# ROYAL GOVERNMENT OF AFGHANISTAN

## FINAL DESIGN REPORT

ON

## KAJAKAI DAM, ARGHANDAB DAM, AND BOGHRA CANAL PROJECTS

PREPARED FOR

MORRISON-KNUDSEN AFGHANISTAN, INC.

BY

INTERNATIONAL ENGINEERING COMPANY, INC.

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SAN FRANCISCO 5, CALIFORNIA, U.S.A.

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FINAL DESIGN REPORT

ON

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AND BOGHRA CANAL PROJECTS

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## SECTION I

## INTRODUCTION

### 1.01 PURPOSE

The purpose of this Report on the Kajakai Dam, Arghandab Dam, and the Boghra Canal Projects is to describe the design criteria, preliminary and final designs and to present the "as constructed" drawings of the completed structures.

### 1.02 AUTHORITY

a. Preliminary Investigations - Preliminary surveys and engineering studies of Kajakai Dam, Arghandab Dam, and the Boghra Canal Projects were made as per contract agreement, dated March 14, 1946, between the Kingdom of Afghanistan and Morrison-Knudsen Afghanistan, Inc. Field surveys and field investigations were made by Morrison-Knudsen Afghanistan, Inc., and engineering studies were conducted by International Engineering Company, Inc., under a subcontract agreement. This preliminary phase of design occured between 1946 and 1949.

b. Final Design - The continuation of preliminary investigations and final design of the three projects were made as per contract agreement, dated April 19, 1950, between the Royal Government of Afghanistan and Morrison-Knudsen Afghanistan, Inc. As in the preliminary stage, the engineering studies and preparation of construction drawings were subcontracted to International Engineering Company, Inc.

A Board of Consultants, under a separate contract with the Royal Government of Afghanistan, collaborated with Morrison-Knudsen Afghanistan Inc., and International Engineering Company, Inc., on the final design phase of the subject projects.

## SECTION II

## DESCRIPTION OF PROJECTS

### 2.01 GENERAL

The design studies for the Kajakai Dam, Arghandab Dam, and Boghra Canal Projects were divided into two major parts, i.e., preliminary and final.

a. Preliminary Design - As a basis for the preparation of contract drawings and specifications, which were included in the preliminary design, the following listed collection of field data and engineering studies were made:

(1) Topographic surveys of dam and reservoir sites, and canal locations.

(2) Subsurface investigations of Kajakai and Arghandab damsites, including core drilling and geologic interpretation.

(3) Collection of streamflow records on the Helmand and Arghandab Rivers.

(4) Investigations to determine sources and types of construction materials.

(5) Irrigation and power studies related to the Helmand and Arghandab Valleys.

(6) Project planning and economic feasibility studies.

In this phase of the design, the economic and engineering feasibility of the projects was evaluated and the general location, size, and type of main features were determined 1/. The preliminary design work was concluded in 1949.

b. Final Design - The final design work included additional collection of field data, revaluation of preliminary conclusions, and the preparation of

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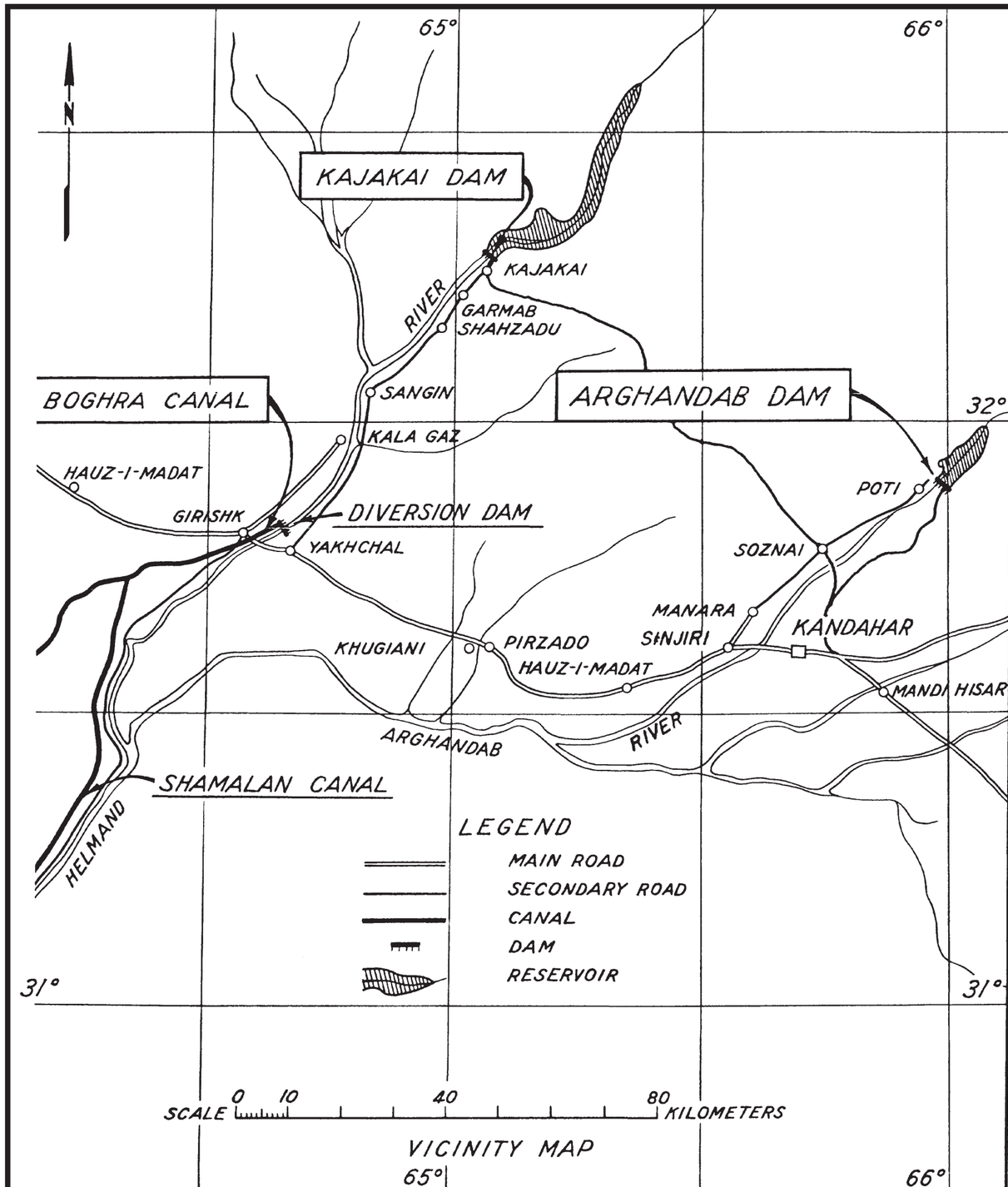
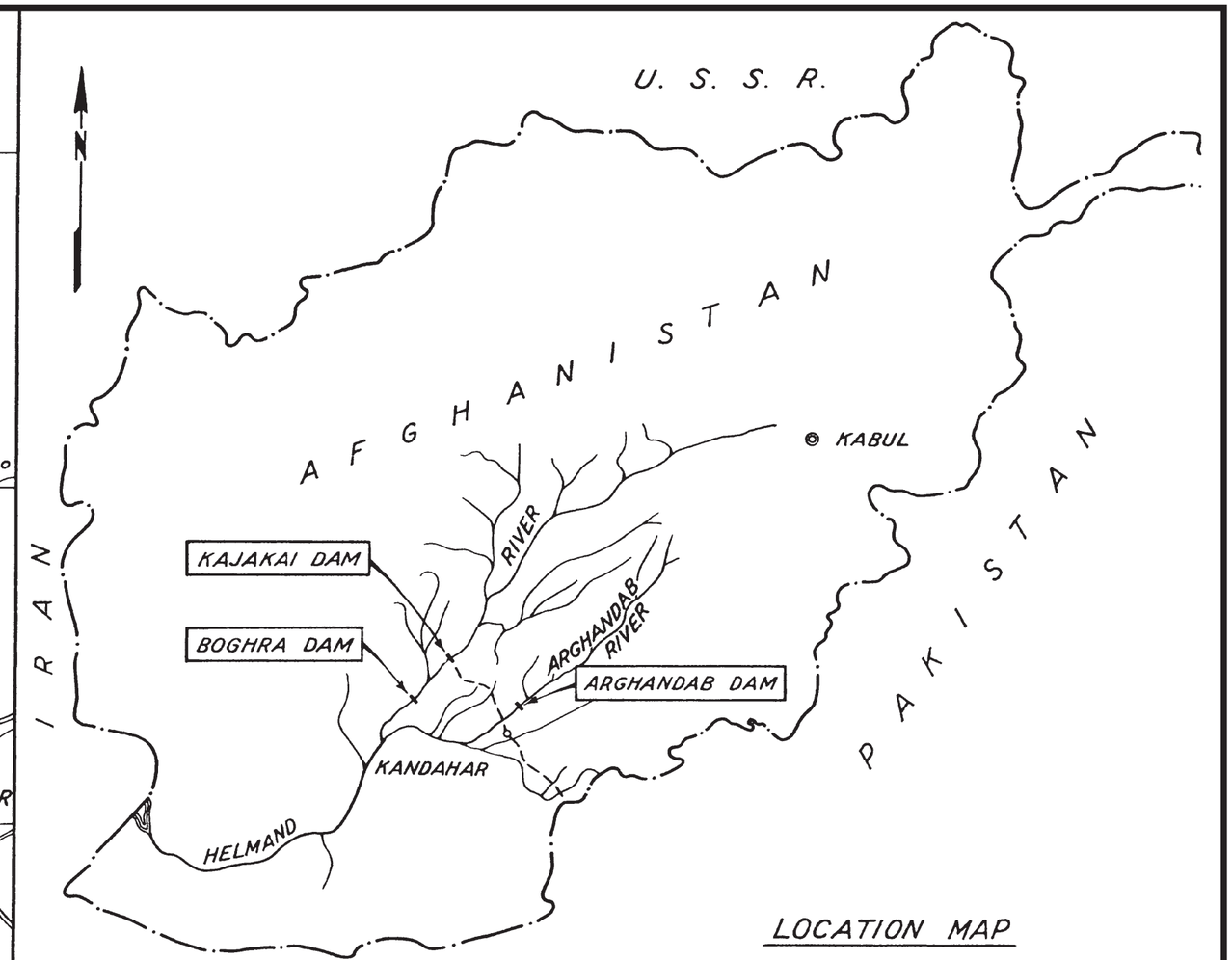
1/ " On Kargha, Seraj, Kharwar, Kajakai, Arghandab, Surkhab, and Boghra Dams, Afghanistan" by J. L. Savage, dated June 30, 1948 (Appendix A)

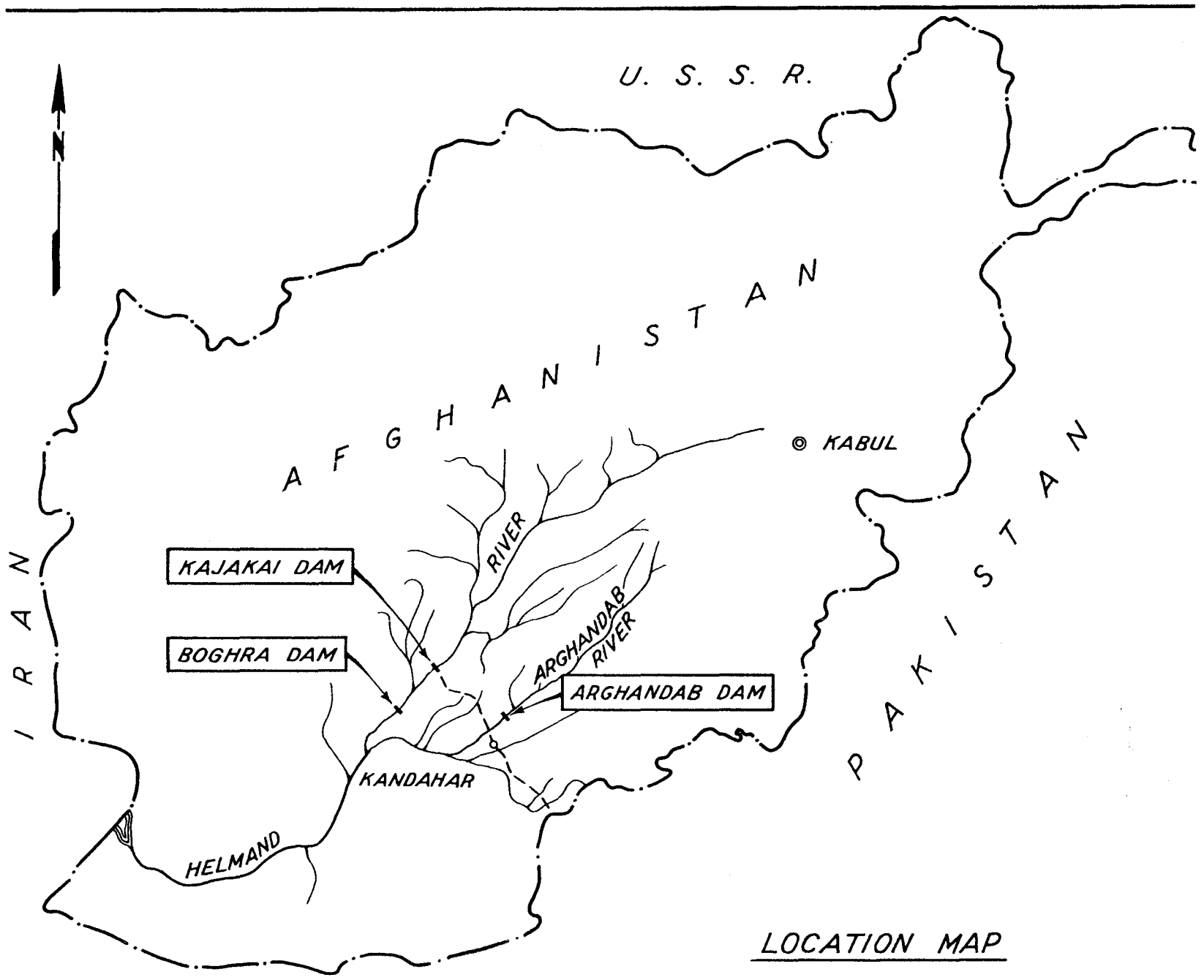



LOCATION AND VICINITY MAPS

KAJAKAI DAM, ARGHANDAB DAM, AND BOGHRA CANAL PROJECTS.

SAN FRANCISCO, CALIFORNIA
AUGUST 29, 1956.

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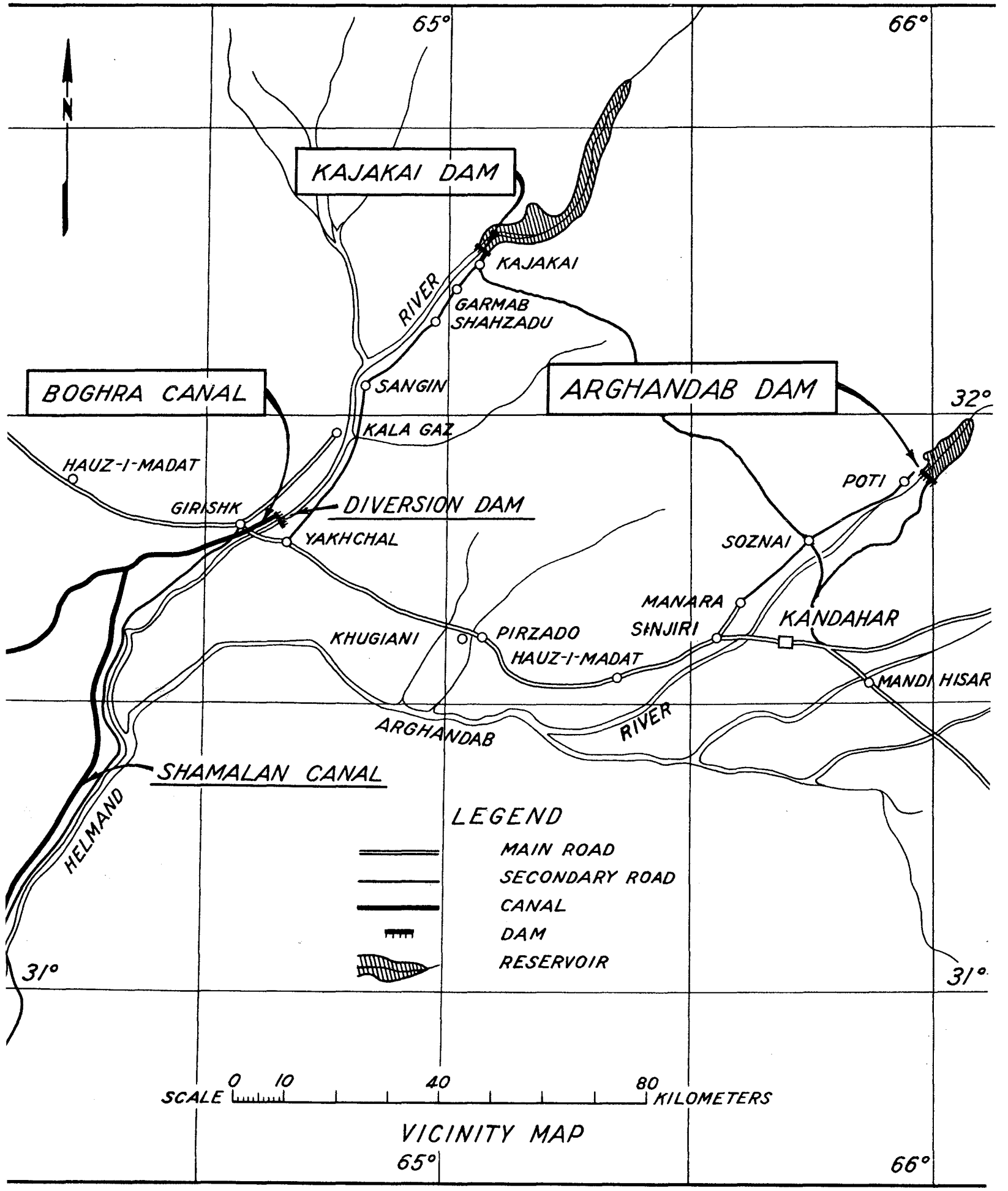

65°
66°
N
KAJAKAI DAM
KAJAKAI
RIVER
GARMAB
SHAHZADU
BOGHRA CANAL
SANGIN
ARGHANDAB DAM
32°
KALA GAZ
HAUZ-I-MADAT
POTI
GIRISHK
DIVERSION DAM
YAKHCHAL
SOZNAI
MANARA
SINJIRI
KANDAHAR
KHUGIANI
PIRZADO
HAUZ-I-MADAT
MANDI HISAR
ARGHANDAB
RIVER
SHAMALAN CANAL
HELMAND
LEGEND
MAIN ROAD
SECONDARY ROAD
CANAL
DAM
RESERVOIR
31°
31°
SCALE 0 10 40 80 KILOMETERS
VICINITY MAP
65°
66°

construction drawings. The Board of Consultants, staffed by individually prominent engineers, was actively engaged in problems involving major engineering decisions in connection with the subject projects.

In a number of instances, the final design of project features differs from the preliminary layouts shown on 1950 Contract Drawings 2/. These differences were based upon additional information from the field, and review of preliminary layouts by the design engineers and the Board of Consultants. Considerations leading to the final designs and reasons for changes, etc., in the preliminary layouts will be discussed as the Report proceeds.

A list of special reports, consultants' reports containing specific recommendations on design of project features, and specifications for construction of the various project features, appears in the Appendixes.

## 2.02 INDIVIDUAL DESCRIPTION

a. Kajakai Dam - The main features of this project are an earth and rockfill dam impounding a reservoir of 2.35 cubic kilometers (1,900,000 acre-ft) gross storage, an ungated spillway, and a reservoir outlet works. Water releases will be made mainly to satisfy irrigation requirements in the Helmand Valley.

Future development of this project will involve the construction of a gated spillway. This would result in an increase of the storage capacity to 3.35 cubic kilometers (2,700,000 acre-ft). Construction of a hydroelectric power plant and transmission facilities is also a future possibility.

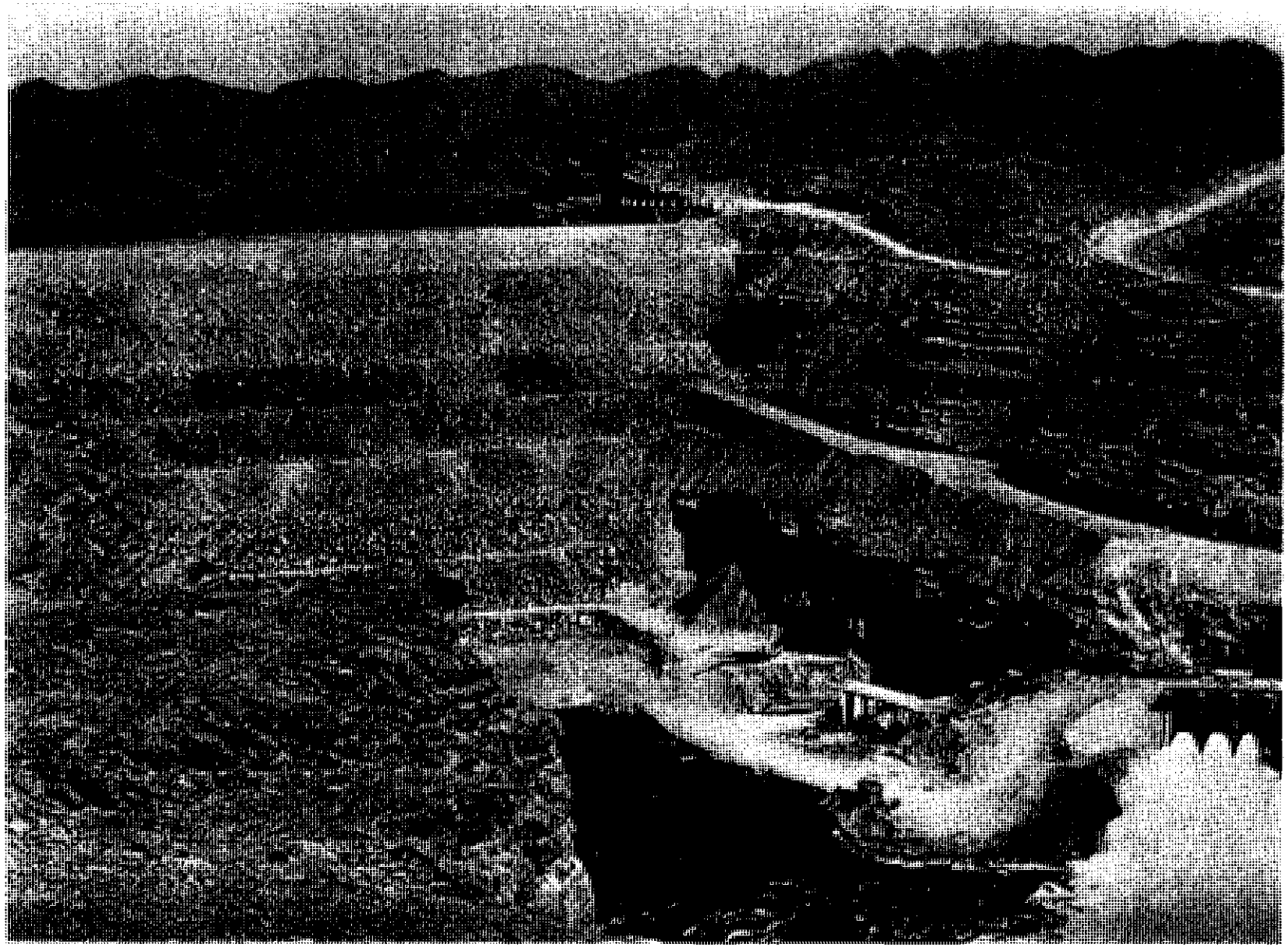
b. Arghandab Dam - The main features of this project are an earthfill dam impounding a reservoir of 0.492 cubic kilometers (398,000 acre-ft) of

2/ "Contract Drawings, Afghanistan" by International Engineering Company, Inc., dated 1948 and 1950 (Appendix C)

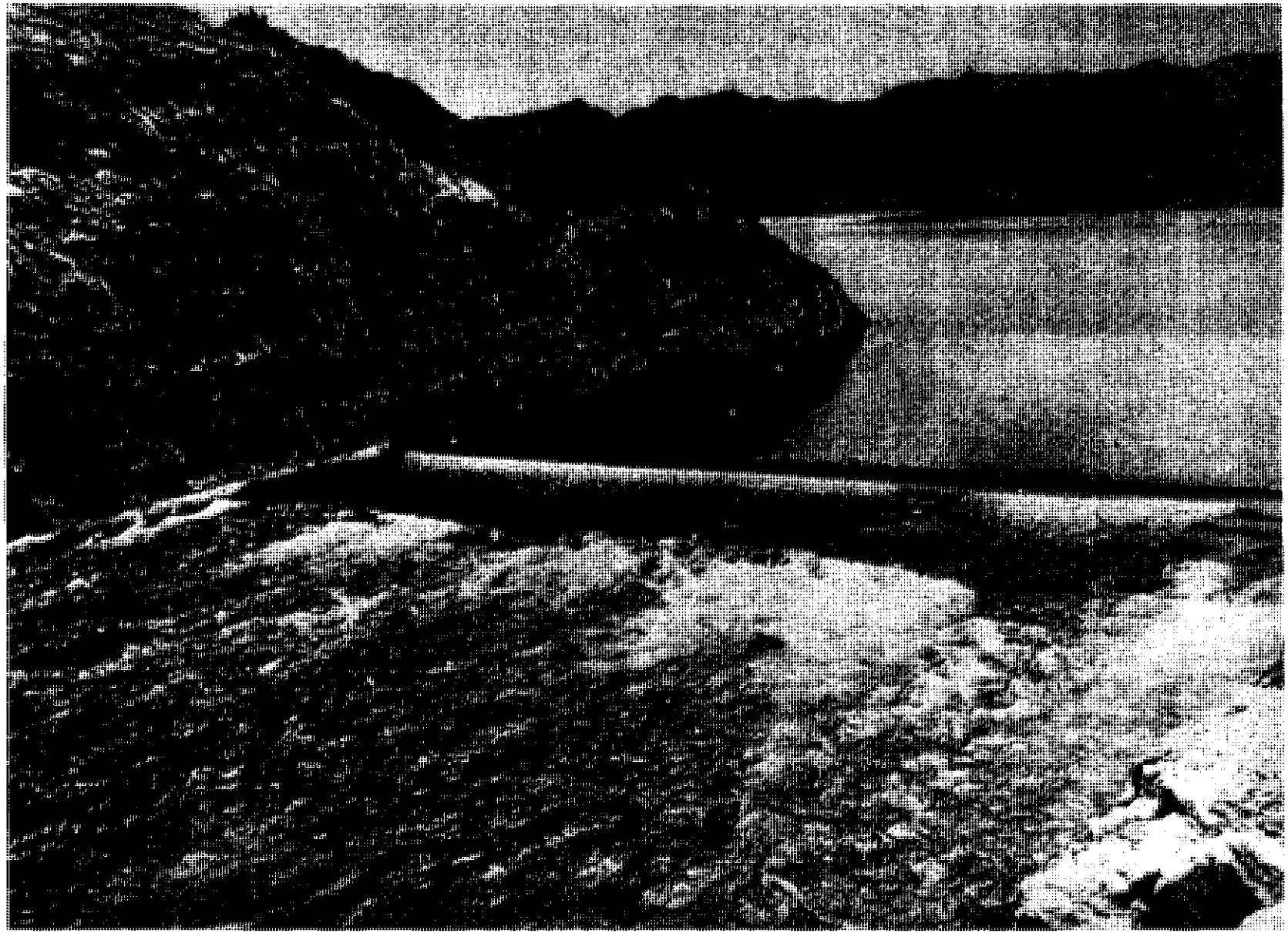
gross storage, two (2) ungated spillways, and a reservoir outlet works. Water releases will be made to satisfy irrigation requirements in the Arghandab Valley, including the Tarnak area.

Full development of this project will involve the future construction of a hydroelectric power plant at the damsite, a switchyard and substation at Kandahar, and transmission facilities.

c. Boghra Canal Project - This project was conceived and started before Morrison-Knudsen Afghanistan, Inc. began operations in Afghanistan. The main features are a diversion dam and a canal system with appurtenant structures. The latter part consists of the Boghra, East Marja, and Shamalan Canals, serving an irrigable land area of 447 square kilometers (108,950 acres).



DOWNSTREAM VIEW OF KAJAKAI DAM AND IRRIGATION OUTLETS



KAJAKAI DAM – SPILLWAY

## SECTION III

## KAJAKAI DAM PROJECT

### 3.01 GENERAL FEATURES

a. Preliminary Design - The preliminary phase of design of the Kajakai Dam had been completed in 1950, and the results of these earlier studies are shown on the Contract Drawings 3/. The following description by main features pertains to the design as shown on these drawings and not necessarily to the project as subsequently constructed.

1. Dam - The dam was designed as an earth and rockfill embankment with a maximum height of 98 meters above original streambed. The top at El 1,050 meters was 10 meters wide and 270 meters long. The computed slopes were 2.5:1 and 3:1 upstream, and 2:1 downstream. The section was designed with a central impervious core of rolled earth flanked by semi-pervious and pervious zones. Riprap protection was shown as dumped rockfill on both upstream and downstream faces. A cutoff trench under the impervious central core was shown to extend through sand and gravel to sound rock and backfilled with impervious rolled earthfill.

2. Spillway - An open channel spillway was located adjacent to the right abutment of the dam. It was to include a 101 meter long uncontrolled concrete section with crest at El 1,035.25 meters, a downstream apron, and training walls. It was designed with provisions for the future installation of crest gates to raise the maximum reservoir level to El 1,045 meters.

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3/ "Contract Drawings, Afghanistan" by International Engineering Company, Inc., dated 1948 and 1950 (Appendix C)

3. Tunnels

(a) Diversion Use - Two (2) 10 meter diameter, horseshoe-shaped tunnels were shown through the left abutment. The tunnels were concrete-lined, spaced 60 meters apart, and were intended to serve as diversion conduits during dam construction and as irrigation and power tunnels after diversion. Outlet works for release of irrigation waters were located in the left or outer tunnel, or irrigation tunnel.

(b) Irrigation Use - The tunnel intake was shown to include a submerged trashrack with stoplog slots for emergency unwatering. A concrete tunnel plug was to be located on an extension of the dam axis. Release of irrigation water was to be made through three (3) steel conduits passing through the plug. Regulating valves were located downstream from the plug and would discharge into the tunnel. Emergency gates were located upstream from the valves. A concrete deflector was shown at the downstream portal, designed to prevent scour in the vicinity of the tunnel outlet.

(c) Power Use - A simple portal structure was shown at the upstream end of the tunnel. Following diversion, the portal would be closed by stoplogs. A concrete tunnel plug was located a short distance downstream. A future high-level intake for power release was shown, connected by an inclined shaft to the power tunnel below the plug. Provisions were made in the design of the tunnel for the future installation of a 6.1 meter diameter penstock. An emergency high-pressure gate was shown at the upstream end of the plug.

4. Power Plant - A future power plant was planned as a conventional installation of four (4) 40,560 metric horsepower (40,000 hp) vertical-shaft reaction turbines, each direct-connected to a 30,000 kw

generator. The plant was located between the tunnel portals. The contract drawings do not show the power plant.

b. Final Design - Final design of the Kajakai Dam project differed from the preliminary design. These differences occurred partly because of the revaluation of additional data received over several ensuing years, and partially because of the Consultants' independent suggestions. The following descriptions by main features are of the changes in design made between the preliminary and final phases:

1. Dam - Design changes included the following:

   (a) Upstream main embankment slope was reduced to 2.5:1.

   (b) Central impervious core thickness was reduced. Required depth of excavation for the cutoff trench was accurately determined at eight (8) meters to sound rock.

2. Spillway - Design changes included the following:

   (a) A 113 meter long unpaved rock-weir was selected.

   (b) Crest was reduced to El 1,033.5 meters.

   (c) Initial reservoir capacity was decreased but kept large enough to satisfy irrigation requirements.

   (d) Future gated spillway would be a separate concrete structure located downstream from the initial rockfill weir. This would require excavation at the initial weir site to El 1,030 meters.

Spillway location was more accurately determined through a saddle about 0.8 kilometers upstream from the dam axis.

3. Tunnels

   (a) Diversion Use - Design changes included the following:

1. Concrete lining was eliminated.

2. Diameter of the tunnels was increased to 10.5 meters.

Combined length of the diversion tunnels was established at 1,296 meters.

(b) Irrigation Use - Design of this feature was more accurately determined and changed. The irrigation tunnel length was established at 720 meters. A combined trashrack and gate structure at the upstream portal of the tunnel was equipped with a concrete bulkhead and a fixed-wheel gate to permit closing. The gates were to be controlled from El 1,050 meters. An overhead traveling crane was provided for operating the gate and bulkhead and for servicing the trashracks. Access to the operating deck was provided by a structural steel bridge.

Diameter of the three (3) steel conduits was established at 2.12 meters (84-in.). The entrances were located at the upstream face of the concrete plug, about 130 meters above the downstream portal. Below the plug the pipes were encased in concrete to discharge through regulating valves at the downstream portal. An emergency shutoff valve for each conduit was located at a concrete-lined valve chamber immediately downstream from the plug.

(c) Power Use - The length of the power tunnel was accurately determined at 576 meters. The entrance to the future steel penstock was located at a concrete-lined transition about 100 meters above the downstream portal.

4. Power Plant - No changes were made during final design.

## 3.02 DESIGN CONSIDERATIONS

a. Dam

1. Description of the Damsite - The Kajakai damsite lies in a narrow steep-walled gorge, which was cut by the Helmand River through a deposit of stepped dolomitic limestone. The bedding planes of the limestone are gently tilted. Short distances upstream and downstream from the site, the gorge opens into broad alluvium-filled valleys. The streambed at the damsite is composed of sand and gravel to a maximum depth of about eight (8) meters.

2. Geology - A geologic investigation of the damsite was made by Dr. F. A. Nickell, Consulting Geologist, June 1950 4/. His report concluded that the site was suitable for the proposed dam. The report also suggested methods of treating the embankment foundation.

The most important geologic features of the limestone are two (2) main systems of vertical joints, each at an angle of roughly 45 degrees with the dam axis. These joints are closely spaced near the intake portals of the tunnel and more widely spaced elsewhere. Sealing of these joints to prevent excessive leakage from the reservoir presented a major construction problem (see paragraph 3.08, b). In addition, a five (5) meter wide fault zone below the streambed had been uncovered by core-trench excavation. Special treatment adopted for this condition is described in paragraph 3.08, c.

There is a seismically active area in Pakistan, nearly 320 kilometers (200 miles) from the damsite. Dr. Nickell conservatively recommended that structures be designed to resist inertia forces produced by a ground-wave acceleration equivalent to 0.05 gravity.

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4/ "Geology of the Kajakai Damsite" by F. A. Nickell, dated August 1950 (Appendix A)

3. Foundation Treatment - Foundation treatment included a trench to be excavated through the sand and gravel overburden in the river. It would extend to sound rock beneath the central foundation area. The trench would be backfilled with impervious rolled earth. Pervious overburden throughout the foundation area would be stripped and replaced with free-draining material.

A grout curtain was considered along the longitudinal centerline of the core trench, extending the cutoff into the underlying rock. This curtain would extend 40 meters into rock at the bottom and about half-way up on the abutments, from which point the grout penetration would decrease to a minimum depth of 16 meters until the top of the dam was reached.

Preliminary estimates were made of the number and spacing of the grout holes. The required quantity of grout was estimated from preliminary data of core borings and geologic studies. It was estimated that primary holes extending to the bottom of the curtain would be spaced at 20 meters and grouted under a maximum pressure of 14.1 kg per sq cm (200 psi). Secondary holes would be drilled, in turn, midway between the primary holes to a depth of 20 meters. Intermediate tertiary and quaternary holes would be drilled in sequence as required. All holes were required to be approximately normal to the abutments.

Spacing of primary holes for upper part of the abutment was estimated at 10 meters and minimum depth reduced to 12 meters. Secondary grout holes would extend to a maximum depth of 20 meters at the bottom and 12 meters at the abutments. Details of the grout treatment are described in

grouting specifications prepared by James B. Hays, Consultant on Grouting Operations 5/. Treatment of fault zones beneath the streambed and on the right abutment is discussed in Paragraph 3.08 c.

4. Materials - During preliminary investigations, test pits were excavated in alluvial deposits both upstream and downstream from the damsite. The pits indicated that abundant materials were available for the construction of an earth and rockfill dam embankment. Field laboratory tests, including grain-size analyses, compaction tests, and permeability determinations established the suitability of available materials.

Borrow area materials were of two (2) general types: a silty clay suitable for impervious fill and a pervious gravel containing a small amount of silt. In addition to these materials, a considerable volume of rockfill was available from required spillway and tunnel excavations. The above preliminary investigations were extended later to reveal additional information required for final design.

Laboratory tests were required to establish the structural properties of the earth materials. Since complete testing facilities were not available in Afghanistan at that time, three (3) representative samples of impervious borrow area material were sent to a soils testing laboratory in the United States. The following tests were performed:

(a) Compaction and penetration

(b) Specific gravity

(c) Atterberg limits

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5/ "Grouting - Kajakai Dam" - Letter by James B. Hays, dated September 22, 1950 (Appendix A)

(d) Mechanical analyses

(e) Triaxial shear

(f) Permeability

Results of these tests appear in a report prepared by O. J. Porter & Company, Consulting Engineers 6/.

Triaxial shear tests were performed on the impervious materials. The values of the shearing resistance obtained by several procedures were used in stability analyses for different loading conditions. The quick shear value was used for analyzing the embankment immediately after placement. The quick-consolidated value of the shearing resistance was used for stability analyses under conditions applying during reservoir drawdown. A more complete discussion of the significance of the various shear values is presented in a report on earth dams and rockfill dams prepared for the Government of Afghanistan by International Engineering Company, Inc. 7/.

Shear tests were performed on materials to be placed in the gravel or rockfill zones of the embankment. Equipment for determination of shear strengths of very coarse soils is being developed by laboratories of the United States Corps of Engineers and the United States Bureau of Reclamation, principal dam-building agencies of the United States Government. Results of these tests to date by these agencies on materials similar to those available in Afghanistan have indicated that an angle of internal friction between 34 and 45 degrees may be expected. A conservative value of 35 degrees for the angle of internal friction was assumed in stability analyses.

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6/ "Laboratory Tests on Soil Samples from the Borrow Area, Kajakai Dam, Afhganistan" by O. J. Porter & Company, dated January 23, 1950 (Appendix A)

7/ "Report on Earth Dams and Rockfill Dams" by International Engineering Company, Inc., dated May 1953 (Appendix A)

5. Embankment Design - At the time preliminary designs were prepared for work under 1950 contract, test results of the structural properties of borrow materials were not available. Consequently, for estimating purposes a tentative embankment section with conservative slopes was proposed pending completion of laboratory tests and embankment stability studies. The preliminary section used was typical of numerous earth and rockfill dams which have been successfully designed and built by the United States Bureau of Reclamation.

Results of subsequent laboratory tests indicated that borrow area materials have exceptionally good structural properties. The volume of rockfill available from required excavations was increased when the spillway was enlarged to handle a greater flood. The embankment was, therefore, redesigned to obtain a more economical section.

Several cross-sections, incorporating recommendations of members of the Board of Consultants, were studied. The development of the final embankment section is described in the following paragraphs.

After review of preliminary designs by the Board, Mr. J. P. Growdon recommended that the embankment section be revised 8/. He proposed that two (2) cofferdams be constructed of dumped rockfill to top El 1,010 meters at the upstream and downstream toes of the dam. The rockfills were to be dumped at the natural angle of repose. Since laboratory tests had established that the permeability of the core material was low, he suggested that a core thickness of 20 meters would be ample to prevent measurable leakage. The remainder of the dam was to be constructed of free-draining sand and gravel with an upstream slope of 2.5:1 and a downstream slope of 2:1 above the cofferdams.

8/ "Kajakai Project" - Letter by J. P. Growdon, dated March 11 and 13, 1950 (Appendix A).

The principal purpose of the cofferdams was to increase the available head on the tunnels to compensate for increased friction losses resulting from omission of tunnel lining (see Section 3.05). The cofferdams would also minimize damage caused by the occurrence of a flood exceeding the diversion capacity of the tunnels, by creation of a quiescent pool over the embankment area between the cofferdams.

The Board of Consultants considered the following factors:

(a) The volume of rockfill from required excavation was insufficient and it is necessary to obtain additional rock-fill material from a quarry.

(b) Embankment construction would require a two (2) year schedule. Therefore, excavation of the entire spillway during the first season to provide rock for the cofferdams would result in an unbalanced construction schedule and would require additional rock-handling equipment and result in higher costs.

It was proposed to incorporate in the design desirable features of the preliminary section and the recommendations of the Board of Consultants. To reduce the quantity of rockfill placement required during the first construction season, the upstream cofferdam, which would eventually become a permanent part of the main dam, was eliminated and replaced by a smaller temporary cofferdam.

In addition, the top of the cofferdam was lowered ten (10) meters to El 1,000 meters. Rockfill blankets, tapering in thickness toward the top of the dam, were included on both slopes. The computed upstream slope of the embankment was 2.25:1. The downstream slope was 2:1 above top

To obtain materials meeting the grain-size distribution requirements for graded filters, it would be necessary to crush, screen, and wash materials obtained from the borrow areas. This expensive procedure was avoided by substituting unprocessed material for graded filters in the transition zones. Within a horizontal distance of 20 meters from each abutment, the transition zones were constructed entirely of impervious material, thus increasing the length of the seepage path along the abutment contact. The graded filter between the downstream gravel and rockfill cofferdam section was eliminated. Seepage below the core was estimated to be negligible.

6. Stability Analysis - The stability of the embankment slopes was investigated by the Swedish Method in which failure is assumed along a circular arc. For conservative analysis, the lowest values of the shearing resistance obtained from triaxial shear tests were used for impervious materials. A conservative angle of internal friction was assumed for the gravel and rockfill zones, based on large-scale tests of similar materials by U. S. Government agencies.

The factors of safety of the slopes, as determined by these analyses, and for all conditions of loading, exceeded the minimum values followed by standard American practice.

A set of specifications and design criteria was prepared 9/. The shearing resistance and coefficient of permeability of materials, and the various zones assumed in design studies, are set forth in these specifications. Periodic tests on materials would be made during construction

9/ "Design Criteria, Specifications, Laboratory Procedures" (for the Embankment Foundation of the Kajakai Dam) by International Engineering Company, Inc., dated October 1950, revised March 1952 (Appendix A).

of the embankment to insure that these criteria were met.

7. Piezometer Installation - Design of the main dam included details of a piezometer installation which would permit the measurement of pore pressures in the completed embankment. This type of data will allow a check to be made on the accuracy of stability study assumptions.

Thirteen (13) plastic piezometer tips, embedded in the embankment, were located at cross-section at Station 0 + 290 on the dam axis. The majority were located on the impervious core. Two (2) plastic tubes from each tip were carried through the embankment to individual compound-altitude, Bourdon-type pressure gages located in a terminal cabinet on the left abutment at the toe of the dam. The tubes were to be filled with water so that a closed hydraulic circuit would transmit water pressures at the tip to the gage in the cabinet. Since air bubbles would not transmit full pressure from the tip, means were provided to remove air from the circuit. The piping and valve system in the cabinet would allow a comparison of individual gage readings against a more precise master gage.

To facilitate shipment of the large number of small items in the terminal well, such as gages and pipe fittings, the entire terminal well unit is assembled in a permanent steel-insulated cabinet. The installation is similar to that developed by the U. S. Bureau of Reclamation.

b. Spillway

1. Preliminary Investigations - Collection of streamflow records on the Helmand River near Girishk was started by Morrison-Knudsen Afghanistan, Inc., in July 1946. No previous long-term flow measurements were available. The 1939 flood, reported to be the largest in recent years, was investigated. The peak discharge was estimated at 4,000 cubic meters per second (140,000 cfs)

The cost of additional excavation was outweighed by the other factors and the alternate location was finally selected.

Based on the flood criteria previously mentioned, the required maximum spillway capacity after routing was found to be approximately 12,200 cubic meters per second (430,000 cfs). Several layouts providing this capacity were studied and are described below:

(1') With the crest elevation at about 1,035.5 meters, the length of crest was found to be about 140 meters. The required spillway excavation would be about 840,000 cubic meters.

(2') Layouts were prepared for a main and an auxiliary spillway. Final evaluation showed the proposed layouts to be comparatively unfavorable from both economic and construction viewpoints.

(3') The spillway crest was lowered about two (2) meters, thereby reducing the required length about thirty percent. The reservoir capacity for the initial ungated spillway was reduced but there still remained adequate storage to provide regulation for many years.

The third alternative was adopted on the basis of economy of first construction. By routing the inflow design flood through the reservoir, with a spillway crest elevation of 1,033.5 meters and a crest length of 113.6 meters, it was found that a spillway with a capacity of 12,200 cubic meters per second would carry the outflow safely.

A tentative layout for the future gated structure was then prepared. The future gated spillway would be regulated by eight (8) radial gates, 12.19 meters (40-ft) wide by 11.22 meters (37-ft) high. Hoists would be located on an operating deck at El 1,050 meters. The crest structure would be located at the downstream end of the cut where minimum additional rock excavation would be required.

This spillway should be re-examined in later years before construction. By that time, additional hydrologic data will be available for evaluation.

c. Tunnel Size and Alignment

1. General - The tunnels were designed for the following purposes:

(a) To divert streamflow around the damsite during construction.

(b) To serve as an outlet for release of irrigation water.

(c) To provide a power conduit for use by the future powerhouse.

2. Diversion Capacity - Streamflow records indicated that runoff could be separated as follows:

(a) A period of low runoff from about July through January, during which a maximum diversion capacity of about 140 cubic meters per second (5,000 cfs) would be required.

(b) A season of flood discharges from about February through June. The proposed construction schedule required two (2) years for building the embankment. Thus, the diversion tunnels would be required to pass the spring floods. The determination of the diversion capacity

The diversion capacity of the tunnels was lowered because of increased friction losses. However, a study of the productive capacity of the equipment available for the first construction season indicated that the dam could be brought up to El 990 meters during this period. The maximum diversion capacity under these conditions was computed as 1,800 cubic meters per second (63,500 cfs) without overtopping the embankment.

3. Tunnel Alignment - In establishing the alignment of the tunnels the following requirements were considered:

(a) The upstream and downstream tunnel portals must be sufficiently removed from the dam construction area to leave ample space for cofferdamming.

(b) The tunnels must be carried sufficiently far into the abutment to provide ample horizontal and vertical rock cover. This cover must be adequate to restrain internal water pressures.

(c) The portal locations must result in a minimum length of tunnel consistent with the other requirements. Portal locations in poor rock requiring heavy timbering are to be avoided.

(d) Discharge from the tunnels must be directed to avoid scour at the toe of the embankment.

(e) The downstream portal of the power tunnel should be located at a site suitable for the future powerhouse.

(f) The centerline distance between the two (2) tunnels should be sufficient to avoid excessive stresses in the intervening rock.

The final alignment differed in only minor respects from the preliminary layout. Use of a steeper upstream slope on the dam (see paragraph 3.03 e) permitted the upstream tunnel portals to be moved further downstream, thereby reducing the tunnel lengths. At the recommendation of the Board of Consultants, the tunnel alignment was moved further into the cliff so that the inside (power) tunnel was approximately 50 meters from the face 12/. Minor adjustments in the portal location were made by the field engineering forces on the basis of geologic and topographic conditions not apparent on topographic sheets.

4. Permanent Outlet Arrangement - A number of possible arrangements for power and irrigation outlets were investigated. Basically, these arrangements were classified as divided-purpose schemes or dual-purpose schemes. In the divided-purpose schemes, irrigation outlets were placed in one (1) of the two (2) tunnels and the other was reserved for future power releases In the dual-purpose schemes, each tunnel was used as a combined power and irrigation outlet. Economic comparison indicated a decided advantage for the divided-purpose scheme, which was therefore adopted.

d. Irrigation Outlets

1. General - The layout shown on the Contract Drawings 13/ has been described previously (see subparagraph 3.01, 3 (b) ).

2. Intake - General - The preliminary design of the submerged trashrack structure at the intake to the irrigation tunnel was made similar

12/ "Kajakai Project" - Letters by J. P. Growdon, dated March 11 and 13, 1950 (Appendix A)

13/ "Contract Drawings, Afghanistan" by International Engineering Company, Inc. dated 1948 and 1950 (Appendix C)

to numerous installations on U S. Bureau of Reclamation projects. In general, this type of installation has given satisfactory service. Removal of debris from the trashracks, if necessary, would be performed by divers. Divers would also be required for placing stoplogs at the portal opening should it become necessary to unwater the tunnel upstream from the plug. Stoplog closure could be effected only if there were very little or no flow through the tunnel.

Mr. Growdon of the Board of Consultants recommended the following changes in design of the intake structure:

(a) Provide a means for closure of the upstream portal with the reservoir at the maximum water surface El 1,045 meters, and with a flow of 100 cubic meters per second (3 500 cfs) through the tunnel.

(b) Permit the removal of trashracks with the reservoir at maximum elevation.

These recommendations were eventually adopted.

3. Intake Closure - Several methods for closure of the tunnel at the upstream portal were investigated. These methods included the use of:

(a) A steel or concrete stoplog installation.

(b) A steel hemispherical bulkhead to close the entire portal.

(c) A fixed-wheel gate to close the entire portal

(d) A stoplog installation for sealing a portion of the opening and a fixed-wheel gate for complete closure.

To prevent excessive head losses at the portal, a sufficient area must be maintained at the opening to limit entrance velocities to about 6 to 7.5 meters per second (20 to 25 fps). This amounts to an area of about 40 square meters (430 sq ft) for maximum discharge conditions.

In the first scheme, closure of the portal by stoplogs was not considered practicable. The high-velocity jet entering the tunnel during placement would produce negative pressures on the underside of the last and next-to-last sections, causing the last section to hang up. Even a hoist or dashpots could not be expected to eliminate these balancing difficulties with certainty.

The second scheme, a one-piece bulkhead would be likely to hang up in the guides because of frictional resistance due to unbalanced hydrostatic forces on the upstream face.

In the third scheme, a fixed-wheel gate of sufficient size to close the tunnel portal would have been about 4.27 meters (14-ft) wide and 9.15 meters (30-ft) high. This method was found to be workable, but the large gate and hoist would probably prove too expensive.

In the fourth scheme, the portal was divided into two (2) openings by a concrete pier. One opening was 3.05 meters (10-ft) wide by 9.15 meters (30-ft) high, and the other opening 1.98 meters (6.5-ft) wide by 6.10 meters (20-ft) high, the height of the latter opening being reduced by a sill. The larger area could be closed under a small unbalanced head by a reinforced concrete bulkhead. The smaller opening would then be closed under unbalanced head conditions by a fixed-wheel gate. This scheme proved to be most economical and was ultimately adopted.

During diversion, it would be desirable to maintain the entire cross-sectional area of the tunnel at the portal. The portal was therefore designed for two-stage construction. The pier and sill were omitted in the first stage and were planned for construction following diversion. Keyways would be provided for this purpose.

4. Tower and Portal Structure Arrangement - Investigations were made to determine the most economical arrangement for operating the intake gates and removing the trashracks from an operating level above the maximum reservoir surface. Among the plans investigated were:

(a) An inclined structure supported on a uniform rock slope excavated above the portal.

(b) A closed reinforced concrete tower.

(c) An open rigid-frame concrete tower.

The topography above the portal was not sutiable for an inclined intake structure. It would have been necessary to excavate a large volume of rock in order to obtain a uniform slope. In addition, closure of the portal under flow conditions would be more difficult with an inclined gate. Only the component of the weight of the gate in the direction of the incline would exert a closing force. Also, at the same time, increased frictional resistance would develop due to the component normal to the guides.

Since the geologist had recommended that all structures should be designed to resist an earthquake acceleration of 0.05 gravity (see subparagraph 3.02 a 2), the resulting inertia forces governed the design of the high structure. These forces would be very large if the structure were submerged since inertia forces of the water would also be resisted. A closed structure would expose a large surface area to external hydraulic forces and would be subject to very large forces in the event of an earthquake.

An open rigid-frame concrete structure proved to be the most economical of the various schemes and was selected for final design.

The elements of the tower and portal structure were as follows:

(a) Portal structure

(b) Tower

(c) Crane operating deck

(d) Bridge

5. Portal Structure - The portal structure extended upstream from the tunnel and would serve as a foundation for the tower. It also extended approximately 30 meters downstream to support the rock arch in the vicinity of the portal. The upstream and downstream portions were designed to act together as a rigid base for the tower.

The trashrack area at the upstream end was divided into three (3) 3.5 meter wide sections by vertical beams which also served as guides for rack sections. The vertical beams were supported by horizontal girders which would carry reactions into side walls. The portion of the portal structure upstream from the gates was designed for an external hydrostatic head of 3.05 meters (10-ft). This allowed for head losses which might be caused by clogging of the racks. The trashracks were designed to fail in the event of excessive clogging, in order to protect the concrete structure from damage.

A transition from rectangular gate and stoplog openings to the horseshoe tunnel section was formed by the pier and tunnel lining. The lining was to be extended about 15 meters downstream from the transition with a minimum concrete thickness of 0.3 meters. A high-pressure grout curtain was to be included at the downstream end of the transition and the surrounding rock for the entire length of the lining. The transition was designed to resist the full external hydrostatic pressure of the reservoir in order to permit unwatering of the tunnel.

6. Tower - The tower frame was supported on the roof of the portal structure at El 983.3 meters and extended upward to the operating deck at El 1,050 meters. The total height was 66.7 meters. The width of the tower was 12.7 meters in both directions. This made the width-to height ratio equal to 1:5.2. The tower was analyzed by the method of moment distribution by treating it as pairs of two-dimensional frames in both directions. The crane bents were analyzed separately since they were not framed together at the top and would not act integrally with the tower.

A study was made to determine the most economical story height for the frame. Concrete volume and formwork for six (6) and eight (8) story towers were estimated and it was found that the story height had no significant effect on the cost. The six (6) story height was selected as more conservative.

Torsional rigidity at the base was provided by the portal structure and at the top by the slab and beams of the operating deck. Additional rigidity was supplied by cross-bracing at the fifth (5th) story level.

7. Crane Operating Deck - The electrically operated lifting hoist had a rated capacity of 75 tons with a 275 percent overload feature. The crane could be moved to any position over the deck to operate the intake gate and bulkhead or to handle trashrack sections. Lateral movement of the hoist could be accomplished by use of a device for manual operation.

The lifting beams were provided with a semi-automatic engaging block that would permit them to engage and disengage the gate or

bulkhead under 70 meters (230-ft) of water. Thus the single crane could be used to handle the gate, bulkhead, and the trashracks.

8. Bridge - A structural steel bridge was designed to connect the operating platform of the intake tower with the dam abutment. The bridge would have an over-all length of 71.54 meters (234-ft 8-1/2 in.) and a clear-width walkway of 1.52 meters (5-ft). It would consist of a deck-truss span and a deck-plate girder span 25.68 meters (83-ft 10-1/2 in.) long. A 24.09 meter (81-ft) high steel bent would serve as the intermediate support.

9. Tunnel Plug - In the contract drawings, the tunnel plug was located approximately at the intersection of the extended dam axis and tunnel centerline. The decision was subsequently made to locate the outlet regulating valves at the downstream end of the tunnel and to carry irrigation releases from the plug to the regulating valves in steel penstocks. Therefore, the tunnel plug location was moved as far downstream as possible to reduce the length of the steel penstocks. A minimum of 30 meters of sound rock cover, vertically above the portion of the tunnel upstream plug, was required 14/. This cover was analyzed by the theory of elasticity and found to be sufficient to permit the rock to withstand the full internal hydrostatic pressure of the reservoir without causing joints in the rock to open. In the final design, the tunnel plug was located at the beginning of the tangent below the downstream curve in the tunnel alignment where the sound rock cover is above minimum requirements.

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14/ "Kajakai Project" - Letters by J. P. Growdon, dated March 11 and 13, 1950 (Appendix A)

The plug was designed in two (2) sections. Each section was keyed into the surrounding rock and thoroughly grouted. Either of the two (2) sections was capable of resisting the maximum headwater pressure with low shearing stresses on the periphery. Conservative practice was used in the design of the tunnel plugs, as they represent only a minor portion of the project cost.

10. Conduits and Valves - The regulating valves were located on the Contract Drawings 15/ in a chamber immediately downstream of the tunnel plug and near the center of the tunnel. Velocities in the tunnel downstream from the valves were found to be high. The continued resistance to erosion under continuous velocities in this range could not be depended upon. The decision was made finally to locate regulating valves at the downstream portal of the tunnel. Steel conduits would carry flow from the tunnel plug to the regulating valves.

The optimum number of irrigation conduits were determined from a study of the following considerations:

(a) Number and size of conduits and valves.

(b) Hollow-jet vs. fixed dispersion-cone valves for normal operation.

(c) Ring-follower gates vs. plug valves (rotovalves) for emergency shutoff.

Arrangements requiring more than three (3) conduits could not be accommodated within the limiting dimensions of the tunnels. Selection of number and size of conduit was therefore restricted to arrangement of three

15/ "Contract Drawings, Afghanistan" by International Engineering Company, Inc., dated 1948 and 1950 (Appendix C)

(3) or fewer conduits. A scheduled or emergency closure of a single conduit would shut off all irrigation releases if only one conduit were used. The discharge capacity of the tunnel would be reduced by 50 percent in a two (2) conduit arrangement. Project operation would not permit a 50 or 100 percent temporary shutoff of water releases. The arrangement was confined, therefore, to a three (3) conduit scheme.

Various combinations of conduit and valve diameters were studied. The most economic arrangement was reduced to the following two (2) plans:

(a) Three (3) 84-in. steel conduits with three (3) 84-inch hollow-jet regulating valves.

(b) Three (3) 78-in. steel conduits with three (3) 78-in. cone-type regulating valves.

The coefficient of discharge of hollow-jet valves is about 0.7 compared with 0.85 for fixed-dispersion cone valves. Smaller conduits and valves could therefore be used with cone-type valves. This advantage would be offset by the requirement of a considerably larger outlet structure for the cone valves. With either type of valve, the outlet structure would contain concrete piers with stoplog slots. As a result of the wide angle of spray of cone-type valves, the manufacturer established substantial minimum clearances between valves and adjacent piers. This would necessitate a greater spacing between piers and also a greater length of pier downstream from the valves. Since the jet from the cone valve would impinge directly on the piers, the pier would also be structurally capable of withstanding these forces. In the case of the cone valves, a hood would also be

required downstream from the valves to confine the spray. This would protect the future powerhouse area. An economic study of the alternate arrangements revealed that the first scheme, using hollow-jet valves, was more economical.

The steel conduits were to be encased in concrete for their full length to prevent damage, should rock break off from the tunnel roof arch. The surface of the concrete encasement would serve as a convenient access road to the emergency valve chamber.

An emergency shutoff valve installed at the upstream end of each penstock would allow the conduit to be quickly shut off in case of damage to a penstock or regulating valve. It would also permit maintenance work in a single conduit while continuing irrigation releases through the other two (2) conduits. In the 1950 Contract Drawings 16/, a ring-follower gate was indicated as the emergency shutoff valve. When specifications were issued, manufacturers were permitted to submit alternate proposals for this valve. The S. Morgan Smith Company of York, Pennsylvania, submitted a lower bid for rotovalves than the bid submitted on the ring-follower gates by another manufacturer. Since either type of valve would be equally satisfactory, rotovalves were procured and necessary revisions made in the design of the valve chamber. An overhead monorail crane with a capacity of 16 tons would be provided in the valve chamber for initial installation of the valves and future servicing. The geared trolley chain hoist was to be manually operated and of the rail-hugger (low headroom) type, which would minimize the required height of the chamber. Three (3) tracks with a manually-operated switching device were provided so that the hoist could

16/ "Contract Drawings, Afghanistan" by International Engineering Company, Inc., dated 1948 and 1950 (Appendix C)

service any one of the three (3) valves. The crane was selected to have sufficient capacity to handle the heaviest section of the valve for assembly or maintenance. The interior of the chamber would be 12 meters by 7 meters high and 14.5 meters long. It would be completely lined with reinforced concrete one (1) meter thick. The roof was designed as a concrete arch.

11. Tunnel Ventilation - The ventilation system was provided in the emergency valve chamber for general ventilation purposes, including the relief of humidity conditions. A 0.508 by 0.914 meter (20 by 36 in.) duct would extend from a fan room located in the valve house to the emergency valve chamber. Air was to be circulated through the duct by an electric-driven centrifugal fan with a capacity of 142 cubic meters per minute (5,000 cfm), which would permit seven (7) complete air changes per hour.

Another ventilation system was designed for the valve house in which air would be circulated by an electrically-driven centrifugal fan with a capacity of 47 cubic meters per minute (1,650 cfm).

12. Power Supply - A small water-wheel generator located in the outlet-valve house would serve as a source of power for operating the emergency and regulating valves and for lighting and ventilating the tunnel and valve house.

The water-wheel generator, a horizontally mounted three (3)-phase, 50-cycle unit, rated at 93 kva, would be direct-connected to a Francis-type turbine rated at 91 metric hp (90 hp) at 1,500 rpm and 72.38 meter (208-ft) head. A power conduit would convey water from the tunnel plug to the turbine in a 0.3048 meter (12-in.) diameter steel penstock supplied with a 0.3048 meter (12-in.) and a 0.2032 meter (8-in.) gate valve

for emergency and normal shutdown of the unit.

Power to operate the intake-tower crane would be supplied by an emergency 3-phase, 50-cycle, diesel-generator set, rated at 104 kva. It would serve as a source of power for illuminating the generator house and the water-level recorder building. A 400-volt circuit from the emergency generator to the valve house would supply power for operating the outlet valves when the water-wheel generator was shut down.

e. Power Tunnel

1. Initial Installation - Final design of the initial installation of the power tunnel was essentially the same as shown on the contract drawings, except for the omission of the tunnel lining which has been previously discussed (see paragraph 3.02, c). The design of the initial tunnel plug was based on the following considerations:

(a) The intake portal structure was designed to withstand full reservoir head.

(b) The second plug will be installed when the future power installation is made.

(c) Downstream movement of the plug in the initial stage would cause relatively less damage than similar action in the irrigation tunnel plug.

2. Future Installation - A power plant will be installed in the future when there is a sufficient demand in the Kandahar area for electrical energy. It was contemplated that the total plant capacity will be 120,000 kw. The initial construction placed no restrictions upon the future plant capacity or the number and size of the units. These factors can be further evaluated at the proper time.

It was anticipated that the power intake will have its invert at El 994 meters. A bench would be excavated at this level on the abutment. It would be necessary to drawdown the reservoir from El 1,033 to 990 meters immediately after the end of the spring runoff (about June 1) in order to permit construction of this intake. Five (5) to seven (7) months, depending on the inflow, would be required to lower the reservoir with all outlets discharging. This would leave two (2) to four (4) months for construction of the intake. Some work could be performed on the intake structure during the reservoir drawdown period.

An inclined shaft would be excavated from the intake to the power tunnel downstream of the initial plug.

The power plant was expected to be located between the two (2) downstream tunnel portals.

f. Construction

1. General - Only those developments which occurred during construction and which resulted in incorporation of special features, or in design modifications, are presented in this section. Such developments are as follows:

(a) Special treatment of the dam foundation on abutment areas and tunnel grouting.

(b) Substitution of gravel for rock in the dam embankment.

(c) Temporary installations in the irrigation tunnel resulting from a delay in the delivery of the emergency valves.

(d) Low concrete weir in the spillway channel.

2. Special Treatment of the Left Abutment Area and Tunnel Grouting - As construction progressed, conditions were exposed in the abutment rock by tunnelling and stripping operations, which indicated special treatment was necessary.

The site was visited in July 1951 by James B. Hays, Consulting Engineer, who made a detailed inspection of the site and supervised grouting tests. He prepared a set of instructions for treatment of the conditions found which served as the basis for the procedure followed. These instructions, including a detailed description of the conditions encountered, appear in a report by Morrison-Knudsen Afghanistan, Inc., 17/.

The rock in the dam foundation and abutments is broken by two (2) groups of vertical joints. These are at approximately right angles and cross the axis of the dam at roughly 45 degrees in east-west and north-south directions, the former group being the more numerous. It would have been possible to develop leakage from the reservoir, particularly at the left abutment where water entering the east-west system from the reservoir could issue from the north-south joints downstream from the axis. The contact over a large portion of these joints was tight. There were numerous pockets of clay of varying size, indicating the presence of channels to provide access for the water which acted as the weathering agent.

In order to minimize leakage, the grout curtain under the dam was extended to tie into the curtain around the tunnels. Grouting was also performed from deep holes drilled directly above the tunnels.

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17/ "Treatment of Joints and Seams in Tunnels" by Morrison-Knudsen Afghanistan, Inc., (Gilbert Waddell and James B. Hays) dated September 24, 1952 (Appendix A)

Since the irrigation tunnel is plugged near the downstream end, it will be subject to the full internal hydrostatic pressure of the reservoir for almost its entire length. The power tunnel will be subject to similar pressures when the future plug is installed. It was, therefore, decided to seal all joints in both tunnels with gunite or concrete. The power tunnel is expected to be dry for a number of years and the irrigation tunnel will be unwatered occasionally for maintenance purposes. The tunnels upstream from the grout curtain will thus be subjected to unbalanced external pressures. The joint seals upstream from the permanent plugs were therefore designed to resist either internal or external pressures. Clay, decomposed rock, and/or loose rock was removed from each joint to a sufficient depth so that the joint seal could resist the flow of water. Narrow joints were chipped to form a vee or square notch to provide a satisfactory key for the gunite. Dowels were installed as necessary. Large openings in the floor were sealed with concrete, using adequate dowels and reinforcement. Gunite was used above the springline.

Holes were drilled into the tunnel walls for grouting the joints. The holes were drilled to cross the joints at an angle that would increase the area of intersection. Radial grout curtains were constructed to tie into the grout curtain in the left abutment area. A grouting pressure of 10.5 to 21 kg per sq cm (150 to 300 psi) were used.

As a further precaution against excessive leakage between the impervious core and the irrigation tunnel portal, the joints were cleaned out and sealed with gunite to minimize leakage.

wide, is on the abutment slope crossing the axis at El 985 meters at an angle of 28° - 30' (downstream toward the river channel). A shaft, 21 meters in depth, was excavated at a point where the fault intersects a seam 16 meters downstream from the axis, and the shaft was backfilled with concrete. A third shaft was excavated to a depth of 8.5 meters into the fault at its intersection with the grout curtain and similarly backfilled with concrete.

A gorge, 15 meters wide, was found beneath the streambed near the center of the canyon and roughly parallel to it. The gorge could have been formed by erosion along an ancient stream channel. The bottom of the gorge was cut into deep relief and contained pinnacles, potholes, and sinuous roughs up to 1.8 meters in depth, which were filled with densely-packed big boulders, cobbles, sand, and gravel. After cleaning out the area, the rock surface was covered with gunite and depressions were filled with lean concrete to bring the surface to a general level which would permit placing impervious fill.

Further details on treatment of fault zones may be found in two reports, 18/ and 19/.

4. Dam Embankment - Specifications for Kajakai Dam established a permissible range for the slope of the boundary between the gravel and rockfill zones, both upstream and downstream from the impervious core.

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18/ "Quarterly Progress Report" by Morrison-Knudsen Afghanistan, Inc., October 1 to December 31, 1951 (Appendix A)

19/ "Quarterly Progress Report" by Morrison-Knudsen Afghanistan, Inc., January 1 to March 31, 1952 (Appendix A)

These limits were established to permit full utilization of the rock excavated from the spillway. The spillway site contained a fault zone comprised of fine-grained material. As a result, portions of the excavated rock contained excessive amounts of clay. Some of the excavated material was, therefore, unsuitable for use in the rockfill portions of the dam. The yield of rockfill for the dam from the spillway excavation was less than anticipated and the rockfill zones were reduced to the minimum limits permitted by the specifications. The slopes between the gravel and rockfill zones were thus constructed as 1.6:1 upstream and 1:1 downstream.

5. Irrigation Outlets - Delivery of the emergency valves was delayed for over a year as a result of concentration by the manufacturer upon wartime production. Therefore, it was impossible to complete final installation of the irrigation outlets in time for operation of the project during the 1952-1953 irrigation season as had been planned. This led to the development of a temporary installation which would enable the contractor to complete all other project features, fill the reservoir, and release water for irrigation in the absence of the valves. It was further necessary to permit future installation of the emergency valves without shutting off irrigation releases for a prolonged period.

For the temporary installation, three (3) extra sections of steel conduit were fabricated and installed in place of the valves. The sections were provided with flanges at the downstream ends for bolting to the conduit. At the upstream end, the sections were joined by expansion-type couplings to short-conduit sections. These were bolted to the upstream section of the conduit. The regulating valves were operated only in the wide-open position during this period. This last was a precautionary measure and not necessary otherwise.

After delivery of the emergency valves, they were installed, using the following procedure:

(a) Regulating valves were shut, the upstream portal closed with the gate and bulkhead, and the tunnel unwatered.

(b) The temporary section and the short section upstream from one conduit were removed. A bulkhead was bolted to the upstream section of the conduit. The shutdown required for this operation was of short duration.

(c) The gate and bulkhead were removed from the upstream portal, permitting operation of the remaining valves while the first plug valve was installed.

(d) The valve was installed and tested dry.

(e) The regulating valves were shut off. The upstream portal was closed and the tunnel unwatered. This permitted removal of bulkhead and installation of the closure nipple and expansion coupling.

The remaining two (2) valves were installed similarly.

6. Low Concrete Weir - A low concrete weir, with crest at El 1,033.5 meters, was placed across the spillway channel. Additional concrete was placed downstream of the weir to fill in low places in the rock surface.

g. Service Behavior

Since completion of the Kajakai Dam, there has been opportunity to observe the behavior of certain parts of the installation. These observations are as follows:

(a) Dam - Following the spring runoff in June 1953, the reservoir filled to El 1,031.5 meters or 2.0 meters below the spillway crest.

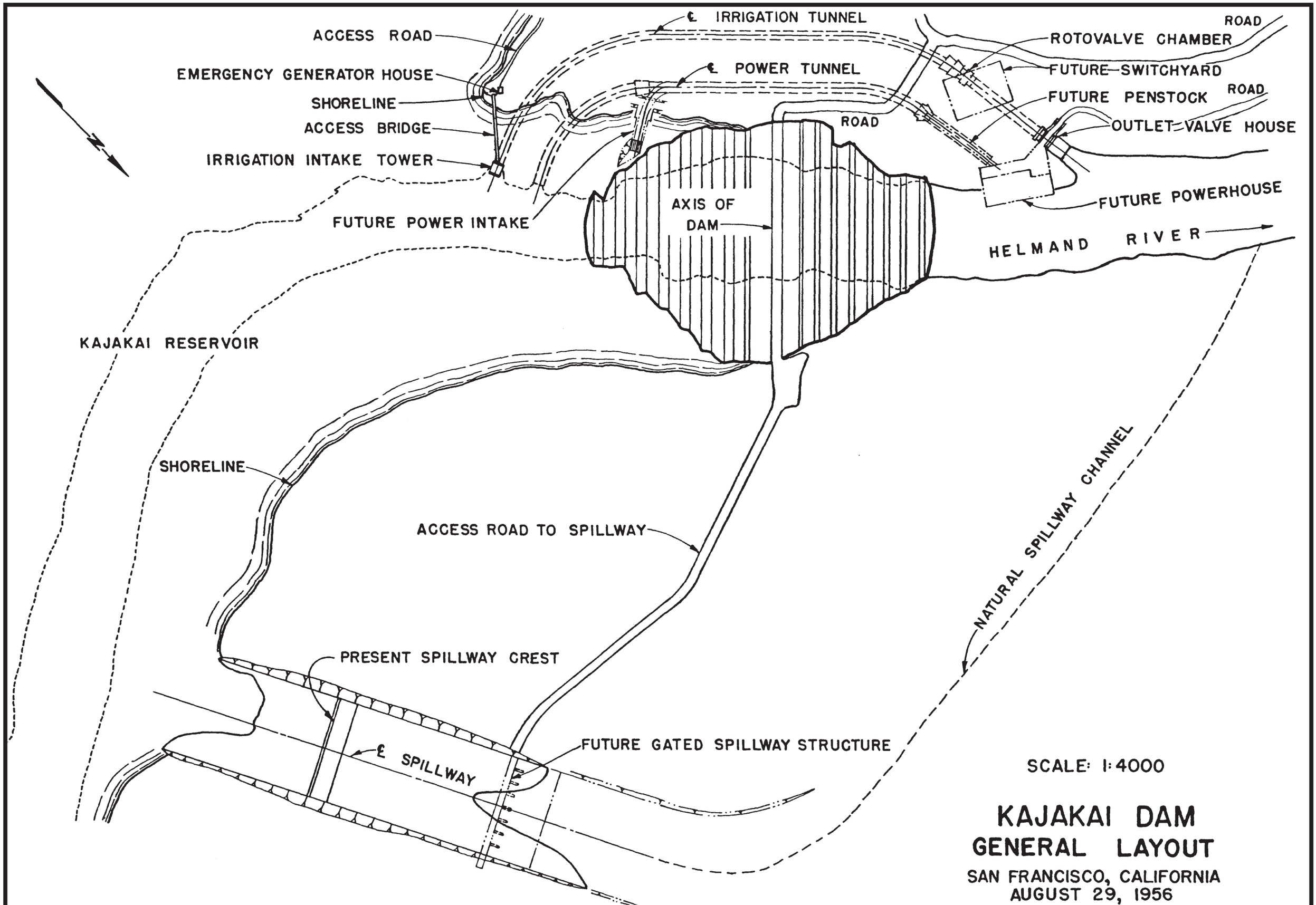
No seepage through the embankment has been observed.

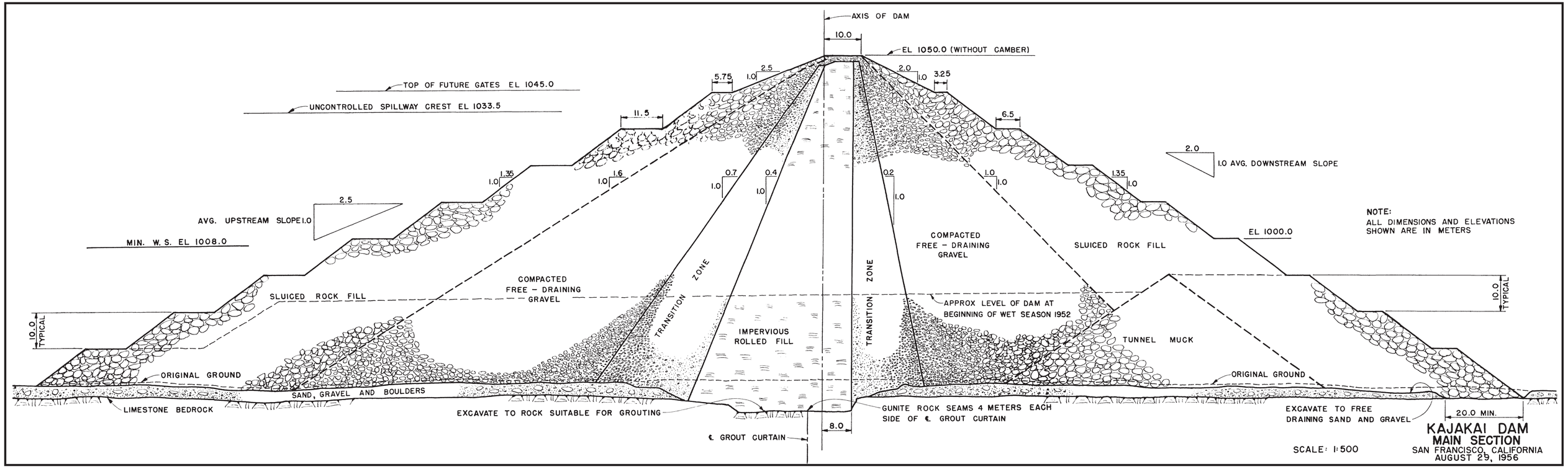
(b) Spillway - The first spillway discharge over the Kajakai occurred on April 18, 1954. The maximum overflow occurred on May 3, when the reservoir rose to El 1,035.8 meters with a corresponding discharge of 678 cubic meters per second (24,000 cfs). Principal effect of this discharge was erosion of overburden materials in the natural spillway channel.

(c) Tunnels - From all indications, the grouting program that was followed is proving to be successful.

(d) Irrigation Outlets - Shortly after January 1953, when the hollow-jet irrigation outlet valves had been in operation for a short time, it was found that the rubber seals had been damaged. The manufacturer investigated the cause of the damage. Model tests by others, of this type of valve, have indicated that a differential pressure between the upstream and downstream faces of the valve seal might be sufficient to cause the seal to stretch and vacate the groove. Several methods for the correction of this condition are under investigation.

The rotovalves were tested after installation for performance under emergency operating conditions. The valves were closed successfully under unbalanced head conditions.


ACCESS ROAD
EMERGENCY GENERATOR HOUSE
SHORELINE
ACCESS BRIDGE
IRRIGATION INTAKE TOWER
FUTURE POWER INTAKE
℄ IRRIGATION TUNNEL
℄ POWER TUNNEL
ROAD
ROTOVALVE CHAMBER
ROAD
FUTURE SWITCHYARD
FUTURE PENSTOCK
ROAD
OUTLET VALVE HOUSE
FUTURE POWERHOUSE
AXIS OF DAM
HELMAND RIVER
KAJAKAI RESERVOIR
SHORELINE
ACCESS ROAD TO SPILLWAY
NATURAL SPILLWAY CHANNEL
PRESENT SPILLWAY CREST
℄ SPILLWAY
FUTURE GATED SPILLWAY STRUCTURE
SCALE: 1:4000
KAJAKAI DAM
GENERAL LAYOUT
SAN FRANCISCO, CALIFORNIA
AUGUST 29, 1956


AXIS OF DAM
10.0
EL 1050.0 (WITHOUT CAMBER)
TOP OF FUTURE GATES EL 1045.0
UNCONTROLLED SPILLWAY CREST EL 1033.5
5.75
1.0
2.5
2.0
1.0
3.25
11.5
6.5
2.0
1.0 AVG. DOWNSTREAM SLOPE
1.35
1.0
1.6
1.0
0.7
1.0
0.4
1.0
0.2
1.0
1.0
1.0
1.35
1.0
2.5
AVG. UPSTREAM SLOPE 1.0
MIN. W. S. EL 1008.0
NOTE:
ALL DIMENSIONS AND ELEVATIONS SHOWN ARE IN METERS
EL 1000.0
COMPACTED FREE - DRAINING GRAVEL
SLUICED ROCK FILL
COMPACTED FREE - DRAINING GRAVEL
SLUICED ROCK FILL
TRANSITION ZONE
TRANSITION ZONE
APPROX LEVEL OF DAM AT BEGINNING OF WET SEASON 1952
IMPERVIOUS ROLLED FILL
TUNNEL MUCK
10.0 TYPICAL
10.0 TYPICAL
ORIGINAL GROUND
ORIGINAL GROUND
SAND, GRAVEL AND BOULDERS
LIMESTONE BEDROCK
EXCAVATE TO ROCK SUITABLE FOR GROUTING
GUNITE ROCK SEAMS 4 METERS EACH SIDE OF ℄ GROUT CURTAIN
EXCAVATE TO FREE DRAINING SAND AND GRAVEL
20.0 MIN.
8.0
℄ GROUT CURTAIN
SCALE: 1:500
KAJAKAI DAM
MAIN SECTION
SAN FRANCISCO, CALIFORNIA
AUGUST 29, 1956

## SECTION IV

## ARGHANDAB DAM PROJECT

### 4.01 GENERAL FEATURES

a. Preliminary Design - The preliminary phase of design of the Arghandab Dam had been completed in 1950, and the results of these earlier studies are shown on the Contract Drawings 20/. The following description by main features pertains to the design as shown on these drawings and not necessarily to the project as subsequently constructed.

1. Main Dam and Saddle Dikes - The main dam was designed as an earth embankment with a maximum height of 48 meters above the original streambed. The crest of El 1,113 meters was to be 10 meters wide and 600 meters long. Computed slopes were 3:1 upstream and 2:1 downstream. The section was designed with a central impervious core of rolled earthfill flanked by zones of sand and gravel. The upstream slope and the base of the downstream slope were to be protected by riprap blankets. Alluvial materials overlying bedrock were to be removed from the foundation area.

Two (2) saddle dikes with an aggregate length of 650 meters were located at low points in the reservoir perimeter, approximately two (2) and three (3) kilometers from the main dam. The cross-section of the dikes was similarly designed as the main dam, except that the downstream slope was 2.5:1, and the slopes of the impervious core were steeper.

2. Spillway - A low concrete gravity weir was planned at a saddle about 1.3 kilometers from the dam. Spillway discharge would be controlled by five (5) radial gates, 12.19 meters (40 ft) wide by 9.75 meters (32 ft) high,

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20/ "Contract Drawings, Afghanistan" by International Engineering Company, Inc., dated 1948 and 1950 (Appendix C)

separated by concrete piers. The top of the gates, shown at El 1,110 meters, allowed three (3) meters of freeboard on the dam.

3. Outlets - A 220-meter long reinforced concrete conduit was located in the right side of the dam embankment. The upstream section of the conduit was shown as 5.18 meters (17-ft) in diameter. A steel liner with an inside diameter of 4.57 meters (15-ft) was located in the portion of the conduit downstream of the impervious core.

A reinforced-concrete intake structure located at the upstream end of the conduit would incorporate a semi-circular trashrack structure. Emergency closure of the portal would be made by a high-pressure Broome gate. The gate would be operated from above the maximum water surface by a hoist located in a control room on top of the intake structure.

Two (2) branch conduits were shown as an extension from the downstream end of the main conduit to an irrigation outlet structure. Releases were to be regulated by two (2) 1.524-meter (60-in.) valves, each with a 1.372-meter (54-in.) emergency control valve upstream. The downstream end of the conduit was shown to be temporarily closed by a steel bulkhead. A future power plant would incorporate a turbine manifold at this location.

b. Final Design - Final design of the Arghandab Dam project varied from the preliminary design. These variations occurred partly because of the revaluation of additional data received during later years, and partially because of the Board of Consultants' independent evaluations. The following descriptions by main features are of the changes in design made between the preliminary and final phases.

1. Main Dam and Saddle Dikes - Design changes included the following:

(a) The maximum height of the main dam was increased to 50 meters above the present streambed, thus the crest was raised to El 1,115 meters.

(b) The crest width was reduced to 8 meters and its length increased to 540 meters.

(c) The upstream main embankment slope was reduced to 2.5:1.

(d) A cutoff trench was provided through alluvial materials below the core.

(e) Only the silty top layer of these alluvial materials was removed from the foundation area and the underlying clean sand and gravel would remain in place.

(f) The number of saddle dikes was increased to six (6), with an aggregate length of 1,660 meters. Dikes No. 1 through 5 would have identical cross-sections and a maximum height of five (5) meters. The crests at El 1,115 meters were to be 6.4 meters wide, and the embankment slopes would be 2:1 upstream. They were reduced to 1.75:1 downstream. The sections were primarily gravel with an inclined impervious core of rolled fill. A one (1)-meter thick riprap blanket was to protect the upstream slope. Dike No. 6, with a maximum height of 20 meters, was a rockfill section. The 145-meter long crest was 6.4 meters wide. Embankment slopes were 2.25:1 upstream and 1.75:1 downstream.

2. Spillway - Design changes included the following:

(a) Two (2) open-channel spillways were cut through saddles in the reservoir perimeters, one located about 1.5 kilometers

from the main dam between Dikes Nos. 5 and 6, the other about 2.2 kilometers between Dikes Nos. 3 and 4.

(b) Both crests were raised to El 1,110 meters.

(c) The crest structure representing Spillway No. 1 would be a low ungated concrete weir with a riprap-paved apron on downstream slope. Flood flows passing over the 240-meter long crest would branch into two (2) channels, each 36 meters wide. The channels were designed to merge about 250 meters downstream from the crest.

(d) Spillway No. 2, a 100-meter long ungated concrete crest structure, would be a small weir with a short downstream apron. A hand-placed stone apron of variable length was designed to extend a maximum of 15 meters further downstream. Stone masonry walls were provided to retain the dike fills at both ends of the weir.

3. Outlets - Design changes included the following:

(a) The length of the circular tunnel was extended to 254 meters.

(b) The unlined diameter was increased to 5.4 meters.

(c) A 0.15-meter thick slab was placed over tunnel muck remaining on the invert.

(d) For 10 meters below the upstream portal and for 60 meters above the downstream portal, the tunnel would be lined with reinforced concrete to make the inside diameter 4.60 meters.

(e) An inclined trashrack structure with guides extending above the maximum reservoir surface, was provided at the upstream portal.

(f) Emergency gates were located upstream from the dam axis near the extension of the cutoff grout curtain. Closure was to be made by a 3.4-meter wide by 4.6-meter high fixed wheelgate.

(g) Transitions from the rectangular gate opening to the circular tunnel section were made in a short section of concrete lining which extended upstream and downstream from the gate structure.

(h) Two (2) branch conduits at the downstream end of the tunnel led to an irrigation valve house where releases were regulated by two (2) 1.22-meter (48-in.) diameter valves. Two (2) emergency 1.22-meter (48-in.) gates were also provided.

## 4.02 DESIGN CONSIDERATIONS

### a. Main Dam and Saddle Dikes

1. Description of the Site - The damsite is located in an approximately 360-meter wide valley of the Arghandab River. The natural streambed occupies the left side of the valley. The right abutment slopes upward at an angle of about 40 degrees with the horizontal. The left abutment slopes at about 30 degrees. A number of low saddles exist in the range which forms the southwest rim of the reservoir.

2. Geology - Dr. F. A. Nickell, Consulting Geologist, visited the damsite in June 1950. His findings, based on site inspection and logs of drill holes prepared by Morrison-Knudsen Afghanistan, Inc., are presented in a report 21/, which concludes that the site is suitable for the proposed

21/ " Geology of the Arghandab Damsite" by F. A. Nickell, dated August 1950 (Appendix A)

type and size of dam. The report also recommends grout curtain treatment for various minor foundation defects.

Subsurface explorations at the damsite showed the presence of a coarse-grained granite formation intruding into metamorphic rock beyond the left abutment. The rock was friable and deeply weathered at the left abutment, and blocky and irregularly decomposed on the right abutment. Field investigations showed a gradual transition from completely weathered rock at the surface to sound rock at a considerable depth. No well-defined contact between the two (2) rock conditions was evident. The rock was found to be jointed in many directions, the dominant patterns, including shear zones, being in N-S, E-W, and NW-SE directions. The dips ranged from largely vertical to around 45 degrees. Basaltic dikes which were found to be present on both sides were also largely vertical and correlated with the NW-SE system of joints.

The presence was noted of a 1.22-meter (4-ft) to 6.1-meter (20-ft) wide curving shear zone behind the promontory which forms the left abutment. This zone, unless sealed, would permit seepage in a direction parallel to the river. At the geologist's recommendation the axis of the dam was moved about 30 meters upstream so that the impervious core of the dam would blanket the shear zone.

The valley floor was overlain by alluvium consisting generally of a layer of silt, about one (1) meter thick, overlying sand, gravel, and cobbles, and extending to bedrock at a maximum depth of about 12 meters.

The geologist also reported that the site is apparently not in a region which had been exposed to recent seismic disturbances. The nearest active area is at Quetta in Pakistan, 320 kilometers (200 miles) from the damsite. He recommended that structures should be designed to resist inertia

forces produced by a ground wave-acceleration equivalent to 0.05 gravity.

3. Foundation Treatment - Preliminary test pits were carried only to shallow depths limited by the ground-water table. The material removed from these pits was primarily silt and would have been unsuitable for the dam unless embankment slopes were flattened appreciably. The depth of foundation deposits was found, by diamond drill holes, to average 10 to 12 meters. Preliminary studies indicated that it would be more economical to strip the overburden from the entire foundation area than to flatten the embankment slopes which would increase the embankment volume and the length of the outlets.

Subsequent foundation investigations revealed that the silty material was a relatively shallow deposit overlying clean sand and gravel. Since sand and gravel are structurally adequate for a dam of the proposed cross-section, it was decided to leave this material in place, except under the impervious core. A trench under the entire foundation area was excavated through sand and gravel to sound rock. It was backfilled with rolled earth. Instructions for foundation excavation were incorporated in specifications issued to the Chief Engineer of Morrison-Knudsen Afghanistan, Inc., dated September 1950. 22/

Instructions for grouting the rock were prepared by James B. Hays, Consulting Engineer. 23/ He recommended grout curtain be placed along the centerline of the cutoff trench, through primary grout holes at about twenty (20) meter centers and approximately 20 meters in depth near the base of the dam.

Secondary holes, approximately midway between the primary holes, would extend ten (10) to twenty (20) meters below the surface as required.

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22/ "Design Criteria, Specifications, Laboratory Procedures" for the Embankment Foundation of Arghandab Dam by International Engineering Company, Inc., dated September 1950 (Appendix A)

23/ "Arghandab - Foundation Grouting", Letter by James B. Hays, dated July 12, 1950 (Appendix A)

Intermediate tertiary (between primary and secondary) and quaternary (between secondary and tertiary) holes were to be grouted if necessary, to depths of eight (8) to ten (10) meters and five (5) meters, respectively.

Grout holes were to be water-tested under a maximum of 150 percent of the ultimate head of water above the area to insure the effectiveness of the grout curtain.

4. Materials - Preliminary field investigations showed that materials suitable for construction of an earthfill dam, and in sufficient quantities, were available near the main damsite.

It was estimated that five (5) borrow areas would furnish ample materials for embankment construction. These areas are listed as follows:

(a) Borrow Areas Nos. 1 and 5, located upstream.

(b) Borrow Areas Nos. 2 and 3, located on the right bank.

(c) Borrow Area No. 4, located on the downstream right bank.

As the upstream borrow areas would be submerged during the second stage of construction, it was planned that Borrow Area No. 4 would furnish all materials for this stage of operations. Later explorations in Area No. 4 revealed that sufficient quantities of satisfactory materials were not available from that source. Therefore, additional Borrow Areas Nos. 6 and 7, and Borrow Area "D", about 1.5 and 2.5 kilometers respectively, on the downstream right bank were explored and found to contain suitable materials. Another Borrow Area "B" was found about 4 kilometers downstream from the main dam and would furnish material for saddle dike construction. Rockfill for Dike No. 6 was to be obtained from spillway excavation.

The following tests were performed in field laboratories by Morrison-Knudsen Afghanistan, Inc., on samples obtained from preliminary test pits in

the borrow areas:

(a) Mechanical analysis (including hydrometer analysis).

(b) Specific gravity.

(c) Compaction.

(d) Penetration resistance.

(e) Moisture content determination.

(f) Direct shear (box shear)

(g) Bearing ratio.

(h) Swelling.

The available materials were generally of two (2) types which were suitable together for a zoned embankment:

(a) An impervious silty clay.

(b) An pervious sand and gravel.

To prepare the final design of the embankment, detailed tests were required. Triaxial shear tests were made to determine the shearing resistance of the materials for stability analyses of the embankment slopes. Since facilities for triaxial testing were not available in Afghanistan when design investigations were under way (though later installed), three (3) samples of typical borrow area materials were shipped to a laboratory in the United States. Two (2) of these samples were impervious silty clay materials, and the third was a somewhat coarser material classified as semi-pervious. The following tests were performed on these samples:

(a) Mechanical analysis (including hydrometer analysis).

(b) Specific gravity.

(c) Atterburg limits.

(d) Solubility.

(e) Compaction.

(f) Consolidation.

(g) Permeability.

(h) Triaxial shear.

A description of the testing procedures and a summary of test results appear in a report by Abbot A. Hanks, Inc., 24/.

Consolidated quick triaxial shear tests were performed on saturated specimens of compacted materials. A discussion of the various types of triaxial testing procedures and their application appear in a special report by International Engineering Company, Inc., 25/.

Riprap for slope protection was selected from spillway excavation.

5. Embankment Design - A tentative cross-section for the main embankment was prepared on the basis of preliminary data from borrow area explorations. The section was ten (10) meters wide at the tope with conservative slopes of 3:1 upstream and 2.5:1 downstream. The section showed a central impervious core of compacted silty clay and outer shells of sand and gravel. Riprap protection was indicated.

Upon subsequent completion of laboratory testing high values were obtained for the shearing resistance of impervious materials in triaxial tests. Subsequent stability analyses indicated that slopes of 2.5:1 upstream and 2:1 downstream were adequate and the section was revised accordingly. An access road to the top of the dam was provided by construction of an inclined berm on the downstream face. Random zones were introduced on both sides of

24/ "Test of Soil Samples for Arghandab Dam, Afghanistan" by Abbot A. Hanks, Inc., dated August 21, 1950 (Appendix A)

25/ "Report on Earth Dams and Rockfill Dams" by International Engineering Company, Inc., dated May 1953 (Appendix A)

the core in which materials were to be distributed so as to form gradual transitions between core and shell materials. The random zones would serve as filters trapping fine particles which tend to migrate from the core into the shell.

At the recommendation of J. P. Growdon of the Board of Consultants, the design crest of the dam was raised two (2) meters to El 1,115 meters. The increase in height was obtained without appreciably increasing the total volume of the embankment, by steepening the slopes above El 1,105 meters to a maximum of 1.905:1 upstream and 1.524:1 downstream.

6. Stability Analysis - Studies of the stability of the embankment slopes were made by the sliding-block or wedge-method of analysis in which plane surfaces of failure are assumed. For purposes of analysis, the assumed shearing resistances of impervious and semi-pervious materials were the minimum values obtained from triaxial shear tests. A conservative figure of 34 degrees was assumed for the angle of internal friction of shell material based on large-scale tests on similar materials which have been performed in laboratories of United States Government agencies. Internal pore pressures in the impervious core were assumed as equal to 100 percent of the maximum reservoir head. Since internal pore pressures of this magnitude are extremely unlikely, computed factors of safety only slightly in excess of unity were considered ample.

Specifications and design criteria for Arghandab Dam, dated September 1950 26/, set limiting values for the shearing resistance and coefficient of permeability of material to be placed in the embankment. The

26/ "Design Criteria, Specifications, Laboratory Procedures" (for the Embankment Foundation of the Arghandab Dam) by International Engineering Company, Inc., dated September 1950 (Appendix A)

usual periodic control tests were also required.

b. Spillway

1. Preliminary Investigation - Collection of streamflow records at the Arghandab damsite began in November 1947. Since no prior records were available, only two (2) years of flow records were available when final design studies were started.

2. Design Flood - A design flood hydrograph was developed by analytical methods. Studies were made of flood records from snow-fed streams in other parts of the world having comparable watersheds. From these investigations it was concluded that the spillway should be capable of handling the outflow from a design flood with a peak inflow rate of 6,370 cubic meters per second (225,000 cfs) and a volume of about 355,000,000 cubic meters (287,000 acre-ft) over a 48-hour period.

3. Earlier Layout Considerations - A site for an open-cut spillway at a saddle in the southwest rim of the reservoir was selected for preliminary studies. The economic maximum storage level had previously been established by power and irrigation studies at El 1,110 meters. This storage level could be maintained either by a gated spillway with top of gates at El 1,110 meters or by an uncontrolled weir with its crest at the same elevation.

Estimated costs of the alternative schemes were compared. The presence of low saddles, which must be diked in any case, favored the gated scheme. After study of various gated arrangements, a spillway structure was proposed with five (5) radial gates, 12.19 meters (40-ft) wide by 9.75 meters (32-ft) high. Routing the design flood through the reservoir indicated that the peak outflow would be 4,390 cubic meters per second (155,000 cfs), with the maximum water surface at El 1,111.5 meters. It was also planned to

locate a fuse-plug dike at one of the saddles with its crest elevation below that of the main dam. Should the gates fail to operate or the design flood be exceeded, the fuse plug would be washed out. After the flood, the dike could be repaired.

Mr. Growdon approved the design flood, but recommended modifications of the spillway layout. He proposed that the top of the fuse plug be located 1.5 meters above the maximum nominal water surface (El 1,111.5 meters) and that the crest of the dam be raised two (2) meters to provide 3.5 meters of freeboard above the top of the fuse plug. With this arrangement, the design flood could be passed through open gates without washing out the fuse plug. Should the gates fail to operate, a very minor flood could be passed over the gates without damage to the fuse plug. In case of a major flood, sufficient freeboard is provided over the top of the fuse plug to insure its operation.

The fuse plug arrangement had some disadvantages. The plug could be damaged by a moderate flood if the gates were not opened. This might result in a sudden increase in discharge and create a downstream flood greater than that which would have occured under natural conditions. Field investigations had indicated questionable foundation conditions at all suitable and economical fuse plug locations. Thus, removal of the fuse plug might result in erosion of a channel to depths below El 1,110 meters, with resultant loss of storage and increased cost for reconstruction.

The recommendation that the dam be raised two (2) meters was adopted. This altered the comparative economics of gated and ungated spillways, eliminating the advantages of the former. A compromise arrangement was proposed, using a combined gated and uncontrolled overflow spillway at the original site. It was desired to retain the advantages of gates, which would

permit controlled releases to make space available for storage in advance of a flood occurence. The proposed scheme included two (2) radial gates, 7.3 meters (24-ft) high and 9.75 meters (32-ft) wide, with top of gates at El 1,110 meters, supplemented by an uncontrolled overflow crest at El 1,110 consisting of fifteen (15) 9.75-meter wide openings between bridge piers for an overall length of 146.25 meters. Routing the design flood through the reservoir resulted in a peak outflow rate of 3,398 cubic meters per second (120,000 cfs) corresponding to a maximum reservoir level at El 1,113.8 meters. It was assumed for this routing study that the gates would be opened as required to maintain the reservoir at El 1,110 meters.

4. Final Layout - Morrison-Knudsen Afghanistan, Inc., in Kandahar, investigated a rock-cut spillway as an alternate to the proposed combined-type spillway. Preliminary cost-comparisons indicated that appreciable savings could be effected by adopting the rock-cut scheme, which was borne out upon further review. The combined spillway was abandoned.

The proposed cut was located adjacent to the saddle at the former spillway site. It was in two (2) sections, each of which consisted of two (2) overflow weirs at El 1,110 meters and would discharge into converging channels. The two (2) sections carried flows into separate outlets. The east section, with crests 60 and 62 meters long, discharged into the original spillway ravine. The west section discharged into a ravine which intersects the river further downstream.

It was subsequently proposed that a rock cut between Dikes Nos. 3 and 4, west of the above location, be substituted for the east section in the previous arrangement. Flow over this spillway section would enter a ravine confluent with the outlet for the west section at the first site.

Thus, all flood flows would eventually re-enter the river at the same location. The proposed location was found to offer the following advantages:

(a) Reduction of rock excavation by 110,000 cubic meters.

(b) Elimination of flood discharges in the vicinity of the valve house and future powerhouse.

Savings resulting from the reduction in rock excavation were partially offset by the cost of retaining walls required at each end of the crest. Estimated overall costs at the second site were lower. Consequently, it was adopted and became a part of the final design. This is designated as Spillway No. 2. The remaining west section at the original location is referred to as Spillway No. 1.

Granitic rock in the spillway ridge cut area is deeply weathered. Although the cut for Spillway No. 1 is 15 to 25 meters deep, the rock is generally soft at the bottom. The cut for Spillway No. 2 is much shallower, and that rock is also generally weathered and seamy. To prevent erosion, concrete crest structures were therefore provided for both spillways.

At Spillway No. 1, the structure consisted of a low concrete overflow weir, 1.5 meters high at maximum section. A cutoff extended into rock a minimum of 0.6 meter. The weir was reinforced with temperature steel and secured to rock by two (2) lines of one (1) meter long steel dowels spaced at two (2) meter centers along both lines. The grouted dowels extended about 0.5 meter below the bottom of the cutoff. A downstream apron of hand-placed stone blocks was designed to have a minimum weight of 91 kilograms. The apron had a variable length of from three (3) to six (6) meters and terminated at a concrete cutoff wall extending a minimum of one (1) meter into soft rock and 0.6 meter into sound rock. When fresh hard rock was exposed, the stone

paving would be omitted. The entire foundation was to be grouted under low pressure through a line of holes downstream from the weir and spaced about five (5) to six (6) meters apart.

The structure at Spillway No. 2 consisted of a 0.6 meter high weir section with a concrete apron extending a minimum of seven (7) meters downstream from the weir. There was a 0.6 meter deep cutoff wall at the upstream end of the structure, and a one (1) meter deep wall at the downstream end. An underdrain of stone masonry was placed beneath the apron. The structure was reinforced with temperature steel and anchored to rock below the upstream cutoff. Transverse joints were located at 9.3 meter intervals along the crest. The entire foundation would be grouted at low pressure along a line of grout holes spaced at intervals of about five (5) meters. A hand-laced stone apron would continue below the concrete apron for a maximum distance of 15 meters.

The dike fills on both sides of the spillway were to be supported by stone masonry walls. Stone masonry was selected on the basis of economy since available labor is especially skilled at this type of construction.

c. Hydraulic Model Testing - Scale models of Spillways Nos. 1 and 2 were constructed in the field laboratories of Morrison-Knudsen Afghanistan, Inc. Both quantitative and qualitative tests were performed to determine hydraulic characteristics of the crest structures and outlet channels. For Spillway No. 1, the discharges were compared for a curved and a straight spillway alignment. Flows were measured in this test by a sharp-edge, V-notch weir. From the results obtained, the theoretical length of straight crest having the same discharge capacity as the proposed curved crest was

established. From tests of the flow pattern for both spillways, the configuration of the crest and channel was modified to produce a streamlined flow. Design changes which resulted from the tests decreased rock excavation by about 122,000 cubic meters, but also increased the volume of rock and concrete masonry by about 1,400 cubic meters.

d. Outlet Works and Diversion

1. General - Outlet works for an earthfill dam may be located either in a tunnel through an abutment or in a conduit through the embankment. For this project, the tunnel or conduit, in addition to serving permanently for irrigation and power releases, was required for stream diversion during construction.

2. Tunnel vs. Conduit - Preliminary studies were made of various tunnel and conduit arrangements. Only limited information was then available on the character of abutment rock which would affect tunneling costs and lining requirements. Data on foundation conditions along potential conduit alignments were limited to logs of a few drill holes. From these earlier studies, it appeared that a conduit scheme would be more economical. Conduits with 3.96, 4.57, and 5.18 meter (13, 15, and 17-ft) diameter circular sections were investigated from the standpoints of permanent use for outlet works and power production, as well as temporary use for stream diversion during construction. A 4.57 meter (15-ft) diameter liner at the base of the right abutment was selected as best adapted for all purposes.

Additional topographic and drill-hole data were subsequently obtained from the site, and a more detailed study was made. The following schemes were investigated:

(a) A conduit at the base of the right abutment as originally proposed.

(b) A conduit at the base of the left abutment.

(c) A tunnel through the right abutment.

The first scheme posed a number of problems. Before discussing these problems, it is desirable to briefly describe the proposed diversion program.

A two (2) year program was planned for Arghandab Dam. This considered diversion in two (2) stages. In the first stage, the tunnel or conduit would be completed concurrently with a portion of the right embankment. The original stream channel on the left side would remain open during this stage. During the second stage, the river would be diverted through the tunnel or conduit during completion of the embankment.

If a conduit were located at the right abutment, as in Scheme (a), placing of the fill on the right bank could not have been commenced until the concrete conduit was completed. This might have delayed the construction program for a year. The time required for delivery of steel at the jobsite was uncertain.

Unwatering at a left abutment location, as required by Scheme (b), for construction of a conduit would be complicated by the close proximity of the river channel. Because of the small working area, it would be necessary to excavate well into the abutment to allow sufficient space for construction of cofferdams. Consequently, this scheme required a considerably larger volume of rock excavation than at the right abutment location. The powerhouse site for this location did not appear suitable. The left bank

location had the advantage of eliminating long-approach and tailrace channels to the tunnel portals.

In Scheme (c), construction of a tunnel through the right abutment would not interfere with fill placing on the right bank. This would serve to cut down construction delays. An economical powerhouse site was available at the right abutment. The right abutment tunnel scheme was also lower in estimated cost than either of the other schemes. The preliminary conduit scheme was therefore abandoned and the above tunnel scheme adopted.

Further investigations were then made to establish the economic tunnel size. The following factors were considered:

(a) Excavation and lining costs.

(b) Height of cofferdam to provide necessary diversion capacity.

(c) Surge tank requirements.

Lined tunnel diameters of 3.96, 4.57, and 5.18 meters (13, 15, and 17-ft) were compared. The corresponding unlined diameters were 4.57, 5.18, and 5.79 meters (15, 17, and 19-ft).

For the three (3) tunnel sizes, two (2) general arrangements of lined and unlined tunnel sections were considered.

Case I - A 50 meter concrete-lined section at the upstream end; a 50 meter steel-lined section with concrete backfill at the downstream end, with remainder of tunnel unlined.

Case II - The entire tunnel concrete-lined, except for a 50 meter section of steel lining with concrete backfill at the downstream end.

The estimated savings resulting from reduction in cofferdam height due to increased capacity of the lined tunnel section did not compensate for the cost of lining the entire tunnel. Case II was, therefore,

eliminated for cost reasons. A 4.57 meter lined (5.18 meter unlined) diameter proved to be the economic tunnel size, and the arrangement as described in Case I was consequently adopted.

3. Intake and Control - The intake and control arrangement was required for two (2) principal functions:

(a) To permit closure of the tunnel at some location upstream from the axis of the embankment for unwatering and maintenance.

(b) To provide space for trashracks for removing debris.

It was required that the intake gate be capable of being closed in emergency situations under full unbalanced head conditions, and be operable from a level above the maximum reservoir surface. To prevent excessive head losses at the trashracks, it was necessary either to provide for removal of the racks for maintenance, or to install mechanical raking facilities.

Two (2) alternative general arrangements were studied as follows:

(a) An intake tower at the upstream tunnel portal incorporating the intake gate and trashracks.

(b) An intake gate shaft and tower immediately upstream from the axis of the embankment and a separate, inclined trashrack structure at the upstream portal.

A tower at the portal would have to be designed to resist forces due to earthquakes (see Paragraph 5.03, b). The resulting moments would be quite large for a tower 40 to 50 meters in height. The principal advantage of this scheme was that it would permit emergency unwatering of the entire tunnel, although the need for this would be very unlikely. As this scheme was found to be considerably more costly than the alternate shaft and tower,

the latter was therefore selected.

Two (2) independent structures for intake and control were provided in the adopted arrangement. The inclined trashrack at the intake was supported on the rock slope at an angle of about 45 degrees. The portal opening, which was designed for a maximum velocity of less than 0.92 meters per second (3 fps) under maximum flow conditions, was divided by a center pier into two (2) openings, 2.6 meters wide by 12.37 meters high. Guides for the rack sections extended one (1) meter above the normal high-water level to El 1,111 meters. The concrete frame at the portal was designed to withstand the full hydrostatic head of the reservoir, with water surface at the minimum operating level of El 1,088 meters. This would permit the frame to support a bulkhead at the portal, to permit unwatering of the tunnel above the control gate.

The trashrack bars were designed for stresses under an externally applied differential head of 6.1 meters (20-ft). To facilitate shipping and handling, the bars for each opening were designed in two (2) units, 6.15 meters (20-ft) high, which would be bolted together before being lowered. The rack sections were to be raised and lowered by a 10-ton (US) capacity chain lifting device. No permanent hoist was installed. It was intended that the lifting device would be operated by a truck.

The intake-gate structure would consist of a concrete-lined shaft through rock above the tunnel, daylighting at El 1,098 meters. Above this level, the structure was extended through the reservoir as a closed, reinforced-concrete tower. The structure was located so that the above-ground portion would be outside the embankment. This avoided the need for designing the tower for external soil pressure.

The gate opening was designed with a width of 3.4 meters and a height of 4.6 meters. The transition between circular and rectangular gate opening sections in the tunnel was to be constructed with reinforced concrete. The reinforced section would extend seven (7) meters from either side of the opening. Grout curtains were to be constructed around the upstream transition and tied into the main dam cutoff.

A 0.508 meter diameter vent was provided immediately below the gate opening to admit air when the downstream portion of the tunnel was unwatered under ordinary conditions, or when the gate was closed under emergency conditions with the tunnel discharging. The intake to the vent was set above the maximum water surface. Air was carried through the lower section in a steel pipe, and through the shaft in a formed opening.

The minimum excavated diameter of the shaft was 5.2 meters. It was to be lined with concrete having a minimum thickness of 0.30 meter. Temperature reinforcement was provided.

The tower was designed to withstand the following independent loads:

(a) Wind and wave action.

(b) Earthquake.

(c) External hydrostatic pressures.

The tower walls were 0.40 meters thick below El 1,111 meters and 0.30 meters thick above that level. A gate maintenance deck was located at El 1,111 meters and a hoist deck at El 1,119 meters.

The selected gate was of the fixed-wheel type with a total weight of 17,690 kilograms (39,000 lb). The hoist had a rated capacity of 64,397 kilograms (71 US tons) with a lifting speed of about 0.61 meters per

second (2 fps) and a normal lift height of 47.85 meters (157 ft). It was to be operable by a 15.2 metric hp 15-hp electric motor equipped with an air-fan brake, to permit emergency lowering of the gate without power at a maximum speed of 1.83 meters per minute (6 fpm). The hoist was operable from a control panel on the tower operating deck.

A structural steel foot-bridge with a 16.307 meter (53 ft-6 in.) span would provide access to the towern. The main structural members were designed as two (2) side-flange beams, 0.45 meters (18 in.) deep. The 1.22 meter (3 ft-11 in.) wide deck was to be made of sectionalized steel grating.

4. Outlet Structure - The reservoir outlet below the tunnel portal was designed as a cut-and-cover conduit. The 4.6 meter diameter steel tunnel liner extended 12.55 meters below the portal and was surrounded by heavily reinforced concrete encasement. Two (2) 1.524 meter diameter conduits branched from the main conduit to the outlet valve house. Immediately below these branches, the conduit was to be closed temporarily by a steel bulkhead. When the future power plant is installed, the bulkhead will be removed and the conduit extended to the turbine manifold.

Operation studies of the Arghandab Reservoir established the acreage which could be irrigated and the required capacity of the outlets. The monthly irrigation demands used in these studies were based on reconnaissance investigations of soil characteristics in the Arghandab Valley, studies of the types of crops which might be raised, and total water requirements. On the basis of these preliminary studies, it was estimated that storage behind Arghandab Dam would permit irrigation of 485 square kilometers (120,000 acres) of arable land in a normal year. Later, more detailed studies of soils and crops in relation to probable irrigation water requirements indicated that

the construction of the Arghandab Dam would permit irrigation of 749 square kilometers (185,000 acres) of arable land in a normal year 27/. It was found that two (2) 1.22 meter (48-in.) fixed-dispersion cone (Howell-Bunger) valves could satisfy the irrigation demand. At least two (2) valves were considered necessary so that shut-down of a single valve for maintenance would not cut off all irrigation releases. Either valve could meet the total irrigation requirement about two-thirds of the time and satisfy approximately two-thirds of the total requirement during the period of maximum demand. Each valve was to be operable by a separate 7.6 metric hp (7-1/2 hp) electric motor.

After the future power plant has been installed, a large portion of the irrigation demand could be satisfied by turbine discharges.

A 1.22 meter (48-in.) ring-follower emergency gate was installed upstream from each regulating valve. These emergency gates could be operated under full unbalanced head conditions and allow either regulating valve to be maintained independently.

The gates were furnished with integral hydraulic hoists which were both operable by a single hydraulic power unit consisting of a rotary-type pump, an electric motor, a pressure-relief valve, and a tank.

The operating mechanisms for the control gates and regulating valves were to be installed in a reinforced concrete valve house located to the left of the future powerhouse site. The mechanisms were operable from a single electrical control panel in the valve house. An overhead chain hoist was provided for servicing the installation. A doorway in the left wall of the valve house would provide access to the future powerhouse.

To confine the jet and protect the future powerhouse from the spray

27/ "Arghandab-Tarnak Area - A Study of Water Allocations and Use for Maximum Irrigation and Power Benefits" by Morrison-Knudsen Afghanistan, Inc., Land Development Department, dated July 1956 (Dr. Fly).

action of the Howell-Bunger valves, a rectangular reinforced concrete hood was to be constructed around the valves and extended 1.55 meters downstream. The hood was lined with 0.8 centimeter (5/16-in.) steel plate for a length of 5.89 meters, beginning a short distance downstream from the valves. This plate would receive the impact of the emerging jet. A 0.6 meter high concrete sill, placed around the interior periphery of the hood at the end of the steel liner, would serve both as an energy dissipator and for liner anchorage.

5. Station Power Supply - Power to operate the regulating and control valves would be supplied by a small hydroelectric unit installed in the valve house. The turbine was a horizontal Francis unit rated at 90 hp, designed for a net effective head of 32 meters (105-ft). A 0.46 meter (18-in.) diameter steel penstock would branch from the main conduit downstream from the tunnel. This penstock would be embedded in concrete. The turbine was directly connected to a 75-kva, 3-phase, 50-cycle generator. The unit would also supply power for the valve house illumination.

A Diesel generator set would provide power for operation of the intake gate hoist, and for emergency operation of control gates and regulating valves if the hydro-unit was shut down. The set was housed in a rubble masonry structure on the right abutment.

6. Developments Subsequent to the Completion of First-Stage Design

Since the first-stage design was completed, additional data concerning the agricultural potentialities of the Arghandab Valley was made available. Further studies were made of the types of crops that might be raised, and their water requirements. The estimated maximum irrigation demand can be met by the combined discharge through two (2) 1.22 meter (48-in.) regulating valves presently in service and, later, also through discharges from the

future power plant.

The future power plant will probably have a peak capacity of around 6,400 kw, and could supply energy to the City of Kandahar and vicinity.

e. Service Behavior - Since the completion of construction and the subsequent operation of Arghandab Dam, the following observations have been made:

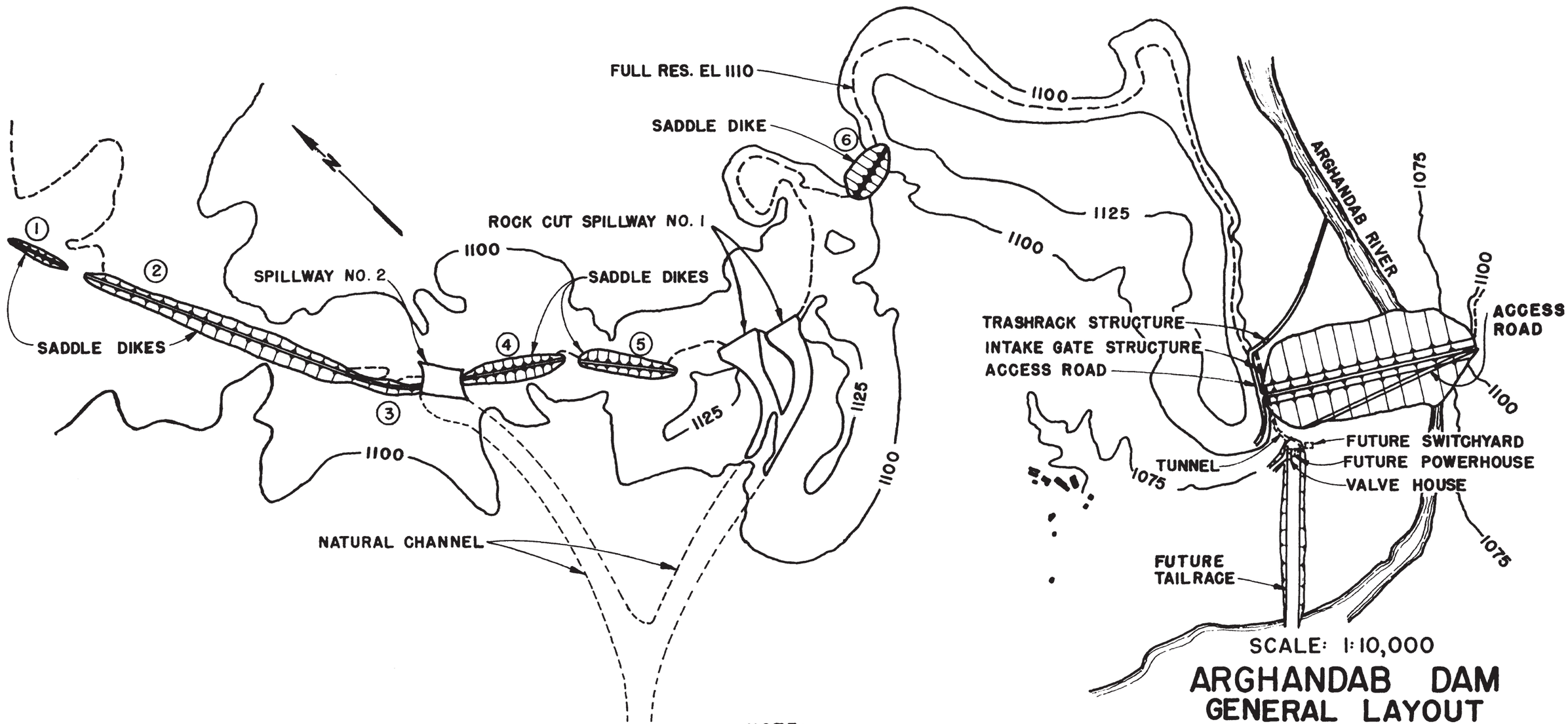
1. Dam - Performance has been generally satisfactory. As the reservoir was filled, minor seepage occurred at the downstream toe. The effluent water was observed to be clear. This indicated that no piping was taking place. It was decided to install a collector drain designed as an inverted filter to collect seepage. With the reservoir at El 1,110 meters the observed quantity of seepage was 0.0085 cubic meters per second (0.3 cfs).

2. Spillway - First discharge over the Arghandab spillway occurred on May 26, 1954. Discharge reached a peak of 269 cubic meters per second (9,500 cfs) on March 30, with the reservoir at El 1,110.66 meters. Erosion occurred in the disintegrated granite to depths from 1.83 to 2.44 meters (6 to 8 ft) in the lower reaches of the channel and to depths up to 3.66 meters (12 ft) in the upper reaches. Some erosion was anticipated, and its effect was given consideration during the design. Rock riprap below the spillway weir was relatively undisturbed. In a few spots ravelling took place.

3. Outlet Works - The outlet regulating valves have been in constant service since February 1952. They have been operated under varying conditions of head and openings. In March and April of 1954, the valves were discharging continously in a full-open position, with reservoir water service above the spillway-crest level. The maximum discharge under these conditions was 52.6 cubic meters per second (1,860 cfs). Performance of the valves has

been satisfactory in all respects.

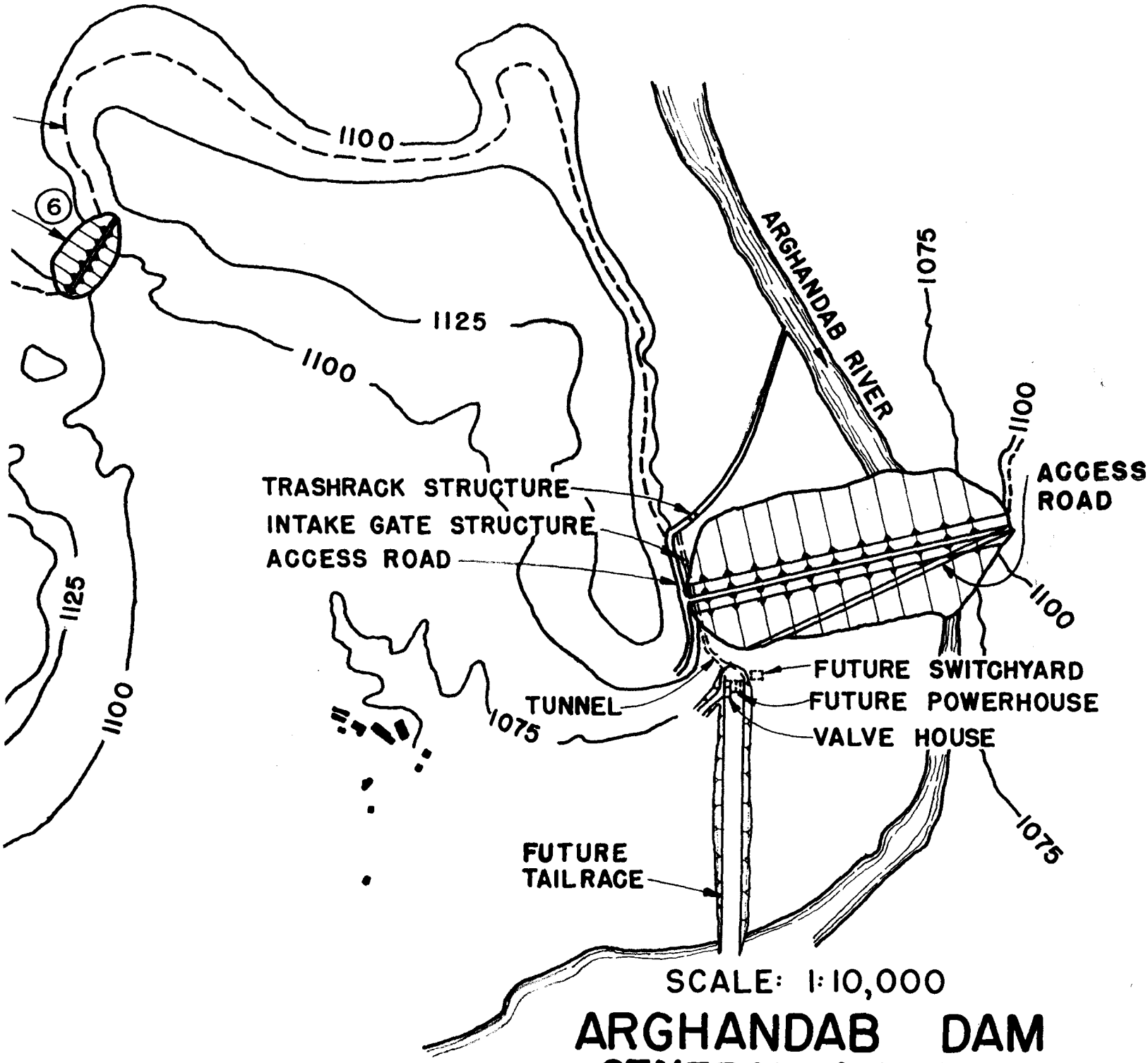
The fixed-wheel intake gate and ring-follower gates have been tested periodically after installation and have functioned as expected.




NOTE:
ALL ELEVATIONS SHOWN ARE IN METERS.

ARGHANDAB DAM
GENERAL LAYOUT
SAN FRANCISCO, CALIFORNIA
AUGUST 29, 1956

**Figure 2.5.5.3-1**



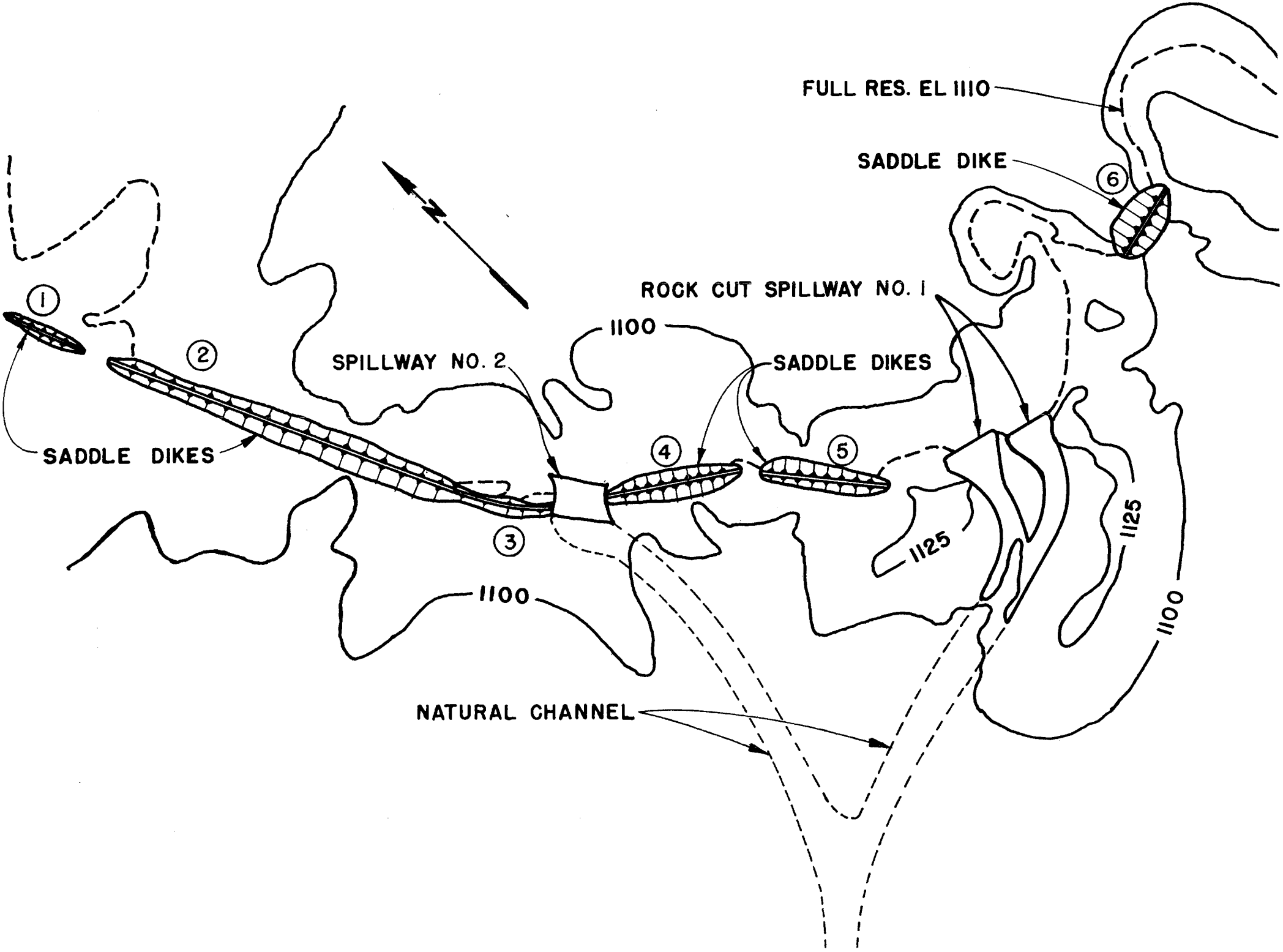

SCALE: 1:10,000

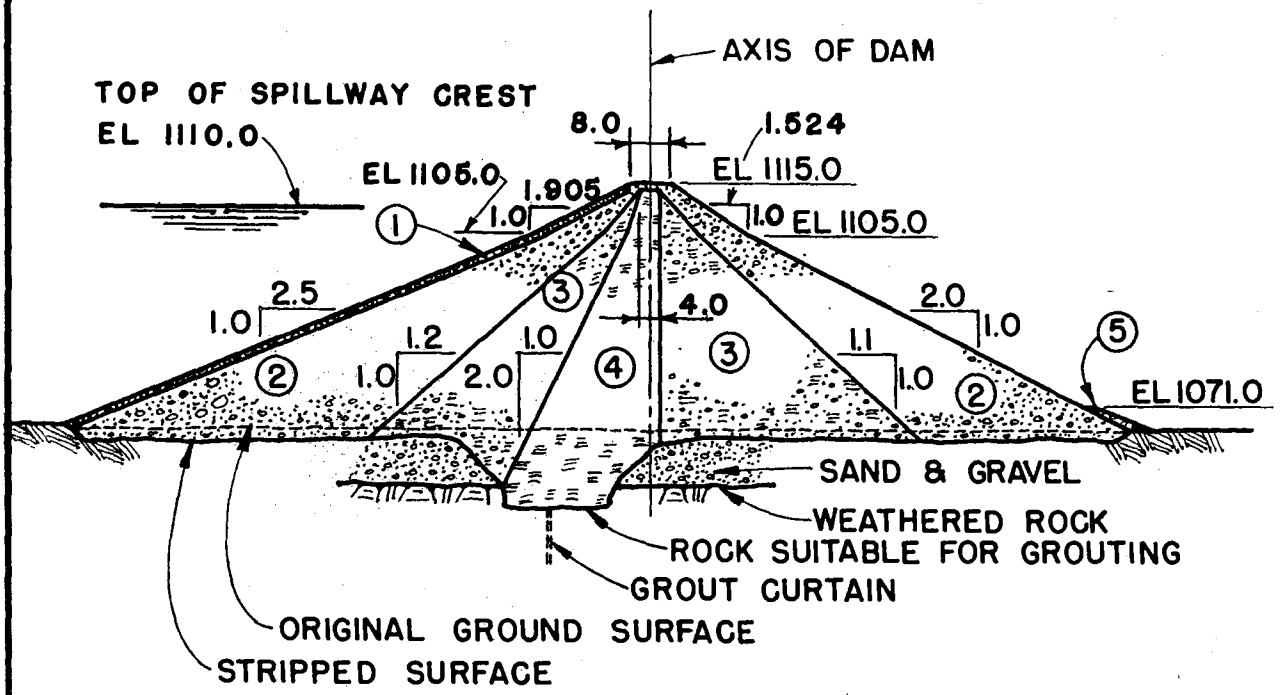
# ARGHANDAB DAM GENERAL LAYOUT

SAN FRANCISCO, CALIFORNIA

AUGUST 29, 1956

NOTE:
ALL ELEVATIONS SHOWN ARE IN METERS.


FULL RES. EL 1110
SADDLE DIKE
6
ROCK CUT SPILLWAY NO. 1
1100
SADDLE DIKES
SPILLWAY NO. 2
1
2
SADDLE DIKES
4
5
3
1125
1125
1100
1100
NATURAL CHANNEL




1. 1.0 METER DUMPED RIPRAP
2. COMPACTED PERVIOUS SAND AND GRAVEL FILL
3. RANDOM ROLLED FILL
4. IMPERVIOUS ROLLED FILL
5. DUMPED ROCK

NOTE:
ALL DIMENSIONS AND ELEVATIONS SHOWN ARE IN METERS

SCALE: 1:1600

# ARGHANDAB DAM
## MAIN SECTION

SAN FRANCISCO, CALIFORNIA
AUGUST 29, 1956

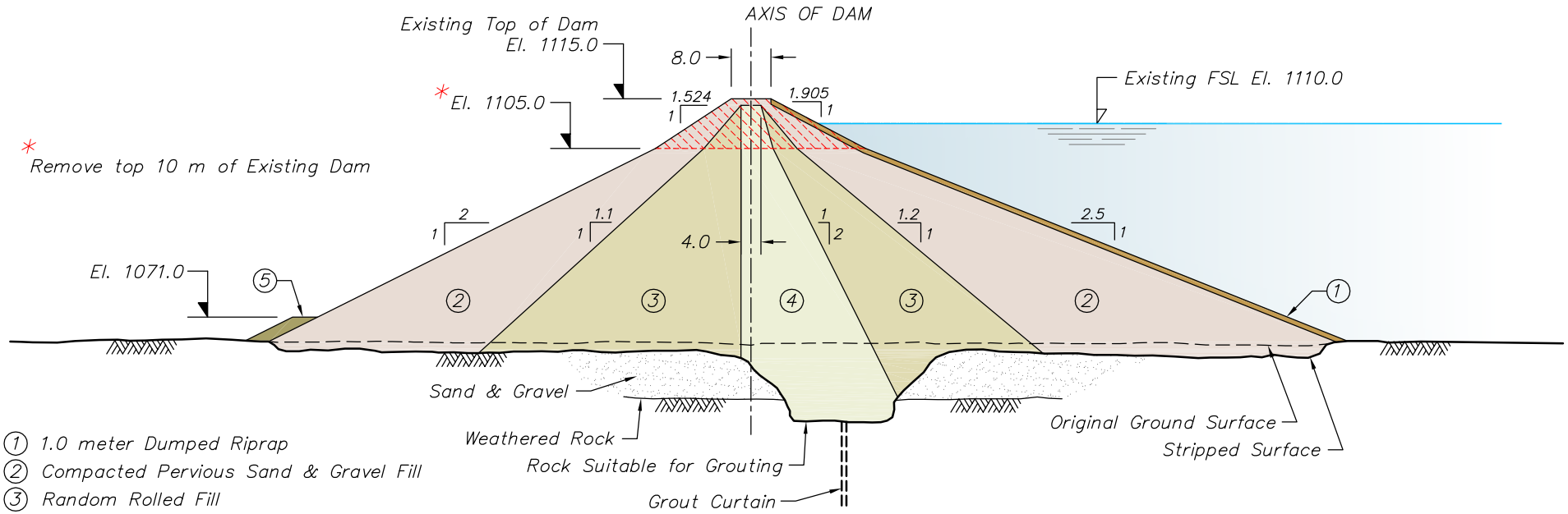
# ARGHANDAB IRRIGATION REHABILITATION PROJECT

**CIDA AFGHANISTAN TASK FORCE**

**APRIL, 2007**

**TECHNICAL APPRAISAL MISSION**

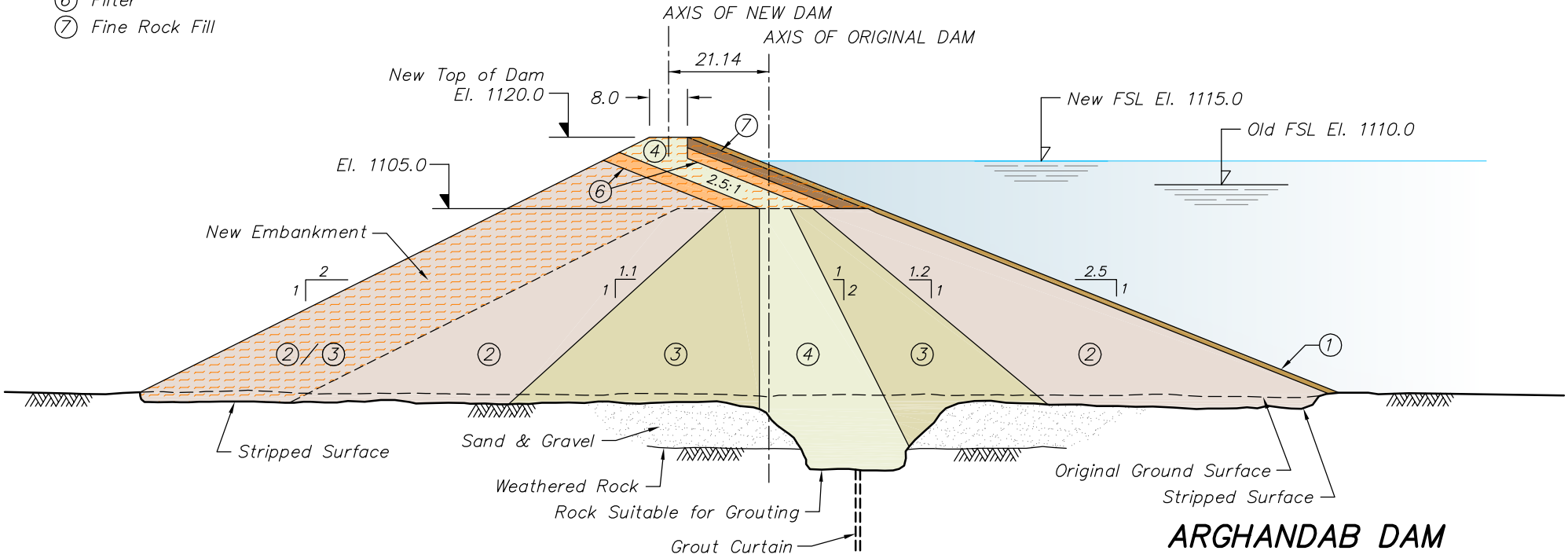
**MISSION REPORT**




1. 1.0 meter Dumped Riprap
2. Compacted Pervious Sand & Gravel Fill
3. Random Rolled Fill
4. Impervious Rolled Fill
5. Dumped Rock ( Not Continous )
6. Filter
7. Fine Rock Fill

**EXISTING EMBANKMENT SECTION**

N.T.S.




NOTE:
All dimensions and elevations are in metres

**PROPOSED EMBANKMENT SECTION**

N.T.S.

September, 2007

# ARGHANDAB DAM

Existing & Raised Dam Cross Sections
5 m Dam Raise
Figure 3.7.5.2.1–1

# April, 2007

## PHOTO GALLERY

Dahla Dam - Upper Arghandab Reservoir

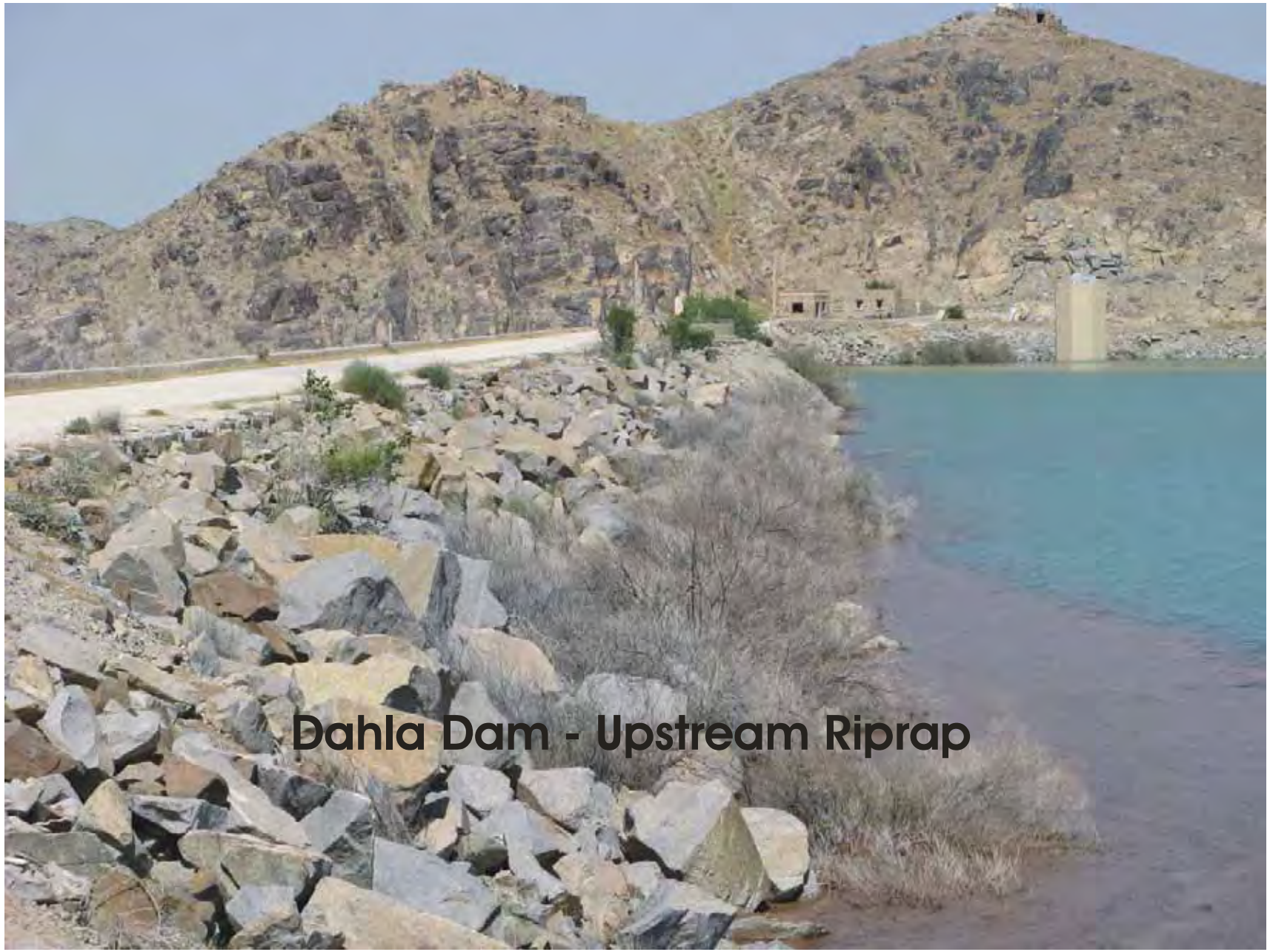
Dahla Dam - Upstream

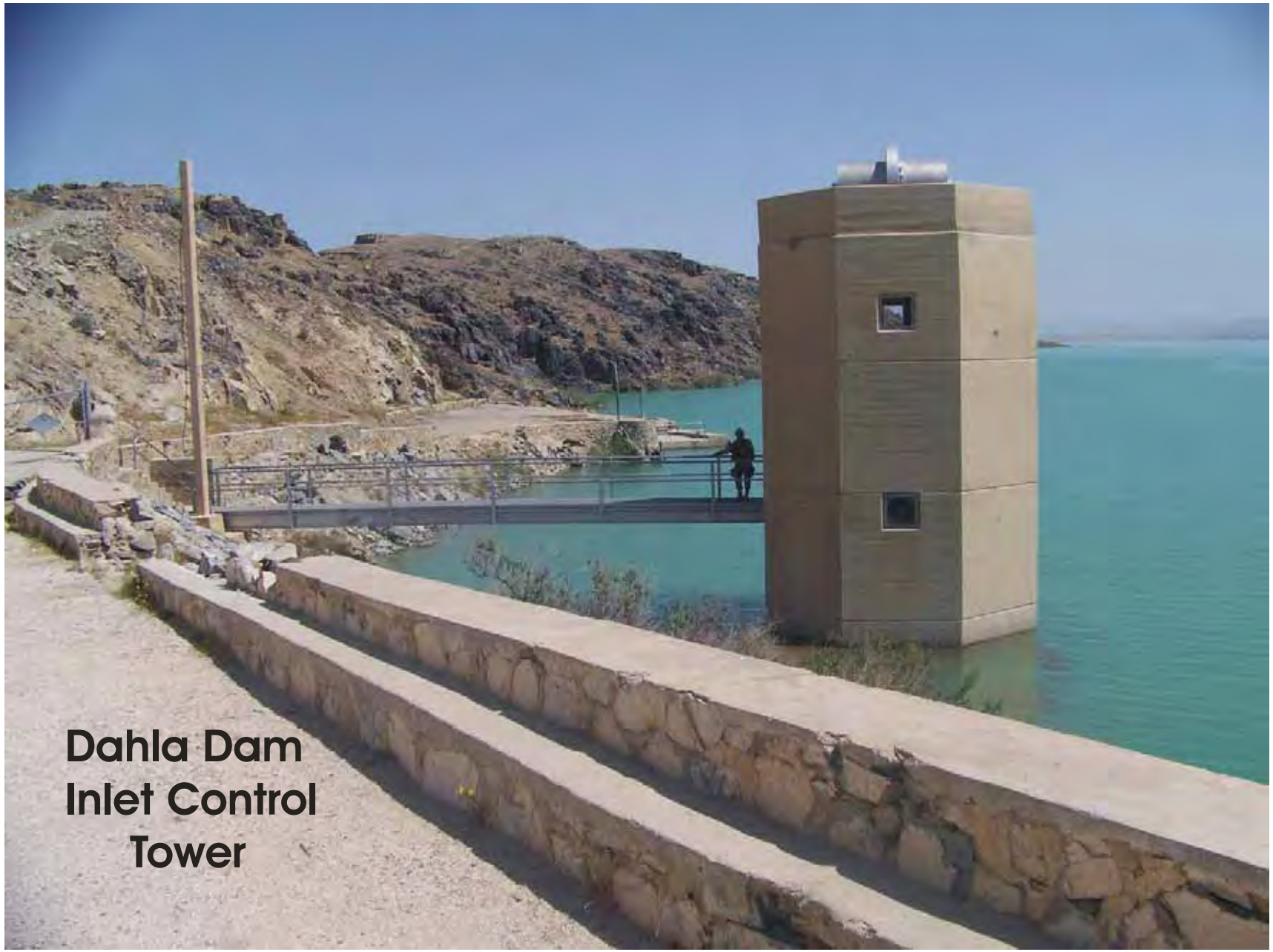
Dahla Dam - Centerline

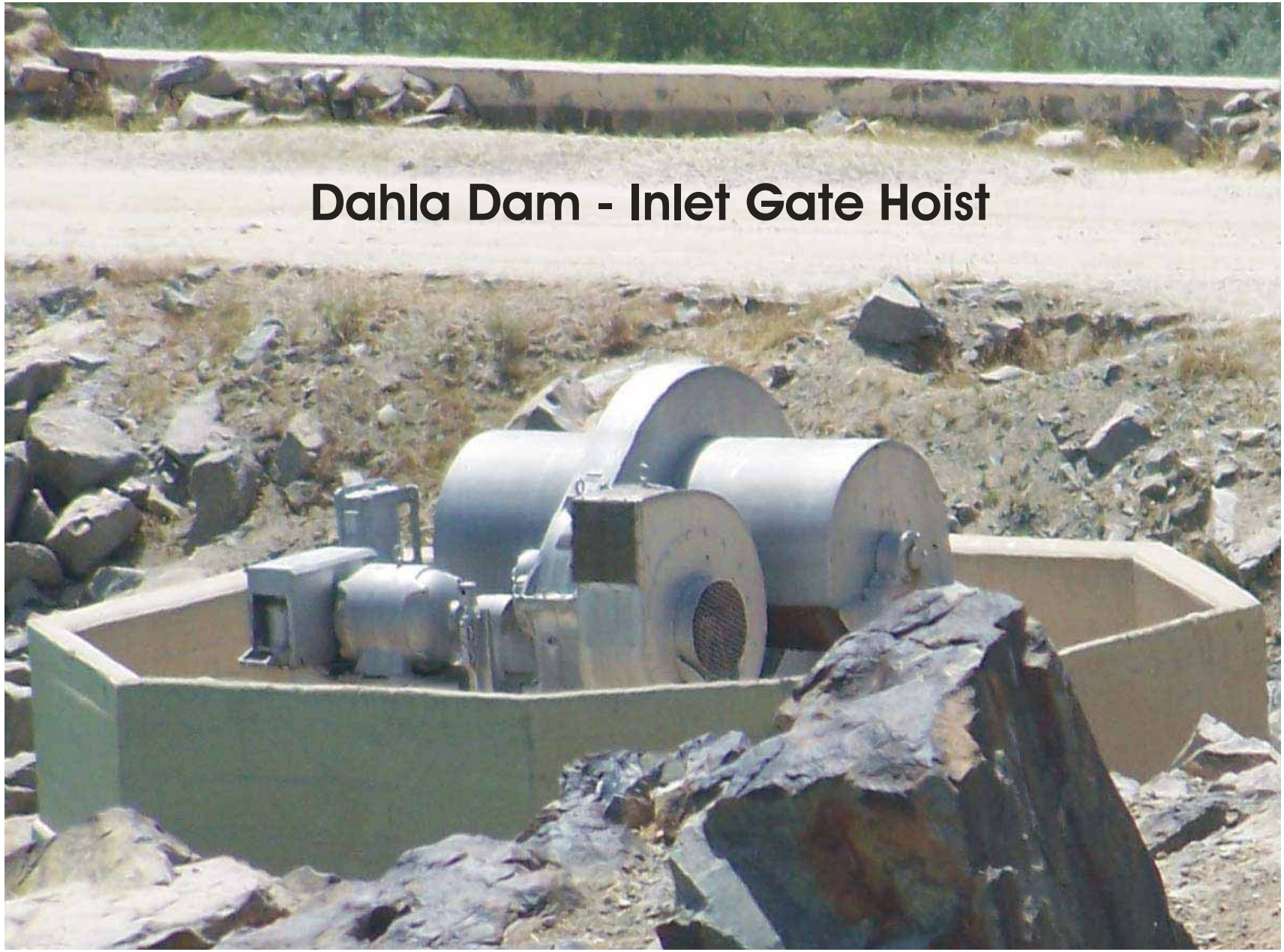
Dahla Dam - Downstream

Dahla Dam
Outlet

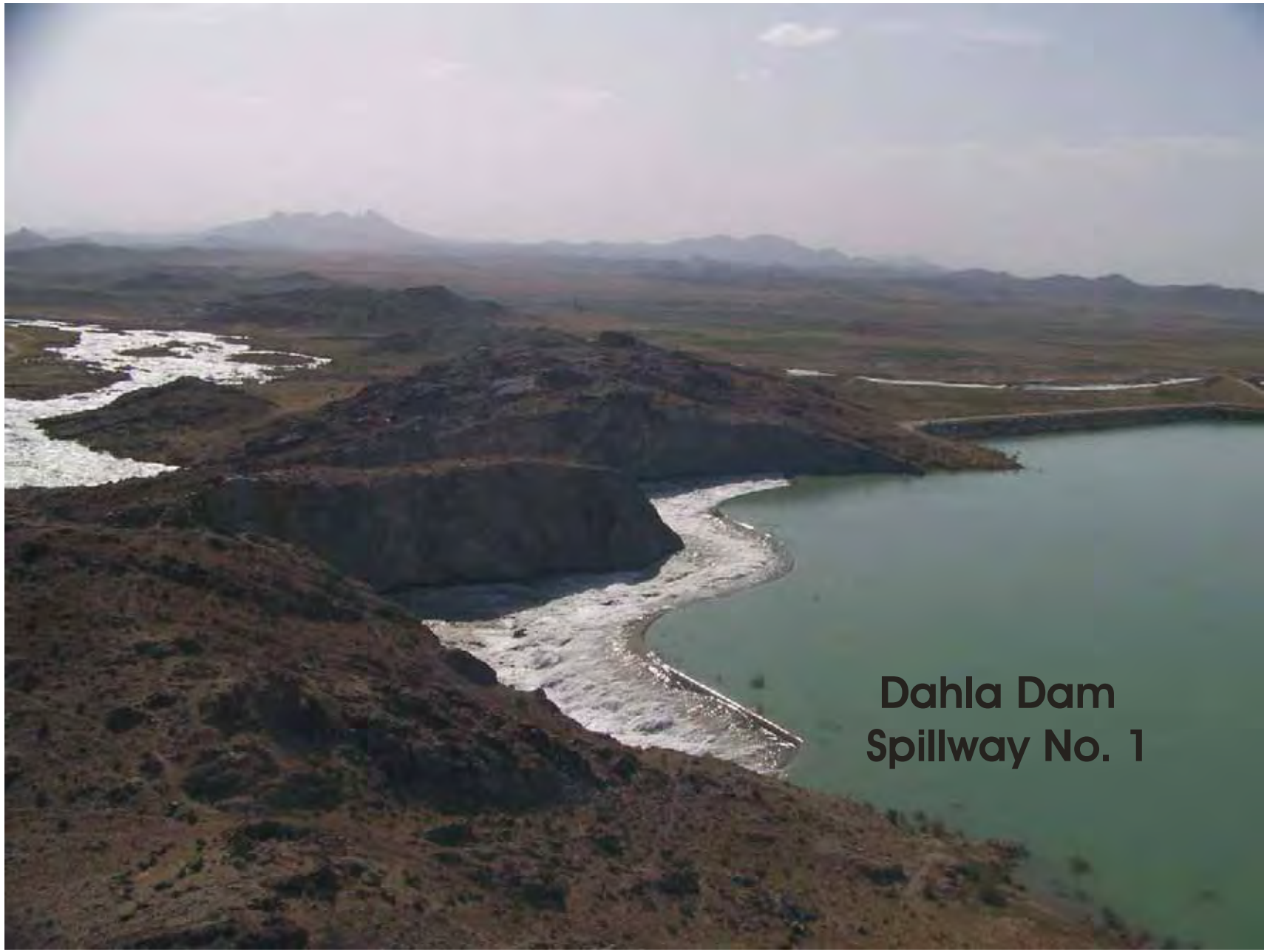
Dahla Dam - Downstream Slope


Dahla Dam - Upstream Riprap

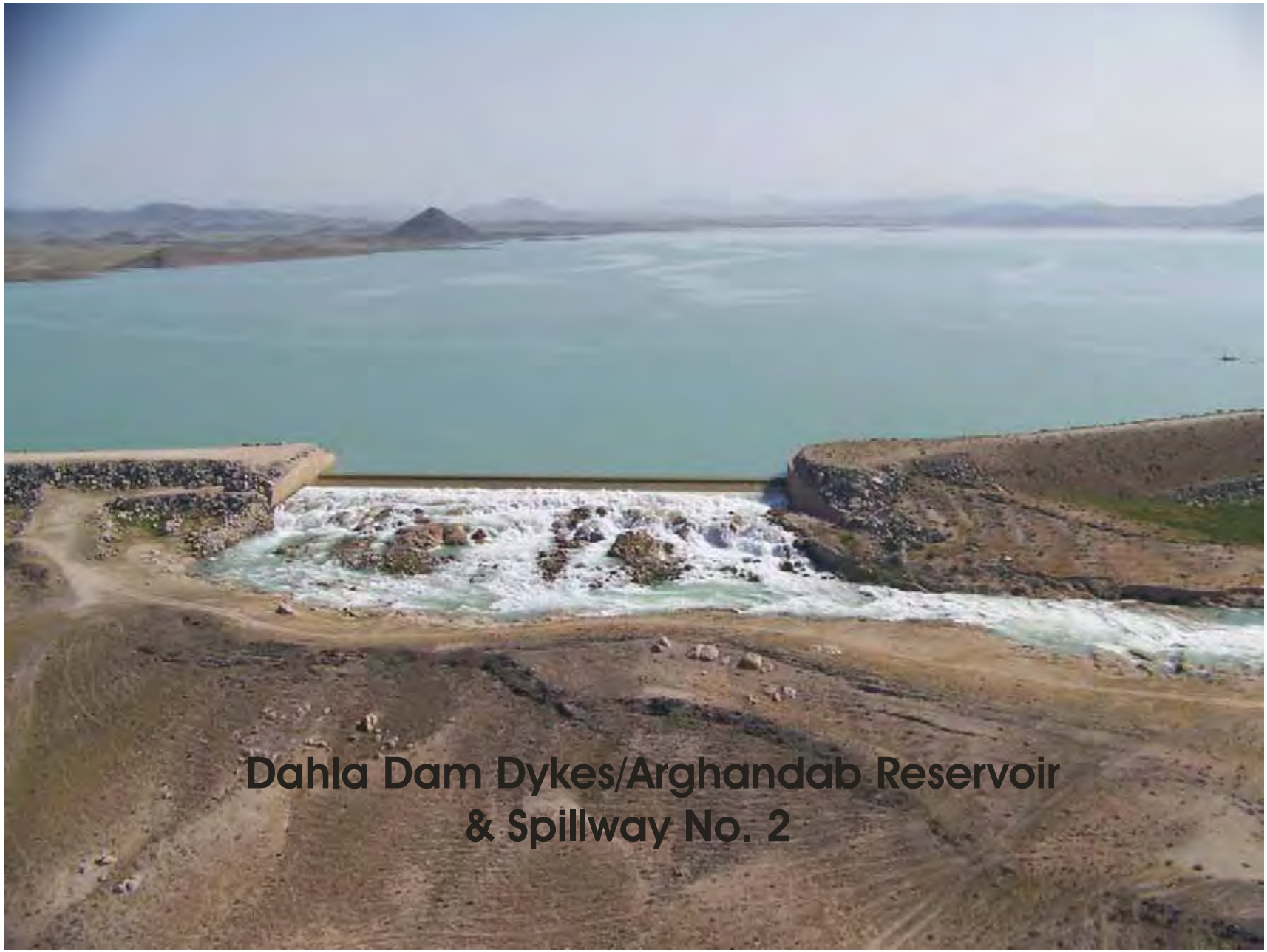

Dahla Dam
Inlet Control
Tower

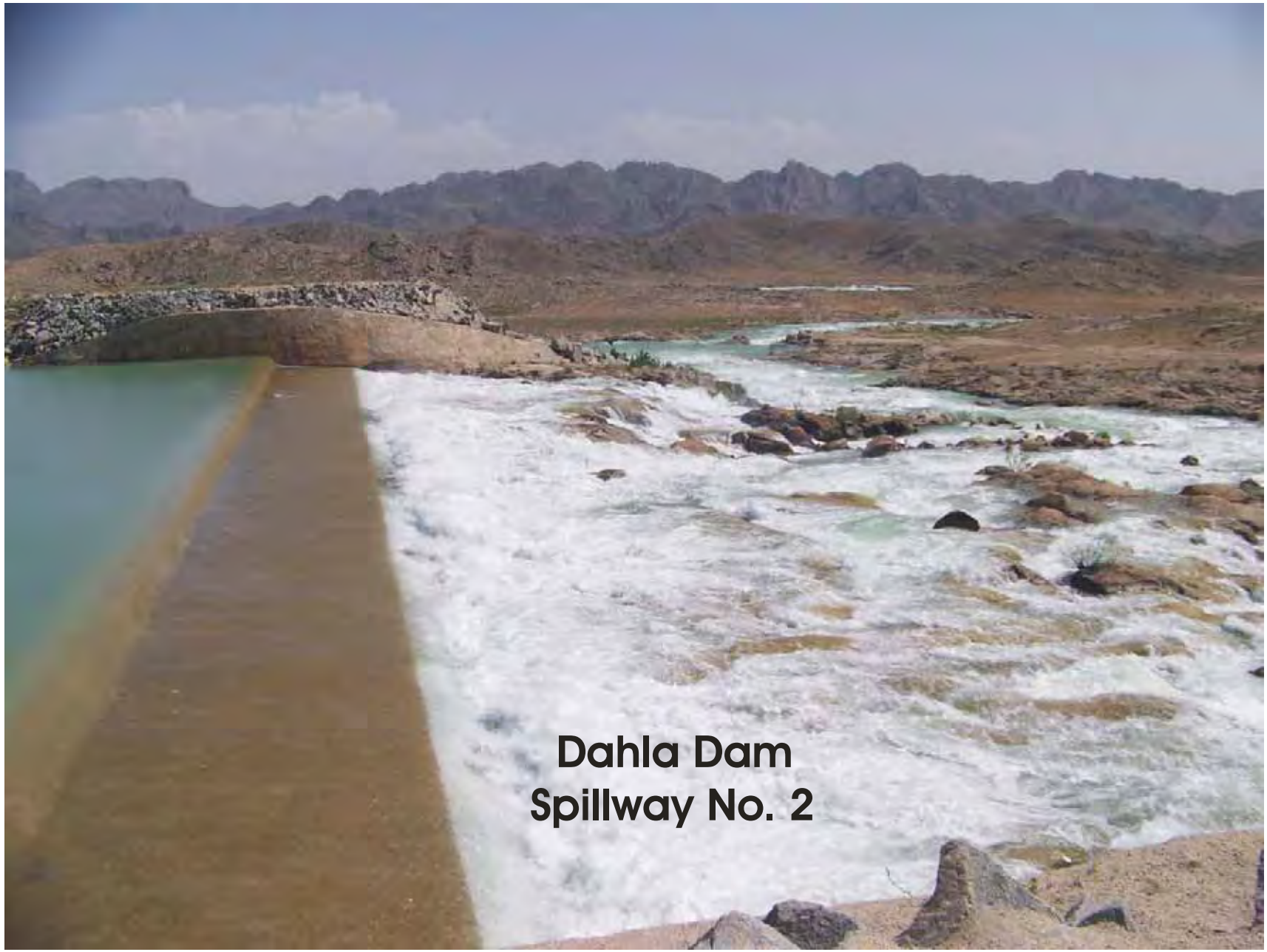

Dahla Dam - Inlet Gate Hoist

Dahla Dam Dykes & Spillways

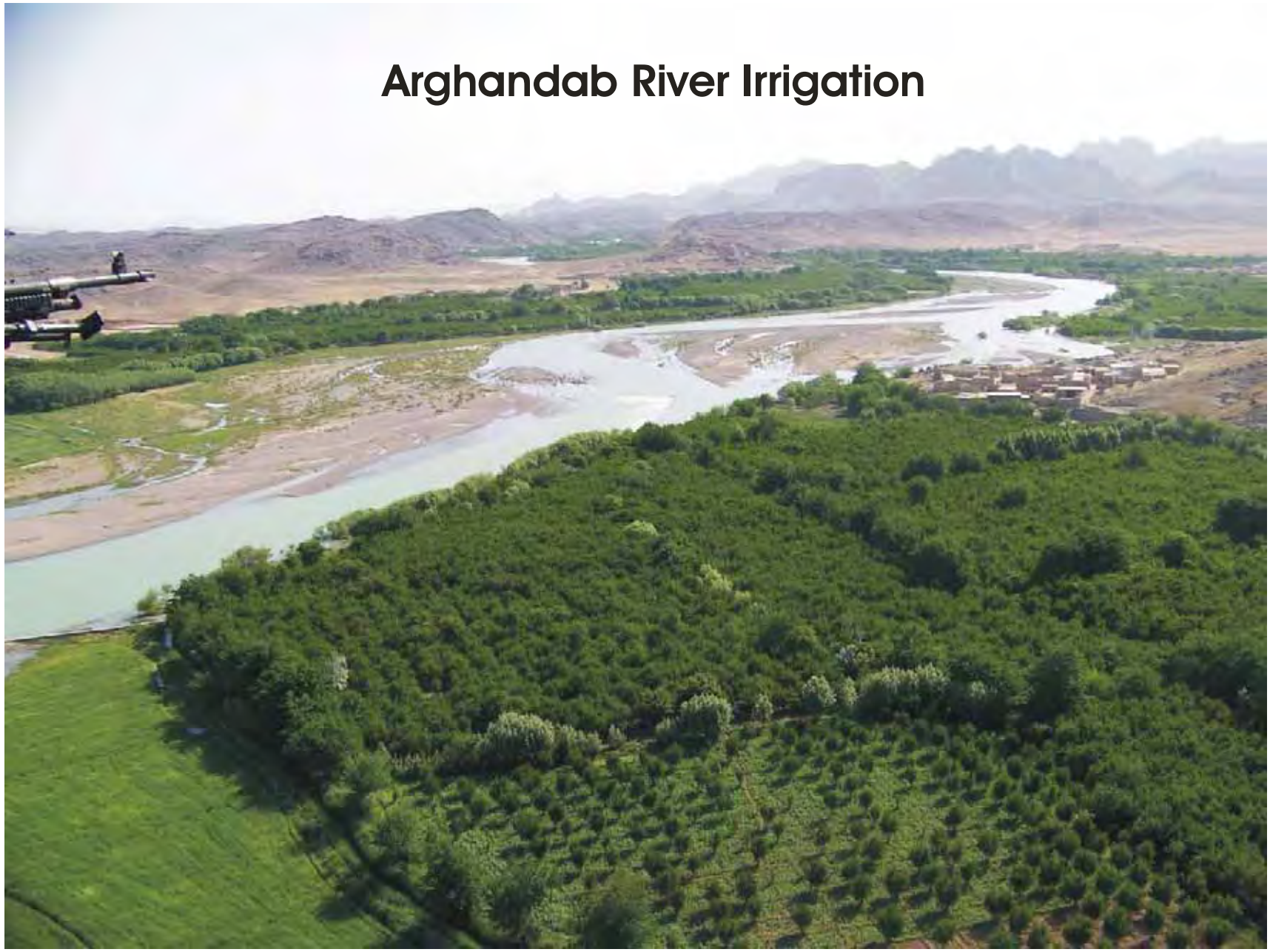

Dahla Dam
Spillway No. 1

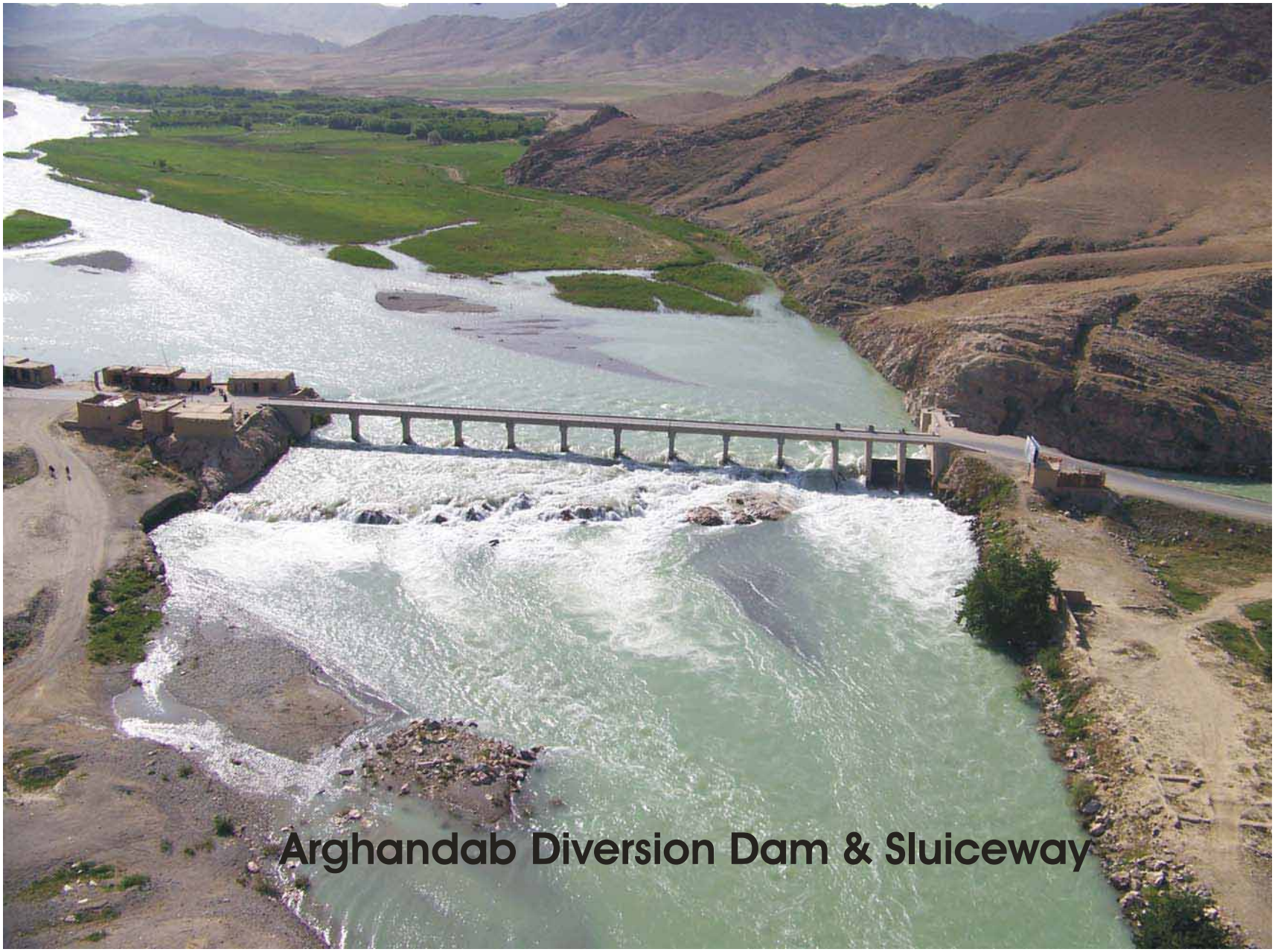
Dahla Dam - Spillway No. 1 Erosion

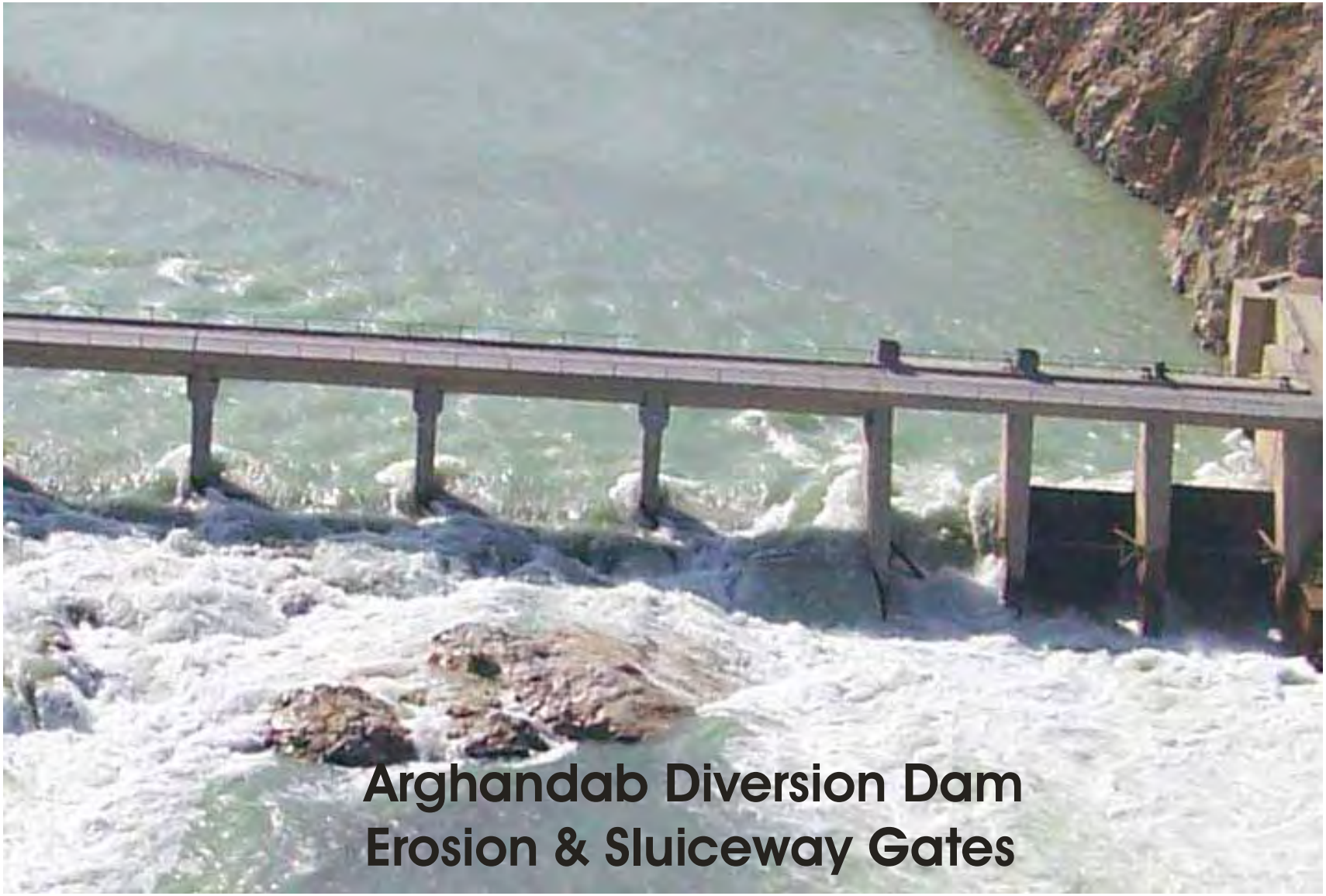

Dahla Dam Dykes/Arghandab Reservoir
& Spillway No. 2

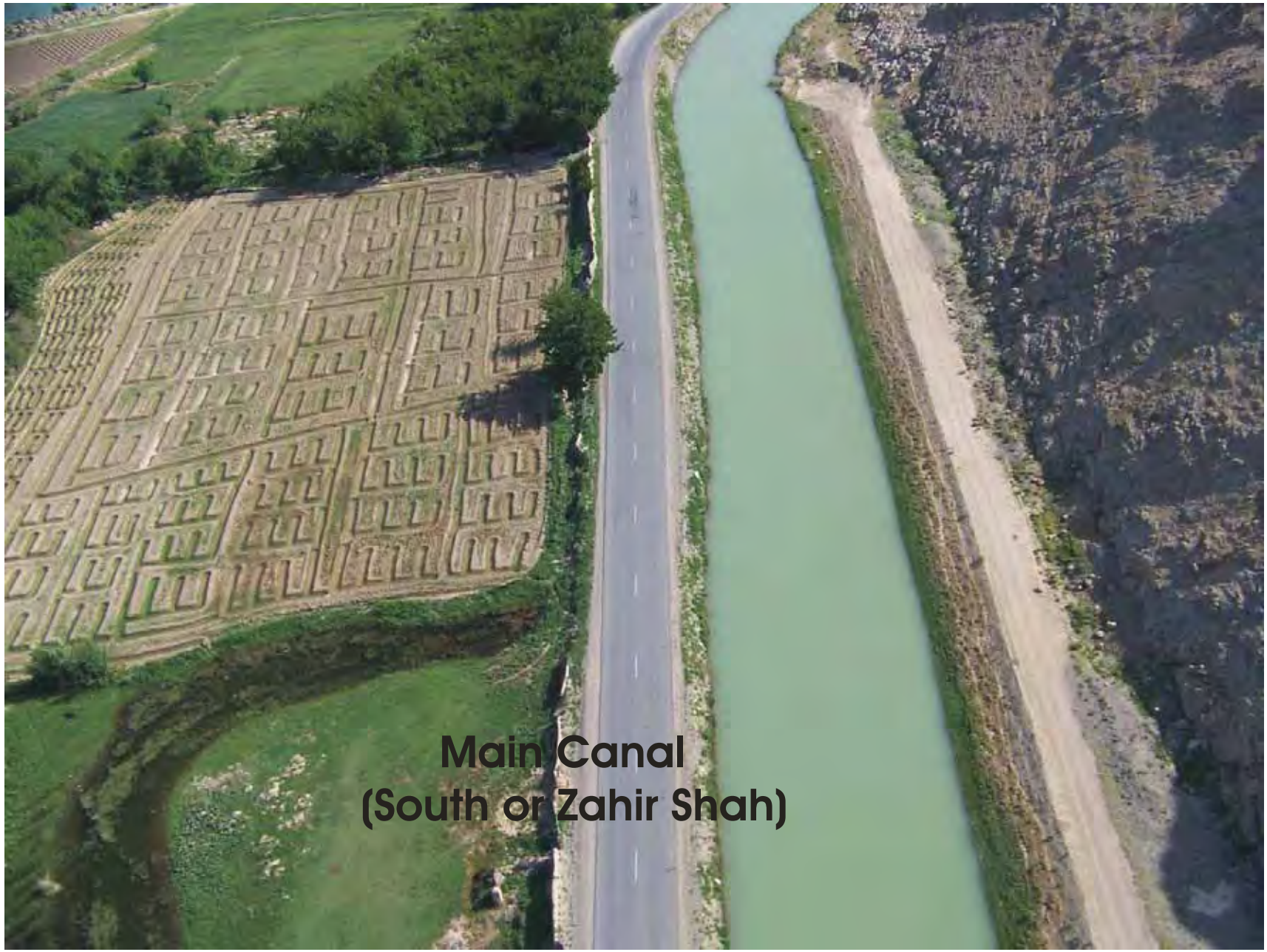

Dahla Dam
Spillway No. 2

# Arghandab River Irrigation




Arghandab Diversion Dam & Sluiceway

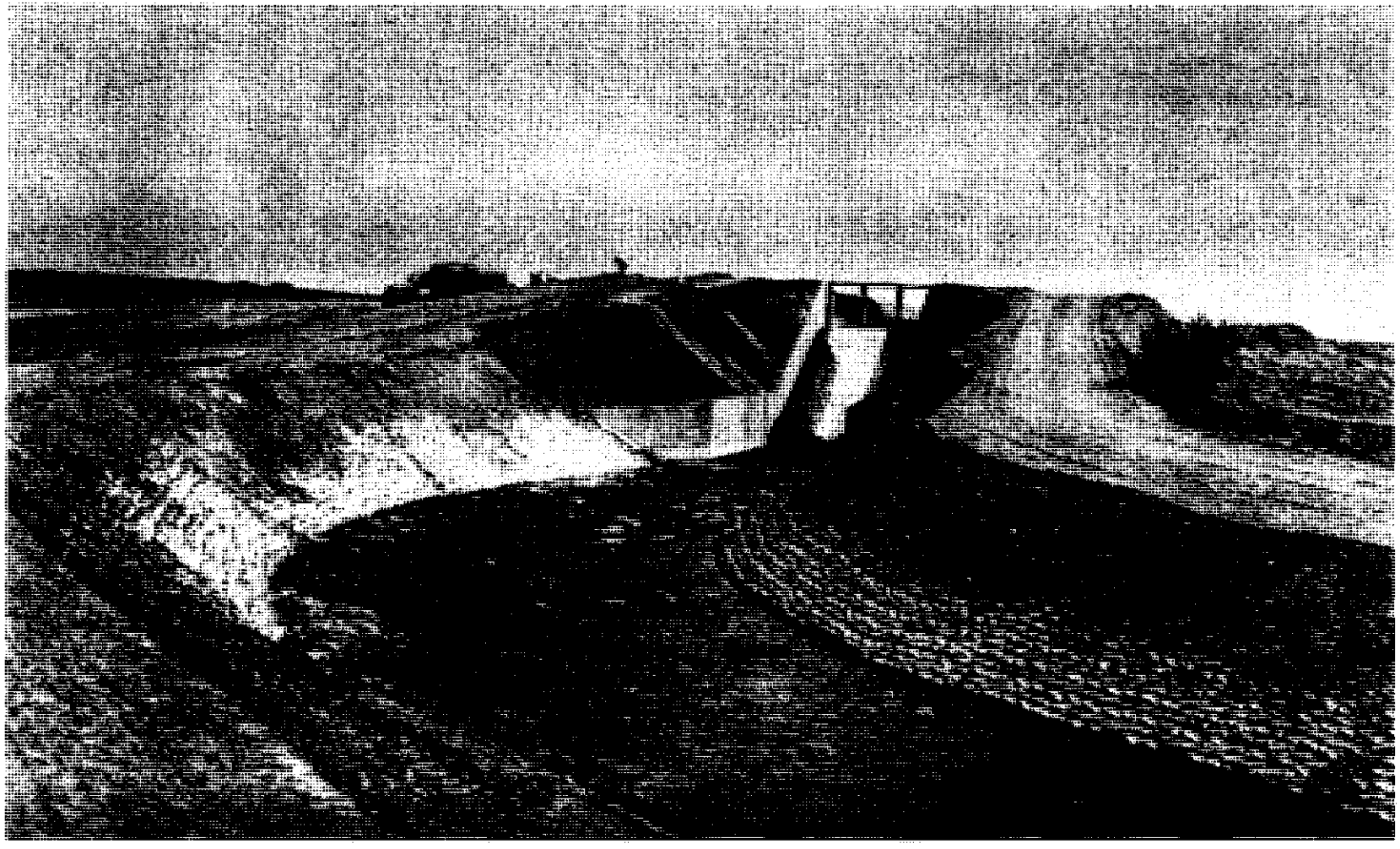

Arghandab Diversion Dam
Erosion & Sluiceway Gates


Main Canal
(South or Zahir Shah)

Irrigation Area
Arghandab River

Irrigation Area
Orchards

Irrigation Area
Kandahar



BOGHRA CANAL PROJECT – SHAMALAN CANAL
COMBINATION CHECK AND 5.13 METER INCLINED
DROP STRUCTURE

BOGHRA CANAL PROJECT — SHAMALAN CANAL
1.5 METER VERTICAL DROP STRUCTURE

## SECTION V

## BOGHRA CANAL PROJECT

### 5.01 GENERAL

Design of the Boghra Canal, from its intake on the Helmand River to Station 56 + 500, has been completed under another contract. Included in this report are the remaining portions of the Boghra Canal from Station 56 + 500 to Station 75 + 100 (Schedule IV); the East Marja Branch (Schedule V); the west Marja Branch (Schedule VI); to Shamalan Branch (Schedule VII); and certain work already designed under the previous contract (Schedule III).

### 5.02 PRELIMINARY DESIGN

All Boghra canals were designed as unlined earth sections. A maximum velocity of 1.27 meters (3.5-ft) per second was anticipated, based on a Kutter's "n" of 0.025. Minimum design freeboard was one (1) meter. An exception occurred at the lower ends of the canals where required capacities were smaller. In these sections the minimum freeboard was 0.60 meters.

The general arrangement of features of the Boghra Canal Project, as shown on the 1950 Contract Drawings 28/, are described below.

a. Boghra Canal (Schedule IV) - Beginning slightly west of the Nad-i-Ali Area at Station 56 + 500, with a design capacity of 23.75 cubic meters per second (842 cfs), the Schedule IV portion of the Boghra Canal extended approximately 19 kilometers in a westerly direction along the northern edge of the Marja Tract. Along this route, the canal serviced numerous turnouts and main canal flow became progressively smaller toward the Marja Division. The Boghra Canal terminated in a division structure at Station 75 + 100.

---

28/ "Contract Drawings, Afghanistan" by International Engineering Company, Inc., dated 1948 to 1950 (Appendix C)

Appurtenant structures of Schedule IV, as shown in Target Estimate, Boghra Canal System of March 1950, included four (4) single-barrel siphons, two (2) drainage inlets, five (5) turnouts, six (6) single-barrel culverts, three (3) double-barrel culverts, six (6) four-barrel culverts, five (5) seven-barrel culverts, one (1) wasteway, and one (1) division structure.

b. East Marja Branch (Schedule V) - The East Marja Branch originated at a division structure on the Boghra Canal. The initial reach was designed to convey a quantity of 12.38 cubic meters per second (436 cfs). The Branch extended approximately ten (10) kilometers along the western edge of the irrigable area. The lower end of the Branch extended nine (9) kilometers in an easterly direction.

Structures included ten (10) single-barrel culverts, three (3) double-barrel culverts, four (4) four-barrel culverts, seven (7) turnouts, one (1) combination check and wasteway, and two (2) chute drops.

c. West Marja Branch (Schedule VI) - The West Marja Branch originated at the main Boghra Canal division structure. The canal then proceeded in a southernly direction along the western edge of the irrigable lands.

Appurtenant structures included 15 turnouts, eight (8) single-barrel culverts, five (5) double-barrel culverts, four (4) four-barrel culverts, three (3) seven-barrel culverts, two (2) siphons, one (1) vertical drop, and one (1) chute drop.

d. Shamalan Branch (Schedule VII) - The Shamalan Branch originated at Station 31 + 680.4 on the Boghra Canal. Capacity was 21.2 cubic meters per second (748 cfs). The Branch extended 66 kilometers in a southernly direction along the west edge of the irrigable land. Periodic changes in canal cross-section were made as flow reductions occurred at turnouts enroute.

Terminal capacity was 2.1 cubic meters per second (74 cfs).

The branch had 35 turnouts, six (6) siphons, four (4) chute drops, sixteen (16) vertical drops, nine (9) single-barrel culverts, five (5) double-barrel culverts, ten (10) four-barrel culverts, four (4) seven-barrel culverts, four (4) wasteways, and four (4) checks.

5.03 FINAL DESIGN

As previously described, all canals were originally designed with unlined earth sections. It was later deemed desirable, along some reaches, to include a 0.30 meter thick compacted-earth lining to aid in seepage control.

The general arrangement of the final project is described in the following paragraphs.

a. Boghra Canal (Schedule IV) - Basically, final design of the Boghra Canal was completed as shown on the Contract Drawings 29/. There were minor changes in alignment as well as in number and location of structures.

Structures incorporated in final design included four (4) single-barrel siphons, three (3) drainage inlets, seven (7) turnouts, eleven (11) single-barrel culverts, four (4) double-barrel culverts, one (1) three-barrel culvert, one (1) four-barrel culvert, one (1) combination check and wasteway, and two (2) overchutes.

b. East Marja Branch (Schedule V) - No substantial change was made in the originally contemplated alignment of the East Marja Branch. Availability of final design data brought minor changes in quantity and location of some structures,

29/ "Contract Drawings, Afghanistan" by International Engineering Company, Inc. dated 1948 to 1950 (Appendix C)

Structures on this Branch included thirteen (13) single-barrel culverts, two (2) double-barrel box culverts, nine (9) turnouts, one (1) wasteway, one (1) underdrain, two (2) chute drops, and one (1) division structure.

c. West Marja Branch (Schedule VI) - It was determined, after investigations had been made, that the West Marja Branch was economically unjustified. Therefore, it was eliminated from the Project.

d. Shamalan Branch (Schedule VII) - The choice of alignment and structures for the Shamalan Branch was influenced by the fact that the canal was to serve lands already under cultivation. Irrigation could not be interrupted during the construction period. These lands were irrigated by an existing, but inadequate, canal system composed of several separate canals, each with its individual diversion dam and intake.

When final data became available, it was found that a more economical design could be achieved by enlarging the old canals and providing interconnections. This scheme produced a system comprising approximately 40 kilometers of old and rehabilitated canals and 26 kilometers of new canals.

The quantity of excavation was thus reduced from that originally estimated. In order to reduce the number of structures, several existing turnouts were combined. Where practicable, drop structures were located just downstream from turnouts. Drop structures were of two (2) types: vertical and inclined. Provision was made in the drops for use of stoplogs.

Checks and wasteways were located along the canal at approximately 16 kilometer intervals. All wasteway channels were extended to the river and were designed for the same capacity as the canal section immediately upstream. Both checks and wasteways were provided with radial gates operable by hand hoists. Some of these structures incorporated vehicular bridges.

Bisecting the irrigable land it serves, the Shamalan Branch Canal included 36 turnouts, one (1) siphon, four (4) chute drops, thirteen (13) vertical drops, fourteen (14) check-drops, five (5) combination checks and wasteways, seven (7) inlet drains, five (5) underdrains, one (1) flume, four (4) bridges, and one (1) division structure.

e. Boghra Canal (Schedule III) - Revisions were made of the design of the Boghra Canal as prepared under a previous contract. A check structure was included at Station 46 + 160 on the Boghra Canal. Schedule III drop structures were revised to incorporate radial gates.

## 5.04 APPURTENANT STRUCTURES

a. Drops - A drop structure on an irrigation canal is a device for absorbing excess head. There are two (2) main types of drop:

(1) Direct vertical drop

(2) Inclined chute drop

At the downstream end of a drop structure, a stilling pool, which sometimes incorporates dentated sills in its design, is utilized for dissipation of the energy created by the falling water.

Economic and hydraulic studies usually indicate the type of drop to be used. In this instance, when the required drop in the water surface was less than 3.05 meters (10-ft), the vertical type was utilized. For drops greater than 3.05 meters, inclined chute drops were used. Model tests indicated that both types of drop structure would operate satisfactorily. The design standards for these structures were similar to those used by the United States Bureau of Reclamation.

In both types of structures, stoplogs or gates were provided to maintain satisfactory water levels in the canals. In the case of inclined drops, as determined by individual requirements, either gates or stoplogs were utilized.

The principal physical parts of a chute drop are the inlet transition, control section, inclined chute channel, stilling pool, and outlet transition. To simplify construction, a "dog-leg" transition was utilized at both inlets and outlets. Economic considerations prescribed the use of a rectangular-shaped chute channel and stilling pool. Dentated sills were employed in stilling pools.

b. Siphon - Under conditions at, or approaching full operating head, a canal siphon barrel becomes a pressure conduit. Through the siphon barrel, the normal canal water surface is depressed. A siphon can be used to take canals across depressions, such as natural drainage channels, ravines, and valleys. Such a structure is also useful for crossing canals beneath railroad or highway rights-of-way.

Hydraulic requirements and available head usually determine the size of a siphon barrel. Barrels can be single or multiple, of either circular or rectangular cross-section. Other design requirements are prescribed by external and internal loading conditions.

On a silt-carrying canal, suspended loads tend to be deposited in siphon barrels. Therefore, siphon design should consider velocities suitable for automatic flusing and ejection, as much as possible, of the silt tending to be deposited. On a long siphon barrel, an air-vent is sometimes included. Maintenance on the siphon barrel will require that the barrel occasionally be pumped dry.

c. Checks - The function of a check is to maintain a proper operating water surface. Excess water may be passed downstream or may be diverted through a wasteway facility.

Design considerations for a canal check include selection of the proper size openings and selection of a means for maintaining required water surfaces. It is usually desirable that the waterway should be large enough so that, at capacity flow, velocity through the check structure will not be materially different from that in the adjacent canal sections. Positive control of flow through a check is usually accomplished by the use of radial gates. In come instances, associated more with smaller canal checks, stoplogs are more practical than radial gates.

d. Wasteways - A wasteway is a facility through which excess canal flow can be removed from the system. It also provides a means for quick dewatering of the main system in case of a downstream bank failure. An occasional full-capacity wasteway on a long canal is essential. An artificial channel leading from the wasteway outlet is sometimes necessary.

The spacing and location of wasteways is a function of canal capacity and other factors, including existence of natural channels for acceptance of wasted waters. For ease of operation and economy of construction, wasteways are usually located upstream from checks.

e. Turnouts - Turnout structures of various sizes are employed to divert operational flows from main or lateral canals. Turnout designs vary. Large-capacity turnouts are best if constructed with an inlet transition and headwalls of reinforced concrete. Use of a commercially available slidegate is desirable. Smaller capacity turnouts, as required for individual farms, are sometimes constructed entirely of timber. Timber turnouts have a limited

life and would be subject to periodic replacement. Turnout barrels can be single or multiple, according to design capacity. The barrels can be of precast concrete pipe, metal pipe, or poured-in-place rectangular boxes. Under the 1950 Contract, all turnout barrels were of precast concrete pipe.

The Boghra turnouts were designed to operate with the main canal at half capacity. Velocities were kept around 1.53 meter per second (5-fps).

f. Drainage Inlets - Drainage inlets are facilities constructed on the upper sides of canals. They are used to accept flows into the canal prism from small natural drainage channels that, under natural conditions, would cross the canal right-of-way. Drainage water is directed down the side slope via an inclined chute to the canal invert. A stilling pool is usually provided at the base of the chute when the main canal prism is unlined.

In the design of drainage inlets, the main problem is determination of discharge. Estimates for drainage discharge at the canal bank are based on empirical formulae which include considerations for drainage basin area, average slope of the basin, vegetative cover on the basin, and estimated rainfall intensity over the basin. Drainage inlets are usually constructed for basins producing relatively small amounts of runoff. Inlets are used for these smaller flows because of their relative low cost in comparison with a structure, such as an overchute or culvert that would be required if runoff were taken across or under the canal prism. Flood runoff usually occurs at periods of the year when irrigation demands and canal flows are low. Thus, flood flows entering into the canal prism does not usually create an operational problem.

g. Culverts (Underdrains) - The function of culverts is to convey drainage water under the canal prism. Drainage culverts may be single or multiple barrel, pipe or box sections, depending upon depth of water, imposed loading, and other factors. They are considered to be under no internal pressures except those due to open-channel flow conditions.

The main problem involved in design of drainage culverts is the determination of discharge capacities. These are estimated as described above for drainage inlets. Design usually follows the same general principles used in the design of canal siphons, except that calculations involve considerations of open-channel flows.

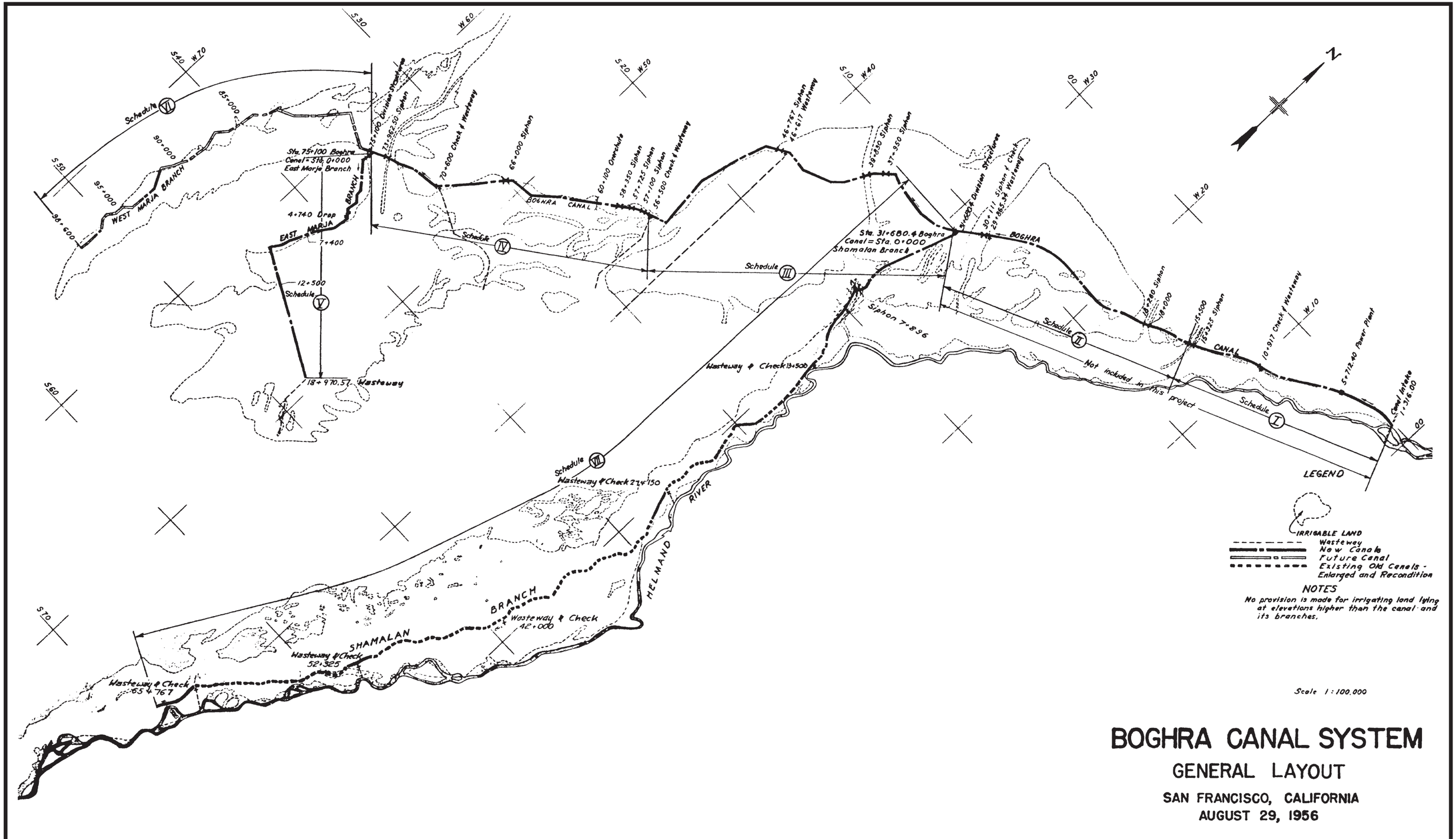
Where natural drainage waters are to be conveyed transversely across the canal prism, a culvert is usually found to be more economical than an overchute.

h. Overchutes - An overchute is an overhead flume crossing the canal transversely.

Overchutes are used where a culvert-type canal crossing is not desirable because of topographic conditions and where heavy loads of silt and debris, carried by flood flows, might be expected to plug under-canal culverts.

An overchute flume cross-section is related to estimated discharge. Drops and stilling pools are necessary at overchute outlets to retard velocity so that erosion will not occur along the outlet channel. Usually, conditions at the overchute inlet involve some channelization and dike work to direct runoff to the structure.

i. Flumes - When necessary to cross one canal over another, and if operating water surfaces are suitable, the crossing can be made via a flume structure. Such a flume is similar in some respects to a drainage overchute, with the exception that a drop and stilling pool are not usually necessary.




# BOGHRA CANAL SYSTEM

## GENERAL LAYOUT

SAN FRANCISCO, CALIFORNIA
AUGUST 29, 1956

## APPENDIX A

## REPORTS AND BACKGROUND DESIGN DATA

## REFERRED TO IN THIS REPORT

a. Comprehensive Reports

(1) "On Kargha, Seraj, Kharwar, Kajakai, Arghandab, Surkhab, and Boghra Dams, Afghanistan" by J. L. Savage, dated June 30, 1948 (page II-1, 1/)

(2) "Report on Earth Dams and Rockfill Dams" by International Engineering Company, Inc., dated May 1953 (page III-8, 7/ and IV-10, 25/)

b. Kajakai Dam

(1) "Laboratory Tests on Soil Samples from the Borrow Area, Kajakai Dam, Afghanistan" by O. J. Porter & Company, dated January 23, 1950 (page III-8, 6/)

(2) "Kajakai Project", letters by J. P. Growdon, dated March 11, March 13, and April 1, 1950 (pages III-9, 8/, III-18, 11/, III-20, 12/ and III-26, 14/)

(3) "Kajakai - Consultants", letter by D. J. Bleifuss, dated May 26, 1950 (page III-14, 10/)

(4) "Geology of the Kajakai Damsite" by F. A. Nickell, dated August 1950 (page III-5, 4/)

(5) "Grouting - Kajakai Dam", letter by James B. Hays, dated September 22, 1950 (page III-7, 5/)

(6) "Design Criteria, Specifications, Laboratory Procedures for the Embankment Foundation of the Kajakai Dam", by International Engineering Company, Inc., dated October 1950, revised March 1952 (page III-12, 9/)

(7) "Treatment of Joints and Seams in Tunnels" by Morrison-Knudsen Afghanistan, Inc., (Gilbert Waddell and James B. Hayes), dated September 24, 1952 (page III-33, 17/)

(8) "Quarterly Progress Report" by Morrison-Knudsen Afghanistan, Inc., October 1 to December 31, 1951 (page III-36, 18/)

APPENDIX A (Continued)

(9) "Quarterly Progress Report" by Morrison-Knudsen Afghanistan, Inc., January 1 to March 31, 1952 (page III-36, 19/)

c. Arghandab Dam Project

(1) "Arghandab - Foundation Grouting", letter by James B. Hays, dated July 12, 1950 (page IV-7, 23/)

(2) "Geology of the Arghandab Damsite" by F. A. Nickell, dated August 1950 (page IV-5, 21/)

(3) "Test of Soil Samples for Arghandab Dam, Afghanistan" by Abbot A. Hanks, Inc., dated August 21, 1950 (page IV-10, 24/)

(4) "Design Criteria, Specifications, Laboratory Procedures" (for Embankment Foundation of the Arghandab Dam) by International Engineering Company, Inc., dated September 1950 (pages IV-7, 22/ and IV-11, 26/)

(5) "Afghandab Tarnak Area - A Study of Water Allocation and Use for Maximum Irrigation and Power Benefits" by Morrison-Knudsen Afghanistan, Inc., Land Development Department, dated July 1956 (Dr. Fly) (page IV-24, 27/)

## APPENDIX B

## REPORTS AND BACKGROUND DESIGN DATA

## NOT REFERRED TO IN THIS REPORT

a. Comprehensive Report

(1) "Preliminary Report on Kajakai, Arghandab, and Seraj Damsites" by F. A. Nickell, dated July 5, 1950

(2) "Documents and Consultants' Reports" (list) by Morrison-Knudsen Afghanistan, Inc., dated March 6, 1952

b. Kajakai Dam Project

(1) "The Kajakai Project and its Relation to the Development of the Helmand Valley" by International Engineering Company, Inc., dated July 18, 1949

(2) "Kajakai Dam - Closure Diversion Tunnels" by B. M. Johnson (to Bleifuss), dated March 1, 1950

(3) "Kajakai Dam - Stability of Proposed Dam Section" by International Engineering Company, Inc., dated April 7, 1950

(4) "Irrigation Tunnel - Valve House - Sump Pump - Installation" (and other design computations) by International Engineering Company, Inc., dated 1950

(5) "Spillway Design" (for the Kajakai Dam) by International Engineering Company, Inc., dated May 22, 1950

(6) "Supplement S" (to the main design report, Kajakai Design Computations) by International Engineering Company, Inc., dated 1950 and 1951

(7) "Kajakai Dam - Stability Analysis, Hydraulics, Spillway, Crest, and Other Computations", by International Engineering Company, Inc., dated 1950 and 1951

(8) "Kajakai Dam - Irrigation Tunnel - Ventilation System" (computations), dated August 1, 1952

(9) "Kajakai Design Report" (correspondence and drawings) by Morrison-Knudsen Afghanistan, Inc., dated 1954

(10) "As-Built Drawings" - Kajakai (list) by Morrison-Knudsen Afghanistan, Inc., dated April 1954

(11) "As-Built Drawings" - Kajakai (list) by Morrison-Knudsen Afghanistan, Inc., dated June 1954

c. Arghandab Dam Project

(1) "Arghandab - Embankment" - Stability, Permeability, Intake Gate, Hoist, and Other Design Computations - by International Engineering Company, Inc., dated 1950

(2) "Arghandab Dam Design Computations" by International Engineering Company, Inc., dated 1950

(3) "Arghandab Design Computations" by International Engineering Company, Inc., dated 1951

(4) "Spillway Design Computations" (for the Arghandab Dam) by Morrison-Knudsen Afghanistan, Inc., dated August 1953

(5) "Arghandab - As-Built - List of Drawings" by International Engineering Company, Inc., dated February 24 and April 27, 1954

d. Boghra Canal Project

(1) "Boghra Design Calculations" by International Engineering Company, Inc., dated 1950 and 1951

(2) "Afghanistan, Boghra Canal Gates & Hoists Computations" by International Engineering Company, Inc., dated August to December 1951

(3) "Design Calculations" by Morrison-Knudsen Afghanistan, Inc., dated 1953 (with photographs)

(4) "Boghra - Shamalan Canal - Turnouts" (and other design computations) by Morrison-Knudsen Afghanistan, Inc., dated 1953

(5) "Boghra Canal - Drawings" by Morrison-Knudsen Afghanistan, Inc., dated 1954

APPENDIX B (Continued)

e. Consultants' Letters

(1) Walter R. Young, dated March 4, June 27 and July 8, 1950

(2) J. L. Savage, dated March 6 and June 22, 1950, August 24, 1951 and October 21, 1952

(3) J. P. Growdon, dated March 11*, 13*, and April 1*, 1950 and March 4, 1951 (3 letters with the same date)

(4) D. J. Bleifuss, dated June 29, 1950 and July 31, 1951

(5) James B. Hays, dated September 22, 1950* and July 12, 1951*

*Listed in Appendix A

## APPENDIX C

## DRAWINGS

a. Contract Drawings (Drawing List not included in this Report)

"Contract Drawings, Afghanistan" by International Engineering Company, Inc., dated 1948 and 1950 (pages II-2, 2/; III-1, 3/; III-20, 13/; III-27, 15/; III-29, 16/; IV-1, 20/; V-1, 28/ and V-3, 29/).

b. "As-Built" Drawings

(1) Kajakai Dam Project, as referred to in Appendix B, b (10).

(2) Arghandab Dam Project, as referred to in Appendix B, c (5).

(3) Boghra Canal Project - (Miscellaneous drawings for each schedule).

# KAJAKAI DAM, ARGHANDAB DAM, AND BOGHRA CANAL PROJECTS

## A. "AS-BUILT" DRAWING LIST (IECO)

### 1. Kajakai Dam Project

#### a. Overall Project

| | Drawing No. | Title |
|---|---|---|
| (1) | 10-F-4 R1 | Area, Capacity and Discharge Curves |
| (2) | 10-F-7 R3 | General Plan |
| (3) | 10-F-8 R2 | General Sections |

#### b. Dam

| | | |
|---|---|---|
| (1) | 11-F-1 R3 | Plan and Sections |
| (2) | 11-F-3 R1 | Piezometer Installation (Sheet 1) |
| (3) | 11-F-4 R1 | Piezometer Installation (Sheet 2) |

#### c. Tunnels

| | | |
|---|---|---|
| (1) | 13-F-1 R10 | Diversion Tunnels; Plan and Profile |
| (2) | 13-F-2 R1 | Irrigation Tunnel; Upstream Portal; General Arrangement |
| (3) | 13-F-3 R4 | Irrigation Tunnel; Upstream Portal Structure, Concrete Outline (Sheet 1) |
| (4) | 13-F-4 R4 | Irrigation Tunnel; Upstream Portal Structure, Concrete Outline (Sheet 2) |
| (5) | 13-F-19 R2 | Irrigation Tunnel; Upstream Portal Structure; Second Stage Concrete |
| (6) | 13-F-25 R4 | Emergency Generator House |

| | | |
|---|---|---|
| (7) | 13-F-26 R3 | Irrigation Tunnel; Res. W. S. Recorder House |
| (8) | 13-F-27 R3 | Irrigation Tunnel; Intake Tower; Concrete Outline (Sheet 1) |
| (9) | 13-F-28 R2 | Irrigation Tunnel; Intake Tower; Concrete Outline (Sheet 2) |
| (10) | 13-F-40 R1 | Irrigation Tunnel; Steel Conduit; Concrete Encasement |
| (11) | 13-F-41 R2 | Irrigation Tunnel; Valve House; General Layout |
| (12) | 13-F-42 R3 | Irrigation Tunnel; Valve House, Concrete Outline (Sheet 1) |
| (13) | 13-F-43 R2 | Irrigation Tunnel; Valve House; Concrete Outline (Sheet 2) |
| (14) | 13-F-44 R3 | Irrigation Tunnel; Valve House; Concrete Outline (Sheet 3) |
| (15) | 13-F-45 R2 | Irrigation Tunnel; Valve House; Concrete Outline (Sheet 4) |
| (16) | 13-F-52 R2 | Irrigation Tunnel; Valve House; Architectural Details |
| (17) | 13-F-53 R2 | Irrigation Tunnel; Upstream Portal; Elevation |
| (18) | 13-F-54 R3 | Power Tunnel; Upstream Portal; Excavation |
| (19) | 13-F-55 R3 | Power Tunnel; Upstream Portal; Outline and Reinforcement (Sheet 1) |
| (20) | 13-F-56 R4 | Power Tunnel; Upstream Portal; Outline and Reinforcement (Sheet 2) |
| (21) | 13-F-57 R3 | Power Tunnel; Plug |
| (22) | 13-F-58 R4 | Irrigation Tunnel; Plug |
| (23) | 13-F-61 R4 | Irrigation Tunnel; Rotovalve Chamber; Concrete Outline |
| (24) | 13-F-68 | Irrigation Tunnel; Pipe Trenches and Equipment Foundations |
| (25) | 13-F-69 | Power Tunnel; Intake Excavation |

| | | |
|---|---|---|
| (26) | 13-F-70 R1 | Irrigation Tunnel; Rotovalve Chamber; Rotovalve Drains |

d. Mechanical

| | | |
|---|---|---|
| (1) | 15-F-1 R1 | Irrigation Tunnel; Stop Log Guides; Details |
| (2) | 15-F-2 R3 | Irrigation Tunnel; Stop Log Guides; Assembly and Anchor Bolts |
| (3) | 15-F-3 R3 | Irrigation Tunnel; Stop Log Steel Frame; Assembly and Details |
| (4) | 15-F-4 | Stop Log; Outline and Reinforcement |
| (5) | 15-F-5 R1 | Irrigation Tunnel; Gate and Stop Log Lifting Device; Assembly and Details |
| (6) | 15-F-6 | Irrigation Tunnel; Trashrack |
| (7) | 15-F-7 | Irrigation Tunnel; Trashrack Guides |
| (8) | 15-F-8 | Irrigation Tunnel; Crane; Clearance Diagram |
| (9) | 15-F-9 R1 | Irrigation Tunnel; Intake Tower; Miscellaneous Metalwork (Sheet 1) |
| (10) | 15-F-10 R1 | Irrigation Tunnel; Intake Tower; Miscellaneous Metalwork (Sheet 2) |
| (11) | 15-F-11 R1 | Irrigation Tunnel; Wheel Gate; Assembly |
| (12) | 15-F-12 R1 | Irrigation Tunnel; Wheel Gate; Leaf |
| (13) | 15-F-13 R1 | Irrigation Tunnel; Wheel Gate; Seals |
| (14) | 15-F-14 | Irrigation Tunnel; Wheel Gate; Wheels |
| (15) | 15-F-15 R2 | Irrigation Tunnel; Wheel Gate Guides; Assembly and Anchor Bolts |
| (16) | 15-F-16 | Irrigation Tunnel; Wheel Gate Guides, Details |
| (17) | 15-F-17 R4 | Irrigation Tunnel; Intake Tower Access Bridge; Assembly and Truss |

| | | |
|---|---|---|
| (18) | 15-F-18 | Irrigation Tunnel; Intake Tower; Miscellaneous Metalwork (Sheet 3) |
| (19) | 15-F-52 | Irrigation Tunnel; Rotovalve Chamber; Hoist and Rail |
| (20) | 15-F-53 R1 | Irrigation Tunnel; Steel Conduit; Assembly and Miscellaneous Details |
| (21) | 15-F-54 R1 | Irrigation Tunnel; Steel Conduit; Details and Sections |
| (22) | 15-F-55 R2 | Irrigation Tunnel; Steel Conduit; Testing, Temporary Installation and Details |
| (23) | 15-F-56 R5 | Irrigation Tunnel; Valve House and Tunnel; Miscellaneous Metalwork |
| (24) | 15-F-57 R1 | Irrigation Tunnel; Valve House and Tunnel; Ladders and Railings |
| (25) | 15-F-59 R0 | Irrigation Tunnel; Ventilating System; Plans and Sections |
| (26) | 15-F-60 R1 | Irrigation Tunnel; Ventilating System; Details |
| (27) | 15-F-61 R1 | Irrigation Tunnel; Valve House; Plumbing and Drain Lines; Details |
| (28) | 15-F-62 | Irrigation Tunnel; Piping; Penstock and Tunnel Conduit Drains |
| (29) | 15-F-64 R1 | Irrigation Tunnel; Miscellaneous Metalwork |
| (30) | 15-F-65 R1 | Irrigation Tunnel; Intake Tower Access Bridge; Bent and Girder |
| (31) | 15-F-66 R1 | Irrigation Tunnel; Rotovalve Control Piping; Plan and Section |
| (32) | 15-F-67 | Irrigation Tunnel; Rotovalve Control Piping; Details |
| (33) | 15-F-101 | Details for Revised Hoist Trolley Frame |
| (34) | 15-F-102 | Details for Revised Lower Sheave Block |
| (35) | 15-F-103 | Hoist Drum with Added Flange |

e. Electrical

| | | |
|---|---|---|
| (1) | 17-F-1 R1 | Irrigation Tunnel; Valve Chamber; Tunnel and Valve House; Grounding Plan |
| (2) | 17-F-2 R1 | Irrigation Tunnel; Valve Chamber and Tunnel; Power and Lighting |
| (3) | 17-F-3 R2 | Valve House; Power and Lighting Plans |
| (4) | 17-F-4 R1 | Valve House; Power and Lighting; Sections and Details |
| (5) | 17-F-5 R2 | Valve House Switchboard; Elementary Wiring |
| (6) | 17-F-6 R2 | Electrical Aerial Distribution Line; Plan and Details |
| (7) | 17-F-7 R1 | Emergency Generator House; Power and Lighting |
| (8) | 17-F-8 | Electrical; Single Line Diagram |

f. Spillway

| | | |
|---|---|---|
| (1) | 21-F-1 R4 | Spillway; Excavation; Plan, Profile and Sections |

g. Roads and Bridges

| | | |
|---|---|---|
| (1) | 81-F-1 R1 | Construction Bridge; Plane and Elevation |
| (2) | 81-F-2 R2 | Construction Bridge; Sections and Details |

h. Miscellaneous and Unnumbered

| | | |
|---|---|---|
| (1) | 505-27-S71 R1 | Irrigation Tunnel; Elliptical Bulkhead |
| (2) | 505-27-S74 | Irrigation Tunnel; Steel Conduit; Conduit Coupling |

2. Arghandab Dam Project

a. General

| | |
|---|---|
| 10-F-18 R1 | General Layout and Typical Sections |

b. Main Dam

| | | |
|---|---|---|
| (1) | 11-F-1 R1 | Main Dam; Excavation Plan and Foundation Treatment |
| (2) | 11-F-2 R2 | Main Dam; Plan and Elevation |
| (3) | 11-F-3 R2 | Main Dam; Typical Sections |

c. Generator House

| | | |
|---|---|---|
| | 13-F-30 R1 | Emergency Generator House |

d. Electrical

| | | |
|---|---|---|
| (1) | 17-F-1 R2 | Intake Gate Structure; Electrical Power and Lighting System |
| (2) | 17-F-2 R1 | Intake Gate Structure; Electrical Grounding System |
| (3) | 17-F-3 R1 | Electrical; One-Line Diagram and Distribution System |
| (4) | 17-F-4 R2 | Valve House; Electrical Power and Lighting System |
| (5) | 17-F-5 R1 | Valve House; Electrical Grounding System |
| (6) | 17-F-6 R1 | Emergency Generator House; Power and Lighting |

e. Valve House

| | | |
|---|---|---|
| (1) | 21-F-1 R2 | Valve House; General Arrangememt |
| (2) | 21-F-2 R1 | Valve House, Excavation, Plan and Sections |
| (3) | 21-F-3 R2 | Valve House; Outline |
| (4) | 21-F-4 R2 | Valve House; Reinforcement Below El 1070 |
| (5) | 21-F-5 R1 | Valve House; Reinforcement Above El 1070 |
| (6) | 21-F-6 R3 | Valve House; Miscellaneous Details |
| (7) | 21-F-7 R2 | Valve House; Embedded Piping |
| (8) | 21-F-8 R2 | Valve House; Handrails and Pipe Ladder |

| | | |
|---|---|---|
| (9) | 21-F-9 R1 | Valve House; Miscellaneous Metal |
| (10) | 21-F-13 R1 | Valve House; Ring Follower Gate Piping and Operating Instructions |

f. Intake Tunnel, Gate and Trashrack Structures

| | | |
|---|---|---|
| (1) | 31-F-1 R2 | Intake Tunnel; Plan, Profile and Typical Sections |
| (2) | 31-F-2 R2 | Intake Tunnel; Rock Trap and Concrete Lining; Outline and Reinforcement |
| (3) | 31-F-5 R1 | Trashrack Structure; Reinforcement (Sheet 1) |
| (4) | 31-F-6 R1 | Trashrack Structure; Reinforcement (Sheet 2) |
| (5) | 31-F-10 R2 | Intake Gate Structure; General Arrangement |
| (6) | 31-F-11 R2 | Intake Gate Structure; Outline Below Elev. 1098 |
| (7) | 31-F-12 R2 | Intake Gate Structure; Reinforcement Below Elev. 1098 |
| (8) | 31-F-13 R2 | Intake Gate Structure; Outline Above Elev. 1098 |
| (9) | 31-F-14 R1 | Intake Gate Structure; Reinforcement Above Elev. 1098 |
| (10) | 31-F-20-R1 | Intake; Trash Rack |
| (11) | 31-F-21 R1 | Intake Gate Structure; Miscellaneous Metal |
| (12) | 31-F-22 R1 | Intake Gate Structure; Access Bridge, Abutment and Ladders |
| (13) | 31-F-23 R1 | Intake Piping |
| (14) | 31-F-30 R2 | Intake Gate; Assembly |
| (15) | 31-F-31 R1 | Intake Gate; Structural Details |
| (16) | 31-F-32 R3 | Intake Gate; Wheels |
| (17) | 31-F-33 R1 | Intake Gate; Seals |

| | | |
|---|---|---|
| (18) | 31-F-34 R2 | Intake Gate Guides; Assembly; Anchor Bolt Details |
| (19) | 31-F-35 R2 | Intake Gate Guides; Details |
| (20) | 31-F-36 R1 | Intake Gate Hoist |
| (21) | 31-F-40 R1 | Conduit and Manifold Encasement & Tunnel Lining at Outlet Portal; Outline |
| (22) | 31-F-41 R2 | Conduit; Steel Liner and Manifold; Replaced by Pacific Coast Engineering Co., Drawing 8715-D |
| (23) | 31-F-42 R1 | Conduit and Manifold Encasement & Tunnel Lining at Outlet Portal; Reinforcement |
| (24) | 8715-D | Conduit and Penstock Manifold (Pacific Coast Engineering Co., Dwg.) |

3. Boghra Canal Project

a. Schedules IV and V

| | | |
|---|---|---|
| (1) | 505-31P-1 R3 | Schedule IV; Plan and Profile (Sheet 1) |
| (2) | 505-31P-2 R3 | Schedule IV; Plan and Profile (Sheet 2) |
| (3) | 505-31P-3 R4 | Schedule IV; Plan and Profile (Sheet 3) |
| (4) | 505-31P-4 R1 | Schedule IV; Plan and Profile (Sheet 4) |
| (5) | 505-31P-5 R3 | Sehedule V; Standard Turnout |
| (6) | 505-31P-10 R3 | Schedule IV; Siphon at Station 73+962.5; General Plan and Section |
| (7) | 505-31P-11 | Schedule V; Plan and Profile (Sheet 1) |
| (8) | 505-31P-12 R2 | Schedule V; Plan and Profile (Sheet 2) |
| (9) | 505-31P-13 R1 | Schedule V; Plan and Profile (Sheet 3) |
| (10) | 505-31P-14 R1 | Schedule V; Plan and Profile (Sheet 4) |
| (11) | 505-31P-15 | Schedule V; Division at Station 75+100; General Plan |
| (12) | 505-31P-16 | Schedule V; Division at Station 75+100; Check on East Marja Branch |

| | | |
|---|---|---|
| (13) | 505-31P-17 | Schedule V; Division at Station 75+100; Check on West Marja Branch |
| (14) | 505-31P-21 R1 | General Layout of Canal System |
| (15) | 505-31P-22 | Schedule IV; Wasteway and Checkworks, Station 70+600 |
| (16) | 505-31P-24 | Schedule IV; Checkworks at Station 70+600 |
| (17) | 505-31P-26 R1 | Schedule IV; Overchute at Station 60+100; General Plan and Section |
| (18) | 505-31P-27 R1 | Schedule IV; Overchute at Station 60+100; Sections |
| (19) | 505-31P-28 | Schedule IV; Overchute at Station 66+000; General Plan and Section |
| (20) | 505-31P-29 | Schedule IV; Overchute at Station 66+000; Sections |
| (21) | 505-31P-30 | Schedule V; East Marja Branch; Drop at Station 4+740; General Plan & Section |
| (22) | 505-31P-31 | Schedule V; East Marja Branch; Drop at Station 10+425; General Plan & Section |
| (23) | 505-31P-32 | Schedule V; East Marja Branch; Drop at Station 10+425; Outline & Reinforcement |

b. <u>Schedule VII (Shamalan Branch)</u>

| | | |
|---|---|---|
| (1) | 505-31P-34 | Plan and Profile (Sheet 1) |
| (2) | 505-31P-35 | Plan and Profile (Sheet 2) |
| (3) | 505-31P-41 | Shamalan Branch; Siphon at Station 7+896 |
| (4) | 505-31P-57 | Check Structure at Station 13+500; General Plan and Sections |
| (5) | 505-31P-58 | Check Structure at Station 13+500; Outline & Reinforcement |
| (6) | 505-31P-61 | Drop at Station 17+750; General Plan & Elevations |

| | | |
|---|---|---|
| (7) | 505-31P-62 | Drop at Station 17+750; Outline & Reinforcement |
| (8) | 505-31P-64 | Drop at Station 16+132, 22+610, 24+150; General Plan and Elevations |
| (9) | 505-31P-65 | Drop at Stations 16+132, 22+610; 24+150; Outline & Reinforcement |
| (10) | 505-31P-67 | Drop at Station 24+400; Outline & Reinforcement |
| (11) | 505-31P-69 | Check & Wasteway Structures st Station 27+150; General Plan |
| (12) | 505-31P-70 | Check & Wasteway Structures at Station 27+150; Outline & Reinforcement |
| (13) | 505-31P-73 | Drop at Station 30+100; Outline & Reinforcement |
| (14) | 505-31P-76 | Drop at Station 32+350; Outline & Reinforcement |
| (15) | 505-31P-78 | Drop at Station 65+767; Outline & Reinforcement |
| (16) | 505-31P-80 | Check & Wasteway Structures at Station 42+000; General Plan & Sections |
| (17) | 505-31P-81 | Check at Station 42+000; Outline & Reinforcement |
| (18) | 505-31P-82 | Wasteway at Station 42+000; Outline & Reinforcement |
| (19) | 505-31P-85 | Radial Gates; 10' x 6' and 8' x 6' |
| (20) | 505-31P 86 | Radial Gates; 10' x 7' and 8' x 7' |
| (21) | 505-31P-87 | Radial Gate; 8' x 4' |
| (22) | 505-31P-89 | Slide Gate; 4' x 5'-6" |

c. <u>Schedule III and VII</u>

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| (1) | 505-31P-90 | Schedules III & VII; Radial Gate Hoists |
| (2) | 505-31P-93 R1 | Schedule III; Check at Station 46+160; General Plan & Elevation |

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| (3) | 505-31P-94 R1 | Schedule III; Check at Station 46+160; Outline & Reinforcement |
| (4) | 505-31P-99 | Schedule VII; Wasteway at Station 65+767; Outline & Reinforcement |
| (5) | 505-31P-104 | Schedule VII; Check and Wasteway Structures at Station 52+325; General Plan & Sections |
| (6) | 505-31P-105 | Schedule VII; Check at Station 52+325; Outline and Reinforcement |
| (7) | 505-31P-106 | Schedule VII; Wasteway at Station 52+325; Outline & Reinforcement |

## B. "AS BUILT" DRAWING LIST (MKA)

1. Kajakai Dam Project

a. General

| | |
|---|---|
| 27-81 | Downstream Bridge; Site Topography |

b. Tunnel Portal Structure & Intake Gate Tower to Elev. 983.30

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|---|---|---|
| (1) | 27-87 | Triangulation |
| (2) | 27-90 | Irrig. Tunnel; Intake Tower; Access Bridge Layout |
| (3) | 27-94 | Topography of Damsite |
| (4) | 27-101 R2 | Foundation & Abutment; Treatment as Constructed (Sheet 1) |
| (5) | 27-102 | Irrig. Tunnel; Intake Tower; Bridge Seat Details |
| (6) | 27-104 R1 | Access Bridge; Plan of Bent; Footing & Abutment |
| (7) | 27-105 R1 | Access Bridge Abutment & Bent Footing Details |
| (7A) | 27-110 | Irrigation Tunnel; Gate and Stoplog Lifting Device |

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| (8) | 27-112 R1 | Rotovalve Chamber; Ventilating System; Duct Support |
| (9) | 27-114 R1 | Irrig. Tunnel; Rotovalve Chamber; Second Stage Concrete Outline |
| (10) | 27-131 R2 | Dam Movement Stations |

2. Arghandab Dam Project

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|---|---|---|
| (1) | 29-80 R1 | Dike; Nominal Section |
| (2) | 29-93 R1 | Valve House; Discharge Chamber Liner Plate |
| (3) | 29-94 | Dike No. 6; Section |
| (4) | 29-98 | Diversion Bulkhead at Valve House |
| (5) | 29-101 | Layout; Dike No. 6 |
| (6) | 29-102 R1 | Plan; Dikes Nos. 2, 3, 4, & 5 |
| (7) | 29-108 | Tunnel Lining End Section & Bulkhead |
| (8) | 29-110 R2 | Spillway No. 1, Weir; Outline & Reinforcement |
| (9) | 29-116 | Valve House; Site Plan, As Built Plan & Sections |
| (10) | 29-118 | Trashrack Structure; As Built Outline |
| (11) | 29-134 | Main Dam; Downstream Toe Filter & Outfall Drain |
| (12) | 29-135 | Spillway No. 1; As Excavated |
| (13) | 29-136 | Emergency Generator House |
| (14) | 29-138 | Valve House, Ring Follower Gates; Piping Diagram |
| (15) | 29-140 | Spillway No. 2; As Built Plan & Miscellaneous |
| (16) | 29-142 | Conduit & Manifold Encasement & Tunnel Lining at Outlet Portal; Outline |
| (17) | 29-143 | Spillway No. 1; Concrete Weir; Excavation & Grout Holes |

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| (18) | 29-159 | D/S Topography along River Channel |

3. Boghra Canal Project

| | | |
|---|---|---|
| (1) | 31-085 R1 | Schedules III and IV; Typical Drainage Inlet |
| (2) | 31-114 R1 | Schedules III and IV; Siphon at Sta. 46+767 & 73+962.5; Inlet and Outlet Transition (Sheet 1 of 2) |
| (3) | 31-177 R1 | Schedules III, IV, and V; Typical Underdrain |
| (4) | 31-117 AR1 | Schedules III, IV, and V; Typical Underdrain |
| (5) | 31-184 R1 | Schedule VII; Division at Sta. 31+680.54 on Main Canal; General Plan |
| (6) | 31-187 R1 | Schedule VII; Check at Sta. 0+047.40 |
| (7) | 31-194 R1 | Schedule VII; Vertical Drop Structures; Sta. 0+500 to 6+340 |
| (8) | 31-200 R1 | Schedule IV; Siphon at Sta. 57+100; General Plan and Section (Sheet 1) |
| (9) | 31-200 R1 | Schedule IV; Siphon at Sta. 57+100; Inlet & Outlet Transition Details (Sheet 2) |
| (10) | 31-201 R1 | Schedule IV; Siphon at Sta. 57+725; General Plan and Section (Sheet 1) |
| (11) | 31-201 R1 | Schedule IV; Inlet and Outlet Transition Details (Sheet 2) |
| (12) | 31-201 A | Schedule IV; Siphon at Sta. 57+725; Barrel Reinforcement |
| (13) | 31-202 R1 | Schedule IV; Siphon at Sta. 58+350; General Plan and Section (Sheet 1) |
| (14) | 31-202 R1 | Schedule IV; Siphon at Sta. 58+350; Inlet and Outlet Transition Details (Sheet 2) |
| (15) | 31-210 R1 | Schedule VII; Siphon at Sta. 7+896; Inlet & Outlet Transition Structure |

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| (15A) | 31-214 | Siphon at Sta. 46+767; Air Vent Location and Details |
| (16) | 31-216 R1 | Schedule VII; Drop at Sta. 9+850; General Plan and Section |
| (17) | 31-223 R1 | Schedule IV; 70.6 Wasteway Channel; Plan & Profile |
| (18) | 31-224 | Schedule VII; Plan and Profile; Sta. 13+500 to 17+500; Typical Canal Section |
| (19) | 31-229 R2 | Schedule VII; Drop at Sta. 11+700; General Plan and Section |
| (20) | 31-230 R1 | Schedule VII; Drop at Sta. 11+700; Outline and Reinforcement |
| (21) | 31-232 R1 | Schedule VII; Wasteway Structure at Sta. 13+500 |
| (22) | 31-236 R1 | Schedule III; Check at Sta. 46+160; Radial Gate 9'-0" x 9' -9" |
| (23) | 31-237 | Schedule III; Check at Sta. 46+160; Radial Gate 9'-0" x 9'-9"; Structural Steel Details |
| (24) | 31-238 | Schedule VII; Plan & Profile; Sta. 17+750 to 25+360; Typical Canal Section |
| (25) | 31-239 | Schedule VII; Plan & Profile; Sta. 17+750 to 25+360 |
| (26) | 31-241 | Schedule VII; Radial Gates; 8'-0" x 7'-0" |
| (27) | 31-242 | Schedule VII; Radial Gate; 10'-0" x 7'-0" |
| (28) | 31-243 | Schedule VII; Radial Gate; 8'-0" x 6'-0" |
| (29) | 31-244 | Schedule VII; Radial Gates; 10'-0" x 6'-0" |
| (30) | 31-245 | Schedule VII; Radial Gate; 8'-0" x 4'-0" |
| (31) | 31-247 R2 | Schedule VII; Wasteway No. 3; Drop at Sta. 2+250; General Plan and Elevations |
| (32) | 31-248 R3 | Schedule VII; Wasteway No. 3; Drop at Sta. 2+250; Outline and Reinforcement |

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| (33) | 31-249 R2 | Schedule VII; Chute Drop at Sta. 63+600; General Plan and Section |
| (34) | 31-250 R2 | Schedule VII; Chute Drop at Sta. 63+600; Outline and Reinforcement |
| (35) | 31-253 | Schedule VII; Slide Gate; 4'-0" x 5'-6" |
| (36) | 31-254 R1 | Schedule VII; Sta. 27+138R; Turnout Details |
| (37) | 31-255 | Schedule VII; Concrete Riprap |
| (38) | 31-256 R2 | Schedule VII; Details of 36" Dia. Turnout |
| (39) | 31-257 R1 | Schedule VII; Details of 24" Dia. Turnout |
| (40) | 31-258 R2 | Schedule VII; Details of 18" Dia. Turnout |
| (41) | 31-259 R2 | Schedule VII; Wasteway No. 4; Drop at Sta. 1+100; General Plan & Elevation |
| (42) | 31-260 R2 | Schedule VII; Wasteway No. 4; Drop at Sta. 1+100; Outline & Reinforcement |
| (43) | 31-261 | Schedule VII; Slide Gate for Dwg. 31-254 |
| (44) | 31-262 R1 | Schedule VII; Q=25 to 50 cfs; Inlet Drain Details |
| (45) | 31-268 R1 | Schedule VII; Details of 18" Dia. Turnout Outlet Headwall |
| (46) | 31-272 R2 | Schedule VII; Drop at Layout Sta. 0+400; Wasteway No. 2; Outline and Reinforcement |
| (47) | 31-274 R1 | Schedule VII; Sta. 48+480 R; Turnout Details |
| (48) | 31-275 R1 | Schedule VII; Details of 24" Dia. Turnout Inlet Structure |
| (49) | 31-277 R1 | Schedule VII; Sta. 63+215 R; Turnout Inlet Structure |
| (50) | 31-278 R1 | Schedule VII; Flume Details at Sta. 10+517; General Plan & Elevation |

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| (51) | 31-279 R1 | Schedule VII; Flume Detail at Sta. 10+517; Outlet and Reinforcement; Details |
| (52) | 31-280 R2 | Schedule VII; Typical Details of Under-drain (Sheet 1 of 2) |
| (53) | 31-280 R2 | Schedule VII; Typical Details of Under-Drain (Sheet 2 of 2) |
| (54) | 31-282 R1 | Schedule VII; Q=50 to 100 cfs; Inlet Drain Details |
| (55) | 31-290 R3 | Schedule VII, Footbridge with Precast R. C. Beams |
| (56) | 31-297 | Schedule VII; Flume Details at Sta. 10+517; Barrel Superstructure; Outline & Reinforcement Details |
| (57) | 31-310 R1 | Schedule IV; Typical 36" Pipe Turnout |
| (58) | 31-318 R1 | Additions to Drop at Sta. 1+100; Wasteway No. 4 and Chute Drop at Sta. 63+600 |
| (59) | 31-319 R2 | Check Drop at Sta. 58+800; Plan, Elevations and Sections |
| (60) | 31-322 R2 | 1.35 m & 1.10 m Std. Check Drop; Plan, Elevations and Sections |
| (61) | 31-323 R2 | 1.35 m & 1.10 m Std. Check Drop; Reinforcement Details |
| (62) | 31-324 R1 | 2.00 m, 1.50 m & 1.40 m Std. Check Drops; Plan, Elevations and Sections |
| (63) | 31-326 | Radial Gate; Addition for Check & Wasteway, Sta. 27+150 |
| (64) | 31-327 | Radial Gate; Addition for Check & Wasteway, Sta. 42+000 |
| (65) | 31-328 R1 | Addition to Wasteway & Check Structures; Sta. 42+000 |
| (66) | 31-329 | Addition to Wasteway & Check Structures; Sta. 27+150 |
| (67) | 31-330 R1 | Wasteway No. 3; Addition to Drop; Sta. 2+250 |

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| (68) | 31-331 R1 | Schedule VII; Turnout Data |
| (68A) | 31-332 R1 | Schedule VII; Precast Sills for Vertical Drops |
| (69) | 31-333 | Schedule VII Gate Trunnion Details |
| (70) | 31-334 | Schedule III; Gate Trunnion Details |
| (71) | 31-335 R1 | Schedule V; 36" Precast Underdrain |
| (72) | 31-336 R1 | Schedule VII Check Drop at Sta. 55+878; Plan, Elevations and Sections |
| (72A) | 31-365 R1 | Schedule IV; Precast Sills for Vertical Drops at 70+600 |
| (73) | 31-367 | Schedule VII; Check st Sta. 0+047.40; Bracket Detail for Gate Hoist |
| (74) | 31-371 R1 | Schedule VII; Highway Bridge at Sta. 19+060; General Plan, Elevations and Sections |
| (75) | 31-372 | Schedule VII; Highway Bridge at Sta. 19+060; Concrete Reinforcement & Reinforcement Bar Details |
| (76) | 31-373 R1 | Schedule VII; Drop at Sta. 7+645; Outline & Reinforcement |
| (77) | 31-378 R1 | Schedule IV; Wasteway at Sta. 70+600 |
| (78) | 31-379 | Base Slab; Wasteway at Sta. 70+600; Reinforcing Details |
| (79) | 31-382 | Drop at Sta. 4+740; Outline & Reinforcement |
| (80) | 31-384 R1 | Schedules III & IV; Turnout for 5 to 10 cfs |
| (81) | 31-385 | Schedule V; Bridge at Sta. 10+788.80; Concrete Outline |
| (82) | 31-386 R1 | Schedule V; Wasteway at Sta. 18+981; Outline & Reinforcement |
| (83) | 31-389 | Schedule V; Drainage Ditches at Sta. 1+229; 1+800 & 2+185; Plan & Profile |

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| (84) | 31-390 | Schedule V; Drainage Ditches at Sta. 2+732; 3+130; 3+942 & 4+310; Plan & Profile |
| (85) | 31-391 | Schedule V; Drainage Ditches at Sta. 5+850; 6+288; 6+725; 7+050 & 7+290; Plan & Profile |
| (86) | 31-392 | Schedule V; Drainage Ditches at Sta. 8+300; 8+740; 9+127 & 9+816; Plan & Profile |
| (87) | 31-393 | Schedule IV; Drainage Ditches at Sta. 59+550 & 60+100; Plan & Profile |
| (88) | 31-394 | Schedule IV; Drainage Ditches at Sta. 62+000 & 63+035; Plan & Profile |
| (89) | 31-395 | Schedule VII; Sta. 25+360 to 29+000; Plan & Profile |
| (90) | 31-396 | Schedule VII; Sta. 29+000 to Sta. 35+000; Plan & Profile |
| (91) | 31-397 | Schedule VII; Sta. 35+000 to Sta. 41+000; Plan & Profile |
| (92) | 31-398 | Schedule VII; Sta. 41+000 to Sta. 47+000; Plan & Profile |
| (93) | 31-399 | Schedule VII; Sta. 47+000 to 53+000; Plan & Profile |
| (94) | 31-400 | Schedule VII; Sta. 53+000 to 59+000; Plan & Profile |
| (95) | 31-401 | Schedule VII; Sta. 59+000 to 65+000; Plan & Profile |
| (96) | 31-402 | Schedule VII; Sta. 65+000 to Sta. 65+942; Plan & Profile |
| (97) | 31-403 R1 | Schedule VII, Wasteway Channel at Sta. 13+500; 27+150 & 42+000; Plan & Profile |
| (98) | 31-404 R1 | Schedule VII; Wasteway Channel at Sta. 52+325 & 65+767; Plan & Profile |
| (99) | 31-405 | Schedule IV; Drainage Ditches at Sta. 64+150; 64+825; 65+575 & 66+000; Plan & Profile |

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| (100) | 31-406 | Schedule IV; Drainage Ditches at Sta. 66+854; 67+325; 68+052 & 68+860; Plan & Profile |
| (101) | 31-407 | Schedule IV; Drainage Ditches at Sta. 69+300; 69+500 & 70+280; Plan & Profile |
| (102) | 31-408 | Schedule IV; Drainage Ditches at Sta. 70+900; 72+076; 72+843 & 73+550; Plan & Profile |
| (103) | 31-416 | Schedule IV; 70.6 Wasteway Channel; Drop Structure at Sta. 0+412 & 0+600; Table of Dimensions |