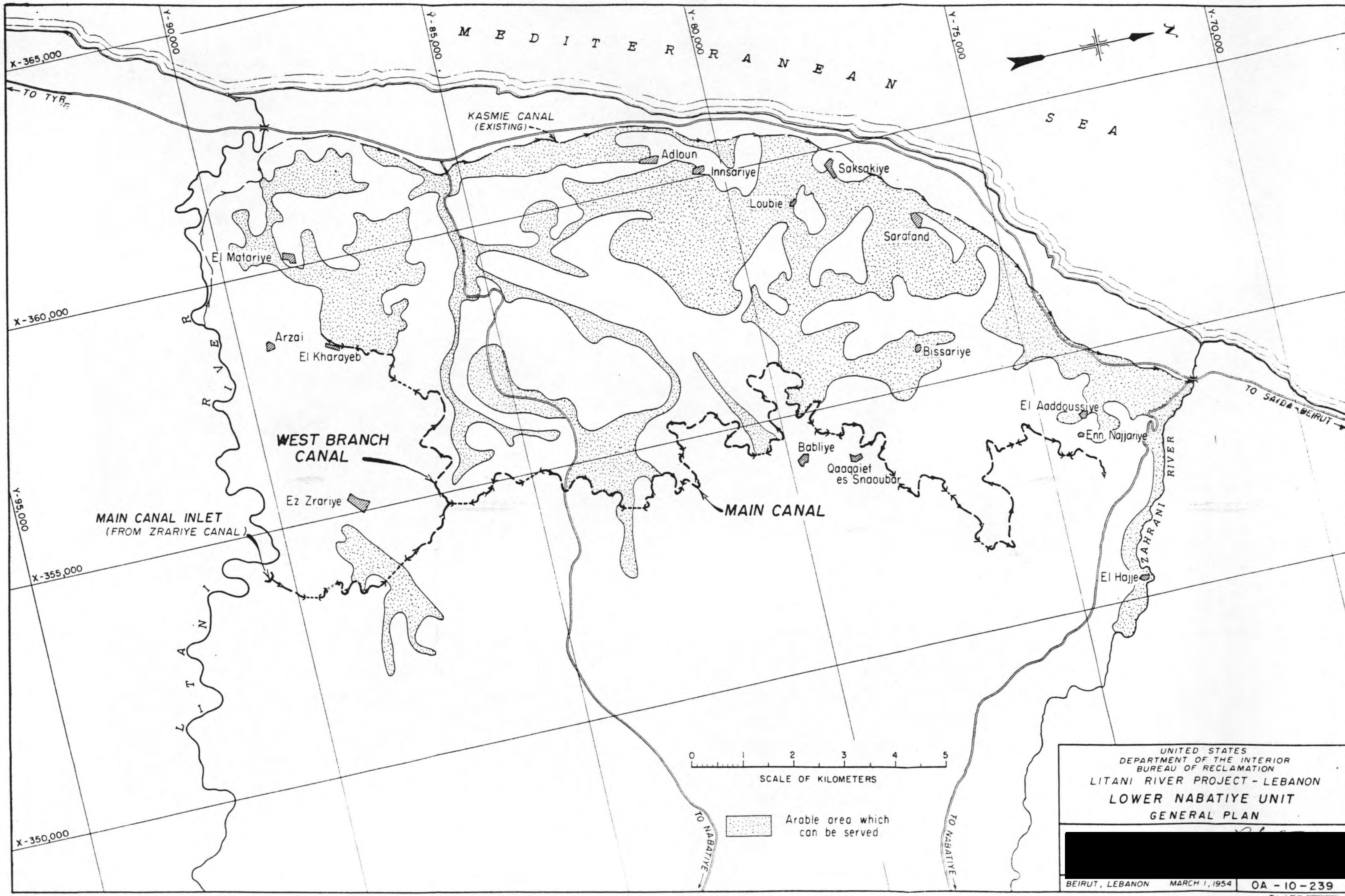


PLATE XXXIII-1

a variable stone content. Soils of the coastal wadis grade into the dark grayish-brown clays of the coastal plain. Below Ez Zrariye the soils are deep clays or clay loams of dark brown, reddish-brown or grayish-brown color. Most of the surface soils in the latter wadi are stony, with the stone content decreasing in the subsoil. The majority of the wadi soils are highly calcareous, having a rather uniform pH of 7.8 to 8.0, and are very low in soluble salts. They appear to be very productive for a wide variety of crops. Climatic conditions in the wadis of the lower plateau, especially those near the coastal plain, are favorable for the growing of bananas and citrus in addition to other types of crops.

The majority of the lands in the larger wadis, and areas of the coastal plain above the Kasmie Canal were given a Class 1 irrigability rating. Areas rated lower than Class 1 were usually due to stony soils or to sloping or uneven topography. Most of these lands are confined to the small, narrow wadis and were included in Class 2.

A tabulation of the various land classes in the lower Nabatiye Unit is contained in Table XXXIII-1. The reconnaissance land classification is shown on Plate XXXIII-2.

TABLE XXXIII-1

LAND CLASSES BY AREA (HECTARES)

Area	General Land Classes			Terrace Classes		6	Total
	1	2	3	1	2	High	Arable
Lower Nabatiye							
Lower Plateau	195	1294	1094	103	677	32	3395
Kasmie Canal to Escarpment	278	222	19	14	8	0	541
Total Area Classified	473	1516	1113	117	685	32	3936
Total Irrigable 1/	470	1497	1113	100	667	0	3847

1/ Gross area which can be reached from proposed canals. Net area in the Lower Nabatiye Unit about 3700 hectares.

Water Supply

The water for the Lower Nabatiye Unit will come from Khardale Reservoir through the Zaiye Tunnel and Canal and the Zrariye Tunnel and will be diverted into the Main Canal from the Zrariye Canal just above the inlet to the Zrariye Power Plant penstock. The water to be used for irrigation represents water stored in Karaoun Reservoir plus the additional water picked up below Karaoun Dam in Khardale Reservoir. As such, it is water which is unused at present. No problem of water rights or prior use is involved. As in the case of other units, irrigation uses will have priority over power uses and a full supply for the Unit is assured each year.

Water Requirements

Because of its lower elevation and consequent different cropping practices, the Lower Nabatiye Unit will have a somewhat different water requirement than does the Upper Nabatiye Unit. Consumptive use was computed by the Blaney-Criddle method, using a weighted consumptive use coefficient of 0.715. This coefficient was derived as follows:

Crop	Area -%	Consumptive Use Coefficient	
		Individual Crop	Weighted
Bananas	30	1.00	0.300
Citrus	35	0.50	0.175
Truck	20	0.60	0.120
Forage	15	0.80	0.120
		Weighted Average	0.715

As in the case of other units, the use of soil moisture available after the winter rains was distributed throughout the summer in such a manner as to reduce peak demands on the Main Canal and provide for a more economical supply and distribution system. Consumptive use is shown in Table XXXIII-2, and Water Requirements in Table XXXIII-3.

TABLE XXXIII-2

CONSUMPTIVE USE
LOWER NABATIYE UNIT 1/

Month	Daylight hours %	Mean Temperature °F	Consumptive Use			Precip-itation cm.	Net consumptive use - cm.
			Factor	in.	cm.		
April	8.77	64	5.6	4.0	10.1	4.4	6
May	9.68	71	6.9	4.9	12.4	1.4	11
June	9.65	64	7.2	5.1	13.0	0.2	13
July	9.84	78	7.7	5.5	14.0	0	14
Aug.	9.30	80	7.4	5.4	13.7	0	14
Sept.	8.35	78	6.5	4.6	11.7	0.4	11
Oct.	7.92	74	5.9	4.2	10.7	2.4	8

1/ Values in table also apply to Saida-Beirut Unit.

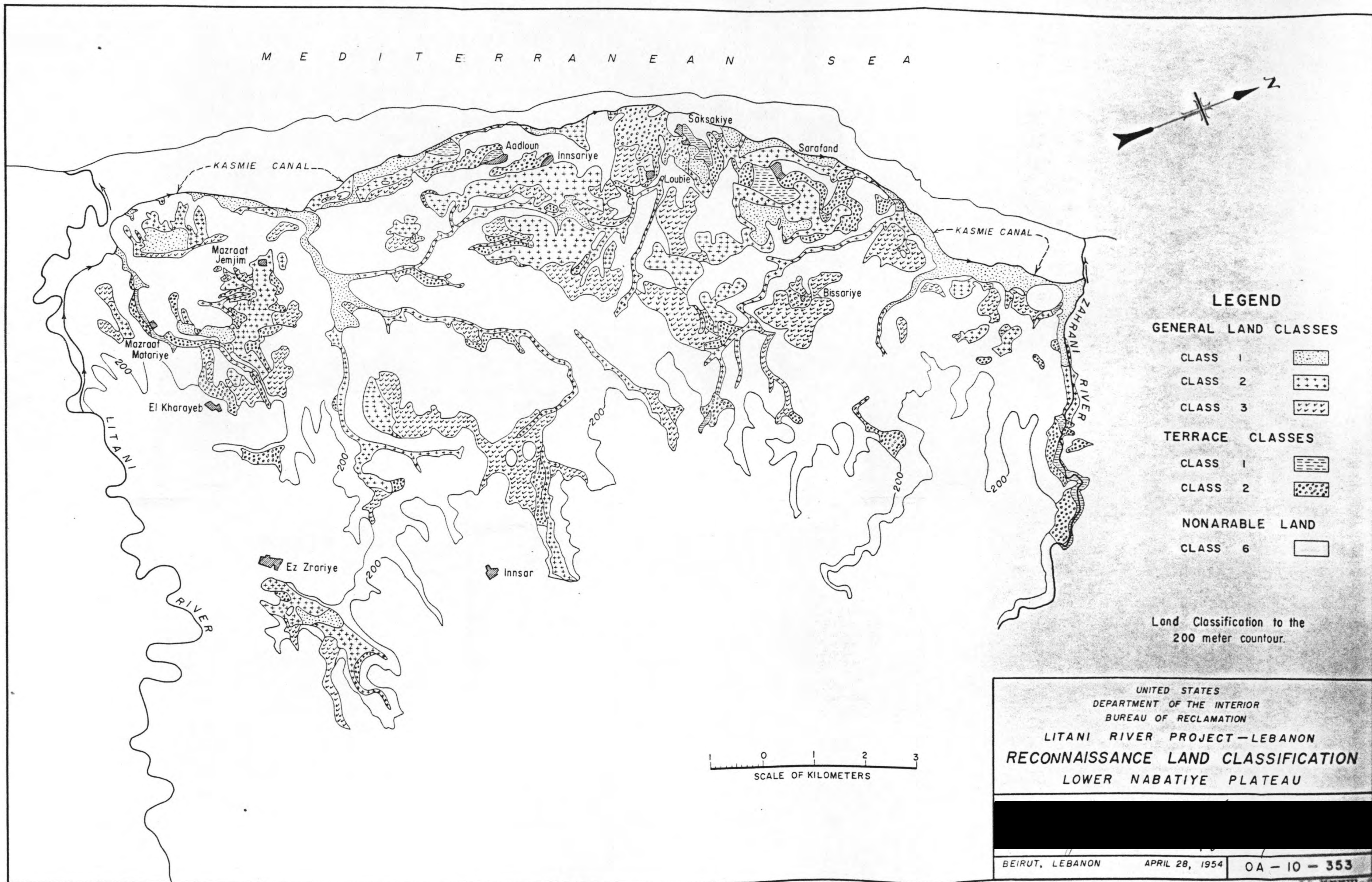


PLATE XXXIII-2

TABLE XXXIII-3

WATER REQUIREMENTS
LOWER NABATIYE UNIT

Month	Net Consumptive use - cm.	Soil Moisture use - cm.	Irrigation Requirement				
				Farm Delivery	Diversion		
			Net cm.	cm.	cm.	M^3/sec.	$M^3 \times 10^6$
April	6	0	6	8	9	1.3	3.3
May	11	1	10	13	15	2.1	5.6
June	13	3	10	13	15	2.1	5.5
July	14	4	10	13	15	2.1	5.6
Aug.	14	4	10	13	15	2.1	5.6
Sept.	11	3	8	11	13	1.8	4.8
Oct.	8	0	8	11	13	1.8	4.8

Description of Features

Main Canal. The Main Canal will begin at approximately elevation 210 meters at the Zrariye Canal just upstream from the inlet to the penstock for Zrariye Power Plant. It will have an initial capacity of 2,100 liters (2.1 meters) per second and will serve 3700 hectares of land. The canal will, for the most part, have a trapezoidal section. In those sections where the alinement is along very steep side hills, a rectangular, masonry, bench flume section will be used. At its beginning, the canal will have a bottom width and a water depth of 100 centimeters. Side slopes will be 5:4, and the slope of the canal will be .0004 throughout its length. Plastered, unreinforced concrete lining will be used on the entire canal.

About one kilometer from the beginning of the canal, a tunnel about 300 meters in length will be required through a low ridge. The tunnel will have a diameter of 170 centimeters and will be lined.

The alinement of the canal is very irregular and is marked by many curves and difficult locations. The land of the unit is cut by many deep gullies and about seventeen siphons, some quite long, will be required. Considerable rock will be encountered in excavation, especially on the steeper hillsides some of which will require the entire section to be excavated in rock. In such locations the local practice is to use a masonry bench flume section. Quite often, the uphill side of the flume can be formed by the face of the excavation itself. All masonry will be smoothly plastered.

A turnout to the West Branch Canal will be provided at kilometer 5.2. Since the capacity of the branch canal is only 370 liters per second the turnout will be similar to the ones used for laterals as shown on Plate XXI-10. Turnouts to about nine laterals will be made and the size of the canal will become progressively smaller as the capacity decreases. The total length of the canal will be 39.6 kilometers and it will terminate at a point near the Zahrani River. Minor structures along the canal will include one bridge on a primary road, one on a village road, nine turnouts ranging in capacity from about 40 to about 500 liters per second, seven checks, seventeen foot bridges, three culverts under roads, twenty five drainage culverts under the canal, and seven livestock watering ramps. Typical designs of minor structures are shown on Plates XXI- through 16.

Siphons. Major structures, other than the tunnel already mentioned, include seventeen siphons. These siphons will be constructed, whenever possible, of reinforced concrete pipe. For those of larger size which have excessive static heads, monolithic reinforced concrete barrels will be used. The sizes and lengths of the various siphons required are shown in Table XXXIII-4.

TABLE XXXIII-4

SIPHON DATA
LOWER NABATIYE UNIT

Siphon No.	Q 1/sec.	Length m.	Static Head m.	Diameter cm.
		MAIN CANAL		
1	2,100	300*	50*	110
2	2,100	650	50	110
3	1,650	475	100	100
4	1,650	140	43	100
5	1,650	440	68	100
6	1,300	425	42	90
7	1,300	45	21	90
8	1,300	30	12	90
9	1,300	280	31	90
10	300	200	25	40
11	300	230	46	40
12	300	240*	60*	40
13	300	185	60	40
14	300	265	80	40
15	100	145	44	30
16	100	130	12	30
17	100	185	55	30
		WEST BRANCH CANAL		
1	300	530	103	40
2	150	200	31	30
3	150	270	40	30

* Approximate. Actual site not surveyed.

West Branch Canal. Although the West Branch Canal will probably have a smaller capacity than some of the laterals from the Main Canal, its route was surveyed in order to determine the amount of land which could be served. The location and land classification surveys revealed that the canal would be smaller than originally anticipated. For the purpose of this report, however, its designation as a canal has been retained.

It will begin at kilometer 5.2 of the main canal, and will flow westward for 7.1 kilometers, terminating at the village of El Khorayeb. It will be lined and will have an initial capacity of 370 liters per second and a terminal capacity of 100 liters per second. Three laterals will be served along its route. Minor structures will include three turn-outs, two checks, four footbridges, three culverts under roads, three drainage culverts under the canal and two livestock watering ramps. Typical designs of these are shown on Plates XXI-1 through 16. Three siphons will also be required. Data regarding the size, length, and head for the siphons are shown in Table XXXIII-4.

Distribution System

The distribution system for the Lower Nabatiye Unit will be the most difficult to layout and construct of any of the units. As pointed out previously, the arable land lies in relatively small, often non-contiguous patches on steeply sloping hillsides and in narrow deep valleys. Very little of it lies well for irrigation, and the layout of the distribution system will present a real problem. It will have to be laid out on a basis of topography only, with little or no regard for property and village boundaries. It will involve a multitude of small, long sub-laterals employing considerable amounts of pipe and numerous drops. No pattern for the system can be established as each area will have to be served in whatever manner the topography permits. Since a great deal of the land to be irrigated will be terraced, the construction of the terraces themselves must be planned along with the distribution system in order to work out the most efficient method of actually getting water onto the land. Because of their great length, all laterals, down to the smallest, will be lined in order to hold transmission losses and the size of ditches to a minimum.

Drainage System

No comprehensive drainage system will be required for the unit. Since the land to be irrigated lies in relatively small patches and usually has a steep slope no drainage problem should ever develop with the possible exception of a few localized areas in valley bottoms. These may need light treatment, but nothing more than shallow surface drains.

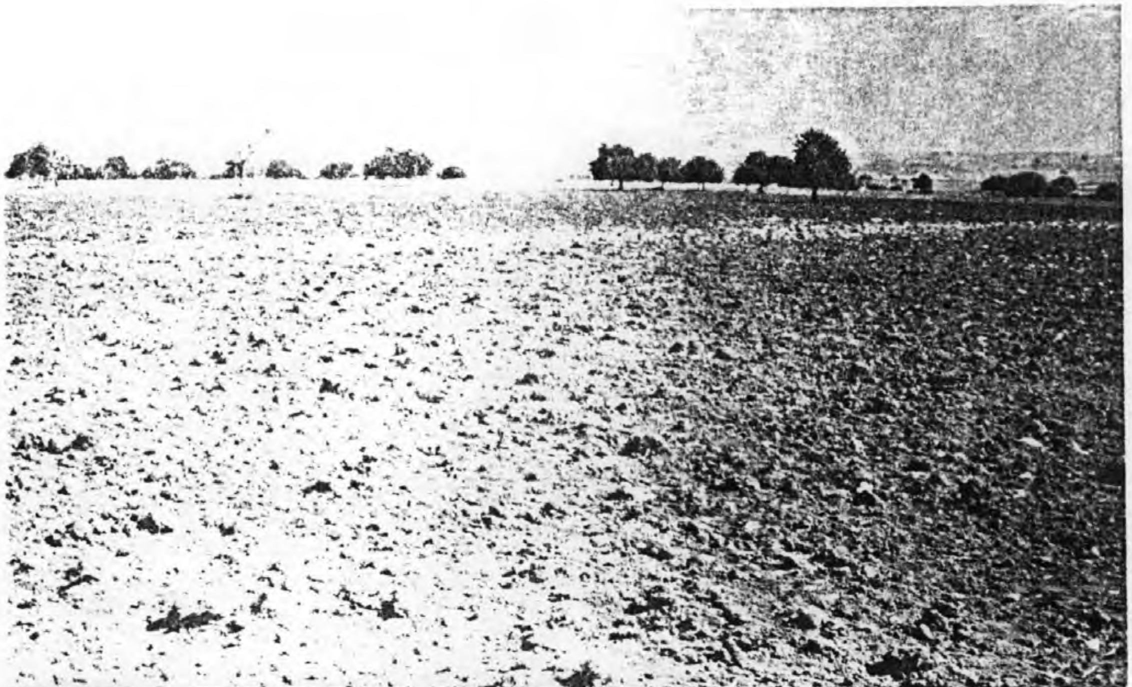
Cost Estimate

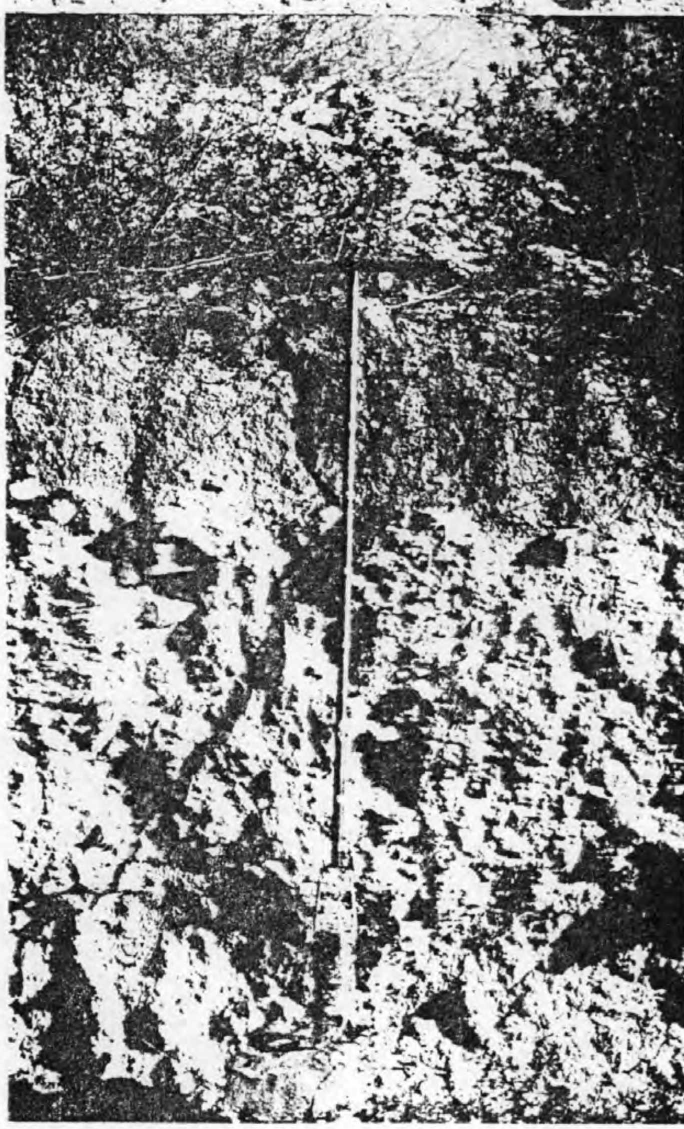
The estimate of cost for the unit is shown in Table XXXVII-21. Unit costs used are the same as those used for the Bekaa Gravity Unit, Volume II, and are based on local costs for similar work. A summary of the cost estimate is as follows:

Main Canal	3,409,000 LL.
West Branch Canal	331,000
Distribution System	2,553,000
Drainage System	70,000
Subtotal	6,363,000
Engineering and Overhead	637,000
TOTAL	7,000,000

General View - Lower Nabatiye Plateau, showing the hilly topography and marly soils that comprises much of this area.

Class 1 Wadi or Valley Land - Lower Nabatiye Plateau.





Smooth upland Area - Lower Nabatiye Plateau. This land was placed in class 2 because of a comparatively shallow depth to marl.

Typical soil type found on the smooth uplands in the Lower Nabatiye Plateau - Dark grayish brown clay over marl.

Class 2 Terrace Land - Lower Nabatiye Plateau. Class 6 land in center and left background.

Class 6 nonirrigable land in the Lower Nabatiye Plateau being cleared for dry land cultivation. The rock is marly limestone.

Class 6 Land - Lower Nabatiye Plateau - Much of the land in both the Upper and Lower Plateau is unfit for cultivation and better suited for grazing.

SECTION XXXIV

SAIDA-BEIRUT UNIT

Description of the Unit

The Saida-Beirut Unit lies along the seacoast between the Awali (Bisri) River just north of Saida and the Beirut River, east of Beirut. The Main Canal will be located along the 200 meter contour for the first half of its length and a somewhat lower elevation along the last portion. An area of 3,900 hectares of arable land lying between the canal and the sea will be served by the Main Canal as shown on Plate XXXIV-1. Water from Joun Tunnel will be diverted to the Main Canal at the inlet of the Joun Power Plant penstock, and will flow northward along the western slope of the coastal hills for 55 kilometers to the Beirut River. Numerous laterals will be served along the route of the canal to serve the several separated areas of arable land. The canal will cross about twenty stream valleys where siphons, some quite large, will be required.

Land Classification

The Saida-Beirut area is part of the narrow coastal plain that characterizes much of western Lebanon. In general, the principal physiographic features consists of a sandy or dune covered coastal strip, and gently sloping coalesced alluvial fans which increase in gradient and terminate in rocky, often precipitous hills or mountains. In many sectors the hills rise abruptly from the sea and the coastal plain is nonexistent.

Due to the favorable climatic conditions, a great variety of both irrigated and dry land crops are grown, ranging from temperate to subtropical types.

Soils and Land Classes. Many sources of parent material have contributed to the formation of the soils of the Saida-Beirut area varying from the wind blown sands of the coastal region to the limestones, sandstones, and marls comprising the hills and mountains. The major source, however, has been alluvium washed from the latter areas. In general, a cross section of the wider portion of the coastal plain as typified by the south Beirut area would include the following sequence of soils - a coastal strip of beach sand or dunes; a flat to very gently sloping belt of reddish-brown loamy sands or sandy loams; gently to moderately sloping coalesced alluvial fans consisting of an association of reddish-brown and dark brown clay loams; and adjoining steeper hilly lands made up largely of shallow, stony, reddish-brown clay loams developed from limestone, or to smaller extent grayish silt loams formed from marl. These areas terminate in rocky hills or mountains. Within this sequence occur diversified soil patterns with variations in color, texture and depth. The majority of the soils are moderately to highly calcareous. Of the samples analyzed, the pH range was from 7.3 to 8.5, with most of the soils being around 7.7. Total soluble salts are very low ranging from a trace to 0.11%.

Five classes of irrigable land were mapped in the Saida-Beirut area - Classes 1 and 2 arable lands under the general standards; Classes 1 and 2 arable under terrace standards; and Class 6 nonarable land. Class 3 land under the general standards was not included due to the preliminary nature of the survey, and the relatively small extent of this type. Class 2 (General), therefore, includes some lands of marginal quality for irrigation that would have been delineated as Class 3. A description of the land classes will be found in the section on land classification - Volume I. Classification standards are presented in Table XXXIV-1. While the survey was concerned primarily in evaluating the rocky, uncultivated uplands which might be brought into production, an appraisal of the presently irrigated lands was also included. A brief discussion of the various land classes by physiographic position is given in the following paragraphs.

Lowlands. This region includes the flat to moderately sloping lands adjacent to, and in some areas extending a considerable distance from the coast. It comprises about 53% of the arable land classified in the survey, including almost all of the Class 1

TABLE XXXIV-1

RECONNAISSANCE LAND CLASSIFICATION STANDARDS
SAIDA - BEIRUT AREA
GENERAL STANDARDS

Land Characteristics	Class 1 - Arable	Class 2 - Arable
Soils		
Texture	Sandy loams to slowly permeable clays	Sands with fines (5% plus) to very slowly permeable clays.
Depth		
To limestone bedrock	120 cm. plus	60 cm. plus
To Firm Marl (penetrable)	90 cm. plus	45 cm. plus
To very soft penetrable marl or raw marly soil	60 cm. plus	30 cm. plus
To clean sand, gravel or cobble 1/	60 cm. plus	30 cm. plus
Alkalinity (pH) 2/	8.6 or less	9.0 or less
Salinity (Total salts) 2/	0.2% or less	0.2 to 0.6%
Topography		
Slopes 3/	Uniform slopes to 2%	Uniform slopes to 2%
Surface	Even enough to require only small amount of leveling or grading	Moderate to heavy grading may be required
Cover (Rocks)	Loose rock may be present, clearing cost small.	Present in sufficient amounts to require moderate to high clearing costs
Drainage	No drainage problem anticipated.	Drainage may be required at moderate to high costs

Class 6 - Nonarable. Includes lands considered permanently nonarable for irrigation

1/ At the minimum depth the overlying soil should be a sandy loam or heavier.

2/ May be slightly higher in open permeable soils and under good drainage conditions.

3/ In this area slopes in excess of 2% are normally terraced.

TABLE XXXIV-1 (Continued)

TERRACE STANDARDS

Land Characteristics	Class - Arable	Class 2 - Arable
Soils		
Texture	Sandy loams to slowly permeable clays	Sands with fines (5% plus) to very slowly permeable clays
Depth		
To limestone bedrock 1/	90 cm. plus	45 cm. plus
To firm marl (penetrable) 1/	60 cm. plus	30 cm. plus
To very soft penetrable marl or raw marly soil 1/	45 cm. plus	--
To clean sand, gravel or cobble 1/ 2/	60 cm. plus	30 cm. plus
Alkalinity 3/	pH 8.6 or less	9 or less
Salinity 3/	Total salts not to exceed 0.2%	Total salts not to exceed 0.6%
Topography		
Slopes 4/	Uniform slopes to 20%	Uniform slopes in excess of 20%
Cover	Loose rock may be present; should not exceed amount required to build terrace walls	Loose rock present in excess of amount to build terrace walls. Moderate to expensive clearing costs.
Drainage	No drainage problems anticipated	

1/ Average depth of unterraced land or minimum depth of terraced land.

2/ At the minimum depth the overlying soil should be a sandy loam or heavier.

3/ May be higher in open permeable soils and under good drainage conditions.

4/ Soil depth will normally determine the upper slope limit. Almost all class 1 lands in this category, in the Saida-Beirut area are presently terraced.

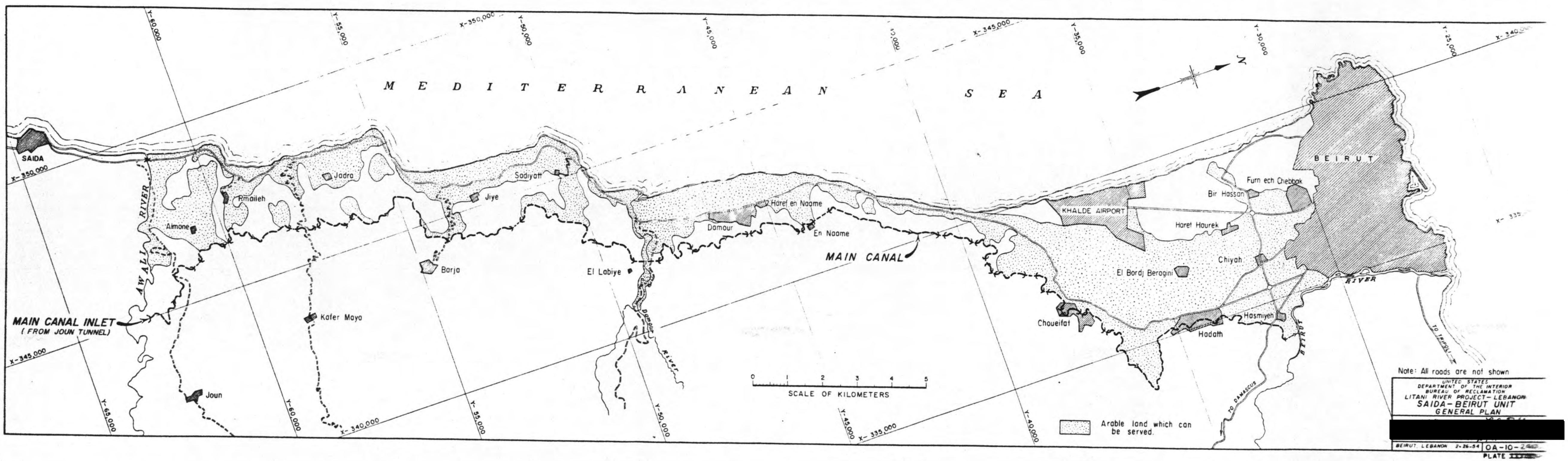
and 2 lands under the general standards. Approximately 30% of this total, although surveyed, should be excluded from future project plans since it apparently is receiving an adequate supply of irrigation water at the present time. Excellent crops of bananas, citrus and vegetables are being produced on these lands. The flat or very gently sloping areas having deep medium textured soils were given a Class 1 (General) irrigability rating. Lighter textured soils (loamy sands and sands with fines) lying adjacent to dunes or beach sands have been included in Class 2 (General). These soils have a very high water consumption under irrigation due to their sandy nature. They are subject to wind erosion, and in some areas are exposed to the deposition of sand from adjacent dunes. In the vicinity of Beirut much of this land type is covered with pine trees, while further south plantings of citrus, vegetables and olives appear to be producing reasonably well.

The gentle to moderately sloping lands comprise the majority of the better arable lands of the coastal area. The soils are dark brown or reddish-brown loams and clay loams of considerable depth. Almost all of these areas are terraced, and have been included in a Class 1 terrace category. About 35% of the lands in this class are irrigated. It is interesting to note that even the gentle slopes of the coastal plain have been terraced. At the lower slope limits these lands are of comparable quality to the Class 1 lands in the general class. Extensive olive groves are located on the dry lands of this type south of Beirut, with bananas and citrus predominating on the irrigated areas near Damour. According to some land owners in the former area, the provision of water for irrigation would cause a shift in the cropping system from olives to citrus or bananas. Small tracts of nonterraced land in this class include areas of rocky, but deep soils, where the existing rock may be utilized to build terrace walls. Nonirrigated Class 1 terrace lands should be very productive for any of the climatically adapted crops with the addition of irrigation water.

Uplands. Included in this category are the moderately sloping to steep lands lying above the coastal plain proper. They are characterized by an association of terraced lands; areas of relatively shallow, rocky, stony soils; and extremely rocky, nonarable lands. The better lands of this type have been rated as Class 2 (Terrace). This class comprises about 65% of the nonirrigated lands in the Saida-Beirut area susceptible of irrigation. It should be emphasized that these lands are extremely marginal for crop production in their present state. Considerable work and expense involving the removal of rocks and the construction of terraces will have to be undertaken before they can be made suitable for irrigation. Unless terraced, most of these areas should be considered as being nonarable from an irrigability standpoint.

On areas already terraced the limitations are usually due to soils of shallow depth over bedrock, or those having steep slopes with narrow field boundaries and high terrace walls which would be more difficult to irrigate. Most of the cultivated, terraced lands in this class are found south of Beirut and consist of rather extensive olive groves.

The deficiencies on uncultivated or semi-cultivated lands are due largely to relatively shallow, very stony soils, and moderately steep slopes. Usually from 30 to 80% of the land surface is covered by stones and rocks, 50% of which are approximately 5 to 15 centimeters in diameter, and 50% being 1/2 to 1 meter in diameter. Rock outcrop or immobile rock may comprise 10 - 20% of the latter figure. This degree of stoniness usually persists below the land surface as well. The soils are reddish-brown, very stony - rocky friable clay loams. Surface soils may be a dark reddish-brown where slight organic accumulation has occurred, but normally the soil profile is quite uniform in appearance. Depth to bedrock is extremely variable, ranging from a few centimeters to a meter or more. The parent limestone is usually weathered into individual blocks and fragments. In some areas alternate layers of hard and soft limestone are found near the surface. Weathering of the softer strata has often resulted in soil formation underneath the harder surface or subsurface formations. The soils are slightly calcareous, have a rather uniform pH of 7.8, and are very low in soluble salts. Drainage is good due to the friable stony nature of the soils, the fractured condition of the underlying limestone, and the land slope. Pedologically these soils could be included in the Terra Roassa soil group. A few localized areas are made up of shallow to fairly deep gray silt loams developed from Senonian marls.


MEDITERRANEAN SEA
N
SAIDA
Jadra
Sadiyatt
Rmaileh
Jiye
Aimone
AWALI RIVER
Barja
El Labiye
Kafer Mayo
Joun
MAIN CANAL INLET
(FROM JOUN TUNNEL)
Damour
Haret en Naame
En Naame
DAMOUR RIVER
MAIN CANAL
KHALDE AIRPORT
Bir Hassan
Furn ech Chebbak
BEIRUT
Haret Hourek
Chiyah
El Bordj Beragini
Choueifat
Hadath
Hasmiyeh
BEIRUT RIVER
TO TRIPOLI
TO DAMASCUS
X-350,000
X-345,000
X-340,000
X-335,000
Y-65,000
Y-60,000
Y-55,000
Y-50,000
Y-45,000
Y-40,000
Y-35,000
Y-30,000
Y-25,000
0 1 2 3 4 5
SCALE OF KILOMETERS
Arable land which can be served.
Note: All roads are not shown
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LITANI RIVER PROJECT – LEBANON
SAIDA – BEIRUT UNIT
GENERAL PLAN
[redacted]
BEIRUT, LEBANON 2-26-54
OA-10-2
PLATE

These nonterraced areas, for the most part, lie in sizeable bodies, and represent units which might be brought into production if irrigation water is made available. At the present time, small, scattered areas are being developed through the construction of stone wall terraces, and are producing fair to good yields of wheat, barley, grapes, and other crops under dry land conditions. Vegetables are grown where a water supply is available. On some of the areas having a shallow depth to bedrock, soil is brought in from other localities and spread to the depth desired. From a purely physical standpoint present evidence would indicate that the uncultivated areas classified as Class 2 (Terrace) are susceptible of reclamation and capable of producing good crop yields under irrigation. The development process, which is done manually, is expensive, laborious, and slow. Recent experiments involving the use of heavy equipment for terrace construction have been carried out by the Agriculture Group of the U.S. Foreign Operations Mission to Lebanon. In the Mt. Sanneen district it was found that by using a tractor with an angledozer blade, and a ripper for subsoiling and ploughing, the cost of preparing terraces on steep (25% slope) virgin land could be reduced by 55% over manual operations. On steep, cultivated land, costs were reduced by 35%. Furthermore, the amount of land that could be terraced and placed in production in one year with heavy machinery would require three years with experienced hand labor. Similar experimental work might well be tried on lands in the Saida-Beirut area.

A tabulation of the land classes by geographic area is given in Table XXXIV-2. The reconnaissance land classification is shown on Plate XXXIV-2.

TABLE XXXIV-2

SAIDA - BEIRUT UNIT
LAND AREAS BY CLASS (HECTARES)

Area	General Class 1	General Class 2	Terrace Class 1	Terrace Class 2	Total Arable	Total presently irrigated
South Beirut	426	340	1289	999	3054	1011
Khalde	21	40	6	243	310	4
Damour	100	50	290	213	653	395
Sadiyatt - Jiye - Jadra			118	725	843	70
Rmaileh - Aalmane	50	10	188	392	640	150
Total Area Classified	597	440	1891	2572	5500	1630
Total Irrigable 1/	10	270	1100	2572	3952	0

1/ Area proposed for irrigation from project canals.

Water Supply

Water from Karaoun Reservoir diverted through Bisri Tunnel and stored again, along with the flow of the Bisri River in Bisri Reservoir will pass through the Awali Tunnel and Power Plant, and then the Joun Tunnel to serve the Joun Power Plant. At the entrance to the Joun Power Plant penstock, 2.2 meters (2,200 liters) per second will be diverted to the Main Canal of the Saida-Beirut Unit. The water represents winter runoff of both the Litani and the Bisri Rivers, plus the summer flows. For the most part, it represents presently unused water. There is some irrigation at present along the Awali River in its lowest reaches, but practically all of the presently irrigated land on the north side of the river has been included in the Unit and will be served by the Main Canal.

All of the irrigated land on the south side of the river will be served with Litani River water by the Kasmie Canal now under construction. There will remain only an insignificant amount of land which will have rights to Awali River Water and for which releases from Bisri Reservoir will be required.

The Unit will have a full supply of water each year since irrigation uses will receive preference over power uses. The quality of irrigation water is excellent and contains no dissolved or suspended matter in sufficient amounts to harm crops or canals.

Water Requirements

Because it lies at about the same elevation and will grow similar crops, the consumptive use for the Unit will be identical to that of the Lower Nabatiye Unit discussed in Section XXXIII. Consumptive use for both units is shown in Table XXXIII-2. The water requirement for the Saida-Beirut Unit is shown in Table XXXIV-3.

TABLE XXXIV-3

WATER REQUIREMENT
SAIDA-BEIRUT UNIT

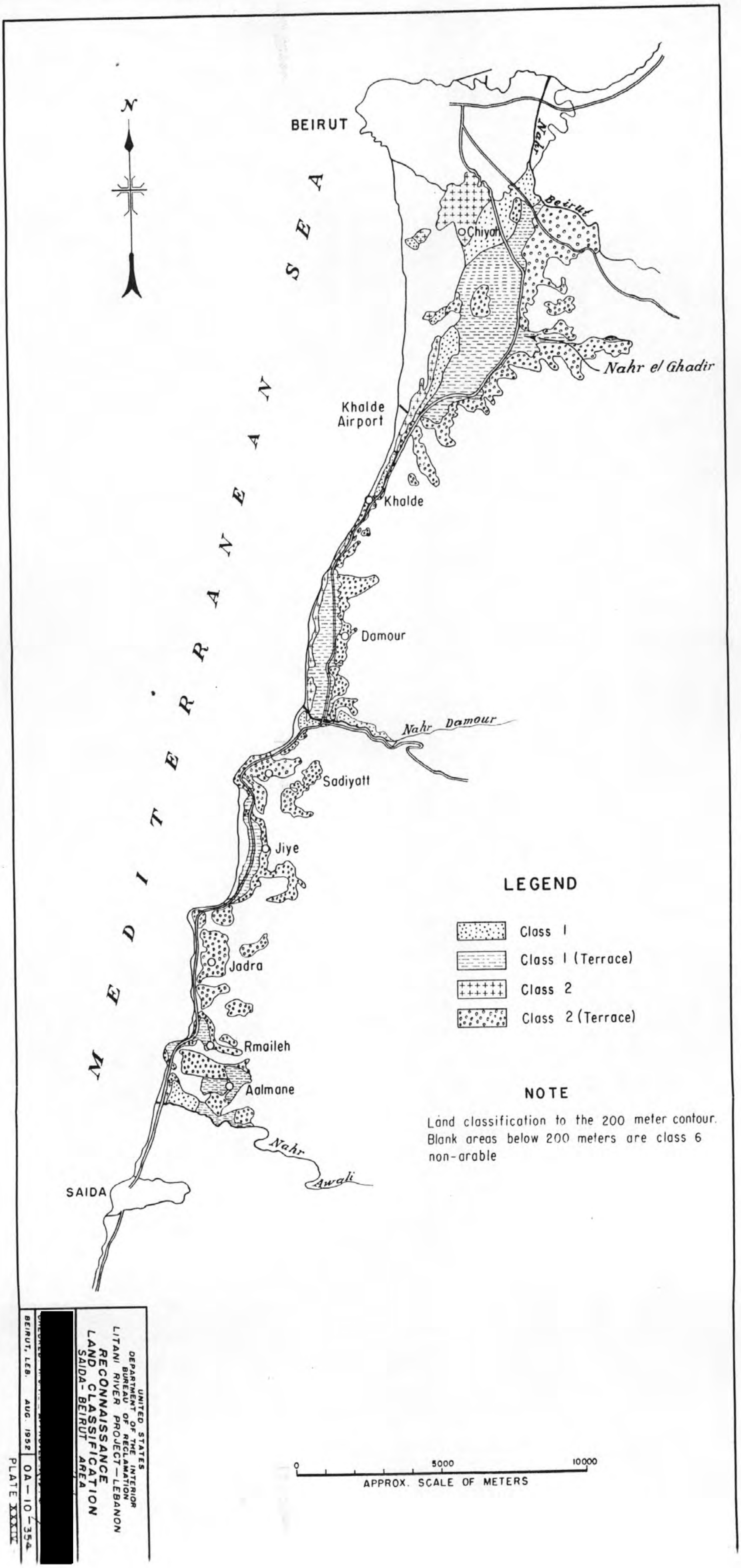
Month	Net Consumptive Use - Cm. 1/	Soil Moisture Use - Cm	Irrigation Requirement Net Cm.	Farm Delivery, Cm.	Diversion Cm.	M^3/sec.	$m^3 \times 10^6$
April	6	0	6	8	9	1.4	3.5
May	11	1	10	13	15	2.2	5.9
June	13	3	10	13	15	2.2	5.8
July	14	4	10	13	15	2.2	5.9
Aug.	14	4	10	13	15	2.2	5.9
Sept.	11	3	8	11	13	1.9	5.0
Oct.	8	0	8	11	13	1.9	5.0

1/ From Table XXXIII-2

Description of Features

Main Canal Inlet. Special consideration will be required for the inlet to the canal because of the use of Joun Tunnel water for both irrigation and power. Joun Tunnel is designed as a pressure tunnel and it will have a maximum static head of about 35 meters at its outlet which is at elevation 173.44 meters. In order to keep the Main Canal as high as possible, advantage will be taken of this head to raise the irrigation water to the canal elevation of about 187 meters. The inlet will therefore require a short length of steel pipe and possibly a small storage reservoir or tank.

Main Canal. The Main Canal will have an initial capacity of 2,200 liters per second and will serve 3,900 hectares of land. With a uniform slope of .0004, the trapezoidal canal will have a bottom width of 100 centimeters and a water depth of 102



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RECONNAISSANCE
LAND CLASSIFICATION
SAIDA - BEIRUT AREA

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BEIRUT, LEB. AUG. 1952 | OA — 10 — 354

PLATE XXXIV

centimeters at its start. It will be lined with plastered concrete with no reinforcing. The alinement of the canal is quite irregular with many curves and short tangents. All will be sidehill location with mild cross slopes, averaging about 15%, prevailing. Maximum cross slopes of about 50% occur for only very short distances, so no bench flumes are likely to be required. Considerable rock will be encountered in excavation of the canal prism. Banks with two meters top widths will be provided on both sides of the canal.

The canal will pass through both the villages of Choueifat and Hadath. In order to keep down the width of the canal through these settled areas and to facilitate crossings, a rectangular masonry canal section will be used through Choueifat and the same or pipe, through Hadath. The capacity of the canal will be about 850 liters per second at Choueifat and about 200 liters per second at Hadath.

The total length of the canal will be 55.2 kilometers, the beginning elevation will be 187 meters and the elevation at the end will be 85 meters. Unfortunately, the larger portion of irrigable land lies near Beirut toward the far end of the canal. This will require the capacity of the canal to be quite large for the major portion of its length. At kilometer 40 the capacity of the canal will still be 1,150 liters per second, or half of the original capacity.

The unit lies in an area which is more populated and more developed than those for other units. Right-of-way will be more costly and more difficult to obtain. Relocation of existing property will also become an important consideration.

Major structures on the canal will include a number of siphons which are discussed separately, nine bridges on primary roads, and two drops. Minor structures will include twenty bridges on village roads and streets, eighteen turnouts and fifteen checks, forty-four footbridges, thirty-five-culverts of less than 500 liters per second capacity under roads, twenty drainage culverts under the canal, and ten livestock watering ramps. Typical designs of minor structures are shown on Plates XXI-9 through 16.

Siphons. The largest and most costly structures on the Main Canal will be siphons. Ranging in length from about 635 meters to about 65 meters, twenty one siphons will be required for crossing streams and ravines. The largest will be across the Damour River at about kilometer 19. This siphon will be 635 meters long, and it will have a static head of 130 meters, and its capacity will be 1,150 liters per second. Fortunately, the canal will drop to a lower elevation at about this point and advantage can be taken of the extra available head by using a small siphon barrel of 60 centimeters. Even so, drops totalling 32 meters will be required on the canal just above the siphon inlet. The Damour River siphon will have a steel pipe barrel and others will have precast reinforced concrete pipe barrels except for those where the head is too great. Monolithic reinforced concrete or steep pipe barrels will be used in those instances or at least for that portion subjected to the higher pressures.

The lengths, sizes and heads on the siphons are listed in Table XXXIV-4.

TABLE XXXIV-4

SIPHON DATA
SAIDA-BEIRUT UNIT

Siphon No.	Q 1/sec.	Length m.	Static Head m.	Diam. cm.
1	2,200	310	125	120
2	2,000	220	81	120
3	2,000	195	40	110
4	2,000	95	26	110
5	1,800	235	95	110
6	1,800	350	97	110
7	1,500	635	130	60
8	1,250	85	28	90
9	1,250	95	31	90
10	1,250	85	24	90
11	1,150	525	73	90
12	1,150	140	44	90
13	1,150	170	43	90
14	1,150	130	43	90
15	1,150	130	33	90
16	1,000	100	25	80
17	1,000	305	27	80
18	850	110	27	80
19	850	75	16	80
20	850	65	17	80
21	850	90	24	80

Distribution System

The distribution system for Saida-Beirut Unit will be relatively simple even though the area is rough and steep. Because the unit is a long narrow strip, a large number of turnouts from the Main Canal will be required. These will serve the main laterals which will run down the slope toward the sea and will branch into sublaterals running parallel to the Main Canal. A number of drops will be required on most of the main laterals. The north end of the unit is much flatter and more uniform than the south and will readily lend itself to the normal type of distribution system.

It will be possible in most cases to provide separate distribution units for each village although in some cases the irrigable area of land of a village is so small that it would not be economical or desirable to provide an individual turnout and distribution system for that village. No particular problem should be encountered in serving these small areas along with the larger area of another village.

Considerable steep terracing will be required in the south and central portions of the unit. The construction of the terraces or the modification of existing ones should be carried out along with the construction of the distribution system so that the most efficient system can be designed.

Drainage System

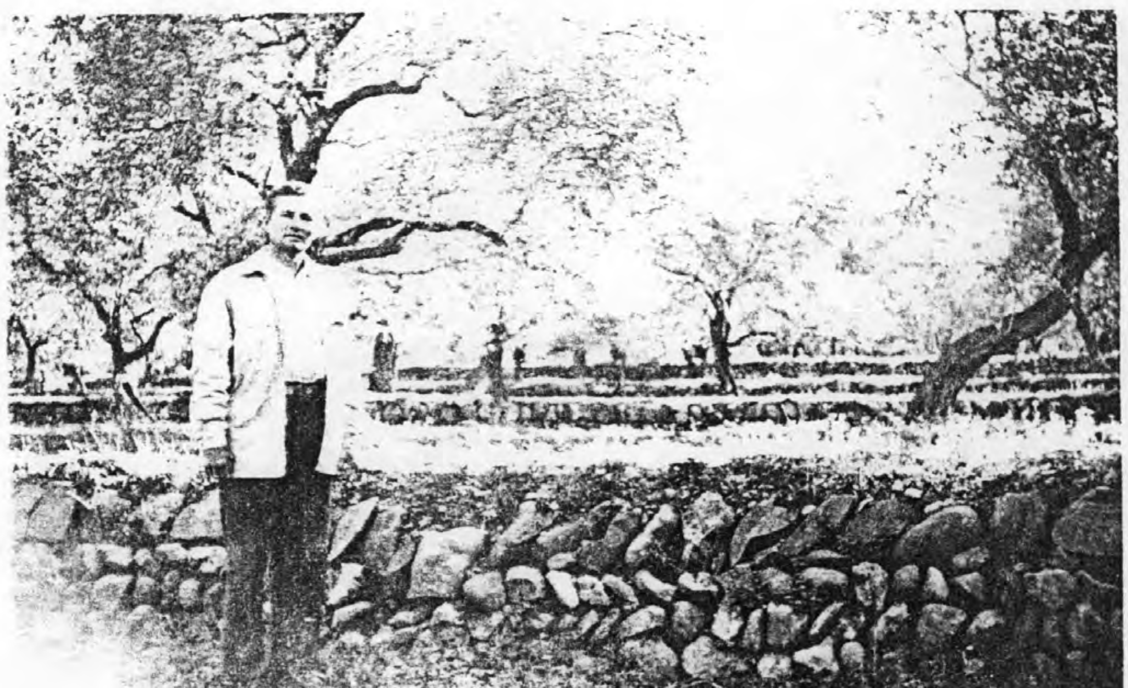
As in the case of other units, drainage on the unit presents no particular problem. In the areas of the unit which are irrigated at present, no drainage problem has occurred as a result of irrigation. In a few localized areas such as valley bottoms and the flats along the coast, some drainage treatment may be necessary. On the steeper hillsides and terraced areas, no treatment whatsoever should be required. At most, a few shallow surface drains should suffice for any of the places that may need treatment.

Cost Estimate

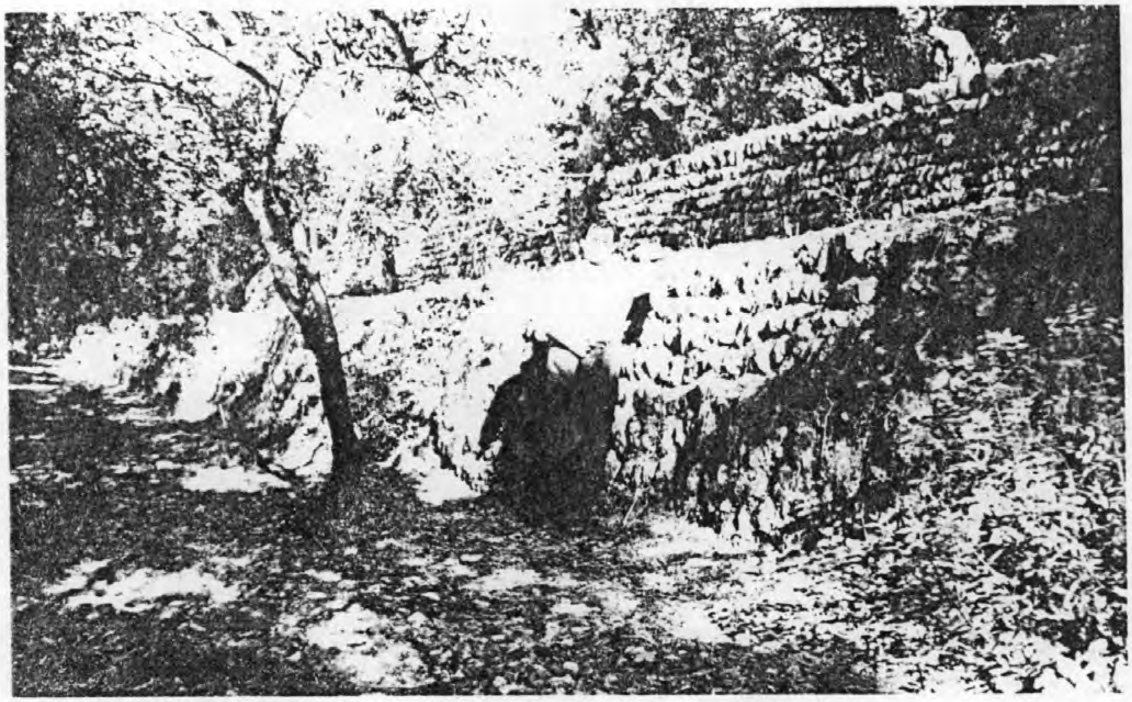
The cost estimate for the Unit is shown on Table XXXVII-22. Unit prices are those used for the Bekaa Gravity unit, and are based on local costs for work of a similar nature. A summary of the cost estimate in Lebanese pounds follows:

Main Canal, including structures	4,489,000
Distribution System	1,346,000
Drainage System	69,000
Subtotal	5,904,000
Engineering and Overhead (10%)	596,000
TOTAL	6,500,000

Class 1 Land (General) in the South Beirut Area. Young orange trees in the foreground.

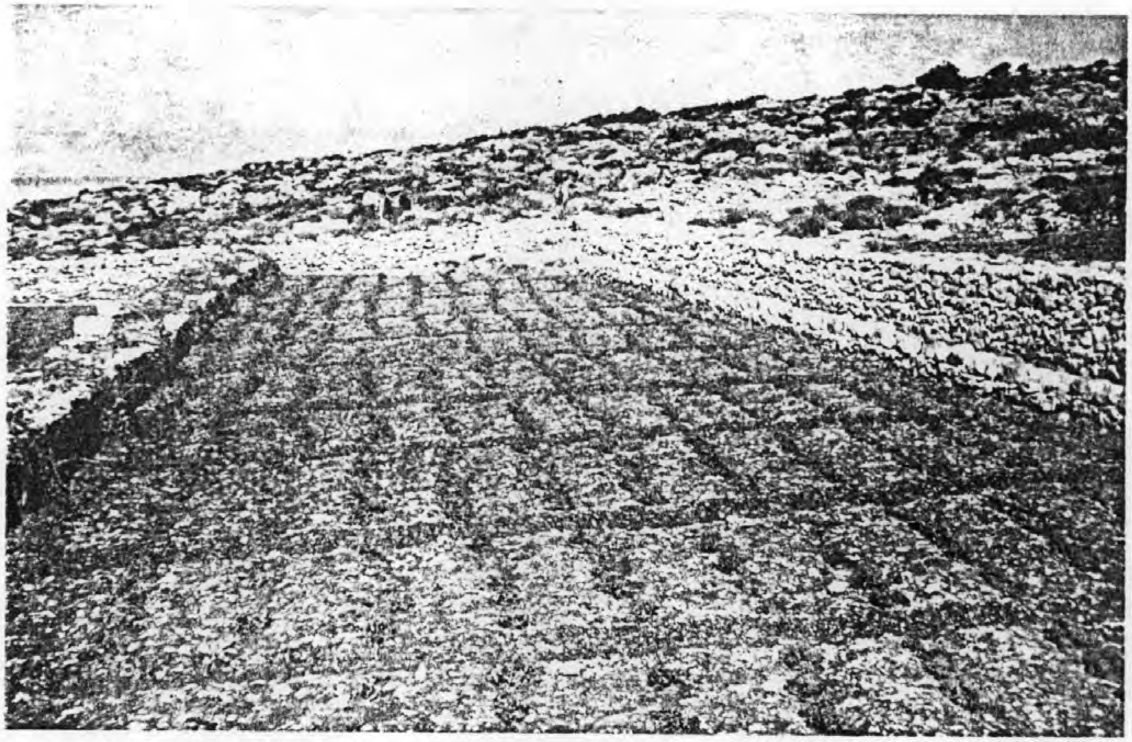


Olive grove on Class 1 Land (Terrace). The land slope is 4%. (South Beirut Area)



Olive grove on Class 2 Land (Terrace). 25% slope.
(South Beirut Area)

Class 2 Land (Terrace) showing uncultivated and Terraced Areas.
(Sadiyatt Area)



Young tomato plants on Class 2 Land (Terrace). Nonterraced (Class 2) and Class 6 Lands in the background. Near Jadra.

Clay soil, beach sand and animal manure to be mixed and spread over shallow surface soil. Class 2 Land (Terrace) near Sadiyatt.

SECTION XXXV

PROJECT OPERATION

Project operation studies have been made to determine the best use of the available water for the multiple purposes included in the plan of development. These operation studies were based upon the assumption that the flows determined for the 31-year period from January 1921 to December 1951 and shown in Appendix III of this report would recur in the same amounts and in the same sequence. They were also made for à hypothetical average year whose flows were assumed to be the same in each calendar month as the 31-year average flow for that month.

Critical Stream Flow Period

The distribution of annual flows of the Litani River at Karaoun gage during the 31-year period (1921-1951) selected for stream flow analysis is shown on Plate XXXV-1. This plate shows that low annual flows occurred in 1925, 1930, 1934, 1936, 1937, and 1951. However, 1925 was followed by the third highest year in the period; 1930 and 1934 by years that were above average; and 1951 by a year above average although it is outside the period considered. The period 1932, 1933 and 1934 constitute a 3-year period when the total for each year is far below average and 1933 is the lowest year in the 31-year period. However, this period is followed by 1935 whose total is approximately equal to the average of the whole period, and whose stream flow was sufficient to fill all the reservoirs considered before the lesser 2-year low water period of 1936-1937 occurred. Therefore the low water period of 1932-1934 was found to be the critical period for reservoir operation.

Hypothetical Average Year

Plate XXXV-1 indicates that the annual stream flow for the calendar year 1935 is approximately equal to the average of the annual flows for the 31-year period. However, comparison of the monthly flow distribution in this year, with the average monthly flow distribution over the period, showed several wide variations, particularly at some of the locations considered. It appeared therefore that a hypothetical average year, made up of the 31-year monthly average flows would generally be a more representative average year. The average monthly flows contained in Appendix III were adopted as the monthly flows, composing this hypothetical average year. All operation studies for the hypothetical average year were made, using these average year flows.

Reservoir Evaporation Losses

Reservoir evaporation losses were determined from Piche evaporimeter records which were adjusted to show reservoir surface evaporation. The method for making such adjustments is discussed in Appendix III. The evaporation losses determined for the complete project are shown in the following table:

TABLE XXXV-1

AVERAGE MONTHLY RESERVOIR EVAPORATION LOSS
COMPLETE PLAN OF DEVELOPMENT

(Millions of Cubic Meters)

Month	Reservoirs		
	Karaoun El. 856	Khardale El. 296	Bisri El. 430
January	+ 1.3	+ 0.1	+ 0.1
February	+ 1.2	+ 0.2	+ 0.1
March	0	- 0.1	0
April	- 0.6	- 0.2	- 0.1
May	- 1.2	- 0.3	- 0.2
June	- 1.8	- 0.4	- 0.2
July	- 2.2	- 0.4	- 0.3
August	- 1.9	- 0.4	- 0.3
September	- 1.5	- 0.3	- 0.3
October	- 1.1	- 0.3	- 0.3
November	- 0.3	- 0.2	- 0.2
December	- 0.7	0	0
Net annual	- 7.4	- 2.3	- 1.7

Reservoir Seepage Losses

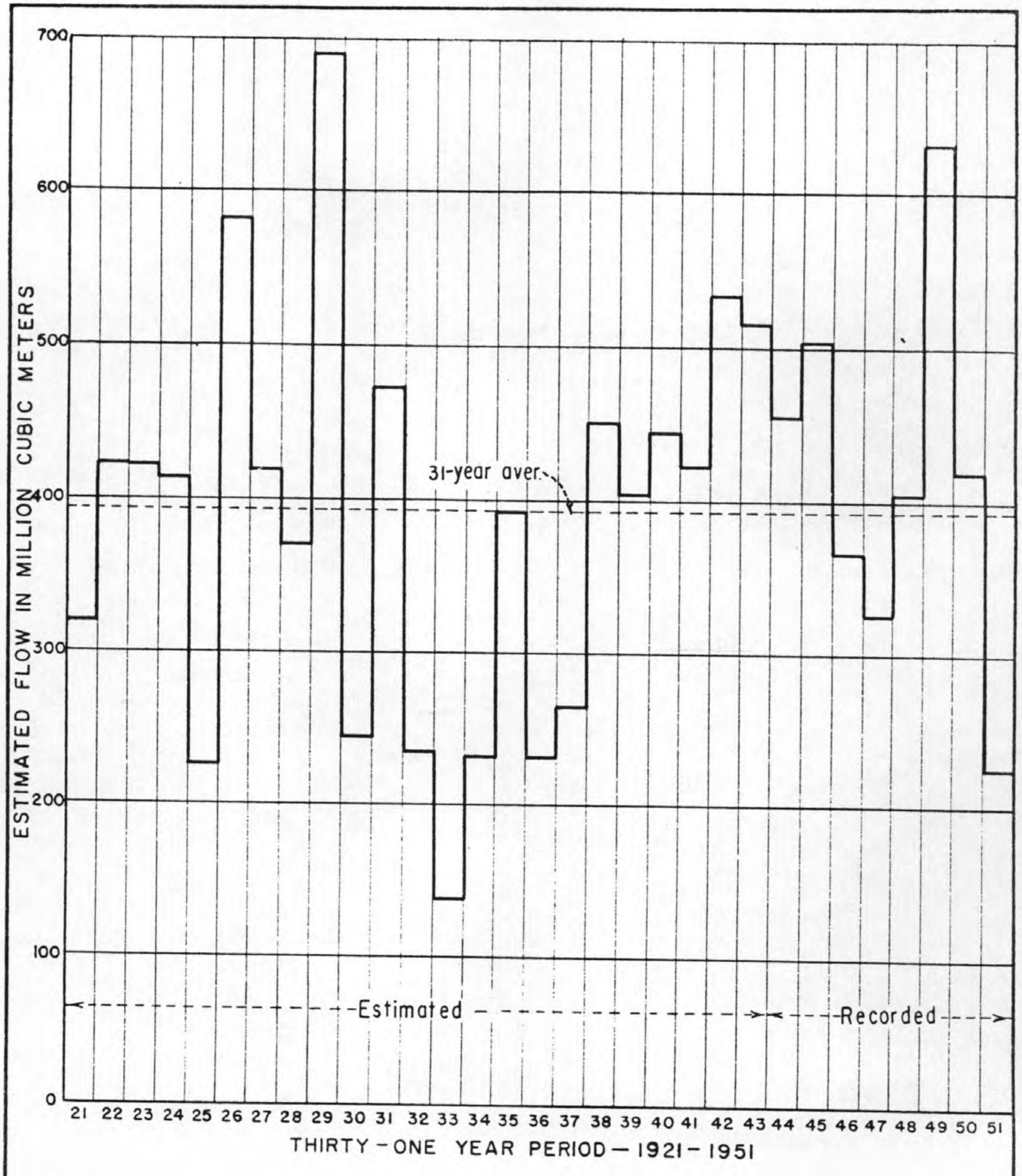
Reservoir seepage losses were estimated for each reservoir and are shown in the following table for purposes of comparison:

TABLE XXXV-2

ESTIMATED RESERVOIR SEEPAGE LOSSES
COMPLETE PLAN OF DEVELOPMENT

(Loss in million cubic meters)

Month	Reservoirs		
	Karaoun	Khardale	Bisri
January	1.3	0.6	0.0
February	1.5	0.8	0.0
March	1.8	1.0	0.1
April	1.9	1.1	0.2
May	1.9	1.0	0.2
June	1.8	0.9	0.2
July	1.7	0.8	0.2
August	1.6	0.6	0.2
September	1.5	0.5	0.2
October	1.4	0.4	0.2
November	1.2	0.4	0.1
December	1.2	0.5	0.0
Total:	18.8	8.6	1.6



Note: The bars shown, represent the total flow each year of the Litani River at the Karaoun Gage during the 31-year period. 1921-1951 under natural flow conditions existing in each year of the period. Flows from January 1944 to December 1951 were recorded at this gage and those prior to 1944 were estimated by correlation with flows at the Mansoura Gage.

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PLATE XXXV-1

It has been assumed that the seepage loss from Karaoun Reservoir will be equivalent to the presently unmeasured inflow into the Litani River between the Karaoun gage and the Karaoun Dam site and it has been omitted from this operation study. It has also been assumed that the seepage loss from Khardale Reservoir will be equivalent to presently constructed local diversions, exclusive of the Kasmie Project, below this dam site and cannot be used to supplement the Kasmie Project requirements. One-half of the seepage losses estimated for Bisri Reservoir have been assumed collectable in the Joun Diversion Pond and available for further downstream use.

Irrigation Requirements

Irrigation has been assumed to have first, or prior right to all the water in the Litani and Bisri River Basins and power production to have only a secondary right. Reservoir storage would be released during the irrigation season first for irrigation use and second for power production, whenever sufficient water was available for both purposes, so as to supplement such releases and meet hydro energy demands. Irrigation demands shown in the following table have been used in this study.

TABLE XXXV-3

ESTIMATED IRRIGATION REQUIREMENTS
LITANI AND BISRI RIVER BASINS

	Projects						
	Bekaa Gravity unit 1/	Bekaa Pumping unit	Saida-Beirut unit	Upper Nabatiye unit	Lower Nabatiye unit	Kasmie project	Total
Area Ha.	5700	4700	3870	3500	3700	6000	29,470
Months	Monthly Requirements - M3 x 10⁶						
Jan.	0	0	0	0	0		
Feb.	0	0	0	0	0	0	0
Mar.	0	0	0	0	0	0	0
Apr.	6.3	5.2	3.5	2.8	3.3	7.2	28.3
May	8.8	7.0	5.8	4.9	5.6	13.4	45.5
June	5.6	6.8	5.7	4.7	5.5	13.0	41.3
July	4.4	7.0	5.8	4.9	5.6	13.4	41.1
Aug.	4.2	7.0	5.8	4.9	5.6	13.4	40.9
Sept.	4.3	6.8	5.0	4.5	4.8	11.4	36.8
Oct.	4.7	6.6	5.0	3.8	4.8	11.0	35.9
Nov.	0	0	0	0	0	0	0
Dec.	0	0	0	0	0	0	0
Total:	38.3	46.4	36.6	30.5	35.2	82.8	269.8

1/ This includes a full supply for 2800 hectares and a partial supply for an additional 2900 hectares.

Usable Reservoir Storage

Capacity curves determined from field topographic surveys for Khardale and Bisri Reservoirs were adjusted for sediment accumulation expected during the 50- and 100-year periods subsequent to their construction, by the methods previous discussed. The sediment accumulation estimated for Karaoun Reservoir was found to be so small that no adjustment was necessary. The volume of dead storage was subtracted from the total storage volume in each case and usable storage curves prepared for each reservoir. These are shown on Plates XXII-2, XXIII-2 and XXIV-2.

Storage Available for Power Production

The above usable storage curves show that Karaoun Reservoir will have a usable capacity of 195.1 million cubic meters when completed to its full height of 856 meters. Khardale Reservoir will have a usable capacity of 70.3 million cubic meters and Bisri Reservoir 13.8 million cubic meters when these reservoirs are placed in operation making a total of 279.2 million cubic meters of storage capacity available in the completed project. This storage capacity was assumed to be operated in each case, first to guarantee a full water supply to all irrigation units, including the Kasmie Project, but excepting the Bekaa Gravity Unit, and second to supply the maximum constribution of hydro power to the combined thermal and hydro system at all times consistent with the operation of these reservoirs for irrigation use.

Karaoun Reservoir is so located that water released from it for use on the Upper Nabatiye Irrigation Unit can be passed through the Sohmor Power Plant, prior to diversion to this unit. Likewise any water released for the Lower Nabatiye Unit can be passed through both Sohmor and Kelia Power Plants, and any replacement storage released to complete the water rights of the Kasmie Project can be passed through Sohmor, Kelia, and Zrariye Power Plants. Water released from this reservoir for diversion through the Bisri (Awali) River for use on the Saida-Beirut Irrigation Unit can be passed through the Bisri and the Awali Power Plants prior to reaching this irrigation project. Additional storage in this reservoir released exclusively for power production on the Bisri River can also be passed through the Joun Power Plant as well as through both the Bisri and the Awali Power Plants. Storage released down the Litani River from Karaoun Reservoir for power production can be used in all three of the plants listed above on that stream.

The Khardale Reservoir is used primarily to store excess inflow originating below Karaoun Reservoir for supplying Kasmie Project and for power production in the Zrariye Power Plant. Such storage may also be used as replacement storage to fill Kasmie Project's primary water right, and to provide additional head for the Zrariye Power Plant.

Usable storage in Bisri Reservoir is limited by the physical features of the reservoir and dam site. It has been designed essentially to provide additional head for Awali Power Plant and to smooth-out the rapid daily fluctuations characteristic of the Bisri River in the winter period. For this reason it is drawn down in October, November and December and held practically empty during January and February and permitted to fill beginning on March 1st. As soon as it fills in March or April it is held full until October to maintain the power head on the Awali Power Plant.

Diversion Tunnel Capacities

Tunnel capacities have been determined on the basis of supplying sufficient water, under minimum head conditions, to operate all the units in each plant at installed capacity. Carrying capacity of freeflow tunnels has been determined by the Manning Formula with a roughness factor assumed at H = 0.014. Head loss in the pressure tunnels has been computed by use of the following head loss formula in English units:

$$H = 2.87\, n^2 \frac{1\, v^2}{d^{4/3}}$$

Where

H = Loss of head due to friction in feet in length 1

1 = Length of tunnel in feet

v = Mean velocity in feet per second

d = Diameter of tunnel in feet

n = Roughness coefficient, assumed as 0.014

The values determined by the above formula were converted into metric units for use in the operation studies, and are shown in the following tabulation for the pressure tunnels.

	Length meters	Average Head Loss meters	Max. Capacity $m^3/sec.$
Sohmor Tunnel	7660	14.0	11.1
Kelia Tunnel	1012	3.5	21.0
Awali Tunnel	3300	10.0	23.2
Joun Tunnel	6400	19.0	18.8

The capacities determined for the free-flow tunnels are shown in the following tabulation:

	Length	Max. Capacity $m^3/sec.$
Markabi Tunnels and Canal	17,300	21.0
Zaiye Tunnel and Canal	20,300	9.0
Zrariye Tunnel and Canal	2,300	9.0
Bisri Tunnel	8,800	13.2

Reservoir Operation Curves

Storage reservoirs must be operated during the non-irrigation season so as to guarantee a full irrigation water supply during the following two irrigation seasons even if such period should be equivalent to the lowest two years in the 31-year period investigated. Therefore, reservoir operation curves were developed from trial operation studies, which would permit the greatest use of storage for power during all the months in the year and still provide sufficient water to meet the full irrigation requirement.

Trial operation studies were made to develop the curves for Karaoun, Khardale and Bisri Reservoirs, which are shown on Plates XXXV-2, 3, and 4. These curves show the storage to which each of these reservoirs may be drawn, at the end of each month during the year, so that they may be expected to provide sufficient storage during the following year to meet irrigation and power requirements, even if the following two years should be equivalent to the lowest two years in the 31-year period investigated. These operation curves have been used in the operation studies for the adopted plan of development.

Reservoir Operating Criteria

It was necessary to develop a limited number of operating criteria for use along with the reservoir operation curves discussed above. Minimum required monthly releases were determined from trial operation studies and were the maximum releases made from each of the reservoirs during all the months that the available storage in such reservoir was below the amount indicated as the minimum for such months by the reservoir operation curves. These releases were unneccessary for the run-of-river operation of Sohmor Plant, since no storage was available for such operation.

Karaoun Reservoir. Minimum required monthly releases were determined for Karaoun, Khardale, and Bisri Reservoirs for use in the operation of the entire project. The releases for Karaoun Reservoir are shown in the following table:

TABLE XXXV-4

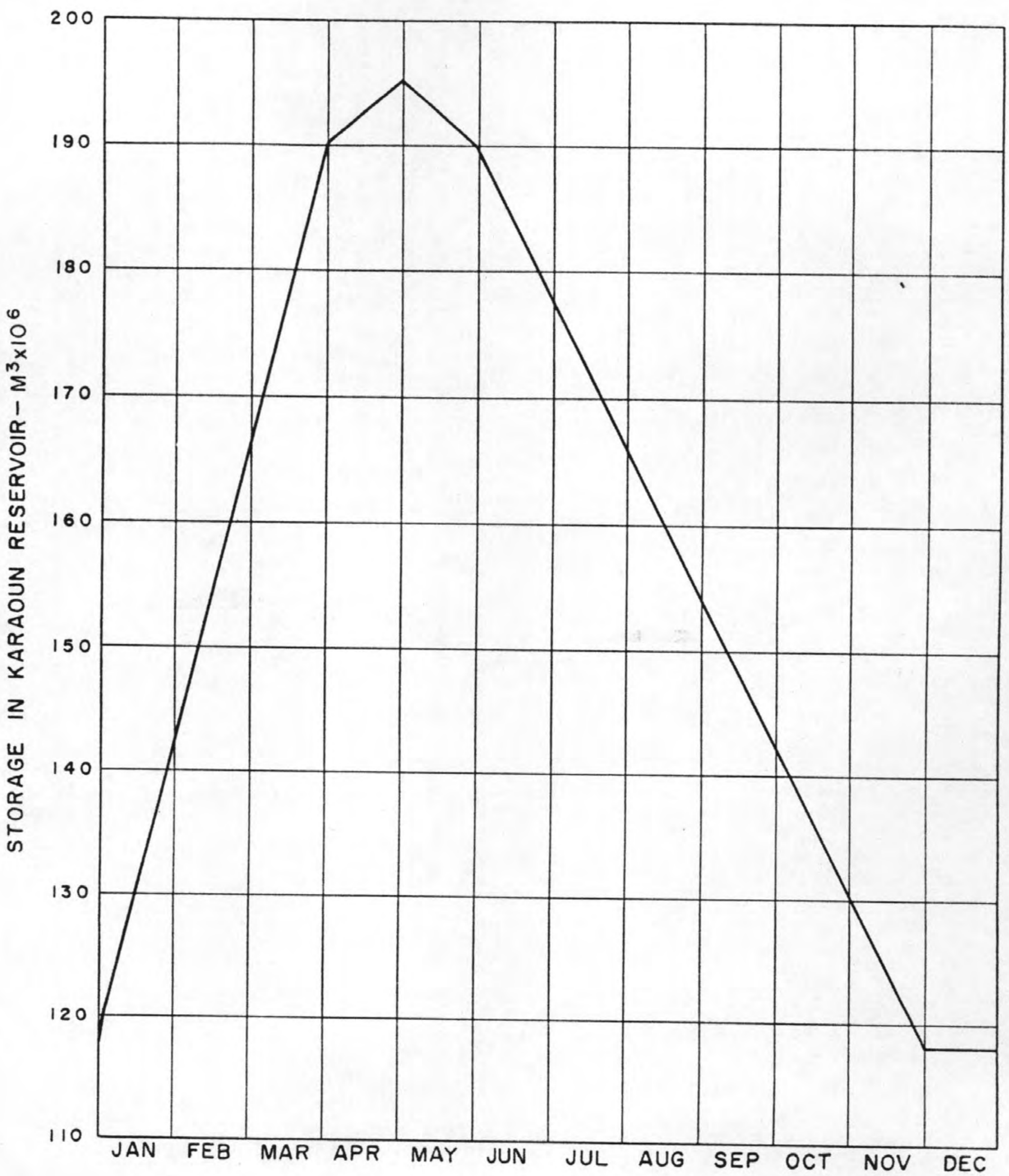
MINIMUM MONTHLY RELEASES - KARAOUN RESERVOIR
COMPLETE PLAN OF DEVELOPMENT

(Units are million cubic meters)

Month	Bisri Tunnel	Bekaa Pumping Unit	Sohmor Tunnel
January	6.7	0	10.3
February	3.3	0	8.1
March	4.9	0	8.5
April	5.3	5.2	6.2
May	6.0	7.0	5.4
June	7.4	6.8	5.2
July	4.9	7.0	5.3
August	5.0	7.0	5.1
September	4.5	6.8	5.0
October	5.2	6.6	6.4
November	5.5	0	8.7
December	9.2	0	12.5
TOTAL:	67.9	46.4	86.7

The above minimum required monthly releases for Karaoun Reservoir, when operated as a part of the complete project development, will provide for a full irrigation supply for the Bekaa Pumping unit; for the release of sufficient water to the Litani River to meet the downstream irrigation and power demands, and at the same time will utilize the full storage of Khardale Reservoir; and the release of sufficient water to the Bisri River to effectively utilize the full storage of Karaoun Reservoir. The reservoir operation curve on Plate XXXV-2 indicates the amount the reservoir can be drawn down in any year and still provide sufficient storage to maintain the minimum releases through critically dry years. In performing the study, the minimum required monthly releases were made in all months that the storage in the reservoir was below the values indicated on the reservoir operation curve. In the other months, when the inflow and usable storage were more than sufficient to provide for the minimum releases, the release of water through the Bisri Tunnel and the Sohmor Tunnel were increased to draw the reservoir down to the values indicated on the reservoir operation curve. A preference was given to the Bisri Tunnel in this release of additional water, with the release to Sohmor Tunnel being maintained at the minimum release until the Bisri Tunnel release was at the maximum value usable by the Bisri Power Plant and then increased up to the capacity of the Sohmor Plant, and any additional inflow beyond the reservoir capacity was spilled down the Litani River.

Khardale Reservoir. The minimum required monthly releases determined for Khardale Reservoir, when operated as a part of the complete project development are shown in the following table:



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RESERVOIR OPERATION CURVE

KARAOUN RESERVOIR PHASE "B"

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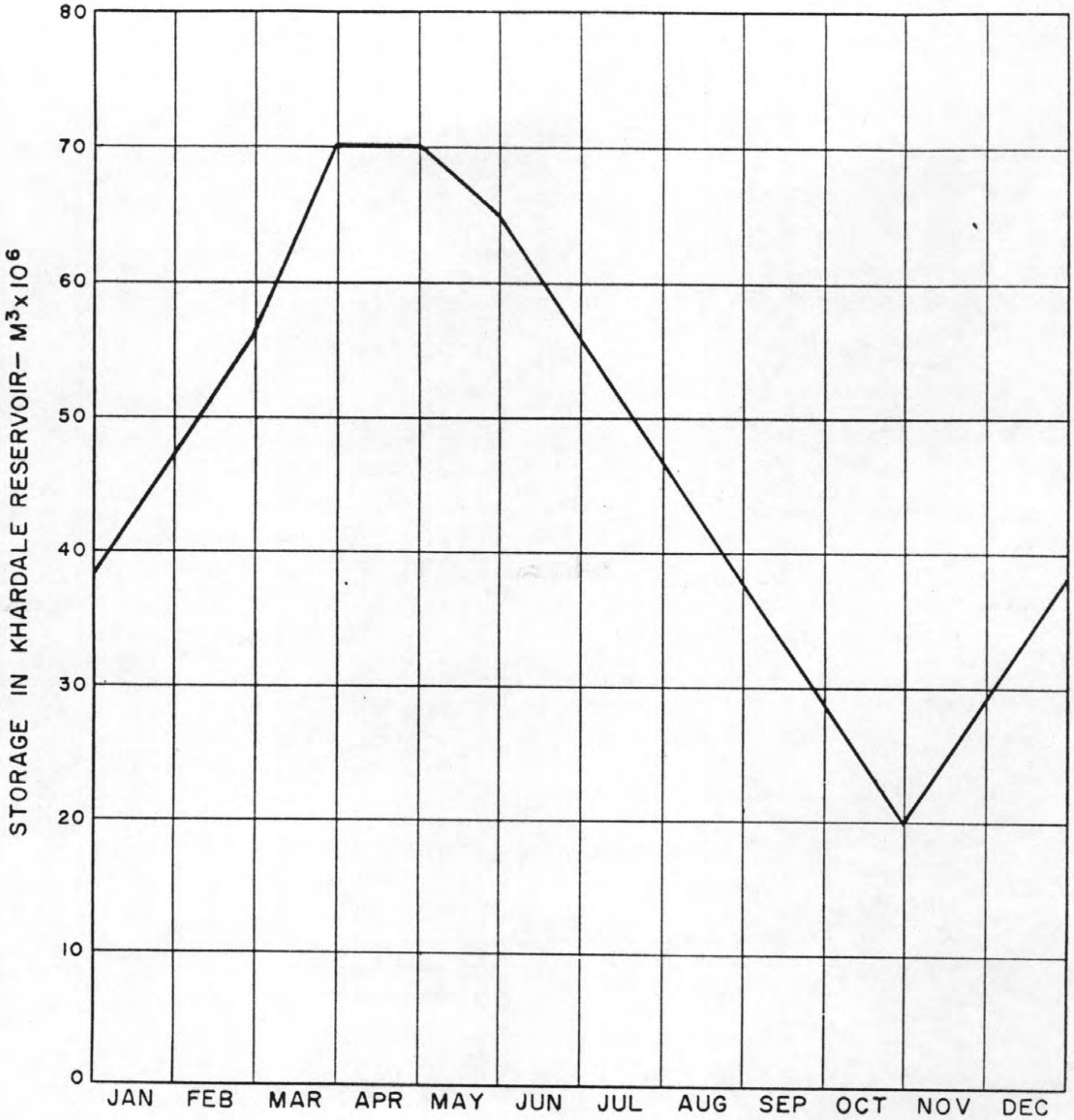
BEIRUT, LEBANON 12-22-53	OA–10–263

PLATE ~~XXXV~~-2



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RESERVOIR OPERATION CURVE
KHARDALE RESERVOIR

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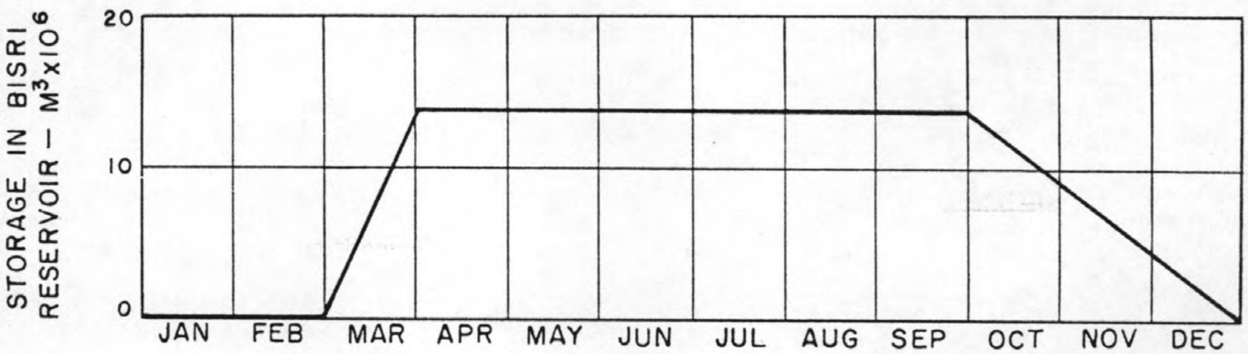
BEIRUT, LEBANON 12-22-53 | 0A–10–264

PLATE XXXV-3



Note: The curve shown indicates the storage level to which Bisri Reservoir may be drawn to provide storage space for the monthly regulation of Bisri River flows.

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RESERVOIR OPERATION CURVE
BISRI RESERVOIR

[redacted]

BEIRUT, LEBANON 12-22-53 | OA-10-269

PLATE XXXV-4

TAELE XXXV-5

MINIMUM MONTHLY RELEASES - KHARDALE RESERVOIR
(All units are million cubic meters)

Month	Lower Nabatiye Unit	Project 1/	Power Water for Zrariye Plant
January	0	0	13.4
February	0	0	12.1
March	0	0	13.4
April	3.3	7.2	5.8
May	5.6	13.4	0
June	5.5	13.0	0
July	5.6	13.4	0
August	5.6	13.4	0
September	4.8	11.4	1.6
October	4.8	11.0	2.4
November	0	0	13.0
December	0	0	13.4
TOTAL:	35.2	82.8	75.1

1/ Kasmie Project water is diverted through the Zrariye Power Plant.

The above minimum required monthly releases provide for a full irrigation supply for the Lower Nabatiye Unit and the Kasmie Project, and an average monthly flow of 5 cubic meters per second through the Zrariye Power Plant. The reservoir operation curve shown on Plate XXXV-3 indicates the amount the reservoir can be drawn down and still provide sufficient storage to maintain the minimum releases through critically dry years. In performing the operation study, the minimum required monthly releases were made in all months that the storage in the reservoir was below the values indicated on the reservoir operation curve. In the other months when the inflow and usable storage were more than sufficient to provide the minimum releases, the release of water through the Zrariye Power Plant was increased in order to draw the reservoir down to the values indicated on the reservoir operation curve. The maximum release of water through the Zrariye Tunnel is limited either by the tunnel capacity of 9 cubic meters per second, or by the amount of water necessary to operate the Zrariye Power Plant at its installed capacity of 12,000 kilowatts.

Bisri Reservoir. The minimum required monthly releases determined for Bisri Reservoir, when operated as a part of the complete project development are shown in the following table:

TABLE XXXV-6

MINIMUM MONTHLY RELEASES - BISRI RESERVOIR
(All units are million cubic meters)

Month	Release through Awali Tunnel
January	Storable inflow
February	Storable inflow
March	4.7
April	5.0
May	5.7
June	5.6
July	5.7
August	5.7
September	4.9
October	Storable inflow plus one-third of storage at end of September.
November	Storable inflow plus one-third of storage at end of September.
December	Storable inflow plus one-third of storage at end of September.

The above minimum required monthly releases provide sufficient water for a full irrigation supply for the Saida-Beirut Unit. The reservoir operation curve shown on Plate XXXV-4 allows the reservoir to be maintained full during the summer months to gain additional head for the Awali Power Plant and permits the evacuation of the reservoir during October, November and December, as well as keeping it essentially empty in January and February so that the capacity of the reservoir is available to smooth out the widely fluctuating winter flows of the Bisri River. In performing the operation study, the minimum required monthly releases were made in all months that the storage in the reservoir was below the values indicated on the reservoir operation curve. Whenever the reservoir was full, the inflow to it was released through the Awali Power Plant, up to its capacity, to keep the reservoir from spilling.

Operation - Complete Project

The multiple purpose plan of development for the Litani and Bisri River Basins is based upon the multiple uses of the available water. It will utilize the average annual flow of 641 million cubic meters in the Litani River Basin and the 114 million cubic meters in the Bisri River Basin to irrigate approximately 21,500 hectares of land and to operate 6 power units for the production of hydroelectric energy. This hydroelectric energy is combined with thermal-produced energy in a coordinated, interconnected, electric transmission system, which will provide much fuller utilization of all the available water for energy production. The joint operation of such facilities eliminates production of secondary, or dump power, since the thermal and hydro facilities will complement each other and only firm power will be produced. The operation study for the complete project therefore has been developed so as to insure filling the irrigation requirements for the plan of development and to produce the maximum amount of electric energy from the six hydro plants while doing so.

It has been assumed that irrigation development has been completed on the following five irrigation units and that each of these require annually the water indicated in Table XXXV-3; Bekaa Gravity Unit, Bekaa Pumping Unit, Upper Nabatiye Unit, Lower Nabatiye Unit, and Saida-Beirut Unit. Karaoun Dam has been assumed to be constructed so that its ultimate reservoir height is 856 meters, Khardale and Bisri Reservoirs completed, and the following power units and their necessary appurtenances completely installed: Sohmor Unit, Kelia Unit, Zrariye Unit, Bisri Unit, Awali Unit, and Joun Unit.

The water estimated as available for each of the months during the 31-year period, 1921 through 1951, has been used in the operation of the above units. Reservoir operation curves developed for each of the storage reservoirs are shown on Plates XXXV-2, 3 and 4 and were used in this study, so as to insure that these reservoirs would not be drawn down so low each year so to prohibit the supplying of the complete irrigation requirements in the following year, even if such year should equal the minimum year in the period considered. Details of this study are contained in Appendix III.

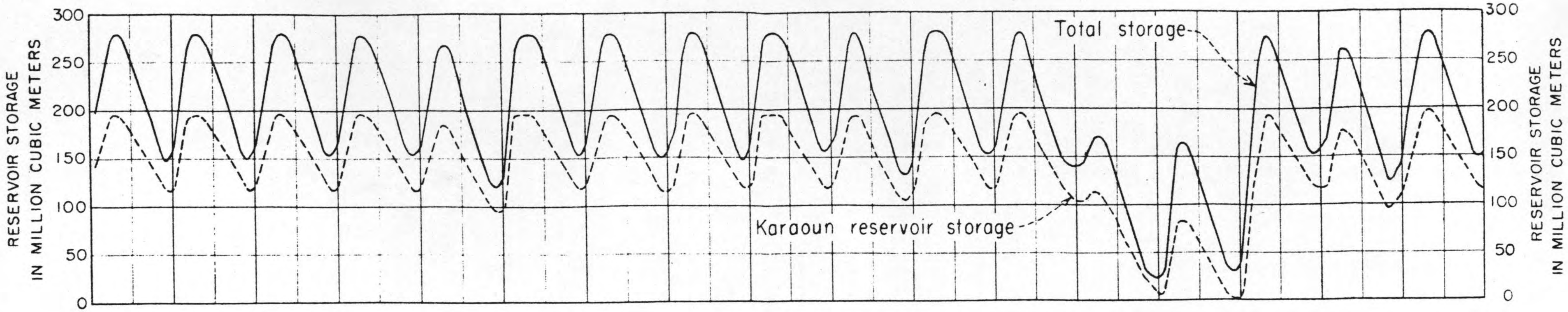
The operation study for the completed project shows that the system operation would supply full irrigation requirements in all of the 31-years, for all of the irrigation units, except the Bekaa Gravity Unit, as well as a full supply for the Kasmie Project. The annual requirement of 38.3 million cubic meters per year for the Bekaa Gravity unit would be supplied in 10 of the 31 years in the period and a shortage less than 10 percent of this amount would occur in 12 additional years. The shortage in the remaining 9 years would vary from about 13 percent to about 34 percent of this amount.

The water available was used to fill all of the 279.9 million cubic meters of available storage in the three reservoirs in March or April of each year, except in 1925, 1930, 1933, 1934, 1935, 1936, and 1951. In these years the maximum amount of storage available varied from 275.1 million cubic meters in 1930 to a minimum of 164.3 million cubic meters in 1933. Carry-over storage in November or December was reduced to a maximum of 151.6 million cubic meters in 6 of the 31 years in the period considered. In other years operations caused it to vary from a maximum of 155.2 million cubic meters in November 1929, to a minimum of 21.6 million cubic meters in December 1933. All reservoirs were essentially full in April 1932 at the beginning of the minimum period and all except Khardale Reservoir were empty at the end of December 1934. At that time Khardale had only 36.6 million cubic meters of storage. These reservoirs were refilled to 274.4 million cubic meters at the end of April 1935 with only Karaoun Reservoir lacking 4.8 million cubic meters to completely fill its full storage capacity. However, there was insufficient water to completely fill all of the reservoirs under this operation plan until April of 1937. The monthly variation in total storage available and in the storage available in Karaoun Reservoir is shown on Plates XXXV-5 and 6.

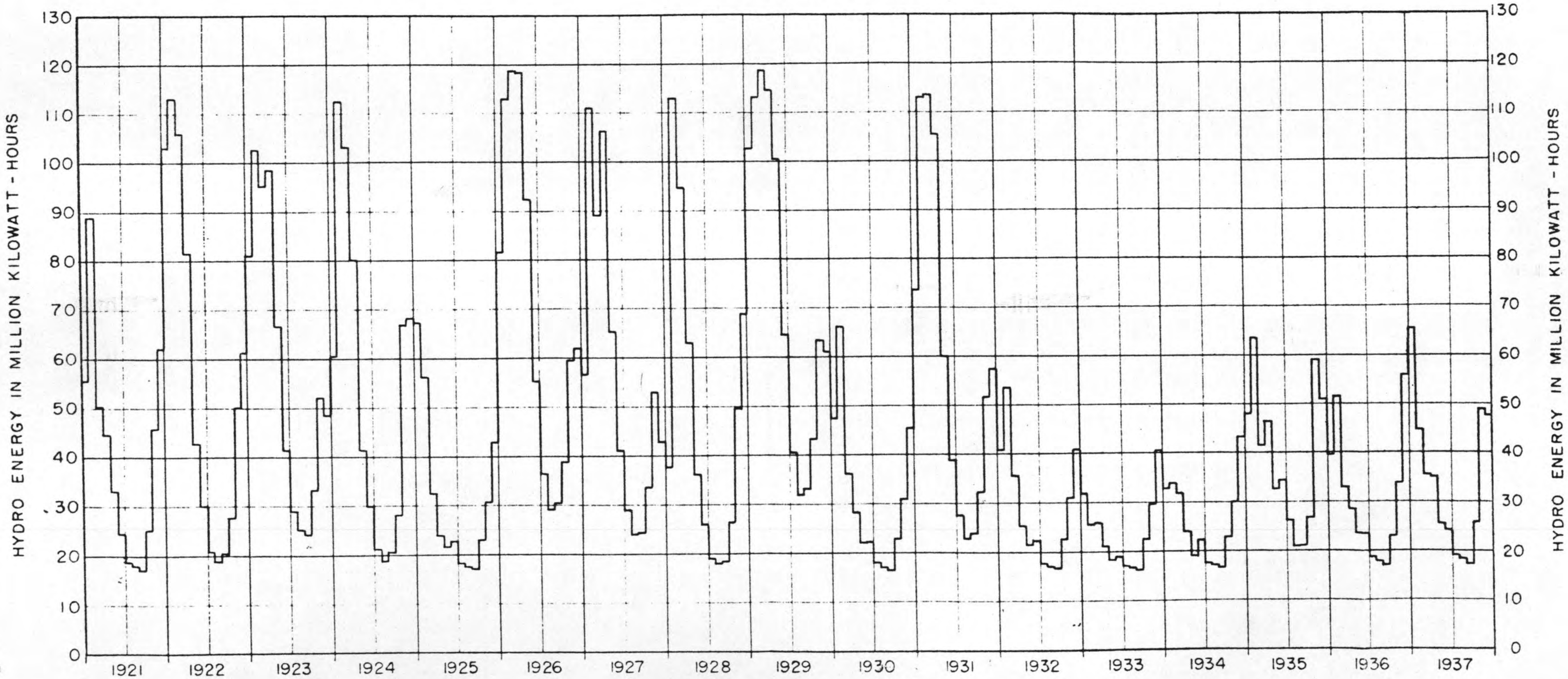
The annual production of hydroelectric energy for use in the coordinated hydro-thermal system under this operation plan would vary from a maximum of 886.5 million kilowatt-hours in 1929 to a minimum of 284.4 million kilowatt-hours in 1933. The average annual production for the 31-year period, 1921 through 1951, would be 606 million kilowatt-hours. Comparison of this average annual energy production with that of 638.9 million kilowatt-hours indicated as available from use of the water assumed available in the hypothetical average year shows it to be 95 percent of the energy determined for the hypothetical average year. The monthly hydro energy produced throughout the 31-year period and its relation to the total estimated monthly requirement of hydro- and thermal-energy at the time the adopted plan of development would be completed are shown on Plates XXXV-5 and 6.

Operation - Bisri Tunnel as a Pressure Tunnel. The operation studies as summarized above and as given in detail in the Hydrology Appendix report were based upon the construction of the Bisri Tunnel as a free-flow tunnel. Additional detailed design studies which were made for this tunnel after completion of the above operation studies show that a pressure tunnel design would be more economical than a free-flow tunnel design for the Bisri Tunnel. The effect of using a pressure tunnel instead of a free-flow tunnel in the operation studies for the Bisri Power Plant would be (to show) an increase in operating head at the power plant which results in an increase in the possible power generation. Studies were made of the probable effect of this change in the tunnel design on the expected generation from the Bisri Power Plant during either an average water year or a critical water year. The results of these studies indicate that the Bisri Power Plant would produce an additional amount of approximately 20 million kilowatt-hours of energy annually during an average water year and approximately 4 million kilowatt-hours of energy annually during a critical water year if the pressure tunnel is used instead of the free-flow tunnel. In addition, the rated capacity of the power plant will be increased from 36,000 to 39,000 kilowatts, but the dependable capacity as available in

December of a critical water year is expected to remain at 36,000 kilowatts. Accordingly, the average annual production of electric energy from the proposed Litani hydroelectric power plants as used in this report has been taken as 626 million kilowatt-hours, and the expected annual production during a critical water period has been taken as 288.4 million killowatt-hours.



RESERVOIR STORAGE



HYDRO POWER PRODUCTION

NOTE

The reservoir storage shown, is the storage available each month as determined by application of the adopted reservoir operation curves and the operation of all the units in the plan of development

The hydro power production shown is that resulting from such operation throughout the 31-year period of the entire project.

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HYDRO POWER PRODUCTION
COMPLETE PROJECT
1921 THRU 1937

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BEIRUT, LEBANON 2-20-54 SHEET 1 OF 2

OA-10-259

PLATE XXXV-5

SECTION XXXVI

FLOOD AND DRAINAGE PROBLEMS

General

The torrential winter rains falling in the Bekaa and the lower slopes of both the Lebanon and Anti-Lebanon Mountains, along with the runoff from melting snow on the higher mountains, have caused flooding of the Litani River in the lower portion of the Bekaa, since time immemorial. Such flooding may be expected each year in the late winter and spring, after the ground has become well saturated. The water overflows the normal banks of the river and spreads out over the low, flat plain of the Bekaa inundating an area of about 4,000 to 9,000 hectares as shown on Plate XXXVI-1. Actual property damage as a result of the flooding is very small. No residences or other buildings are located in the area susceptible to inundation and the shallow depth of the backwater causes no noticeable scouring or erosion of the land. Some damage is caused to crops which are planted early and are covered for a longer period than usual by the flood water but the principal loss results from the wet areas which cannot be planted until too late in the spring to produce good crops.

In addition to the flooding of land adjacent to the Litani there are a few areas of perennial swamps, principally around low lying springs. Only the largest one of these swamps, the Ammik swamp, is of sufficient size to warrant consideration of the possibility of draining and reclaiming the land. The others are of such minor size and importance that the expense of draining would not be offset by the value of the land reclaimed. These smaller swamps will probably be drained as a part of the public health program already undertaken in the Bekaa by the Lebanese Government. Therefore only flood control in the southern section of the Bekaa and drainage of the Ammik swamp have been considered in this investigation. Since the swamp area is contiguous to the area flooded and essentially forms a part of it the two problems have been treated as one in this report.

The Flood Problem

Abd-El-Al 1/ describes the flood situation in the Litani River Basin as follows:

"The confluences of the tributary streams of the Litani in South Bekaa are concentrated in the Cheberkiyet-Ammik area. This results in:

"1. - A permanent marshy area of approximately 600 hectares in the lower part of the plain, at Ammik;

"2. - An area which is flooded during January-February. Neither the drainage ditches of Hafir and Jaher, nor the Litani itself, offer an immediate outlet to the flood waters of that area which is sufficient to carry them off. They back up to the north and spread over both banks of the Litani from Cheberkiyet-Ammik to Haouch el Harimi, covering an area of approximately 4000 hectares;

"3. - Land where the winter water, though it does not remain on the surface for a long time, nonetheless remains close to the surface of the soil and delays seeding, or is otherwise detrimental to crop growth.

"One may conclude that of the 23,000 hectares of irrigable land in South Bekaa, approximately 20 percent requires drainage works to assure a good crop yield.

* * * * * * * * * * * *

1/ Abd-El-Al, Ibrahim, The Litani Hydrologic Study, Beirut, Lebanon, 1948.

"The evil originates, thus, from the localizing at Cherberkiyet-Ammik of the confluence point of all the streams which carry off flood waters.

"At this point in its course the Litani is suddenly swollen by heavy influxes of water which it cannot carry off, due to its inadequate section and gradient. The spread of the flood waters degenerates into a general flood and the duration of their evolution continues on a diminishing scale until the end of spring. Their drain-off is still further slowed by the fact that downstream from the confluence point of the sources, the Litani forms a series of eleven meanders. Moreover, abundant vegetation, occasionally shubbery, obstructs the banks of the normal river channel and substantially hinders the run-off of flood waters.

"Drainage of the area, or the methods which sould be adopted to prevent floods consist, therefore, of giving the Litani the minimum section appropriate to its cross-profile in order to assure the carrying off of maximum flood waters, estimated at 50 cubic meters per second at Mansoura. In those portions where the fall was too gentle, it was necessary to eliminate the bends formed by the river, and the drainage works program provided for the elimination of the aforementioned eleven meanders, as well as cleaning out the river bed between Cheberkiyet and Joub-Jannine. The works program, launched in 1942, was actively pushed, and by 1944 seven meanders had been eliminated and the section of the bed built up to achieve the optimum profile. The length of the channel was thus reduced in the area of the work from 7400 meters to 4800 meters and the embankments constructed exceed 100,000 cubic meters. A very marked improvement has been achieved in the drain-off of flood waters, and the force of the drainage flow is recorded by equations of the curve of the water-flow compared to water-levels on the limigraph at Mansoura:

"$q = 6.773 (h + 0.1316)^{1.374}$ before the works

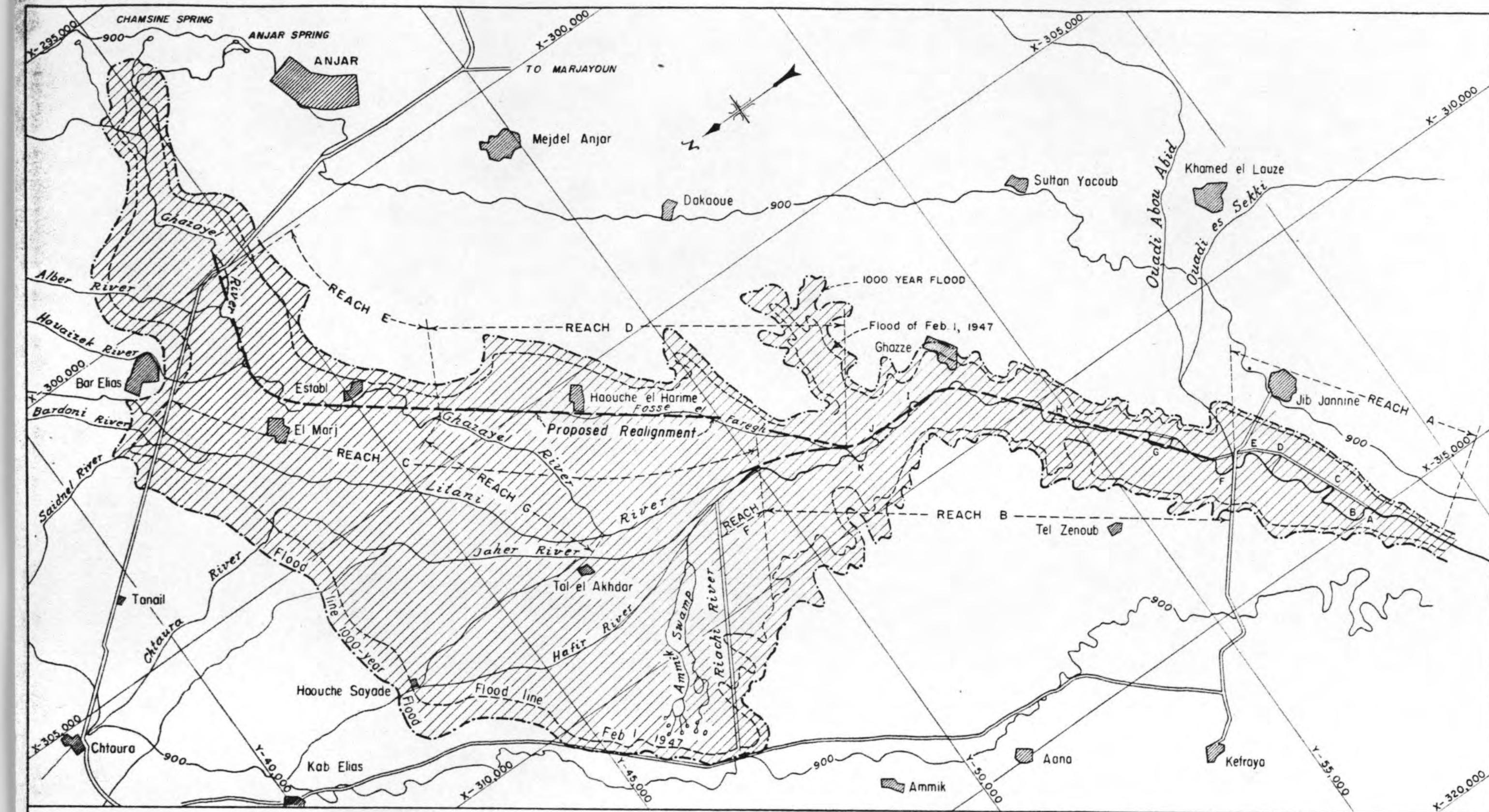
"$q = 8.086 (h + 0.1842)^{1.381}$ after the works

"It is clear, therefore, that for same waterlevel, the flow of water is notably stronger in the second equation."

The flood problem, described above by Abd-El-Al, has been given consideration as a part of this investigation, but studies have been confined essentially to field inspection trips, study of existing topographic and geologic maps, hydrologic studies, and use of survey data collected as a result of the earlier surveys and mapping of the area by the Ministry of Public Works. These limitations were placed on the flood investigation, since the above extracts show that the Ministry of Public Works is well aware of the flood problem and has not only initiated plans to solve this problem, but also has already carried out a part of these plans. Therefore, it has been considered that this flood control and drainage work is part of an existing project that will ultimately be completed and operated by the Lebanese Government. Hence, the following discussion is given only to more clearly define the problems and to suggest a more extensive and coordinated plan to improve the already adopted solution to them. Since this is an existing project that has already been undertaken by the Lebanese Government it has not been included as a part of the plan of development of the Litani and Bisri River Basins covered by this report.

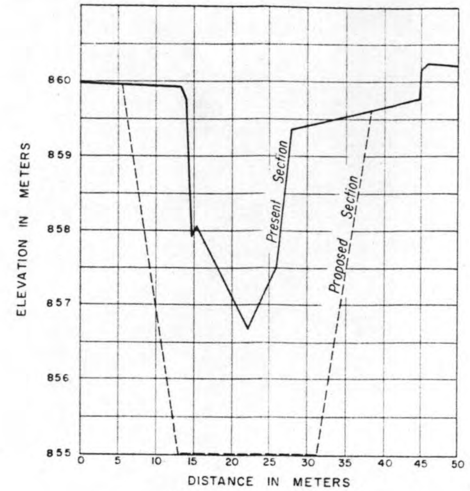
Geology of Area Flooded

The agricultural area, subject to flooding, comprises the downstream portion of the Bekaa that has been filled in geologic time by sediments washed down from the surrounding mountains. This filling appears to have taken place in an ancient lake covering this area and apparently formed by the upfolding of minor anticline ridges across the Litani River Valley. The top of one of these folds appears to be located about two

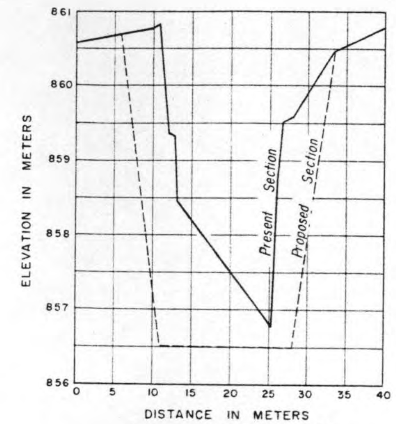


FLOOD AREA SOUTHERN BEKAA

SCALE OF KILOMETERS



TYPICAL SECTION 'A'



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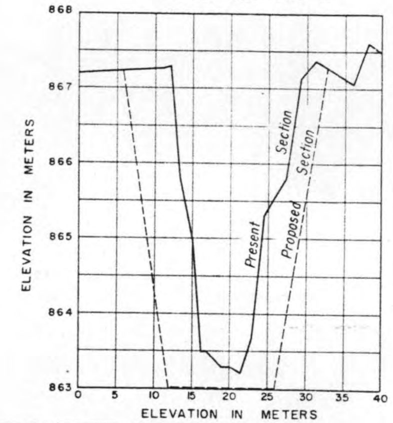

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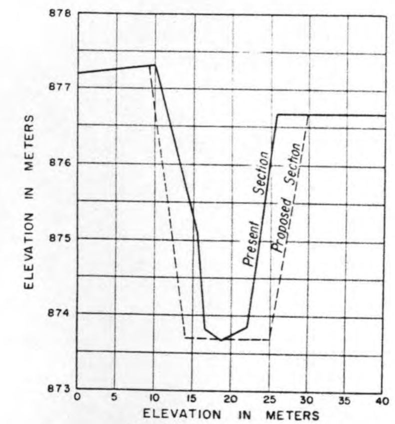
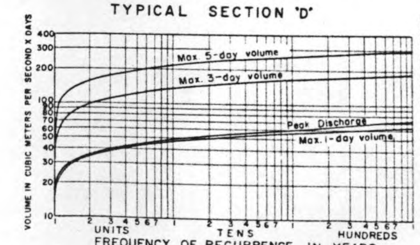
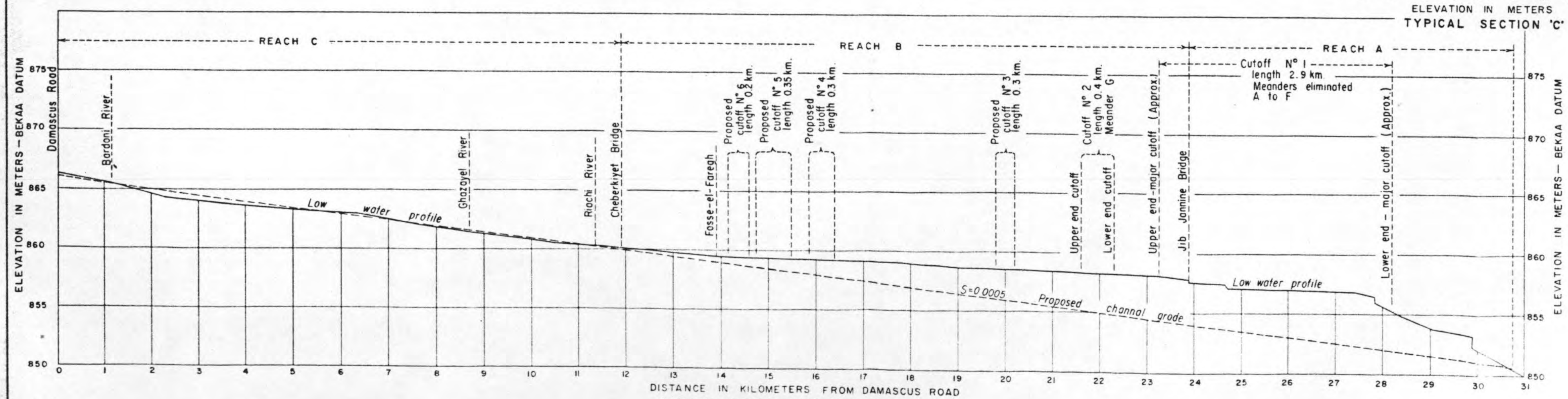


TYPICAL SECTION 'D'



FLOOD FREQUENCY AT MANSOURA



LITANI RIVER—LOW WATER PROFILE

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LITANI RIVER PROJECT— LEBANON

FLOOD AND DRAINAGE PROBLEMS
SOUTHERN BEKAA AREA

[redacted]

BEIRUT, LEBANON | FEB 23, 1954 | OA-10-195

PLATE XXXVI

kilometers downstream from the bridge at Jib Jannine and the top of the other one is in the vicinity of Baalbeck. Between these folds the valley fill is very thick and is believed to be largely unconsolidated. The Litani, the Ghazaiel and the lower portions of the Bardoni and Riachi Rivers have meandered across this alluvial plain and due to its flat slope, have been unable to carve adequate channels to carry off their seasonal flood waters. Below the top of the downstream anticline the Litani River has carved a deep canyon throughout most of its lower reaches, but apparently the top of this fold has successfully resisted the development of this canyon any farther upstream. It is believed, however, that the width of the top of this fold is relatively narrow and that it may consist of only a few narrow limestone ledges, preventing the extension of the canyon making process and the lowering of the gradient of the Litani River through the Bekaa. Detailed geologic explorations should be made in this area to determine the exact location and extent of such ledges before further channel improvement is undertaken below Jib Jannine Bridge.

Contributing Area

The Litani, Bardoni and Ghazaiel Rivers drain 849, 77 and 142 square kilometers of area, respectively, above the Damascus Road. At Cheberkiyet Ammik, the Riachi and Chtaura Rivers bring in the drainage from about 128 additional square kilometers, while an unnamed wadi, with about 57 square kilometers of mountain drainage, joins the Litani River just above the Jib Jannine highway crossing. It appears to be the rapid runoff from this unnamed wadi which produces the very characteristic sawtooth-effect observed on practically all the flood hydrographs recorded at Karaoun gage. The steep slopes of this tributary drainage area produces short, sharp, peak flows which reach Karaoun gage some hours before the peak from the larger drainage areas and produce the sharp peaks noted. The remaining 267 square kilometers of drainage area above Jib Jannine are made up of short wadis which drain the slopes of the Lebanon and Anti-Lebanon Mountains and the Bekaa plain.

Recorded Floods

Recorded and estimated maximum discharges at Mansoura for each of the years during the period of record at this station are shown in the following table:

TABLE XXXVI-1

FLOOD PEAKS AT MANSOURA GAGE - LITANI RIVER

Date	Gage Height m.	Discharge m^3/second	Date	Gage Height m.	Discharge m^3/second
2-7-32	2.98	32.3	3-25-43	3.22	35.8
2-3-33	1.90	18.1	1-22-44	3.07	41.2
2-23-34	2.76	29.3	- - 45	-	45.5 1/
2-3-35	3.05	33.2	2-20-46	3.14	42.4
2-13-36	2.90	31.2	2-1-47	4.15	60.6
- - 37	--	27.8 1/	2-18-48	-	35.4 1/
- - 38	--	34.7 1/	2-8-49	-	41.2 1/
3-6-39	3.15	34.6	1-24-58	-	25.6 1/
1-30-40	3.50	39.6	2-21-51	-	20.9 1/
- - 41	--	39.3 1/	2-5-52	-	41.0 1/
3-23-42	3.56	40.8			

1/ Estimated at 102.5% of mean daily discharge

The above table shows that the largest flood of record at Mansoura occurred on February 1, 1947 and produced a stage of 4.15 meters and an estimated peak discharge of 60.6 cubic meters per second.

Flood Frequency

The data shown in the above table were analyzed by application of the Hazen 1/ annual flood method and the frequency curves shown on Plate XXXVI-1 determined. Application of the maximum recorded discharge on February 1947 of 60.6 cubic meters per second to these frequency curves indicates that the 1947 flood peak has a frequency of occurrence of about once in 100 years. The same frequency study shows that the 500-year, the 1000-year, and the 10,000-year floods at Mansoura may be expected to have peaks of 70, 73, and 83 cubic meters per second, respectively. It has been assumed that ample protection would be provided for the area if control of the 1000-year flood, with a peak flow of 73 cubic meters per second at Mansoura, is effected.

Flood peaks, from drainage areas having somewhat similar characteristics, have been found to vary approximately as the ratio of the square roots of their respective drainage areas. Application of this principle to several points in the flooded area results in the following values for the 1000-year flood at such points:

Stream	Location	Drainage Area (km^2)	1000-year Flood (M^3/sec.)
Litani and Bardoni	Below Damascus Road	926	61
Litani	Mansoura	1323	73
Litani	Below Jib Jannine	1520	78
Ghazaiel	Damascus Road	142	24
Riachi and Chtaura	Mouth of Riachi	128	23

Area Flooded

The area where flooding takes place is essentially contained between the Damascus Road, which crosses the Bekaa at Bar Elias, and the secondary highway which crosses at Jib Jannine. A minor amount of flooding occurs above the Damascus Road, but is due largely to water overflowing the existing channel banks and drains off in a relatively short time. Flooding, in the area considered, results almost exclusively, from the small capacity of the existing channels and the flat gradient of the Litani River, in the lower part of the flood area, which prevents the rapid run-out of the accumulated flood waters from upstream. This backing-up of flood waters produces the secondary damage caused by a high water table and the inability of existing drainage channels to carry off the surplus water from minor depression areas. This effect is also largely responsible for past failures in the attempts to drain the Ammik Swamp area. The approximate area inundated by the flood of February 1, 1947 has been sketched on Plate XXXVI-1 from a consideration of the recorded gage height at Mansoura gage, and the low water profile shown on this Plate. The data available, for the extension of the stage-discharge curve for Mansoura and for the other points are very meager, but indicate that the 1000-year flow would reach about elevation 865 meters at Mansoura and about 870 meters at the Damascus Road. These elevations were used to sketch the approximate flooded area of the 1000-year flood as shown on Plate XXXVI-1. The elevation of the peak flow for both of these floods was considered to be level from Mansoura to the highway crossing at Jib Jannine. Planimetering of these flooded areas indicates that about

1/ Hazen, Allen - Flood Flows - John Wiley & Sons, Inc., New York, 1930

6300 hectares of area were flooded in 1947 and about 9100 hectares would be flooded in the 1000-year flood.

Channel Improvements

A program of straightening and enlarging the channel of the Litani River through this flooded area was undertaken by the Ministry of Public Works in 1942 and the seven meanders shown as "A" to "G" on Plate XXXVI-1 were cut off by new channels between that time and 1944 when the work appears to have been indefinitely suspended. Likewise in recent years artificial drainage channels known as Nahr Hafir and Nahr Jaher were constructed on the west side of the Bekaa to carry the flows of the Chtaura and the Kab Elias River through the low lying area along this side of the Litani River. A new channel about 4.5 kilometers long was constructed some years ago along the southern edge of the Ammik swamp to carry the flows of the Riachi River and in an attempt to drain this swamp. However, neither of these programs have been successful in accomplishing their purposes.

A review of the work accomplished by the 1942 - 1944 program shows that a new river channel constructed on either side of Jib Jannine eliminated meanders "A" to "F" and a short cutoff eliminated meander "G" the next upstream meander. This work shortened the channel length about 2600 meters in this reach, and it appears that the channel may have been cleared of vegetation and debris in this area. However, the marked improvement reported by Abd-El-Al 1/ in 1948 does not appear to have continued and the flooding in the southern part of the Bekaa and the Ammik swamp still continues practically unabated.

Consideration of available data from earlier surveys and reports indicates that the failure of past efforts may be attributed to the following causes:

(a) The channel improvement at Jib Jannine was started too far upstream to permit the construction of the gradient required to drain the southern part of the Bekaa.

(b) The new channel was too small in cross section and had too flat a gradient to be effective in draining the flooded area.

(c) The improvement work apparently did not include any maintenance to keep the new channel, or the existing ones, clear of vegetation and shrubbery.

(d) The channel improvements were not continuous upstream from the new channel at Jib Jannine and the flat gradient between the cutoff meander above this point and the new channel, completely nullified the benefits expected to result from its construction. Similar lack of results would have occurred if the other four cutoffs for meanders "H" "I" "J" and "K" included in the program, had been made.

(e) Since no lowering of the Litani River profile was accomplished in the reach immediately below the Riachi River and the drainage channels for the Ammik swamp, the drainage of this swamp was not accomplished. Likewise the increased growth of vegetation in these channels has still further increased the drainage problem in this area.

Suggested Solution

It is suggested that the following modifications be made in the adopted program for flood control and drainage in the South Bekaa, the details of which are discussed in subsequent paragraphs:

(a) Extend the artificial channel at Jib Jannine downstream to a point about 2400 meters below the Jib-Jannine Bridge where its gradient should intersect the existing low water profile at about elevation 850.5 meters, and lower its gradient to a slope

1/ See footnote on Abd-El-Al on page XXXVI-1.

of 0.0005 from this point upstream. Increase its cross sectional area to that sufficient to carry the 1000-year flood in this area, estimated at 78 cubic meters per second.

(b) Extend the existing artificial channel upstream from the Jib Jannine Bridge by a realignment of the river channel to the lower end of Fosse-el-Faregh. The gradient of this new alignment should be an extension of that in the lower channel and the cross section of this new channel should be sufficient to carry the 1000-year flood, estimated at 73 cubic meters per second. This realignment will reduce the channel length in this reach by about 3000 meters and will require the lowering of the present low water profile, a maximum of about 4 meters immediately above the Jib Jannine Bridge.

(c) Remove all of the existing vegetation from the channel and banks of the Litani River from the lower end of Fosse-el-Faregh to the Damascus Road crossing at Bar Elias and enlarge this channel to carry the 1000-year flood estimated at 61 cubic meters per second. The natural gradient in this reach is approximately 0.0005 and will not have to be changed.

(d) Enlarge the existing drainage canal known as Fosse-el-Faregh throughout its entire length to provide a new extension of the Ghazaiel River and to discharge the water from this stream into the enlarged Litani River channel below the end of Fosse-el-Faregh. This enlarged channel should have a gradient of 0.0005 and a cross section sufficient to carry the 1000-year flood of the Ghazaiel River, estimated at 24 cubic meters per second.

(e) Realign, and enlarge the channel of the Ghazaiel River from the upper end of the Fosse-el-Faregh to the Damascus Road near Bar Elias. This realigned channel should have a gradient of 0.0005 and a cross section sufficient to carry the 1000-year flood from the Ghazaiel River, estimated at 24 cubic meters per second.

(f) Close the present inadequate channel of the Ghazaiel River below the upper end of Fosse-el-Faregh by construction of an impervious dyke across it at this location. This will leave this channel available to serve as a drainage ditch below this point.

(g) Remove the existing vegetation from the channel and banks and enlarge the present inadequate channel of the Riachi River from the lower end of the present improved channel in this river to a new junction with the proposed new channel of the Litani River, just above the lower end of Fosse-el-Faregh. The gradient should be 0.0005 and the enlarged channel should have a cross section sufficient to carry the 1000-year flood, estimated at 24 cubic meters per second. This enlarged channel and those in the Litani below its mouth will do much to provide for the drainage of the Ammik Swamp and the surrounding agricultural area.

For purposes of discussion the various channels mentioned above have been divided into the following reaches, which are shown on Plate XXXVI-1.

<u>Reach A</u> - Litani River downstream from Jib Jannine Bridge for about 2400 meters.

<u>Reach B</u> - Litani River upstream from Jib Jannine Bridge to lower end of Fosse-el-Faregh.

<u>Reach C</u> - Litani River upstream from lower end of Fosse-el-Faregh to the Damascus Road Bridge.

<u>Reach D</u> - Fosse-el-Faregh throughout its length to provide a new outlet for the Ghazaiel River.

<u>Reach E</u> - Ghazaiel River from upper end of Fosse-el-Faregh to the Damascus Road Bridge.

Reach F - Existing channel Ghazaiel River from upstream end of Fosse-el-Faregh to present junction with the Litani.

Reach G - Riachi River from junction with Litani River about 1.5 kilometers upstream.

Improvement Proposed

Reach A - This includes the partially constructed major cutoff below the Jib Jannine Bridge. This cutoff should be enlarged, lengthened and deepened so as to effectively carry the 1000-year flood, estimated at 78 cubic meters per second. The low-water gradient should be constructed with a slope of 0.0005 and the bottom width of the channel should be 18.5 meters. It is estimated that side slopes will be stable with a slope of 1-1/2 horizontal to 1 vertical. This new channel shown as Typical Section "A" on Plate XXXVI-1 will have a length of about 2400 meters and will reduce the length of the existing channel by about 1500 meters. It is estimated it will require the excavation of about 212,000 cubic meters of material, most of which will be lake deposits, but a small portion of which may be limestone ledges at the lower end of this channel.

Reach B - This reach extends upstream from the Jib Jannine Bridge for a distance of about 9250 meters to the vicinity of the lower end of Fosse-el-Faregh. The existing channel in this reach is about 12,500 meters long and has many meanders. The new channel would start by utilizing the upper end of the cutoff already constructed immediately above the Jib Jannine Bridge and would follow a complete new alignment from the end of this cutoff to the upper end of the reach. The 1000-year flood for this reach has been estimated at 73 cubic meters per second based upon the Mansoura records in this reach. This will require a trapezodial channel with a bottom width of 17 meters and side slopes of 1-1/2 to 1 with a gradient of 0.0005 as shown on Typical Section 3 on Plate XXXVI-1. It will reduce the channel length in this reach by about 3000 meters. It is estimated that it will require the excavation of about 550,000 cubic meters of material, all of which is believed likely to be unconsolidated lake deposits.

Reach C - This reach extends along the Litani River from the lower end of Fosse-el-Faregh to the Damascus Road Bridge, a distance of about 10,900 meters. The channel is relatively straight throughout this reach and has an existing low-water gradient of about 0.0005. The 1000-year flood in this reach is estimated at 61 cubic meters per second. It will require a channel with a bottom width of 14 meters, side slopes of 1-1/2:1 as shown by Typical Section "C" on Plate XXXVI-1 and flowing at a depth of 3 meters to carry this flood at this gradient. This can be provided by enlarging the existing channel throughout this reach. It is estimated that such enlargement will require the excavation of about 455,000 cubic meters of material, all of which is believed likely to be unconsolidated lake deposits.

Reach D - This reach includes the existing drainage canal known as Fosse-el-Faregh. It has a length of about 7400 meters from the Litani River to where it joins the Ghazaiel River just downstream from Istabel. Little is known about its origin but it appears to have been an ancient channel of the Ghazaiel River and to have been somewhat enlarged for drainage purposes. On some maps it is shown as the Nahr el Faregh, which strengthens this assumption. Cross sections were not available for this reach but have been assumed to be approximately the same as those for the Ghazaiel River below the head of the Fosse-el-Faregh. The alignment in this reach is excellent and the existing channel needs only to be enlarged, to provide a much better channel for the lower section of the Ghazaiel River. It will provide better flow conditions on the Litani River also, since the Ghazaiel River water will enter this river lower down and at the upper end of the enlarged channel. The 1000-year for this reach is estimated at 24 cubic meters per second and will require a trapezoidal channel 12 meters bottom width, with side slopes 1-1/2:1 and a gradient of 0.0005. It is estimated that it will require the excavation of about 222,000 cubic meters of material to provide this channel. This material is believed to be unconsolidated lake deposits.

Reach E - This reach extends along the Ghazaiel River from the upper end of Fosse-el-Faregh to the Damascus Road Bridge near Bar Elias. The Ghazaiel River is very crooked in this reach and a channel realignment is required. The realigned channel will have a length of about 5000 meters as compared to the present channel length of about 7100 meters. The 1000-year flood for this reach is estimated at 24 cubic meters per second and with a slope of 0.0005 and a depth of 2 meters will require a trapozoidal channel with a bottom width of 12 meters and side slopes of 1-1/2 to 1 as shown by Typical Section D on Plate XXXVI-1. Most of this channel will be outside the present river channel and will require new rights-of-way. It is estimated that its excavation will require handling of about 200,000 cubic meters of unconsolidated lake deposits.

Reach F - This is a short reach of only about 1500 meters to connect the Riachi River and its drainage system, with the Litani River at the upper end of Reach B. Much of the drainage area above this point is controlled by the Ammik Swamp and it is very difficult to estimate the design flood for this reach. It has been assumed however, that a channel with a bottom width of 12 meters and side slopes of 1-1/2 to 1 and with a gradient of 0.0005 will be ample to carry this flow. This channel will require the excavation of about 45,000 cubic meters of unconsolidated lake deposits.

Reach G - This reach is the present channel of the Ghazaiel River from the present month of this river to the upper end of Fosse-el-Faregh. It is about 3800 meters long and has numerous short meanders. It joins the Litani River about 3200 meters above Cheberkiyet-Ammik and its discharge further congests the channel over this area and increases flood damage. It is proposed that this channel be closed by the construction of an earth dike across its upper end, thus leaving it available for drainage purposes throughout its entire length. Ample material will be available for this dike from the channel improvements in the vicinity of the site.

Estimated Cost. The four typical sections, shown on Plate XXXVI-1 have been assumed to represent existing conditions in Reaches A, B, C and E. No cross sections were available for Reaches D and F, or for the proposed new alignment of the channel in these several reaches. However, examination of existing topography indicates that the maximum depth to which such channels will have to be excavated, in all except Reach A will not exceed 3 to 4 meters. It is anticipated that the channel in Reach A will require some sections to be excavated to 4 or 5 meters depth in order to obtain the required gradient for this reach. Consideration of the general geology of the area indicates that the material to be excavated will consist almost entirely of unconsolidated lake bed deposits, the one exception being the lower end of Reach A where some limestone ledge rock may be encountered and may require blasting to provide the required channel. The amount of such material is believed to be less than 50,000 cubic meters and is most likely to occur at the lower end of this reach.

The following tabulation shows the estimated quantities of material to be removed to provide the improved channel for the several reaches:

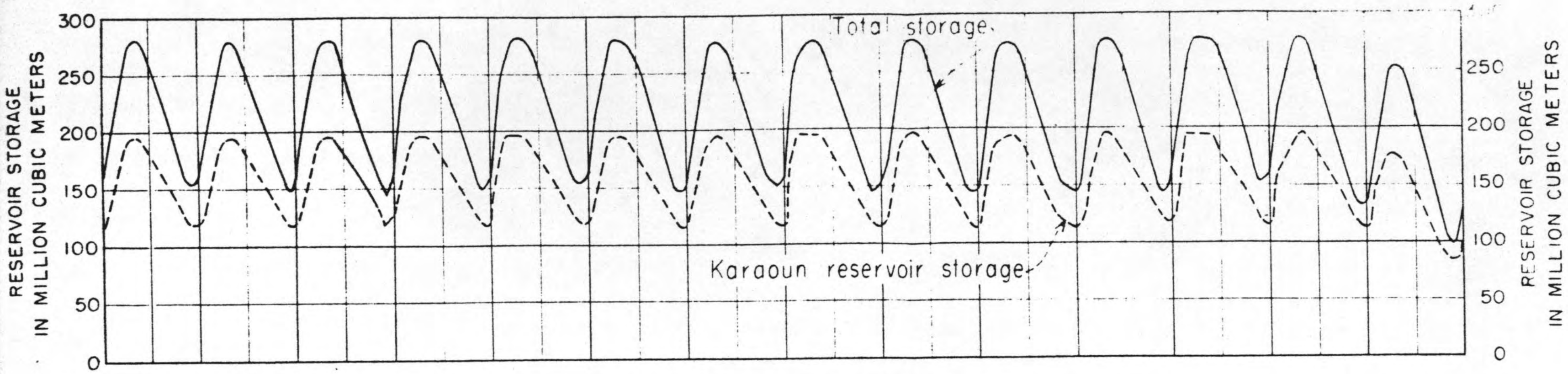
Reach		Quantity cubic meters
A		212,000
B		550,000
C		455,000
D		210,000
E		200,000
F		45,000
	Total	1,672,000

It has been assumed that about 25% of the material in Reach A, or about 50,000 cubic meters may be ledge rock and boulders, but that all of the remaining material will be unconsolidated sediment deposits. These deposits could easily be excavated by use of a dragline, or with a caterpillar-mounted shovel, equipped with a moderately long boom. The estimated cost of excavation, with either type equipment, is one Lebanese pound per cubic meter, for the unconsolidated material, and three Lebanese pounds per cubic meter, for the 50,000 cubic meters of the more difficult excavation in Reach A. Application of these prices indicate that the excavation required will cost about 1,770,000 Lebanese pounds. The channel realignment and rectification will require additional rights-of-way and damages, both for the new channel, and for disposal areas for the excavated material. Some of this material may be used to advantage to fill low-lying areas and improve surface drainage conditions and some of it may be deposited in the meanders and cutoff section of the existing channel. However, although such fills would result in additional land becoming available, their filling would require additional handling of the excavated material, which might make it uneconomical to do so. The length of this channel work is estimated at 36 kilometers and the width of the required rights-of-way assumed at an average of 35 meters over this entire distance. This results in about 125 hectares of land being required for such uses. At an assumed average valuation of 4,000 Lebanese pounds per hectare, these rights-of-way would cost about 500,000 Lebanese pounds. The total cost of excavation and right-of-way is about 2,250,000 Lebanese pounds. An allowance of about 25 percent has been added to provide for surveys, supervision, and contingencies which increases the cost of this entire project to 2,800,000 Lebanese pounds or 800,000 American dollars at an exchange rate of 3.50 Lebanese pounds for one dollar.

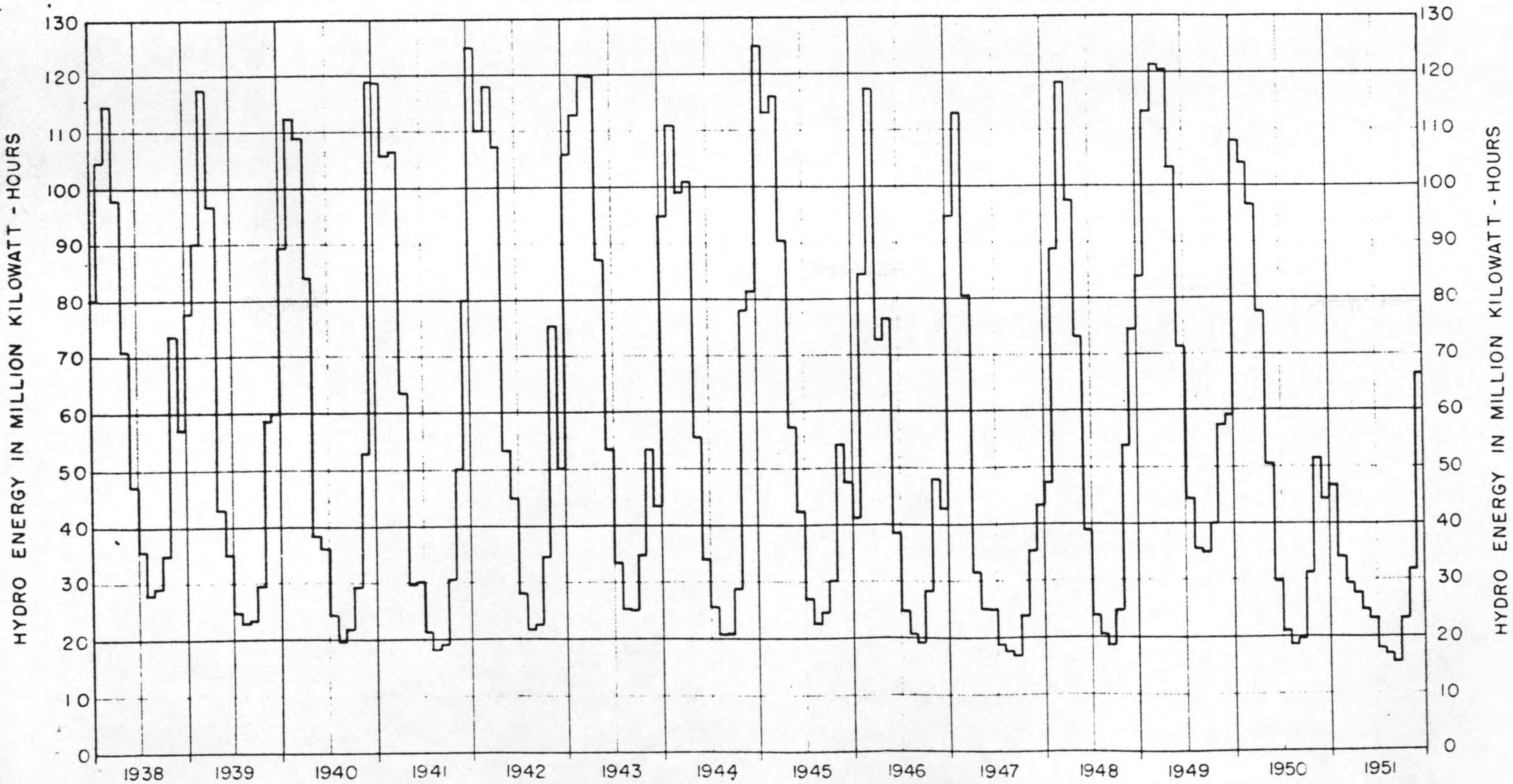
Expected Benefits. The results and benefits from the completion of this project were summarized by Abd-El-Al 1/ in 1948 as follows:

> "The works program being carried on, working upstream from Jib-Jannine to Mansoura, has not yet reached the point of confluence. There is no doubt that when it is completed the condition of the plain from the hydrological, agricultural and sanitary point of view, will be substantially improved. Rapid run-off of winter flood waters, drainage of swampy areas, lowering of the hydrostatic level of the watertable, which is too high for certain taproot crops, soil respiration through drainage, eradication of malaria, general improvement of an area of approximately 5000 hectares on both banks of the Litani - These are the results expected from the drainage works in the South Bekaa."

1/ Abd-El-Al, Ibrahim, The Litani Hydrologic Study, Beirut, Lebanon 1948.



RESERVOIR STORAGE



HYDRO POWER PRODUCTION

NOTE

The reservoir storage shown, is the storage available each month as determined by application of the adopted reservoir operation curves and the operation of all the units in the plan of development.

The hydro power production shown is that resulting from such operation throughout the 31-year period of the entire project.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
LITANI RIVER PROJECT-LEBANON
HYDRO POWER PRODUCTION
COMPLETE PROJECT
1938 THRU 1951

[redacted]

BEIRUT, LEBANON 2-20-54 SHEET 2 OF 2	OA-10-260

PLATE XXXV-6

SECTION XXXVII

COST ESTIMATES

General

The estimates of costs of construction work in this report are based upon material and labor conditions in Lebanon. Due allowances have been made for variations in wages and in the classes of labor required for different kinds of work and the working conditions at the particular location. Local costs have been taken into consideration where appropriate and have been correlated with United States cost data where conditions are at considerable variance with normal local experiences. Where no local data were available, United States cost averages were adjusted for the differential in labor rates currently in force in the two countries.

Adequate unskilled labor is available in the vicinity of the work. Local skilled labor is available but will require some training. Equipment operators with the exception of truck drivers must be trained. In general, Lebanese workmen properly trained, equipped and paid can be expected to produce approximately as well as similar labor in the United States.

Labor wage rates in Lebanon are increasing very slowly as compared with the cost of living and will probably not increase materially within the next few years. On this basis, we have assumed a rate of 5 ŁL. ($1.50) per 8-hour day for common labor and 10 ŁL. ($3.00) per day for skilled labor as compared for adjustment purposes, to $1.75 and $2.50 per hour, respectively, in the United States.

An analysis of the effect of Lebanese wage rates on U. S. costs of doing work requiring mechanization and foreign supervision is to lower the U. S. costs between 15 and 20 percent.

Tunnel and canal work in Lebanon is much cheaper than in the United States because of the extensive use of hand labor at low rate of pay. Tunnel costs as determined for this report will vary considerably, having been adjusted in each specific case to take into account the geological condition of the rock, the probable water conditions, the distances between headings, and the over-all difficulties which would indicate whether the work should be undertaken by a highly specialized and mechanized organization or by a local contractor. The use of a foreign contractor is more costly because of added cost of importing equipment and key personnel.

Cement is available from local mills, but all steel, lumber and items of major equipment will have to be imported. There are many small shops and manufacturing plants in Lebanon that can make small items of equipment, do machine work, fabricate structural items, and make castings.

The estimates of costs include appropriate allowances for engineering and overhead, and interest during construction at six percent per annum.

All estimates include additional sums for contingencies depending upon the hazards involved. For purposes of conversion of Lebanese pounds to dollars or vice-versa the average exchange rate of 3.5 Lebanese pounds (ŁL) to one dollar may be used.

Karaoun Dam and Reservoir

The cost estimate for raising the Karaoun Dam and Reservoir is based upon the assumption that the work will be done by modern mechanized methods of American standards. Unit costs near United States average costs have been assumed for this work. The estimate of cost for the second stage construction of Karaoun Dam is shown in Table XXXVII-1.

Experience during the first stage construction and future cost analyses may indicate the practicality of constructing the second stage embankment by the use of more primitive methods. The cheap labor and the great number of unemployed persons in Lebanon may warrant a long-time construction program for raising the second stage embankment. Present local practices on construction jobs stress a liberal number of common laborers (both men and women), a considerable number of draft animals, and a minimum amount of mechanical equipment. The long-time construction schedule can be adjusted so that additional reservoir storage can be provided as the demand occurs. The height of the embankment required at any time can be determined from a time-storage curve of the combined power-irrigation demand.

Advantages of this slow schedule, over a more rapid mechanized method, are: (a) the reservoir and foundation treatment if required, will be simplified (more time for observation will allow a more beneficial treatment program), (b) the investment loss will be minimized in the event of excessive leakage (progress can be halted until the leakage is safely reduced), (c) the reservoir storage can be more efficiently used, (d) only relatively small annual appropriations will be required of the financing agency, (e) repayments on the investment commence almost immediately (interest during construction is virtually eliminated), (f) the embankment settlement will be better controlled, and (g) the local labor force can be employed for a longer period.

A disadvantage of this slow construction method would be the necessity for passing flood discharges over the embankment during construction. This could be accomplished by providing a low saddle in the embankment at the first stage spillway area. Provisions will be required for adequate protection of the exposed impervious zone against erosion during the passage of floods until such time as the ultimate spillway can be utilized.

Bisri Dam and Reservoir

The construction of Bisri Dam and Reservoir will require the excavation and placement of large quantities of material. It is assumed that the work will be throughly mechanized and that local personnel will be utilized to operate equipment and as common labor. The nature of the work is outside the scope of experience of local contractors and the unknown factors involved in the methods and type of contractor that may undertake the work result in unit costs estimated near United States average costs for similar work. The estimate of cost for this feature is shown in Table XXXVII-2.

Khardale Dam and Reservoir

The construction of Khardale Dam and Reservoir will require the excavation and placement of large quantities of material. It is assumed that the work will be throughly mechanized and that local personnel will be utilized to operate equipment and as common labor. The nature of the work is outside the scope of experience of local contractors and the unknown factors involved in the methods and type of contractor that may undertake the work result in unit costs estimated near United States average costs for similar work. The estimate of cost for this feature is shown in Table XXXVII-3.

Markabi Diversion Dam

Unit costs for construction of Markabi Diversion Dam, as used for this report, are assumed to be similar for United States work of the same type and magnitude. The work may be included in the major contract for the adjacent Sohmor Power Plant and accomplished by a foreign contractor, or the work may be done under a relatively small separate contract by a local contractor. In either event, it is expected that the cost of construction for the dam will be approximately the same. The estimate of cost for Markabi Diversion Dam is shown in Table XXXVII-4.

Joun Diversion Dam

The work involved in the construction of the Joun Diversion Dam appears suitable for accomplishment by local contractors. However, close control of construction procedures and tolerances will be necessary to insure compliance with the designers' assumptions. The unit costs applied to this structure are slightly lower than comparable work in the United States. Even though plentiful local labor will be available, supervision and the procurement of much of the construction materials and equipment from foreign markets will offset the use of cheap labor and result in costs approximating those in countries using higher paid labor. The estimate of cost for Joun Diversion Dam is shown in Table XXXVII-5.

Kelia Forebay

The costs assumed for construction of Kelia Forebay anticipate the use of local contractors using little mechanization and much local labor. Work on this feature should be attractive to local contractors. The estimate of cost for this feature is shown in Table XXXVII-6.

Tunnels

Construction of the Awali, Joun, Markabi, Kelia, and Zrariye Tunnels will be ideal for local contractors as they are expected to be driven through fairly sound rock with no water of serious consequence and will have short reaches between portals or adits. The shortest reach being 1010 meters, the longest 3285 meters and the average for all sections being about 2200 meters. By driving from both headings the haulage distance would be halved. The cost estimates were based on unit prices derived from tunnels recently completed or now under construction in Lebanon. The estimates of cost for these tunnels are shown on Tables XXXVII-7, 8, 9, and 10.

The cost estimate for the Zaiye Tunnel was made somewhat higher than the other tunnels by increasing the unit prices for excavation 33 percent, and the unit price for concrete lining by 18 percent, to adjust for the additional cost resulting from the longer distance between portals, 7680 meters. The estimate of cost for the Zaiye Tunnel is shown on Table XXXVII-11.

Multiple Purpose Canals

The unit prices used for estimating the cost of the Markabi, Zaiye and Zrariye Canals were based on local prices for similar work recently completed in Lebanon except the unit prices for concrete structures and flumes were increased in anticipation of the additional cost that will result when stricter control for mixing and placing concrete is demanded in order to obtain better concrete than is now produced by local practices. The estimates of cost for these canals are shown on Tables XXXVII-12, 13, and 14.

Penstocks and Surge Tanks

The trenches could be excavated by hand methods using local labor or by mechanized equipment depending upon the urgency for their completion. The unit prices using either method will be somewhat less than in the United States for similar items of work. Prices for concrete, reinforcement steel; miscellaneous metal work, etc., will also be somewhat less than United States prices, due to the low wage scale in Lebanon. The steel penstock will have to be obtained in a foreign market, shipped to Lebanon and then installed under strict supervision. This will result in unit prices for furnishing and installing penstocks similar to United States prices. Prices applicable to surge tank construction are the same as these for tunnels. The estimates of cost for the penstocks and surge tanks are shown on Tables XXXVII-15, 16, 17 and 18.

Irrigation Units

The construction of canals, laterals, and canal structures constitutes the greatest share of the cost of the irrigation units. All of the canals to be built are relatively small and are suitable for construction by local methods. It is anticipated that this type of work will be performed by local contractors and that mechanical equipment used will be limited to such items as portable crushers for aggregate, trucks for hauling materials, and concrete mixers. The unit prices used for earthwork and minor concrete work are based on present costs of performing similar work in Lebanon, and are the same as those shown in the estimate of cost for the Bekaa Gravity Unit, Table XIX-5, Volume II.

Costs of such items as pumping plants and steel pipe were based on current United States prices, adjusted slightly as necessary to reflect the differences in local labor costs and site conditions.

Estimating the costs of distribution and drainage systems was made difficult by the lack of accurate detailed topography of the irrigable areas. Typical areas of each unit were selected and a distribution system was laid out for each, using available topographic maps. The cost of the system was then extended to the entire unit on the basis of area to be served. Unit costs for earthwork, masonry, and lining for all laterals are the same as were used for the larger canals.

Estimates of cost for irrigation units are shown in the tables listed below:

Bekaa Pumping Unit - Table XXXVII - 19

Upper Nabatiye Unit - Table XXXVII - 20

Lower Nabatiye Unit - Table XXXVII - 21

Saida-Beirut Unit - Table XXXVII - 22

Power Plants

Material and equipment costs for power plants, switchyards and substations were based upon an analysis of quotations from internationally known manufacturers and United States cost data adjusted to March 1953. It was assumed that the principal items of equipment would be furnished by European manufacturers. Labor costs for construction and installation were found to be generally lower in Lebanon than in the United States, and the over-all cost estimates for the power features take this into consideration. The resulting cost estimates for power plants, substations and switchyards are on a cost per installed kilowatt basis and do not take into consideration differences in individual sites. Except that the cost estimate for Zrariye Power Plant was modified in accordance with the detailed estimates made for the Phase "A" Sohmor Power Plant. Detailed data regarding the sites for the different features and additional cost data with the design date are included in Appendix VI. Cost estimates for the Phase "B" power plants are shown in Tables XXXVII-23 and 24.

Power Transmission System

The short transmission distances weighted against the amount of substation and terminal equipment favored the construction of a transmission system consisting of multiple 69-kv. lines. Transmission line costs were estimated on the basis that the Bureau of Reclamation standard designs for 69-kv. wood pole, H-frame construction would be applicable except for a few specially designed dead-end structures. Studies showed this to be the most economical type of construction. The terrain crossed by the lines is generally mountainous. The resulting estimates considering the cost of materials delivered to Lebanon, the local labor conditions, and right-of-way costs are generally about twenty percent less than the cost of similar lines in the United States. Estimates of cost for the transmission system required for Phase "B" development are shown in Tables XXXVII-23 and 25.

DC-489
(Mar. 54)

Formerly XD-3094

ESTIMATE SHEET
(Dams Branch)

DAM Earth and Rockfill	UNITED STATES	**SPILLWAY** Overflow
CREST EL. 859.9 M MAX. HT.	DEPARTMENT OF THE INTERIOR	MAX. CAP. 720 CMS
MAX. W.S. EL. 858 M CAP. 200,000,000 CM	BUREAU OF RECLAMATION	Crest Length: 125.5 M
NOR. W.S. EL. 856 M CAP. 160,000,000 CM	Litani River Project--Lebanon	Crest Elevation: 856 M
MIN. W.S. EL. 820.0 M CAP. 6,400,000 CM	Karaoun Dam, Spillway, and Reservoir	GATES None
IRRIGATION STORAGE None	Estimate of Cost	**OUTLET WORKS** Same design as Stage I
FLOOD STORAGE	Stage II	CAP. 65 CMS AT W.S. EL. 858 M
	Table XXXVII--1	
COMPUTED............ CHECKED.......		DATE April 7, 1954 REF. DWG............

ITEM No.	(Quantities are in metric units) (Costs are in Lebanese pounds £L.) ITEM	QUANTITY		EQUIPMENT MATERIAL AND LABOR		MISCELLANEOUS		SUMMARY	
		AMOUNT	UNIT	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST
	DAM STRUCTURE--STAGE II								
1	Diversion and care of river during construction and unwatering foundation	Lump Sum	ls	L.S.	15,000				
2	Excavation, stripping borrow area	61,000	cuM	1.10	67,100				
3	Excavation, all classes, dam foundation	88,000	cuM	3.00	264,000				
4	Excavation, rock, in quarries, and transportation to dam embankment	830,000	cuM	8.20	6,806,000				
5	Excavation, common, in borrow area, separation, and transportation to dam embankment	304,000	cuM	3.40	1,033,600				
6	Removing rockfill from downstream face of Stage I dam	34,000	cuM	3.50	119,000				
7	Separating and transporting stockpiled material to dam embankment	96,000	cuM	2.60	249,600				
8	Earthfill in embankment	306,000	cuM	0.80	244,800				
9	Special compaction of earthfill	1,900	cuM	10.00	19,000				
10	Rock fines in embankment	156,000	cuM	0.55	85,800				
11	Rockfill in embankment	1,260,000	cuM	1.15	1,449,000				
12	Concrete in cutoff walls--through Stage I spillway	230	cuM	90.00	20,700				
13	Reinforcement steel	14,000	kg	0.85	11,900				
14	Breaking and removing Stage I spillway wall	120	cuM	45.00	5,400				
15	Grouting Stage I spillway drains	70	cuM	200.00	14,000				
16	Furnishing and handling cement	188	M.T.	88.00	16,500				
	Subtotal				10,421,400				
	Contingencies (20% ±)				2,088,600				
	Field Total--Dam Structure--Stage II				12,510,000				12,510,000
	SPILLWAY--STAGE II								
17	Excavation, rock, in open cut	85,060	cuM	14.00	1,190,840				
18	Concrete in weir crest	655	cuM	95.00	62,225				
19	Concrete in floors and cutoff walls	943	cuM	90.00	84,870				
20	Concrete in retaining walls	115	cuM	160.00	18,400				
21	Concrete in gravity walls and abutments	434	cuM	95.00	41,230				
22	Concrete in side lining	1,410	cuM	120.00	169,200				
23	Concrete in bridge	90	cuM	210.00	18,900				
24	Reinforcement	201,220	kg	0.85	171,037				
25	Drilling holes for anchor bars and grouting bars in place	3,280	lM	18.00	50,040				
26	Cement	1,302	M.T.	88.00	114,600				
27	Gravel blanket	45	cuM	20.00	900				
28	Drilling grout holes 12M deep	492	lM	22.50	11,070				
29	Pressure grouting	92	cuM	160.00	14,720				
30	Sewer pipe drains, 4-inch, with uncemented joints in gravel	370	lM	12.00	4,440				
31	Sewer pipe drains, 6-inch, with uncemented joints in gravel	300	lM	14.00	4,200				
32	Sewer pipe drains, 8-inch, with uncemented joints in gravel	160	lM	18.00	2,880				
33	Pipe handrail	1,500	kg	3.50	5,250				
	Subtotal				1,973,802				
	Contingencies (20% ±)				396,198				
	Field Total--Spillway--Stage II				2,370,000				2,370,000
	Total Field Cost, Dam and Spillway--Stage II				14,880,000				14,880,000
	Engineering, Supervision, Administration, and General Expense (10% ±)				1,488,000				1,488,000
	Subtotal--Estimated Cost of Dam and Spillway without interest				16,368,000				16,368,000
	MISCELLANEOUS--STAGE II								
34	Land and land rights	Lump Sum				L.S.	240,000		
35	Relocation of existing property	Lump Sum				L.S.	20,000		
36	Clearing reservoir area	85	Ha			500	44,000		
37	Roads and bridges	Lump Sum				L.S.	50,000		
38	Camps and housing	Lump Sum				L.S.	100,000		
39	Treating reservoir area	Lump Sum				L.S.	350,000		
	Subtotal for Miscellaneous						804,000		
	Administration and General Expenses (3.5% ±)						28,000		
	Total Miscellaneous						832,000		832,000
	Total estimated cost of dam and appurtenances without interest								17,200,000
	Interest during construction (6% ±)								1,032,000
	Total cost of dam, reservoir, and appurtenances, Stage II								18,232,000

DC-489
(Mar. 54)

Formerly XD-3094

ESTIMATE SHEET
(Dams Branch)

DAM	UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION	SPILLWAY Controlled
CREST EL. 432 M MAX.HT. 42M	Litani River Project--Lebanon	MAX. CAP. 670 CMS
MAX.W.S.EL. 430 M CAP. 32,100,000 cuM	Bisri Dam	Crest El. 422.55 M
NOR.W.S.EL. 430 M CAP. 32,100,000 cuM	Impervious Core--Earth Fill Type	
MIN.W.S.EL 422.55 M CAP. 16,300,000 cuM	Estimate of Cost	GATES Two--7.45 x 7.45 M. Radial
IRRIGATION STORAGE A.F.	Table XXXVII--2	OUTLET WORKS Discharge over spillway
FLOOD STORAGE		CAP. AT W.S.EL.
COMPUTED G.D.--R.J.H.. CHECKED. RJH		DATE Feb. 16, 1954 REF. DWG. OA-10-196

All quantities are shown in metric units
Costs are shown in Lebanese Pounds (Ŀ.L.)

Average annual monetary exchange is assumed as $1 U.S. = Ŀ.L. 3.50

ITEM No.	ITEM	QUANTITY AMOUNT	UNIT	EQUIPMENT MATERIAL AND LABOR UNIT COST	EQUIPMENT MATERIAL AND LABOR TOTAL COST	MISCELLANEOUS UNIT COST	MISCELLANEOUS TOTAL COST	SUMMARY UNIT COST	SUMMARY TOTAL COST
	Items for Dam								
1	Diversion and care of river during construction and unwatering dam foundation	Lump sum		ls	500,000				
2	Excavation, common, for dam foundation	450,000	cuM	2.75	1,237,500				
3	Excavation, common, for spillway and diversion channels	1,060,000	cuM	2.00	2,120,000				
4	Excavation, rock, in spillway and diversion tunnels	56,000	cuM	35.00	1,960,000				
5	Excavation, rock, in open cut, for spillway	7,000	cuM	9.00	63,000				
6	Excavation, common, separation in borrow pits, and transportation to embankment	220,000	cuM	2.30	506,000				
7	Excavation, rock, in quarries, grading, and transportation to embankment	257,000	cuM	8.20	2,107,400				
8	Earthfill in embankment	1,730,000	cuM	0.75	1,297,500				
9	Rock fill in embankment	320,000	cuM	1.25	400,000				
10	Grouting dam foundation and tunnels	Lump sum		ls	1,100,000				
11	Concrete in tunnel linings	12,200	cuM	140.00	1,708,000				
12	Concrete in spillway structure	1,600	cuM	125.00	200,000				
13	Concrete in miscellaneous structures	4,000	cuM	200.00	800,000				
14	Reinforcement bars	710,000	kg	0.85	603,500				
15	Tunnel supports	650,000	kg	1.70	1,105,000				
16	Radial gates and hoists	75,000	kg	3.90	292,500				
17	Miscellaneous metalwork	4,000	kg	2.70	10,800				
	Subtotal for dam and appurtenances				16,011,200				
	Contingencies, engineering, superintendence, and accounts 25% ±				3,988,800				
	Estimated cost of dam and appurtenances without interest during construction				20,000,000				
	Items for Reservoir								
18	Land, land rights, and relocation of existing property	Lump sum		ls	2,000,000				
19	Clearing reservoir area	80	Ha	500.00	40,000				
20	Roads	Lump sum		ls	150,000				
21	Camps	Lump sum		ls	150,000				
	Subtotal for reservoir				2,340,000				
	Administration and general expense 3.5% ±				90,000				
	Estimated cost of reservoir without interest during construction				2,430,000				
	Estimated cost of dam, appurtenances, and reservoir without interest during construction				22,430,000				
	Interest at 6% ± (1.5 years completion)				2,070,000				
	Total estimated cost of dam, appurtenances, and reservoir, including interest during construction				Ŀ.L. 24,500,000				

DC-489
(Mar. 54)

Formerly XD-3094

ESTIMATE SHEET
(Dams Branch)

DAM	UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION Litani River Project--Lebanon Khardale Dam Impervious Core--Earth Fill Type Estimate of Cost Table XXXVII--3	**SPILLWAY** Controlled
CREST EL. 296.40 M MAX.HT. 76.4 M		MAX. CAP. 1,100 CMS
MAX.W.S.EL. 295 M CAP. 82,500,000 [redacted] cuM		Crest El. 288 M
NOR.W.S.EL. 295 M CAP. 82,500,000 [redacted] cuM		
MIN.W.S.EL. 257 M CAP. 5,500,000 [redacted] cuM		GATES Three--9.00 x 7.00 M. Radial
IRRIGATION STORAGE		OUTLET WORKS Two--1.00 M dia. H.J.Valves
FLOOD STORAGE		CAP. 180 CMS AT W.S. EL. 295 M
COMPUTED C.D. ... R.J.H. CHECKED RJH		DATE .... REF. DWG ....

ITEM No.	All quantities are shown in metric units Costs are shown in Lebanese Pounds (L.L.) ITEM Average annual monetary exchange is assumed as $1 U.S. = L.L. 3.50	QUANTITY		EQUIPMENT MATERIAL AND LABOR		MISCELLANEOUS		SUMMARY	
	Items for Dam	AMOUNT	UNIT	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST
1	Diversion and care of river during construction and unwatering dam foundation	Lump sum		ls	350,000				
2	Excavation, rock, in open cut, for dam foundation	50,000	cuM	9.00	450,000				
3	Excavation, rock, in open cut, for spillway structure, diversion channels, and river outlet structures	63,000	cuM	9.00	567,000				
4	Excavation, rock, in diversion tunnel and river outlet shaft	13,000	cuM	35.00	455,000				
5	Excavation, rock, in quarries, grading, and transportation to embankment	303,000	cuM	8.20	2,484,600				
6	Excavation, common, separation in borrow pits, and transportation to embankment	720,000	cuM	2.50	1,800,000				
7	Earthfill in embankment	720,000	cuM	0.80	576,000				
8	Rock fill in embankment	428,000	cuM	1.15	492,200				
9	Grouting dam foundation	Lump sum		ls	1,100,000				
10	Concrete in diversion tunnel and river outlet shaft linings	3,900	cuM	140.00	546,000				
11	Concrete in spillway structure	8,150	cuM	120.00	978,000				
12	Concrete in river outlet trashrack structure and valve house	200	cuM	150.00	30,000				
13	Concrete in miscellaneous structures	1,000	cuM	130.00	130,000				
14	Reinforcement bars	1,047,000	kg	0.85	889,950				
15	Diversion tunnel supports	263,200	kg	1.70	447,440				
16	River outlet pipe	61,000	kg	2.50	152,500				
17	Hollow jet valves and controls	12,000	kg	4.25	51,000				
18	Radial gates and hoists	82,000	kg	3.90	319,800				
19	Miscellaneous metalwork	14,000	kg	2.70	37,800				
	Subtotal for dam and appurtenances				11,857,290				
	Contingencies 15% ±				1,778,710				
	Subtotal				13,636,000				
	Engineering, superintendence, and accounts 10% ±				1,364,000				
	Estimated cost of dam and appurtenances without interest during construction				15,000,000				
	Items for Reservoir								
20	Land and land rights	Lump sum		ls	790,000				
21	Relocation of existing property	Lump sum		ls	1,500,000				
22	Clearing reservoir area	Lump sum		ls	25,000				
23	Roads and bridges	Lump sum		ls	140,000				
24	Camps	Lump sum		ls	300,000				
	Subtotal for reservoir				2,755,000				
	Administration and general expense 3.5% ±				95,000				
	Estimated cost of reservoir without interest during construction				2,850,000				
	Estimated cost of dam, appurtenances, and reservoir without interest during construction				17,850,000				
	Interest at 6% ± (1.5 years completion)				1,610,000				
	Total estimated cost of dam, appurtenances, and reservoir, including interest during construction				L.L.19,460,000				

DC-489
(Mar. 54)

Formerly XD-3094

ESTIMATE SHEET
(Dams Branch)

DAM		UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION Litani River Project--Lebanon MARKABI DIVERSION DAM Concrete--Gravity Type Estimate of Cost	SPILLWAY Uncontrolled
CREST EL.	MAX.HT. 4 M		MAX.CAP. 180 CMS
MAX.W.S.EL. 662.15 M	CAP.		Crest El. 660 M
NOR.W.S.EL. 660 M	CAP.		
MIN.W.S.EL. 660 M	CAP.		GATES -- None
IRRIGATION STORAGE			OUTLET WORKS
FLOOD STORAGE			CAP. AT W.S.EL.
COMPUTED......C.D.......	CHECKED. RJH..	Table XXXVII--4	DATE March 4, 195 REF. DWG. 0A-10-53....

ITEM No.	ITEM All quantities are shown in metric units Costs are shown in Lebanese pounds (Ł.L.) Average annual monetary exchange is assumed as $1 U.S. = Ł.L. 3.50	QUANTITY		EQUIPMENT MATERIAL AND LABOR		MISCELLANEOUS		SUMMARY	
		AMOUNT	UNIT	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST
	Items for Dam								
1	Diversion, care of river during construction and unwatering foundations	Lump Sum		ls	Ł.L. 53,000				
2	Excavation, all classes in open cut, for dam foundation	7,700	cu M	9.00	69,300				
3	Backfilll	65	cu M	16.00	1,040				
4	Riprap	185	cu M	9.00	1,665				
5	Concrete in spillway crest	540	cu M	125.00	67,500				
6	Concrete in spillway retaining walls	190	cu M	200.00	38,000				
7	Concrete in spillway channel lining	115	cu M	150.00	17,250				
8	Concrete in miscellaneous structures	65	cu M	225.00	14,625				
9	Reinforcement bars	55,000	kg	0.85	46,750				
10	Slide gates, frames, and hoists	15,500	kg	4.60	71,300				
11	Miscellaneous metalwork	500	kg	5.00	2,500				
	Subtotal for dam and appurtenances				Ł.L. 382,930				
	Contingencies, engineering, superintendence, and accounts 25% ±				101,070				
	Estimated cost of dam and appurtenances without interest during construction				Ł.L. 484,000				
	Items for Reservoir								
12	Roads and bridges	Lump Sum		ls	Ł.L. 50,000				
	Subtotal for reservoir				Ł.L. 50,000				
	Administration and general expense 3.5% ±				3,000				
	Estimated cost of reservoir without interest during construction				Ł.L. 53,000				
	Estimated cost of dam, appurtenances, and reservoir without interest during construction				Ł.L. 537,000				
	Interest during construction 6% ±				32,000				
	Total estimated cost of dam, appurtenances, and reservoir, including interest during construction				Ł.L. 569,000				

DC-489
(Mar. 54)

Formerly XD-3094

ESTIMATE SHEET
(Dams Branch)

Study J-2

DAM		UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION	SPILLWAY Uncontrolled
CREST EL. 220 M.	MAX.HT. 34 M.		MAX.CAP. 115 CMS (100 yr. flood)
MAX.W.S.EL. 219 M.	CAP. 385,000 cuM		Crest El. 217 M.
NOR.W.S.EL. 217 M.	CAP. 270,000 cuM		
MIN.W.S.EL. 212 M.	CAP. 80,000 cuM	Litani River Project--Lebanon	GATES None
IRRIGATION STORAGE		Joun Diversion Dam	OUTLET WORKS
FLOOD STORAGE	115,000 cuM	Slab and Buttress Type	CAP. 70 CMS. AT W.S.EL. 219 M.
			Two--1.50 M x 2.10 M Slide Gates
COMPUTED.....RJH.......	CHECKED Rjh-CD.	Estimate of Cost Table XXXVII--5	DATE Feb. 23, 1954 REF. DWG. 0A-10-275...

ITEM No.	All quantities are shown in metric units. Costs are shown in Lebanese Pounds (ⱠL.) ITEM Average annual monetary exchange is assumed as $1 U.S. = ⱠL. 3.50	QUANTITY		EQUIPMENT MATERIAL AND LABOR		MISCELLANEOUS		SUMMARY	
		AMOUNT	UNIT	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST
	Items for Dam								
1	Diversion, care of river during construction, and unwatering foundation	Lump Sum		ls	ⱠL. 160,000				
2	Excavation river fill, for foundation of dam	16,000	cuM	2.75	44,000				
3	Excavation, rock for foundation of dam	2,000	cuM	9.00	18,000				
4	Pressure grouting, dam foundation	Lump Sum		ls	125,000				
5	Concrete in foundation cutoff wall	1,300	cuM	90.00	117,000				
6	Concrete in buttress footings	460	cuM	100.00	46,000				
7	Concrete in deck slabs, corbels, and buttresses	4,100	cuM	200.00	820,000				
8	Concrete outlet works	520	cuM	175.00	91,000				
9	Concrete in parapets, walks, and training walls	430	cuM	225.00	96,750				
10	Miscellaneous concrete	50	cuM	175.00	8,750				
11	Reinforcement bars	250,000	kg	0.85	212,500				
12	Slide gates, guides and hoists	17,000	kg	4.00	68,000				
13	Trashrack metal work	8,800	kg	2.30	20,240				
14	Miscellaneous metalwork	7,000	kg	2.50	17,500				
	Subtotal for dam and appurtenances				ⱠL.1,844,740				
	Contingencies, engineering, administration, and accounts 30% ±				555,260				
	Estimated cost of dam and appurtenances without interest during construction				ⱠL.2,400,000				
	Items for Reservoir								
15	Roads	Lump Sum		ls	50,000				
16	Camp	Lump Sum		ls	100,000				
	Subtotal for reservoir				ⱠL. 150,000				
	Administration and general expense				5,000				
	Estimated cost of reservoir without interest during construction				ⱠL. 155,000				
	Estimated cost of dam, appurtenances, and reservoir without interest during construction				ⱠL.2,555,000				
	Interest during construction 9% ±				235,000				
	Total estimated cost of dam, appurtenances and reservoir including interest during construction				ⱠL.2,790,000				

7-1493 (formerly XD-853)
(Apr.-51)
Bureau of Reclamation

ESTIMATE SHEET

DAM
Crest el. 629 M. Max.Ht. 15 M.
Max.W.S.el.627.40 M.Cap.678,000 cuM
Nor.W.S.el.626.50 M.Cap.558,000 cuM
Min.W.S.el.624.50 M.Cap.322,000 cuM

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
Litani River Project--Lebanon
Kelia Forebay
Impervious Core--Rock Fill Type
Estimate of Cost
Table XXXVII--6

SPILLWAY Uncontrolled
Max. Cap. 35.4 C.M.S.
Crest el. 626.50
Gates None
OUTLET WORKS None

ITEM No.	ITEM	QUANTITY		EQUIPMENT MATERIAL AND LABOR		MISCELLANEOUS		SUMMARY	
		AMOUNT	UNIT	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST
1	Diversion and care of intermittent stream during construction	Lump Sum	cuM	L.S.	£L. 10,000				
2	Excavation common, for dam foundation	6,800	cuM	3.00	20,400				
3	Excavation rock for cutoff trench	350	cuM	9.00	3,150				
4	Excavation, common, for spillway channel	2,600	cuM	3.00	7,800				
5	Excavation rock, for spillway channel	3,500	cuM	9.00	31,500				
6	Excavation, common, in borrow pits, separation, and transportation to embankment	1,700	cuM	3.00	5,100				
7	Grading excavated rock from tunnels and transportation to embankment	26,200	cuM	4.00	104,800				
8	Earth fill in embankment	11,100	cuM	0.75	8,325				
9	Rock fill in embankment	30,000	cuM	1.25	45,825				
10	Grouting dam foundation	Lump Sum		L.S.	120,000				
11	Concrete in bridge, pier, parapet, and retaining walls	320	cuM	125.00	40,000				
12	Concrete in spillway crest	120	cuM	100.00	12,000				
13	Concrete in spillway channel lining	280	cuM	80.00	22,400				
14	Reinforcement bars	58,000	kg	0.85	49,300				
	Subtotal for dam				£L.472,275				
	Contingencies, engineering superintendence and accounts 25% ±				117,725				
	Estimated cost of dam without interest during construction				£L 59,000				
	Items for Reservoir								
15	Land and land rights	17	Ha	L.S.	100,000				
16	Roads	Lump Sum		L.S.	12,000				
17	Camps	Lump Sum		L.S.	50,000				
	Subtotal for reservoir				£L.162,000				
	Administration and general expense (3.5% ±)				6,000				
	Estimated cost of reservoir without interest during construction				£L.168,000				
	Estimated cost of dam and reservoir without interest during construction				£L.758,000				
	Interest during construction 6% ±				46,000				
	Total estimated cost of dam and reservoir including interest during construction				£L.804,000				

Notes:
All quantities shown are in metric units.
Costs are shown in Lebanese Pounds (£L.)
Average annual monetary exchange is assumed as $1 = £L. 3.50.

7-1493 (formerly XII-853)
(Apr.-51)
Bureau of Reclamation

ESTIMATE SHEET

Circular pressure tunnel
Concrete lined
Diameter--3.10 M.
Length--3,265 M.
Inlet elevation--417 M.
Outlet elevation--395.60 M.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
Litani River Project--Lebanon
Awali Tunnel
Estimate of Cost
Table XXXVII--7

ITEM No.	ITEM	QUANTITY		EQUIPMENT MATERIAL AND LABOR		MISCELLANEOUS		SUMMARY	
		AMOUNT	UNIT	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST
1	Excavation in tunnel	34,500	cuM	60.00	2,070,000				
2	Concrete lining of tunnel	9,840	cuM	110.00	1,082,400				
3	Timber lagging	200	cuM	300.00	60,000				
4	Steel tunnel supports	143,000	kg	1.30	185,900				
5	Reinforcement bars	160,000	kg	0.85	136,000				
6	Roof bolts	9,400	lm	11.50	108,100				
7	Drilling grout holes	2,200	lm	11.50	25,300				
8	Grout connections	2,800	kg	3.85	10,800				
9	Grouting	520	cuM	275.00	143,000				
10	Intake structure and gate chamber	Lump Sum		L.S.	125,000				
	Subtotal				£L3,946,500				
	Contingencies (20% ±)				789,500				
	Subtotal				£L4,736,000				
	Engineering, administration, and accounts(10%±)				474,000				
	Estimated cost of tunnel without interest during construction				£L5,210,000				
11	Access roads	2,000	M	30.00	60,000				
12	Camps	Lump Sum		L.S.	100,000				
	Subtotal				£L 160,000				
	Administration and general expense (3.5% ±)				6,000				
	Estimated cost of access roads and camps without interest during construction				£L 166,000				
	Item for tunnel				5,210,000				
	Estimated cost of tunnel, access roads and camps without interest during construction				£L5,376,000				
	Interest 6% ± (2 years completion)				324,000				
	Total estimated cost of tunnel, access roads and camps with interest during construction				£L5,700,000				

Notes:
All quantities are shown in metric units
Costs are shown in Lebanese Pounds (£L.)
Average annual monetary exchange is assumed as $1 = £L. 3.50

7-1493 (formerly XR-853)
(Apr.-51)
Bureau of Reclamation

ESTIMATE SHEET

Circular Pressure Tunnel
Concrete lined
Diameter--2.80 M.
Length--6,400 M.
Inlet elevation--208 M.
Outlet elevation--173.44 M.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
Litani River Project--Lebanon
Joun Tunnel
Estimate of Cost
Table XXXVII--8

ITEM No.	ITEM	QUANTITY		EQUIPMENT MATERIAL AND LABOR		MISCELLANEOUS		SUMMARY	
		AMOUNT	UNIT	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST
1	Excavation in tunnel	54,530	cuM	60.00	3,271,800				
2	Concrete lining of tunnel	15,110	cuM	110.00	1,662,100				
3	Timber lagging	270	cuM	300.00	81,000				
4	Steel tunnel supports	200,000	kg	1.30	260,000				
5	Reinforcement bars	200,000	kg	0.85	170,000				
6	Roof bolts	29,200	M	11.50	335,800				
7	Drilling grout holes	4,300	M	11.50	49,500				
8	Grout connection	5,450	kg	3.85	21,000				
9	Grouting	900	cuM	275.00	247,500				
	Subtotal for tunnel				LL6,098,700				
	Contingencies (20%±)				1,219,700				
	Subtotal				LL7,318,400				
	Engineering, administration, and accounts (10%±)				731,600				
	Estimated cost of tunnel without interest during construction				LL8,050,000				
10	Access roads	4,500	M	30.00	135,000				
11	Camps	Lump Sum		L.S.	150,000				
	Subtotal				LL 285,000				
	Administration and general expense (3.5%±)				10,000				
	Estimated cost of access roads and camps without interest during construction				LL. 295,000				
	Items for tunnel				8,050,000				
	Estimated cost of tunnel, access roads, and camps without interest during construction				LL8,345,000				
	Interest 6% ± (3 years completion)				755,000				
	Total estimated cost of tunnel, access roads, and camps with interest during construction				LL9,100,000				

Notes:
All quantities are shown in metric units
Costs are shown in Lebanese Pounds (LL.)
Average annual monetary exchange is assumed as $1 = LL. 3.50
Estimate does not include items for valve structure and penstock.

7-1493 (formerly XD-853)
(Apr.-51)
Bureau of Reclamation

ESTIMATE SHEET

Markabi tunnel	Kelia tunnel	
Free flow, horseshoe	Circular pressure	UNITED STATES
Concrete lined	Concrete lined	DEPARTMENT OF THE INTERIOR
Diameter--3.00 M.	Diameter--2.90 M.	BUREAU OF RECLAMATION
Length--7.750	Length--1,010 M.	Litani River Project--Lebanon
Inlet elevation 657.50	Outlet elevation 617.50	Markabi and Kelia Tunnels
		Estimate of Cost
		Table XXXVII--9

ITEM No.	ITEM	QUANTITY		EQUIPMENT MATERIAL AND LABOR		MISCELLANEOUS		SUMMARY	
		AMOUNT	UNIT	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST
1	Excavation in tunnels	91,600	cuM	60.00	5,496,000				
2	Concrete lining of tunnels	27,210	cuM	110.00	2,993,100				
3	Timber lagging	855	cuM	300.00	256,500				
4	Steel tunnel support	745,000	kg	1.30	968,500				
5	Reinforcement bars	200,000	kg	0.85	170,000				
6	Roof bolts	10,600	lm	11.50	121,900				
7	Drilling grout holes	5,900	lm	11.50	67,800				
8	Grout connection	7,450	kg	3.85	28,700				
9	Grouting behind lining	1,315	cuM	275.00	361,600				
10	Chute stilling pool	Lump sum		L.S.	30,000				
	Subtotal				LL10494,100				
	Contingencies (20%±)				2098,900				
	Subtotal				LL12593,000				
	Engineering, administration and accounts(10%±)				1259,000				
	Estimated cost of tunnel without interest during construction				LL13852,000				
11	Access roads	Lump Sum		L.S.	100,000				
12	Camps	Lump Sum		L.S.	250,000				
	Subtotal				350,000				
	Administration and general expenses (3.5%±)				12,000				
	Estimated cost of access roads and camps without interest during construction				LL 362,000				
	Items for tunnels				13,852,000				
	Estimated cost of tunnel, access roads and camps without interest during construction				LL14,214,000				
	Interest 6%± (2 years completion)				853,000				
	Total estimated cost of tunnels, access roads and camps with interest during construction				LL15,067,000				
	Notes:								
	All quantities are in metric units								
	Costs are shown in Lebanese Pounds (LL)								
	Average annual monetary exchange is assumed as $1 = LL. 3.50								
	Estimate does not include items for valve structure and penstock.								

7-1493 (formerly XI-853)
(Apr.-51)
Bureau of Reclamation

ESTIMATE SHEET

Horsehoe Free Flow Tunnel
Concrete Lined
Diameter : 2.20 M.
Length : 2325 M.
Inlet Elevation : 224.40 M.
Outlet Elevation: 219.75 M.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
Litani River Project--Lebanon
Zrariye Tunnel
Estimate of Cost
Table XXXVII--10

ITEM No.	ITEM	QUANTITY		EQUIPMENT MATERIAL AND LABOR		MISCELLANEOUS		SUMMARY	
		AMOUNT	UNIT	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST
1	Excavation in tunnel	12,950	cuM	60.00	777,000				
2	Concrete lining of tunnel	3,620	cuM	110.00	398,200				
3	Timber lagging	70	cuM	300.00	21,000				
4	Steel tunnel supports	52,000	kg	1.30	67,600				
5	Reinforcement bars	17,250	kg	0.85	14,700				
6	Roof bolts	7,200	M	11.50	82,800				
7	Drilling grout holes	1,560	M	11.50	17,900				
8	Grout connections	1,980	kg	3.85	7,600				
9	Grouting	255	cuM	275.00	70,100				
	Subtotal for tunnel				LL1,456,900				
	Contingencies (20%±)				291,600				
	Subtotal				LL1,748,500				
	Engineering, administration, and accounts (10%±)				174,500				
	Estimated cost of tunnel without interest during construction				LL1,923,000				
10	Access roads	Lump Sum		L.S.	45,000				
11	Camps	Lump Sum		L.S.	25,000				
	Subtotal				70,000				
	Administration and general expense (3.5%±)				2,000				
	Estimated cost of access roads and camps without interest during construction				72,000				
	Items for tunnel				1,923,000				
	Estimated cost of tunnel, access roads, and camps without interest during construction				LL1,995,000				
	Interest 6%± (2 years completion)				120,000				
	Total estimated cost of tunnel, access roads, and camps with interest during construction				LL2,115,000				

Notes:
All quantities are shown in metric units
Costs are shown in Lebanese Pounds (L.L.)
Average annual monetary exchange is assumed as $ 1 = L.L. 3.50
Estimate does not include items for storage canal, valve structure, and penstock

7-1493 (formerly XD-853)
(Apr.-51)
Bureau of Reclamation

ESTIMATE SHEET

Horseshoe, Free Flow Tunnel
Concrete lined
Diameter : 2.2 M.
Length: 7,680 M.
Inlet Elevation : 250 M.
Outlet Elevation : 234.64 M.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
Litani River Project--Lebanon
Zaiye Tunnel
Estimate of Cost
Table XXXVII--11

ITEM No.	ITEM	QUANTITY		EQUIPMENT MATERIAL AND LABOR		MISCELLANEOUS		SUMMARY	
		AMOUNT	UNIT	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST
1	Excavation in tunnel	43,200	cuM	80.00	3,456,000				
2	Concrete lining of tunnel	12,380	cuM	130.00	1,609,400				
3	Timber lagging	330	cuM	300.00	99,000				
4	Steel tunnel supports	240,000	kg	1.30	312,000				
5	Reinforcement bars	22,500	lm	0.85	19,100				
6	Roof bolts	22,250	lm	11.50	255,900				
7	Drilling grout holes	5,150	lm	11.50	59,200				
8	Grout connections	6,500	kg	3.85	25,000				
9	Grouting	850	cuM	275.00	233,800				
10	Inlet structure	Lump Sum		L.S.	50,000				
11	Gate chamber	Lump Sum		L.S.	135,000				
	Subtotal				LL6,254,400				
	Contingencies(20%±)				1,250,600				
	Subtotal				LL7,505,000				
	Engineering, administration, and accounts (10%±)				750,000				
	Estimated cost of tunnel without interest during construction				LL8,255,000				
12	Access roads	Lump Sum		L.S.	60,000				
13	Camps	Lump Sum		L.S.	75,000				
	Subtotal				LL 135,000				
	Administration and general expense (3.5%±)				5,000				
	Estimated cost of access roads and camps without interest during construction				LL 140,000				
	Items for tunnel				8,255,000				
	Estimated cost of tunnel, access roads, and camps without interest during construction				LL8,395,000				
	Interest 6% ± (4 years completion)				1,005,000				
	Total estimated cost of tunnel, access roads and camps with interest during construction				LL9,400,000				

Notes:
All quantities are shown in metric units
Costs are shown in Lebanese Pounds (LL.)
Average annual monetary exchange is assumed as $1 = LL. 3.50

7-1493 (formerly XI-853)
(Apr.-51)
Bureau of Reclamation

ESTIMATE SHEET

Concrete lined
Q = 20.9 C.M.S.
Length: 9,550

United States
Department of the Interior
Bureau of Reclamation
Litani River Project--Lebanon
Markabi Canal
Estimate of Cost
Table XXXVII--12

ITEM No.	ITEM	QUANTITY		EQUIPMENT MATERIAL AND LABOR		MISCELLANEOUS		SUMMARY	
		AMOUNT	UNIT	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST
1	Excavation, all classes, in open cut	185,000	cuM	7.00	1,300,000				
2	Compacted fill	54,000	cuM	0.85	46,000				
3	Gravel backfill	300	cuM	1.00	300				
4	Concrete lining in canal	8,700	cuM	60.00	522,000				
5	Concrete in flume	2,150	cuM	100.00	215,000				
6	Concrete in spillways	30	cuM	125.00	4,000				
7	Reinforcement bars	215,000	kg	0.80	172,000				
8	Circular culverts	17	ea	1500.00	25,500				
9	Single laned concrete bridge	6	ea	10,000	60,000				
10	Double laned concrete bridge	1	ea	20,000	20,000				
	Subtotal				ĿL2,364,800				
	Contingencies 15% ±				355,200				
	Subtotal				ĿL2,720,000				
	Engineering, administration, and accounts 10% ±				272,000				
	Estimated cost of canal without interest during construction				ĿL2,992,000				
11	Road relocation	Lump Sum		L.S.	25,000				
12	Right of way	Lump Sum		L.S.	55,000				
13	Camps	Lump Sum		L.S.	100,000				
	Subtotal				ĿL 180,000				
	Administration and general expense 3.5 % ±				6,000				
	Estimated cost of roads, right of way, and camps without interest during construction				ĿL 186,000				
	Items for canal				2,992,000				
	Estimated cost of canal, roads, right of way, and camps without interest during construction				ĿL3,178,000				
	Interest 6% ± (3 years completion)				286,000				
	Estimated cost of canal, roads, right of way, and camps with interest during construction				ĿL3,464,000				

Notes:
All quantities are shown in metric units
Costs are shown in Lebanese Pounds (ĿL)
Average annual monetary exchange is assumed as $1 U.S. = Ŀ.L. 3.50

7-1493 (formerly XD-853)
(Apr.-51)
Bureau of Reclamation

ESTIMATE SHEET

Concrete lined
Q = 9 C.M.S.
Length: 12,540

United States
Department of the Interior
Bureau of Reclamation
Litani River Project--Lebanon
Beirut, Lebanon, April 26, 1954
Zaiye Canal
Estimate of Cost
Table XXXVII--13

ITEM No.	ITEM	QUANTITY		EQUIPMENT MATERIAL AND LABOR		MISCELLANEOUS		SUMMARY	
		AMOUNT	UNIT	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST
1	Excavation, all classes, in open cut	250,000	cuM	7.	1,750,000				
2	Compacted fill	45,000	cuM	0.85	38,300				
3	Gravel backfill	2,500	cuM	1.	2,500				
4	Concrete lining in canal	7,520	cuM	60.	451,200				
5	Concrete in flume	4,580	cuM	100.	458,000				
6	Concrete in spillway	40	cuM	125.	5,000				
7	Reinforcement bars	457,500	kg	0.80	366,000				
8	Box culvert	8	ea	11,000.	88,000				
9	Circular culvert	10	ea	1,500.	15,000				
10	Single laned concrete bridge	7	ea	10,000.	70,000				
11	Double laned concrete bridge	1	ea	20,000.	20,000				
12	Siphon	Lump sum		ls	75,000				
	Subtotal for canal				£.L. 3,339,000				
	Contingencies 15% ±				501,000				
	Subtotal				3,840,000				
	Engineering, administration, and accounts 10% ±				384,000				
	Estimated cost of canal without interest during construction				4,224,000				
13	Relocation of roads and access roads	Lump sum		ls	85,000				
14	Right-of-way	Lump sum		ls	53,000				
15	Camps	Lump sum		ls	100,000				
	Subtotal				£.L. 238,000				
	Administration and general expenses 3.5% ±				8,000				
	Estimated cost of access roads, right-of-way, and camps without interest during construction				£.L. 246,000				
	Items for canal				4,224,000				
	Estimated cost of canal, access roads, right-of-way, and camps without interest during construction				£.L. 4,470,000				
	Interest 6% ± (3 years completion)				400,000				
	Total estimated cost of canal, access roads, right-of-way, and camps with interest during construction				£.L. 4,870,000				
	Notes: All quantities are shown in metric units. Costs are shown in Lebanese Pounds (£.L.) Average annual monetary exchange is assumed as $1 U.S. = £.L. 3.50								

7-1493 (formerly XD-853)
(Apr.-51)
Bureau of Reclamation

ESTIMATE SHEET

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
Litani River Project--Lebanon
Beirut, Lebanon, April 26, 1954
Zrariye Canal
Estimate of Cost
Table XXXVII--14

Concrete lined
Total storage: 50,000 CM
Length: 1,800 M.

ITEM No.	ITEM	QUANTITY		EQUIPMENT MATERIAL AND LABOR		MISCELLANEOUS		SUMMARY	
		AMOUNT	UNIT	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST
1	Excavation, all classes, in open cut	100,000	cuM	7.00	700,000				
2	Concrete lining in canal	2,880	cuM	60.00	172,800				
3	Concrete in spillway	15	cuM	125.00	1,900				
4	Box culvert	2	ea	11,000	22,000				
5	Circular culvert	5	ea	1,500	7,500				
6	Single laned bridge	1	ea	10,000	10,000				
	Subtotal for canal				£L. 914,200				
	Contingencies 15% ±				136,800				
	Subtotal				£L. 1,051,000				
	Engineering, administration, and accounts 10% ±				105,000				
	Estimated cost of canal without interest during construction				£L. 1,156,000				
7	Access roads	ls		ls	35,000				
8	Camps	ls		ls	30,000				
	Subtotal				£L. 65,000				
	Administration and general expense 3.5% ±				2,000				
	Estimated cost of access road and camps without interest during construction				£L. 67,000				
	Items for canal				1,156,000				
	Estimated cost of canal, access roads, and camps without interest during construction				£L. 1,223,000				
	Interest 6% ± (1.5 years completion)				55,000				
	Total estimated cost of canal, access roads, and camps with interest during construction				£L. 1,278,000				
	Notes:								
	All quantities are shown in metric units								
	Costs are shown in Lebanese Pounds (£L.)								
	Average annual monetary exchange is assumed as $ 1 U.S. = £L. 3.50								

7-1493 (formerly XI-853)
(Apr.-51)
Bureau of Reclamation

ESTIMATE SHEET

Single Penstock
Ave. Diameter: 1.85
Slope Length: 335 M.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
Litani River Project--Lebanon
Beirut, Lebanon, April 6, 1954
Zrariye Penstock
Estimate of Cost
Table XXXVII--15

ITEM No.	ITEM	QUANTITY		EQUIPMENT MATERIAL AND LABOR		MISCELLANEOUS		SUMMARY	
		AMOUNT	UNIT	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST
1	Excavation, all classes, in open cut, for penstock	500	cuM	12.00	6,000				
2	Concrete in supports	155	cuM	230.00	35,600				
3	Reinforcement bars	25,000	kg	0.85	21,300				
4	Steel penstock	247,000		2.50	617,500				
5	Valve structure	ls		ls	93,000				
	Subtotal for Penstock				ŁL.773,400				
	Contingencies 15% ±				116,600				
	Subtotal				ŁL.890,000				
	Engineering, administration, and accounts 10% ±				89,000				
	Estimated cost of penstock without interest during construction				ŁL.979,000				
	Interest 6% ± (1½ years completion)				44,000				
	Total estimated cost of penstock with interest during construction				ŁL.1,023,000				

Notes:
All quantities are shown in metric units
Costs are shown in Lebanese Pounds (ŁL.)
Average annual monetary exchange is assumed as $ 1 U.S. = ŁL. 3.50

7-1493 (formerly XI-853)
(Apr.-51)
Bureau of Reclamation

ESTIMATE SHEET

Diameter of Surge Tank: 10 M
Single Penstock
Average Diameter: 2.60 M
Slope Length: 220 M

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
Litani River Project--Lebanon
Beirut, Lebanon, April 6, 1954
Joun Surge Tank and Penstock
Estimate of Cost
Table XXXVII--16

ITEM No.	ITEM	QUANTITY		EQUIPMENT MATERIAL AND LABOR		MISCELLANEOUS		SUMMARY	
	Items For Surge Tank	AMOUNT	UNIT	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST
1	Excavation, rock, for surge tank	5,400	cuM	68.00	367,200				
2	Concrete lining of surge tank	950	cuM	140.00	133,000				
3	Reinforcement bars	130,000	kg	1.00	130,000				
4	Grouting	30	cuM	450.00	13,500				
5	Grout holes and connections	200	M	17.00	3,400				
6	Miscellaneous steel	2,000	kg	2.00	4,000				
	Subtotal for surge tank				₤L. 651,100				
	Contingencies 15% ±				97,900				
	Subtotal				₤L. 749,000				
	Engineering, administration, and accounts 10% ±				75,000				
	Estimated cost of surge tank without interest during construction				₤L. 824,000				
	Interest 6% ± (1-1/2 years completion)				30,000				
	Total estimated cost of surge tank including interest during construction				₤L. 860,000				
	Items For Penstock								
1	Excavation, all classes, in open cut	600	cuM	12.00	7,200				
2	Concrete in supports	250	cuM	230.00	57,500				
3	Reinforcement bars	40,000	kg	0.85	34,000				
4	Steel penstock	408,000	kg	2.50	1,020,000				
5	Valve structure	Lump Sum		ls	120,000				
	Subtotal for penstock				₤L.1,238,700				
	Contingencies 15% ±				186,300				
	Subtotal				₤L.1,425,000				
	Engineering, administration, and accounts 10% ±				145,000				
	Estimated cost of penstock without interest during construction				₤L.1,570,000				
	Interest 6% ± (1-1/2 years completion)				70,000				
	Total estimated cost of penstock with interest during construction				₤L.1,640,000				

Notes:
All quantities are shown in metric units
Costs are shown in Lebanese Pounds (₤L.)
Average annual monetary exchange is assumed as $1 U.S. = ₤L. 3.50

7-1493 (formerly XD-853)
(Apr.-51)
Bureau of Reclamation

ESTIMATE SHEET

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
Litani River Project--Lebanon
Beirut, Lebanon, April 28, 1954
Kelia Penstock
Estimate of Cost
Table XXXVII--17

Double Penstock
Average Diameter: 1.85 M.
Slope Length: 1,760 M.

ITEM No.	ITEM	QUANTITY		EQUIPMENT MATERIAL AND LABOR		MISCELLANEOUS		SUMMARY	
		AMOUNT	UNIT	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST
1	Excavation, all classes, in open cut	13,500	cuM	12.00	162,000				
2	Concrete in supports	1,220	cuM	50.00	61,000				
3	Reinforcement bars	195,000	kg	0.85	165,800				
4	Steel penstock	5,000,000	kg	1.68	8,400,000				
5	Valve structure	ls		ls	158,000				
	Subtotal				£L. 8,947,000				
	Contingencies 18% ±				1,610,000				
	Subtotal				£L.10,557,000				
	Engineering, administration, and accounts 10% ±				1,056,000				
	Estimated cost of penstock without interest during construction				£L.11,613,000				
6	Camps	ls		ls	50,000				
7	Right of way	ls		ls	30,000				
	Subtotal				£L. 80,000				
	Administration and general expense 3.5% ±				3,000				
	Estimated cost of penstock, camps, and right of way without interest during construction				£L. 83,000				
	Items for penstock				11,613,000				
	Estimated cost of penstock, camps, and right of way without interest during construction				£L.11,696,000				
	Interest 6% ± (2 years completion)				704,000				
	Total estimated cost of penstock, camps, and right of way with interest during construction				£L.12,400,000				

Notes:
All quantities are shown in metric units
Costs are shown in Lebanese Pounds (£L.)
Average annual monetary exchange is assumed as $1 = £L. 3.50

7-1493 (formerly XD-853)
(Apr.-51)
Bureau of Reclamation

ESTIMATE SHEET

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
Litani River Project--Lebanon
Beirut, Lebanon, April 29, 1954
Awali Surge Tank and Penstock
Estimate of Cost
Table XXXVII--18

Surge Tank Diameter: 10 M.
Single Penstock
Average Diameter: 2.60M
Slope Length: 360 M.

ITEM No.	ITEM	QUANTITY		EQUIPMENT MATERIAL AND LABOR		MISCELLANEOUS		SUMMARY	
	Items for Surge Tank	AMOUNT	UNIT	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST
1	Excavation, in rock, for surge tank	4,200	cuM	68.00	285,600				
2	Concrete lining of surge tank	760	cuM	140.00	106,400				
3	Reinforcement bars	103,000	kg	1.00	103,000				
4	Grout holes and connections	175	lm	17.00	3,000				
5	Grouting	25	cuM	450.00	11,300				
6	Miscellaneous metalwork	1,750	kg	2.00	3,500				
	Subtotal				LL. 512,800				
	Contingencies 15% ±				76,200				
	Subtotal				LL. 589,000				
	Engineering, administration and accounts 10% ±				59,000				
	Estimated cost of surge tank without interest during construction				LL. 648,000				
	Interest 6% ± (1.5 years completion)				29,000				
	Estimated cost of surge tank with interest during construction				LL. 677,000				
	Items for Penstock								
1	Excavation, all classes, in open cut for penstock	2,000	cuM	12.00	24,000				
2	Excavation, all classes, in open cut for tailrace	10,000	cuM	8.00	80,000				
3	Concrete in supports	310	cuM	230.00	71,000				
4	Reinforcement bars	50,000	kg	0.85	42,500				
5	Steel penstock	911,000	kg	2.50	2,277,500				
	Subtotal				LL.2,495,000				
	Contingencies 15% ±				375,000				
	Subtotal				LL.2,870,000				
	Engineering, administration, and accounts 10% ±				287,000				
	Estimated cost of penstock without interest during construction				LL.3,157,000				
	Interest 6% ± (1.5 years completion)				143,000				
	Estimated cost of penstock with interest during construction				LL.3,300,000				

Notes:

All quantities are shown in metric units
Costs are shown in Lebanese Pounds (LL.)
Average annual monetary exchange is assumed as $1 = LL. 3.50

7-1493 (formerly XII-853)
(Apr.-51)
Bureau of Reclamation

ESTIMATE SHEET

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
Litani River Project--Lebanon
Bekaa Pumping Unit
Estimate of Cost
Table XXXVII--19

ITEM No.	(Costs are in Lebanese Pounds) ITEM	QUANTITY		EQUIPMENT MATERIAL AND LABOR		MISCELLANEOUS		SUMMARY	
		AMOUNT	UNIT	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST
	KARAOUN PUMPING PLANT								
1	Acquisition of Right of Way	(Included		in Karaoun Dam)					
2	Steel pipe intake line	38,000	kg	2.20	83,600				
3	Pumping plant including all electrical and mechanical equipment and building	Lump Sum		L.S.	1,750,000				
4	Steel pipe discharge line	88,800	kg	2.20	195,400				
5	Transition to Main Canal	Lump Sum		L.S.	5,000				
	Field Cost				2,034,000				
	Contingencies (15% ±)				306,000				
	Total Cost				2,340,000				2,340,000
	MAIN CANAL (14.1 km)								
6	Acquisition of Right of Way	15	Ha	1000	15,000				
7	Excavation for canal	60,000	cuM	5.00	300,000				
8	Compacting Embankment	50,000	cuM	3.00	150,000				
9	Trimming	85,000	sM	0.50	43,000				
10	Placing lining	85,000	sM	4.00	420,000				
11	Bridges, 2 meters wide	3	ea	900	2,600				
12	Footbridges, 1 meter wide	12	ea	100	1,200				
13	Drain Culverts	5	ea	400	2,000				
14	Stock watering ramp	1	ea	200	200				
	Field Cost				934,000				
	Contingencies (15% ±)				98,000				
	Total Cost				1,032,000				1,032,000
	BIFURCATION WORKS								
15	Constructing bifurcation structure	Lump Sum		L.S.	10,000				
	Field Cost				10,000				
	Contingencies (20% ±)				2,000				
	Total Cost				12,000				12,000
	LITANI RIVER SIPHON								
16	Acquisition of Right of Way	Lump Sum		L.S.	10,000				
17	Excavation for structure	3,330	cuM	3.00	10,000				
18	Compacting backfill	1,666	cuM	3.00	5,000				
19	Constructing inlet and outlet transitions	Lump Sum		L.S.	10,000				
20	Placing 80 cm. precast reinf. conc. pipe	2,200	M	100.00	220,000				
21	Care of river during construction	Lump Sum		L.S.	10,000				
	Field Cost				265,000				
	Contingencies (15% ±)				42,000				
	Total Cost				307,000				307,000
	EAST CANAL								
22	Acquisition of Right of Way	25	Ha	1000	25,000				
23	Excavation for canal	40,000	cuM	5.00	200,000				
24	Compacting embankment	30,000	cuM	3.00	90,000				
25	Trimming	77,000	sM	0.50	38,500				
26	Placing lining	77,000	sM	4.00	308,000				
27	Placing precast concrete pipe	4,100	lM	30.00	123,000				
28	Bridges, 7 meters wide	2	ea	2000	4,000				
29	Bridges, 2 meters wide	8	ea	900	7,200				
30	Turnouts	7	ea	800	5,600				
31	Checks	5	ea	400	2,000				
32	Footbridges	17	ea	100	1,700				
33	Road culverts	6	ea	200	1,200				
34	Drain culverts	8	ea	400	3,200				
35	Stock watering ramps	6	ea	200	1,200				
	Field Cost				810,600				
	Contingencies (15 % ±)				85,400				
	Total Cost				896,000				896,000
	WEST CANAL								
36	Acquisition of Right of Way	15	Ha	1000	15,000				
37	Excavation for canal	20,000	cuM	5.00	100,000				
38	Compacting embankment	20,000	cuM	3.00	60,000				
39	Trimming	53,000	sM	0.50	26,500				
40	Placing lining	53,000	sM	4.00	212,000				
41	Bridges, 7 meters wide	2	ea	2000	4,000				
42	Bridges, 2 meters wide	2	ea	900	1,800				
43	Turnouts	6	ea	800	4,800				
44	Checks	5	ea	400	2,000				
45	Footbridges	11	ea	100	1,100				
46	Road Culverts	6	ea	200	1,200				
47	Drain Culverts	8	ea	400	3,200				
48	Stock watering ramps	5	ea	200	1,000				
	Field Cost				432,600				
	Contingencies (15% ±)				45,400				
	Total Cost				478,000				478,000
	DISTRIBUTION SYSTEM								
49	Construction of laterals	4,700	Ha	300	1,410,000				
	Field Cost				1,410,000				
	Contingencies (15% ±)				211,000				
	Total Cost				1,621,000				1,621,000
	DRAINAGE SYSTEM								
50	Construction of drains	2,000	Ha	60	120,000				
	Field Cost				120,000				
	Contingencies (15% ±)				12,000				
	Total Cost				132,000				132,000
	CONSTRUCTION COST								6,818,000
	ENGINEERING AND OVERHEAD (10% ±)								682,000
	TOTAL COST OF UNIT								7,500,000

7-1493 (formerly XI-853)
(Apr.-51)
Bureau of Reclamation

ESTIMATE SHEET

United States
Department of the Interior
Bureau of Reclamation
Litani River Project--Lebanon
Upper Nabatiye Unit
Estimate of Cost
Table XXXVII--20

ITEM No.	Costs are in Lebanese Pounds ITEM	QUANTITY		EQUIPMENT MATERIAL AND LABOR		MISCELLANEOUS		SUMMARY	
		AMOUNT	UNIT	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST
	MAIN CANAL (19.7 km)								
1	Acquisition of right of way	20	Ha	1000	20,000				
2	Inlet structure	Lump sum		L.S.	10,000				
3	Excavation for canal	60,000	cuM	5.00	300,000				
4	Compacting embankment	57,000	cuM	3.00	171,000				
5	Trimming	86,000	sM	0.50	43,000				
6	Placing canal lining	86,000	sM	4.00	344,000				
7	Bridges - 7 meters wide	1	ea	2000	2,000				
8	Bridges - 2 meters wide	2	ea	900	1,800				
9	Turnouts	7	ea	800	5,600				
10	Checks	5	ea	400	2,000				
11	Footbridges	13	ea	100	1,300				
12	Road culverts	2	ea	200	400				
13	Drain culverts	9	ea	400	3,600				
14	Drops	5	ea	500	2,500				
15	Stock watering ramps	5	ea	200	1,000				
	Field Cost				908,200				
	Contingencies (15±%)				91,800				
	Total Cost				1,000,000				1,000,000
	LITANI RIVER SIPHON								
16	Acquisition of right of way	2	Ha	1000	2,000				
17	Constructing transitions	Lump sum		L.S.	10,000				
18	Installing 90 cm. dia. steel pipe	650,000	kg	2.50	1,625,000				
19	Excavation, foundations & supports	Lump sum		L.S.	245,000				
20	Care of river during construction	Lump sum		L.S.	10,000				
	Field Cost				1,892,000				
	Contingencies (15±%)				288,000				
	Total Cost				2,180,000				2,180,000
	DISTRIBUTION SYSTEM								
21	Construction of Laterals	3,500	Ha	435	1,522,500				
	Contingencies (15±%)				227,500				
	Total Cost				1,750,000				1,750,000
22	DRAINAGE SYSTEM								
	Construction of drains	1,000	Ha	60	60,000				
	Contingencies (15±%)				10,000				
	Total Cost				70,000				70,000
	CONSTRUCTION COST								5,000,000
	ENGINEERING AND OVERHEAD (10%)								500,000
	TOTAL COST								5,500,000

ESTIMATE SHEET

United States
Department of the Interior
Bureau of Reclamation
Litani River Project--Lebanon
Lower Nabatiye Unit
Estimate of Cost
Table XXXVII--21

ITEM No.	Costs are in Lebanese pounds ITEM	QUANTITY		EQUIPMENT MATERIAL AND LABOR		MISCELLANEOUS		SUMMARY	
		AMOUNT	UNIT	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST
	MAIN CANAL (39.6 km.)								
1	Acquisition of right of way	40	Ha	1000	40,000				
2	Constructing inlet to Main Canal	Lump sum		L.S.	5,000				
3	Constructing 2.7 m. dia. tunnel	350	M	260	91,000				
4	Excavation for canal	84,000	cuM	6.00	504,000				
5	Placing compacted embankment	70,000	cuM	3.00	210,000				
6	Trimming	117,000	sM	0.50	58,500				
7	Placing lining in canal	117,000	sM	4.00	468,000				
8	Constructing masonry flumes	3,000	cuM	30.00	90,000				
9	Placing unreinforced concrete in flumes	800	cuM	40.00	32,000				
10	Plastering masonry in flumes	6,000	sM	3.00	18,000				
11	Placing 80 cm. precast pipe, unreinforced	2,800	M	50.00	140,000				
12	Placing 110 cm. precast pipe for siphons	950	M	400	380,000				
13	Placing 100 cm. precast pipe for siphons	150	M	300	45,000				
14	Placing 90 cm. precast pipe for siphons	800	M	200	160,000				
15	Placing 40 cm. precast pipe for siphons	900	M	50	45,000				
16	Placing steel pipe for siphons	250,000	kg	2.20	550,000				
17	Supports and foundations for steel pipe	Lump sum		L.S.	83,000				
18	Transitions for siphons	34	ea	500	17,000				
19	Bridges, 7 meters wide	1	ea	2000	2,000				
20	Bridges, 2 meters wide	1	ea	900	900				
21	Footbridges	17	ea	100	1,700				
22	Turnouts	10	ea	800	8,000				
23	Checks	8	ea	400	3,200				
24	Road culverts	3	ea	200	600				
25	Drainage culverts	25	ea	400	10,000				
26	Stock watering ramps	7	ea	200	1,400				
	Field Cost				2,964,300				
	Contingencies (15±%)				444,700				
	Total Cost				3,409,000				3,409,000
	WEST CANAL (7.1 km)								
27	Acquisition of right of way	7	Ha	1000	7,000				
28	Excavation for canal	7,000	cuM	6.00	42,000				
29	Placing compacted embankment	6,000	cuM	3.00	18,000				
30	Trimming	18,000	sM	0.50	9,000				
31	Placing lining in canal	18,000	sM	4.00	72,000				
32	Placing 80 cm. precast pipe, unreinforced	1,100	M	50.00	55,000				
33	Placing 30 cm. precast pipe for siphons	500	M	50.00	25,000				
34	Placing steel pipe for siphons	20,000	kg	2.20	44,000				
35	Supports and foundations for pipes	Lump sum		L.S.	7,000				
36	Transitions for siphons	6	ea	500	3,000				
37	Footbridges	4	ea	100	400				
38	Turnouts	3	ea	800	2,400				
39	Checks	2	ea	400	800				
40	Road culverts	3	ea	200	600				
41	Drainage culverts	3	ea	400	1,200				
42	Stock watering ramps	2	ea	200	400				
	Field Cost				287,800				
	Contingencies (15±%)				43,200				
	Total Cost				331,000				331,000
	DISTRIBUTION SYSTEM								
43	Constructing laterals	3,700	Ha	600	2,220,000				
	Contingencies (15±%)				333,000				
	Total Cost				2,553,000				2,553,000
	DRAINAGE SYSTEM								
44	Constructing Drains	1,000	Ha	60	60,000				
	Contingencies (15±%)				10,000				
	Total Cost				70,000				70,000
	CONSTRUCTION COST								6,363,000
	ENGINEERING AND OVERHEAD (10%)								637,000
	TOTAL COST								7,000,000

7-1493 (formerly XD-853)
(Apr.-51)
Bureau of Reclamation

ESTIMATE SHEET

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
Litani River Project--Lebanon
Saida-Beirut Unit
Estimate of Cost
Table XXXVII--22

ITEM No.	(Costs are in Lebanese Pounds) ITEM	QUANTITY		EQUIPMENT MATERIAL AND LABOR		MISCELLANEOUS		SUMMARY	
		AMOUNT	UNIT	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST	UNIT COST	TOTAL COST
	MAIN CANAL (55.2 km.)								
1	Acquisition of right of way	55	Ha.	1500	82,500				
2	Inlet from Joun Tunnel	Lump Sum		L.S.	5,000				
3	Excavation for canal	150,000	cuM	5.50	825,000				
4	Compacting embankment	120,000	cuM	3.00	360,000				
5	Trimming	214,000	sM	0.50	107,000				
6	Placing canal lining	214,000	sM	4.00	856,000				
7	Constructing masonry flumes	Lump Sum		L.S.	115,000				
8	Constructing 21 siphons	4,200	M	350	1,470,000				
9	Bridges, 7 meters wide	9	ea	2000	18,000				
10	Bridges, 2 meters wide	20	ea	900	18,000				
11	Footbridges	44	ea	100	4,400				
12	Turnouts	18	ea	800	14,400				
13	Checks	15	ea	400	6,000				
14	Drops	2	ea	2000	4,000				
15	Road culverts	35	ea	200	7,000				
16	Drainage culverts	22	ea	400	8,800				
17	Stock watering ramps	10	ea	200	2,000				
	Field Cost				3,903,100				
	Contingencies (15% ±)				585,900				
	Total Cost				4,489,000				4,489,000
	DISTRIBUTION SYSTEM								
18	Construction of Laterals	3,900	Ha.	300	1,170,000				
	Contingencies (15% ±)				176,000				
	Total Cost				1,346,000				1,346,000
	DRAINAGE SYSTEM								
19	Construction of drains	1,000	Ha.	60	60,000				
	Contingencies (15% ±)				9,000				
	Total Cost				69,000				69,000
	CONSTRUCTION COST								5,904,000
	ENGINEERING AND OVERHEAD (10%)								596,000
	TOTAL COST								6,500,000

ESTIMATE SHEET

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
Litani River Project--Lebanon
Bisri Powerplant, Bisri Switchyard, and Sohmor Switchyard
Estimate of Cost
Future Construction
Table XXXVII--23

ITEM No.	ITEM (Quantities are in metric units) (Costs are in Lebanese pounds LL)	QUANTITY AMOUNT	UNIT	EQUIPMENT MATERIAL AND LABOR UNIT COST	EQUIPMENT MATERIAL AND LABOR TOTAL COST	MISCELLANEOUS UNIT COST	MISCELLANEOUS TOTAL COST	SUMMARY UNIT COST	SUMMARY TOTAL COST
	BISRI POWERPLANT--Future Construction								
	Water Wheels, Turbines, and Generators								
1	Turbine, same as Phase A, 1 each	125,000	kg	5.09	636,250				
2	Generator, same as Phase A	1	ea	L.S.	840,938				
3	Exposed piping	5,000	kg	3.15	15,750				
	Subtotal				1,492,938				
	Contingencies (15% ±)				277,062				
	Field Total--Water Wheels, Turbines, and Generators				1,720,000				1,720,000
	Accessory Electric Equipment								
4	Generator protective equipment cubicle	1	ea	L.S.	14,700				
5	Generator neutral grounding equipment cubicle	1	ea	L.S.	10,275				
6	Main control board	Lump Sum	ls	L.S.	28,604				
7	Cable, 1/c, 2,000MCM, 15-kv, VCL	340	M	84.00	28,560				
8	Potheads, 15-kv, for 2,000MCM, 1/c, VCL cable	12	ea	423.75	5,085				
9	Transite conduit and fittings, 9 cm	320	M	13.50	4,320				
10	Miscellaneous	Lump Sum	ls	L.S.	3,550				
	Subtotal				95,094				
	Contingencies (15% ±)				14,906				
	Field Total--Accessory Electric Equipment				110,000				110,000
	Total field cost--Bisri Powerplant--Future Construction				1,830,000				1,830,000
	Engineering, supervision, administration, and general expense (15% ±)								275,000
	Subtotal--Cost, Bisri Powerplant--Future construction, without interest								2,105,000
	Interest during construction (6% ±)								126,000
	Total estimated cost of Bisri Powerplant--Future Construction								2,231,000
	BISRI SWITCHYARD--Future Construction								
1	Transformer, 3-phase, 16,250-kva, 6.6 to 69/39.9-kv with 69-kv lightning arresters	1	ea	)	693,000				
2	Single breaker switchyard bays, 69-kv	4	ea	)					
	Subtotal				693,000				
	Contingencies (15% ±)				107,000				
	Field Total--Bisri Switchyard--Future Construction				800,000				800,000
	Engineering, supervision, administration and general expense (17% ±)								136,000
	Subtotal--Bisri Switchyard--Future Construction, without interest								936,000
	Interest during construction (6% ±)								56,000
	Total estimated cost, Bisri Switchyard--Future Construction								992,000
	SOHMOR SWITCHYARD--Future Construction								
1	Single breaker switchyard bays, 69-kv	2	ea		231,000				
	Contingencies (15% ±)				39,000				
	Field Total--Sohmor Switchyard--Phase B				270,000				270,000
	Engineering, supervision, administration and general expense (17% ±)								46,000
	Subtotal--Sohmor Switchyard--Future Construction, without interest								316,000
	Interest during construction (6% ±)								19,000
	Total estimated cost--Sohmor Switchyard--Future Construction								335,000

TABLE XXXVII-24

Construction Cost of Phase "B" Power Plants

Power Plant	Cost of Structures and Improvements £L	Cost of Equipment £L	Total Cost
Bisri (Completion)		2,230,000	2,230,000
Kelia	3,730,000	9,480,000	13,210,000
Zrariye	2,180,000	3,070,000	5,250,000
Awali	3,200,000	8,100,000	11,300,000
Joun	2,180,000	5,350,000	7,530,000
Total in Phase "B"	11,290,000	28,230,000	39,520,000

TABLE XXXVII-25

Cost of Phase "B" Transmission System

	Lebanese Pounds (£. L.)
Transmission Line - Sohmor-Kelia	520,000
Transmission Line - Bisri-Awali	260,000
Transmission Line - Hazmie-Les Pins	130,000
Transmission Line - Awali-Joun	260,000
Transmission Line - Joun-Les Pins I	950,000
Transmission Line - Kelia-Zrariye	730,000
Transmission Line - Joun-Zrariye	780,000
Transmission Line - Bisri-Hazmie	910,000
Transmission Line - Joun-Les Pins II	900,000
Sending Substations and Switchyards - Sohmor	335,000
Sending Substations and Switchyards - Kelia	3,000,000
Sending Substations and Switchyards - Bisri	992,000
Sending Substations and Switchyards - Awali	2,390,000
Sending Substations and Switchyards - Joun	2,150,000
Sending Substations and Switchyards - Zrariye	1,170,000
Receiving Substations and Switchyards - Les Pins	4,040,000
Receiving Substations and Switchyards - Hazmie	1,540,000
TOTAL	21,057,000