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OMANI-AMERICAN JOINT COMMISSION
FOR ECONOMIC AND TECHNICAL CO-OPERATION

FEASIBILITY REPORT

THE WADI AL KHAWD
AQUIFER RECHARGE PROJECT

SULTANATE OF OMAN
MINISTRY OF AGRICULTURE AND FISHERIES

DECEMBER 1981

STANLEY CONSULTANTS

STANLEY CONSULTANTS

PREMIER BUILDING, SUITE 306
1725 EYE STREET, N.W.
WASHINGTON, D.C. 20006
TELEPHONE: 202/331-7770
CABLE: STANLEY WASHINGTON DC
TELEX: WUI 64290 ITT 440294

December 23, 1981

Mr. Daniel Mackell
Bureau for the Near East (NE/PD/ENG)
Agency for International Development
Washington, DC 20523

Dear Mr. Mackell:

We are pleased to present herewith our "Feasibility Report, The Wadi Al Khawd Aquifer Recharge Project."

This report has been prepared in accordance with your contract No. AID/otr-c-1628 Work Order No. 8.

Our project team has sincerely enjoyed working with you, your staff and the many people associated with this project in Oman.

Respectfully submitted,

STANLEY CONSULTANTS

Robert H. Anderson, V.P.
Project Manager

SRY:gjm:8069

SYNOPSIS

An expanding economy in Oman has led to a rapid pace of development in the capital area. Water is critical to sustain this rapid growth.

New sources of water are being developed to meet increasing demand. Capturing stormwater runoff and recharging aquifers by infiltration presents an attractive potential source.

This report examines the feasibility of recharging the aquifer underlying the Wadi Al Khawd with stormwater.

The Wadi Al Khawd is located approximately 50 kilometers northwest of Muscat near Seeb. The project site is presently adjacent to two groundwater well fields which furnish part of the potable water supply for the capital area. Increased use of water during recent drought conditions is reported to have caused an increase in the concentration of salts in the well water used for irrigation in the gardens near Seeb.

Three alternative designs were analyzed to retard stormwater runoff and recharge it to the aquifer. They are:

1. Construction of two pervious gravel and rock fill structures in series across the wadi to spread the water over leveled basins for infiltration.
2. Construction of two impervious gravel and rock fill structures in series across the wadi with culverts which release the water into existing wadi channels for infiltration.
3. Construction of a single impervious gravel and rock fill structure across the wadi with culverts which release the water into existing wadi channels for infiltration.

Our analysis concludes that alternative number three is the best alternative. It is the least expensive, takes less time to construct, is an equally effective method of recharge, and is the recommended design on which the feasibility study is based.

The cost of construction is estimated to be 4,027,000 Rials Omani. The project will take approximately 16 months to construct.

The recharged water stored in the aquifer can be used in different ways. It can be pumped out and used for potable water supply in the capital area, or for irrigating crops.

The average additional groundwater recharge produced by this project will range between three and five million cubic meters of water per year. Because of weather patterns, some years will produce more, others less. A groundwater recharge of four million cubic meters per year was used for economic analysis.

The economic analysis compared estimated project benefits to estimated project costs for agriculture and for urban water supply. The estimated internal rate of return for agriculture is 5.9 percent and for urban water supply 18.2 percent. Additional benefits derived but not accounted for in the rate of return computations include:

1. Experimental value of the project for recharging aquifers. Data collected from this project can be very useful regarding design, operation, and management of future recharge projects elsewhere in Oman.
2. Reduction in flood flows and related reduction in flood damages.
3. Possible sale of sediment removed in the dam maintenance program.

Although there is a reduction in flood flows, this project is not intended for flood control. The downstream area should be set aside as a flood hazard area and development limited.

Successful continued operation of the hydraulic structure will require regular maintenance. The structure should be monitored for damage by erosion, and repairs made as required. Trash which accumulates around the culvert inlets should be removed. In addition, sediment deposits should be removed which might hinder recharge.

Complete utilization of the additional groundwater in the garden area for agricultural purposes will be made more difficult because intermittent recharge events are not coordinated with the seasonal nature of crop water requirements. Some of the additional water may therefore be lost as groundwater discharge to the ocean. A three part monitoring program is outlined to manage this resource wisely.

The construction of this project will not change the general character and wildlife habitat of the Wadi Al Khawd, nor will there be significant reduction in flora. No endangered species are affected by this project. Therefore, the effect on the physical environment is not significant.

There are no religious, social, or cultural barriers to the construction of the dam, or use of the recharge water in connection with this project. However, proper communication strategies will be required to prevent misunderstandings of project purposes and potentials.

The need for water in Oman is substantial. Aquifer recharge can be an important source of water supply. This project is not only economically, sociologically, and environmentally feasible, it is an important demonstration project for possible application elsewhere in Oman.

We recommend its construction and development.

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PART 1 - INTRODUCTION

1.1 Purpose and Authority

The purpose of this report is to discuss the design, construction, and maintenance of the proposed project, to determine economic feasibility, to evaluate impacts on the physical environment, and to assess project sociocultural and socioeconomic feasibility.

The Government of Oman and the U.S. Agency for International Development acting through the Joint Commission on Economic and Technical Cooperation has authorized this study.

1.2 Project Background and Description

An expanding economy in Oman has led to a rapid pace of development in the capital area. Water is critical to sustain this rapid growth. Estimates show the demand for water tripling in the next ten years.

The capital area is in an arid region of the world, and water is scarce. New sources of water are being pursued to meet increasing demand. Currently, studies are evaluating the use of desalinization plants, capture of runoff by high dams and reservoirs, additional groundwater well fields, and the recharging of aquifers by infiltration of storm water runoff.

This project carries to completion a concept and preliminary design which has been considered by the Oman Public Authority for Water Resources. This report examines the feasibility of recharging the aquifer underlying the Wadi Al Khawd with storm water. Construction plans, specifications, and bid documents for the project are being prepared concurrently with this study.

The Wadi Al Khawd is located on the southeast Batinah coast of Oman approximately 50 kilometers west of Muscat (see Figure 1.1). This area is in the path of existing and planned development of the capital

area. Already plans have been prepared for a housing development in the vicinity of the project site.

Wadi Al Khawd is underlain by an unconfined aquifer and it presently is adjacent to two groundwater well fields which supply potable water to the capital area. Over the last few years, exploitation of the aquifer and agricultural use practices have increased the salinity of shallow wells near the coast. These wells provide water to the agricultural areas around Seeb.

By constructing a recharge dam with controlled outlets across the wadi, storm water is slowed allowing time for infiltration to the water table.

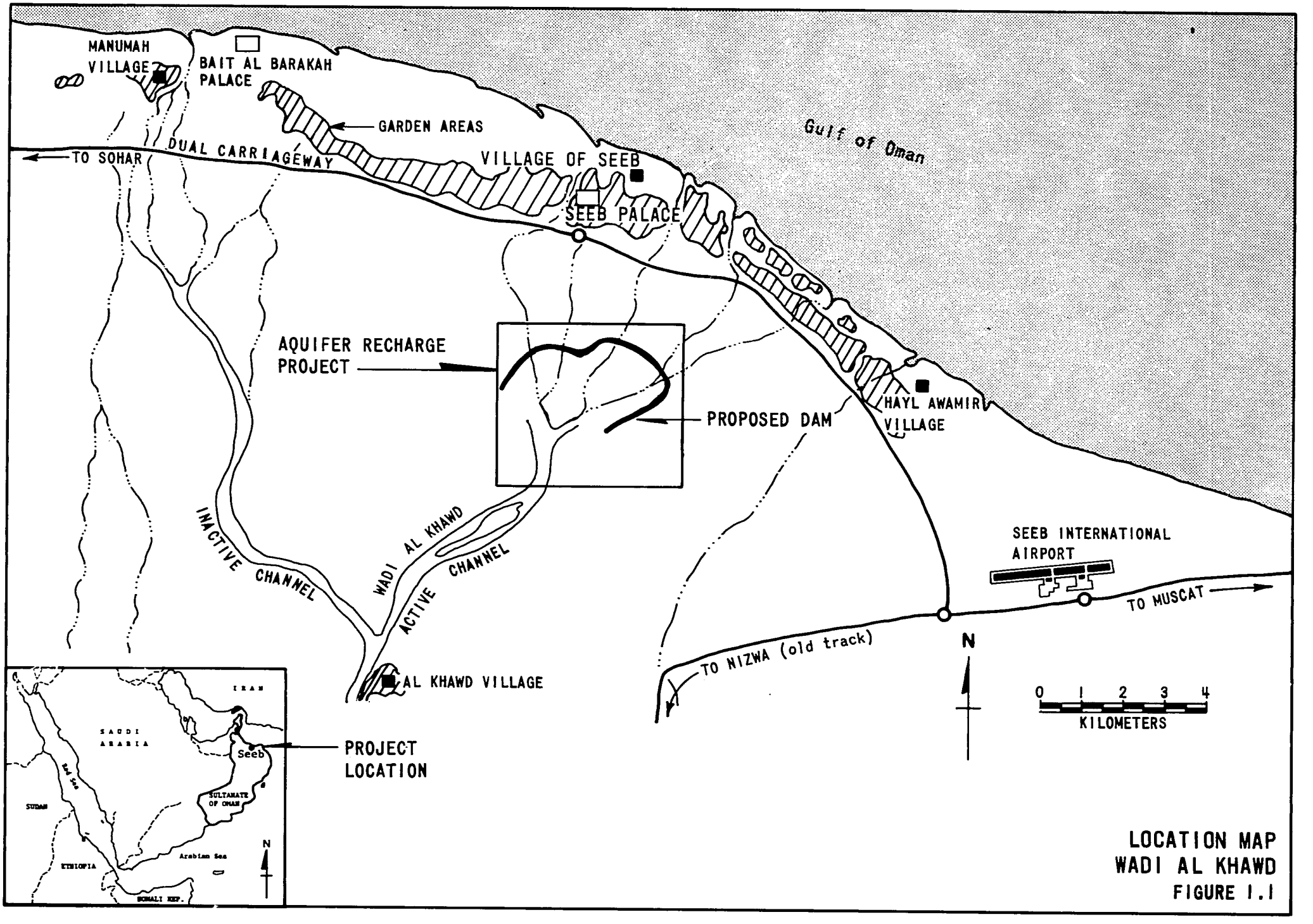
The water, once captured and stored in the aquifer, can be used in different ways. It can be pumped out and used for potable water supply for the capital area, used for irrigation, or to improve the quality of existing wells in the gardens at Seeb.

1.3 Development Objectives

The objective of this project is to increase the amount of freshwater available for use. This objective can be met by capturing Wadi Al Khawd runoff which is normally lost to the sea and recharging the underground aquifer by infiltration. The basic plan for recharge was presented in the 1980 report by Tetra-Tech International, Inc., "Preliminary Engineering Design for Wadi Al Khawd Recharge Scheme." Tetra-Tech serves as consultant to the Oman Public Authority for Water Resources.

The objectives of the Engineering Services Agreement in this current study are to:

1. Perform a soils and materials survey necessary for project analysis, final design, and to establish sources from which construction materials can be obtained.
2. Develop a plan for the construction of the project which includes a description of construction operations, methods of construction supervision and control, and construction schedule.



LOCATION MAP
WADI AL KHAWD
FIGURE 1.1

3. Perform an analysis to determine the economic justification for the construction of the proposed project.
4. Assess the sociocultural and socioeconomic feasibility of the project to determine the compatibility of the project with its sociocultural environment in which it is to be introduced, and assess the distribution of benefits among different groups.
5. Determine the present environmental conditions in the wadi with regard to major plant species, wildlife, endangered species, and prepare proposals for minimizing any adverse effects.
6. Prepare a report which presents the analysis of the above items along with project recommendations and conclusions.
7. Prepare final design plans, specifications, cost estimates, and bidding documents in order for the Government of Oman to advertise and execute a competitive bid construction contract in accordance with AID Handbook II.

1.4 Report Organization

The report is divided into ten parts and five appendices:

Part 1 - Defines the project.

Part 2 - Outlines the methodology used to do the study and design.

Part 3 - Discusses the project design elements and criteria.

Part 4 - Outlines procedures to maintain the project after its construction.

Part 5 - Presents an economic analysis of the proposed project.

Part 6 - Discusses the sociocultural and socioeconomic feasibility of the project.

Part 7 - Assesses the impact on the physical environment.

Part 8 - Contains a construction plan, schedules, and methods of supervision.

Part 9 - Contains the construction cost estimate and discusses the criteria used in its formulation.

Part 10 - Provides study results and recommendations.

Appendix A - Tabulates plant and animal species which may be found in the study area.

Appendix B - Contains supporting data for cost estimates.

Appendix C - Contains design computations.

1.5 Acknowledgements

The project team responsible for the preparation of the design drawings and report was composed of staff members selected from Stanley Consultants' offices in Muscatine, Iowa, U.S.A.

In gathering and evaluating the basic data required for preparation of this report, Stanley Consultants' staff relied heavily upon the cooperation and assistance of a large number of official agencies and individuals associated with the Government of Oman.

The consultants are pleased to acknowledge this assistance and express their gratitude for the support and cooperation extended by these individuals, organizations, and their staffs.

PART 2 - WORK PROGRAM

2.1 Existing Reports and Data Collection

The following studies discuss recharging the Wadi Al Khawd aquifer with storm runoff:

A. "Report on Water Resources Study, Phase II, and Technical Proposal for Construction of Water Recharge Projects," by the Government of the United States, U.S. Army Corps of Engineers Division, Middle East - November, 1979.

B. "Evaluation of Alternative Ground-Water Development Schemes for the Wadi Samail (Al Khawd) Aquifer," by Tetra-Tech International, Inc. - March, 1980.

C. "Preliminary Engineering Design for the Wadi Al Khawd Recharge Scheme," by Public Authority for Water Resources.

Report "A" addresses the planning, design, and construction of structures for recharging the shallow aquifers of the Batinah Plain. It also presents a conceptual design for a pilot project in the Wadi Samail (Al Khawd) to reduce surface flow losses to the sea and increase recharge to the alluvial gravels.

Report "B" evaluates the hydrologic effects in terms of water level changes and salinity increase in the Wadi Al Khawd of:

1. Continued development from the aquifer at the projected 1983 pumping rates.
2. Additional development from the aquifer with alternative schemes.

Report "C" is a preliminary design of an aquifer recharge scheme using two permeable dams for the Wadi Al Khawd.

Three members from Stanley Consultants (an engineer, an economist, and an environmental analyst) were in Oman between May 24 and June 11, 1981. While in Oman they conducted interviews and gathered maps,

photos, statistics, field information, previous studies, and other data to assist in the design and analysis of the project.

Various government agencies and government consultants were contacted to obtain information. Data was collected from the following:

Ministry of Diwan Affairs
Ministry of Communication
Ministry of Agriculture and Fisheries
Ministry of Social Affairs and Labor
Ministry of Commerce and Industry
Ministry of Electricity and Water
Ministry of Land Affairs and Municipalities
Ministry of National Heritage
Public Authority for Water Resources
Food and Agriculture Organization (United Nations)
Petroleum Development in Oman PD(O)
Development Council
U.S. Embassy in Oman

2.2 Engineering Survey

An engineering survey was required for siting the dams, computation of earthwork quantities, locating borrow materials, and setting of field reference points (both horizontal and vertical). B.K.S. Surveys Limited of Oman was engaged to produce topographic mapping of the project site by aerial methods. In addition, aerial photography was supplemented with ground surveys to insure required accuracy and detail and to set field reference points.

The engineering survey was done according to the following requirements:

1. Mapping was produced at 1:2,000 scale with contours at 1 meter intervals, accurate to plus or minus 1/2 meter. Spot elevations were required in flat areas. Relative accuracy of contours was to be within 25 percent of the contour interval.
2. Bench marks were set and referenced close to the project site for vertical control during construction.
3. Horizontal control points, based on Universal Transverse Mercator, Zone 40, Clarke 1880 Spheroid Projection, were set and referenced in the field for use during construction.

2.3 Soils Investigations

A soils investigation was undertaken to determine the permeability of the site for aquifer recharge, establish material sources, identify areas of unsuitable material, assess foundation stability and safety, and determine costs of excavation and embankment.

Trow-Oman, Materials Consultants, was engaged to do a soils investigation program consisting of the following:

1. Fifteen soil borings to a depth of 10-20 meters. In four selected locations, borings were continued to groundwater (24± meters) plus two meters.
2. Four surface infiltration tests.
3. Four infiltration tests taken at one and one-half meters depth.
4. Test pits to identify borrow areas for material to be used in dam construction.
5. Laboratory tests conducted in the following areas:
   a. Sieve analysis.
   b. Soil classifications (unified system) of all major soil types encountered in borings and test pits.
   c. Field moisture, modified proctor (density) and permeability.
6. Soils report which includes location data, boring logs, tests run and a summary of all results.

2.4 Summary

Existing reports, data collection, survey and soils investigations were used to develop design concepts discussed in the next section.

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PART 3 - DESIGN

3.1 Design Development

Investigations of plans for augmenting urban water supplies have been actively pursued for the past decade. As the desirability of using surplus flood flow from Wadi Samail became more apparent, additional investigation and hydrologic measurement programs were initiated to assist in design development.

3.1.1 Gibb and Partners - Sir Alexander Gibb and Partners (1)* recommended investigation of the possibility of impounding wadi flood flows in the coastal plain for increasing groundwater recharge. They pointed out the need for hydrological and meteorological data in order to assess the potential of such a project. A tabulation of estimated average monthly discharges at Al Khawd based upon occasional gaging by PD(O) Ltd. was included in their report.

3.1.2 Corps of Engineers - The United States Army Corps of Engineers (2) after having recommended development of conceptual designs and specific sitings for recharge projects in their Phase I report of the previous year, undertook more detailed study leading to selection of Wadi Samail as an initial recharge development site. The Corps estimated that six million cubic meters of additional water might be made available from such a project. They presented preliminary discharge curves from which estimates of maximum possible and 50-year peak discharges might be estimated. Application of these curves to Wadi Samail yields estimates of about 1,800 cubic meters/second for the maximum probable flood and 1,050 cubic meters/second for the 50-year flood.

The Corps proposed construction of a concrete weir to divert flood flows down an abandoned channel leading to the west. Recharge would be effected by reducing flood flow down existing channels by about 50 percent and diverting this flow down presently inactive channels.

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*See page 3-22.

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3.1.3 Authority for Water Resources - The Public Authority for Water Resources, Oman (3) proposed two retention structures acting in series over that portion of the alluvial fan presently used by the stream. The two dams would be designed to impound the estimated 25-year flood of from 9 to 10 million cubic meters. The dams would be of permeable construction and would augment discharge by diverting wadi flow over specially prepared leveled spreading basins. This would be accomplished by permitting permeable flow through the dams and by distributing flood overflow over spillways extending almost completely across the alluvial fan. The project would have an estimated cost of 2.6 million R.O. No estimate of increased recharge was included.

3.1.4 Tetra Tech - Tetra Tech International, Inc. (4) assumed a total retention capacity of four million cubic meters and an annual increased recharge rate of 1.33 million cubic meters for a project similar to that proposed by the Authority for Water Resources. Their study consisted of an evaluation of effects of long-term uniform recharge upon water levels in the aquifer. A mathematical model of the aquifer was formulated and calibrated using the best available information. Alternative groundwater development plans utilizing the assumed uniform recharge rate were examined. The best plan appeared to be the interception of recharge by means of a collecting gallery about one kilometer south of the Coast Road. Presumably this intercepted supply would be diverted to urban water use. Additional study of the effectiveness of the spreading basins was recommended.

3.1.5 Authority for Water Resources - Subsequently, the Public Authority for Water Resources (5) compared proposed plans. They outlined five design objectives:

1. Maximization of existing subsurface storage potential
2. Preservation of historical water resources for current users.
3. Minimization of requirement for new (groundwater) recovery and transport facilities.
4. Exploitation of aquifer hydraulic properties to maximize retention of water in storage.

5. Minimization of wetted areas and subsequent evaporation loss during periods of recharge.

Consideration of these objectives led to recommendation of a plan along the lines of that proposed by the Water Authority rather than that of U.S. Army Corps of Engineers.

3.1.6 Current Study - While not called upon to duplicate the work which led to selection of the general impoundment scheme recommended by the Water Authority, Stanley Consultants has endeavored to realize the design objectives outlined above. These objectives will be met to the extent possible consistent with the Scope of Work for this study. Efforts have been made to accomplish further development of objectives 1 and 5 without diminishing the others.

3.2 Hydrologic Data

Water resource planning has produced a need for more hydrologic information which, as it is accumulated enables more effective plans to be formulated. An additional need which is partly hydrologic and partly engineering research is now being felt. Actual operating experience of a recharge project is needed to provide information upon which plans for further projects may be based. Consequently, the Wadi Al Khawd project has been brought into the design phase at a time when adequate hydrological information is only just becoming available.

The monthly discharge measurements at Al Khawd reported in (6) enabled approximate water balances to be deduced. However, no special attention was paid to flood flows. Previous studies have attempted water balances using extremely limited data on surface water supplies. Because of the limited data base, estimates of recharge potential are lacking in most of these studies.

Other investigators (7) have employed correlations of wadi channel geometry with flood flow using relations derived for streams in New Mexico and Wyoming in the USA. Although conditions cannot be expected to be exactly the same in Oman, this approach was considered to be a reasonable alternative until sufficient flood discharge measurements have been made for independent analysis or to calibrate the channel geometry equation for Oman. Autographic stage records began at Al Khawd

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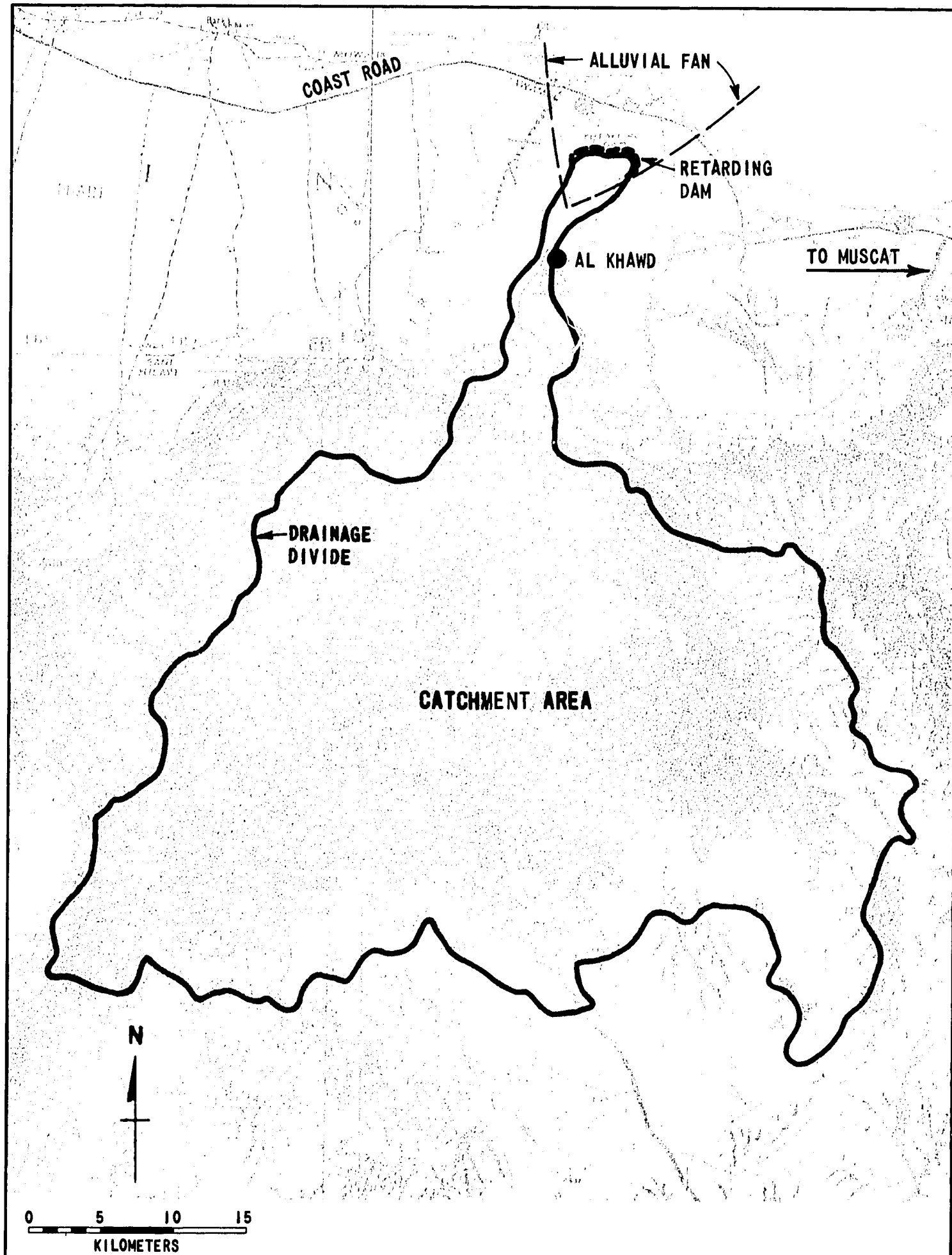
in April, 1975. The location of Al Khawd in relation to the dam is shown on Figure 3.0. Regular current meter discharge measurements started in 1977. Current meter discharge measurements have also been made at the Coast Road since May, 1977, when floods occurred.

The continuous records (1974 through 1977) have enabled us to make preliminary discharge frequency estimates. While four years of data are inadequate for accurate frequency analysis, they provide results showing a conservative margin of safety over previous estimates. The tentative flood frequency distribution on Figure 3.1 is derived from this limited data base using the extremal (Gumbel) distribution. The standard error of estimate of the 100-year flood based upon the 4 years of data assuming extremal distribution is 275 cubic meters/second. This error would be reduced to approximately 177 cubic meters/second if 10 years of flood measurements were available.

Probable maximum flood (PMF) flows for emergency spillway design are best estimated from long-term rainfall records. Since the appropriate data is lacking, the Corps of Engineers (2) presented a provisional curve (Figure I-5) using Creager's Formula with coefficient C=20 for estimating the PMF. For Wadi Al Khawd with 1,800 km^2 drainage area the PMF estimate is 1,900 m^3/s. This flow requires a depth of approximately 0.5 m on the 3,000 m long spillway. The actual capacity of the spillway, including freeboard of 1.0 m above the PMF level is therefore very nearly five times the estimated PMF flow. This is a very conservative design considering the rainfall and basin characteristics for this watershed.

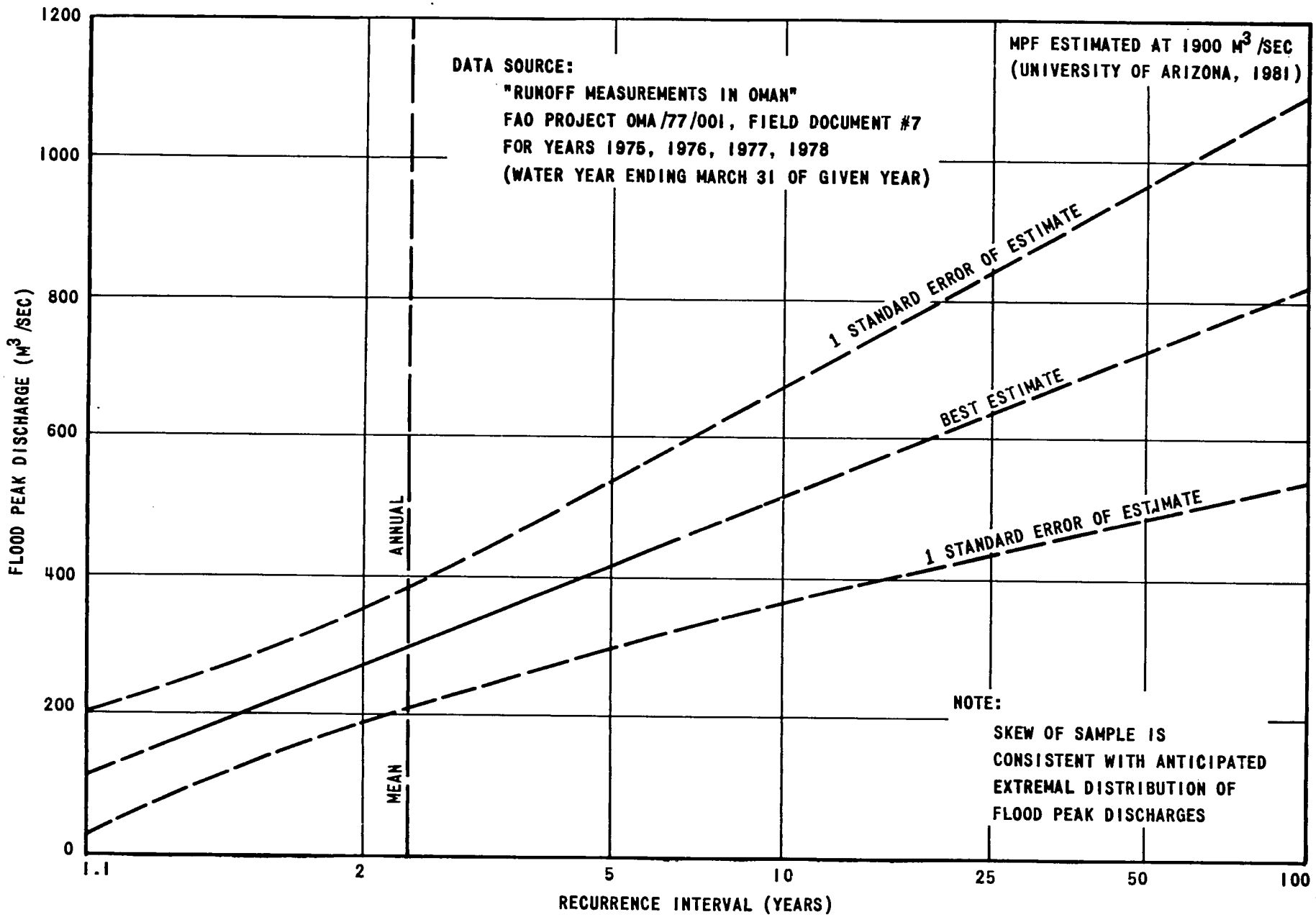
Records of runoff measurements made in Wadi Al Khawd as reported in (6) contain some interesting data on simultaneous flood discharges at the gaging station at Al Khawd and at the Coast Road. Table 3.1 summarizes this information.

Flood volume reductions range from 53 to 200 million cubic meters in Table 3.1, but the relationship between volumes at the two stations



RECHARGE SYSTEM CATCHMENT AREA
WADI AL KHAWD

FIGURE 3.0



ANNUAL PEAK DISCHARGE-FREQUENCY AT AL KHAWD
WADI AL KHAWD
FIGURE 3.1

TABLE 3.1

MEASUREMENTS EVIDENCING INFILTRATION BETWEEN AL KHAWD AND COAST ROAD

Date	Flood at Al Khawd		Flood at Coast Road	
	Peak[1]	Volume[2]	Peak[1]	Volume[2]
24/25.5.77	73	2.90	Less than 20	0.60
17/18.11.77	230	3.00	30	0.50
20/21.11.77	21	0.32	Trace[3]	Trace[3]
25/26.11.77	105	2.70	34	0.78
8/10.3.78	170	6.60	51	2.32

[1]Peak discharge in cubic meters/second.

[2]Flood volume in millions of cubic meters.

[3]May have been due to local rainfall.

Source: F.A.O. Field Document No. 7 (6)

is complex and cannot be accurately derived from these events. The data were used, however, to derive a simple linear relationship:

$$V_c = 0.48\ (V_A - 1.8) \text{----------} (1)$$

Where:

V_c = Estimated flood volume at the Coast Road.

V_A = Measured flood volume at Al Khawd.

Application of Equation (1) to the four years of flow measurements at Al Khawd and assuming all flow at the Coast Road reaches the sea, resulted in the following:

Average flood volume at Al Khawd = 16.8 million cubic meters/year.

Average flood volume at Coast Road = 4.8 million cubic meters/year.

Hence:

Natural recharge from flood flow = 12.0 million cubic meters/year.

These estimates are in reasonable agreement with those of P. M. Horn (6). It should be emphasized that these estimates, based as they are on short duration data, contain a possibility of error which would require many years of continued measurement to significantly reduce. The standard deviation of measured annual flood volumes indicates that our estimate of average annual values contain a standard error of nearly 50 percent. This means that should we be able to compare the above estimate with one based upon 30 or more additional years of measured data, we would not expect the difference between the two estimates to exceed about 50 percent of the values given (either upwards or downwards). It is interesting to note that this standard error obtained from short-term measurements of a gaged stream, is comparable to that to be expected from typical "Regional Equations" based on long term records when applied to ungaged streams.

The values given and those obtained subsequently for average annual recharge derived from the retarding dam are the best estimates possible based upon available data for use in examining economic feasibility of the project.

For the purpose of providing a range of values for sensitivity analysis of the economics of the project we have assumed that possible water supply drived from the project may range from 3.0 to 5.0 million cubic meters/year with a most probable value of 4.0 million cubic meters/year.

Comparison of estimated flood volumes shows an additional very important fact. There is an existing mechanism for groundwater recharge, principally through the natural wadi channel beds. If channel flow can be restricted to less than approximately 50 cubic meters per second, almost no flow will be lost to the sea. Since this flow will be discharged to the normal channels, it will not be necessary to wet dry ground in artificial spreading basins. Consequently, by concentrating flow to the "wet" channels, losses due to wetting and subsequent evaporation will be minimized (see Design Objective No. 5, described above). Design Objective No. 1, to maximize existing subsurface storage potential, is realized by diverting as much floodwater as possible into

the ground. This will be accomplished by controlling flood outflow from the retarding structures to below 50 cubic meters per second as much as possible. To accomplish this, impoundment volume must be maximized but water must be stored in reservoirs for the least possible time to minimize evaporation loss.

If maximum practical storage is developed at the location previously selected by the Authority for Water Resources (with minor relocation of the dam to reduce flooding of the Seeb well field) 11.5 million cubic meters of flood flow can be stored and outflow controlled to about 50 cubic meters/second up to the 10-year flood level. Allowing for infiltration loss between Al Khawd and the retarding basin, the average annual spillway overflow from the dam is estimated to be 0.2 million cubic meters. Assuming approximately 0.2 m cubic meters of spillway overflow is lost to the sea, construction of the retarding dam will make an additional 4.8 - 0.2 = 4.6 million cubic meters of recharge available. This volume will be realized as additional water supply only if provision is made for its extraction.

Figure 3.2 is a frequency plot of flood inflow volume derived from the Al Khawd gage data. The data consists of 11 flood events recorded in the four-year period. These individual floods show significantly greater volume than those estimated from channel cross section data in (7).

On Figure 3.3, the flood inflows for each of the four years are totaled. Recharge estimates based on annual totals are considerably greater than those derived from consideration of isolated peak floods. This is the reason that the estimate of available recharge of 4.6 million cubic meters exceeds that used in the groundwater modeling study (4), although it is not as large as the preliminary estimate by the Corps of Engineers (2) of 6 million cubic meters per year.

3.3 Retarding Structure

Mapping at a scale of 1:2,000* with one meter contour interval and spot elevations made for this project, has enabled better dam alignment and storage/area curves (see Computations - Appendix C) than was possible in the preliminary studies.

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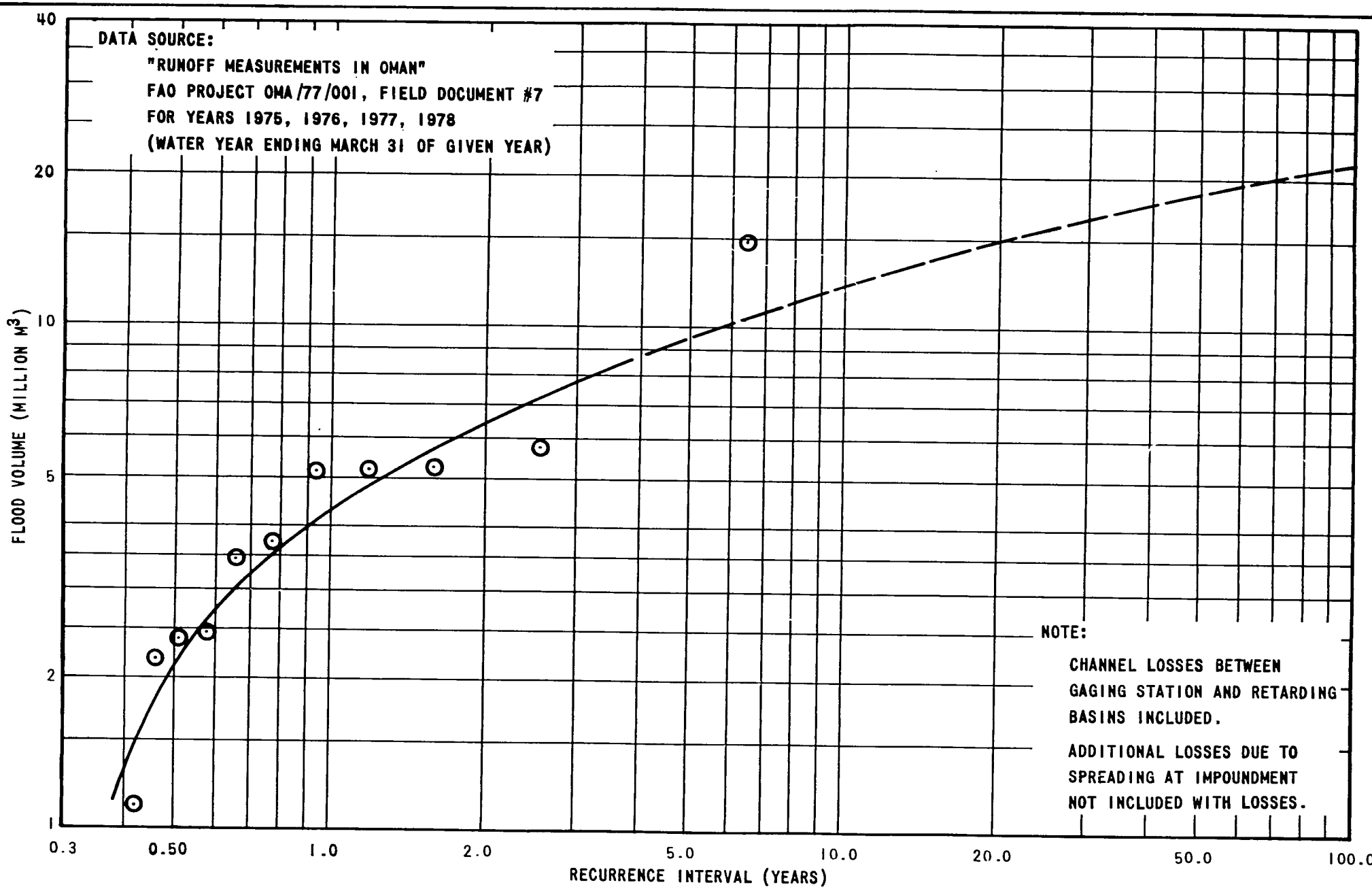
*Design drawings are included at end of report.

Approximate locations outlined in the preliminary engineering design report have been retained. Storage at spillway elevation has been increased to 11.5 million cubic meters with a corresponding combined lake surface area of 260 hectares (see design drawings).

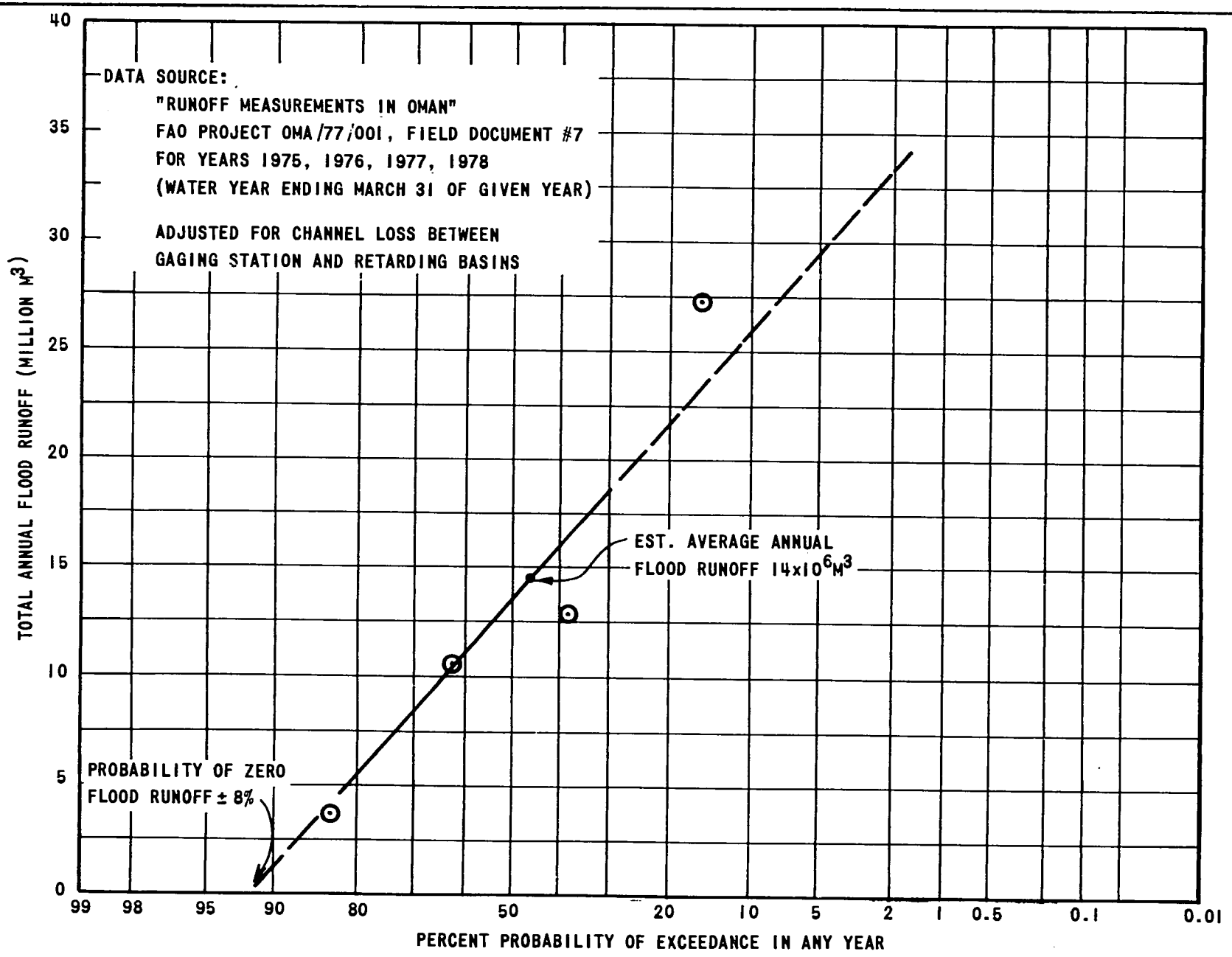
The storage volume is about as large as practically possible in this location, and is sufficient to ensure that floodwater losses to the sea will be very small. The resulting dam height is 7.5 meters, the spillway crest elevation is 38.5 meters (based on mean sea level datum), and the spillway crest length is 3,000 meters. Maximum pool elevation when passing the maximum probable flood is 39.0 meters.

Consideration of several factors led to abandoning the permeable dam concept. Sediment load during a large flood would plug a permeable dam within a few hours. Plugging would begin at filter zones within the dam, necessitating rebuilding of the dam to restore the original permeable condition. The intent of the permeable flow as well as the long spillways was to spread thin sheet flow over leveled recharge areas. This would involve wetting extensive volumes of sand causing considerable water loss. Large floods would speedily erode the leveled recharge areas necessitating frequent restoration maintenance. Fine suspended sediment would also speedily seal the spreading areas. In order to construct a dam with a reasonable prospect of remaining pervious, the stone used should be of sufficient size that all suspended sediment would pass through readily. It is estimated that such stone should be at least 50 millimeters and above. Reservoir loss rates would then exceed the infiltration capacity of the downstream area leading to continued loss to the sea. Sieve analysis of typical wadi deposits shows that material of this size constitutes only about 5 percent of the total material available locally. Consequently, about 20 times the volume of the proposed dam would have to be excavated and screened and then reexcavated and hauled to the structure in order to construct a pervious dam.

The need for culvert outlets would remain for a "pervious dam" to provide for the possibility of ultimate plugging of the dam by sediment. A slight reduction in number of culverts over that included



SINGLE EVENT FLOOD VOLUME~FREQUENCY AT RETARDING BASINS
WADI AL KHAWD
FIGURE 3.2



ANNUAL TOTAL FLOOD RUNOFF~FREQUENCY AT RETARDING BASINS
WADI AL KHAWD
FIGURE 3.3

in our design might be realized but this would be offset by the necessity to provide control gates, control towers and access. In this location, a permeable dam design would not be economically attractive. Our design ensures a long service life of a dam made "pervious" by uncontrolled culvert outlets at a cost less than would be achievable by the original concept.

Economic considerations have also led to selection of a single impoundment for the final design in place of the twin reservoirs originally conceived. The chief factor influencing this choice has been the high cost of constructing two spillways of equal capacity required to pass the spillway design flood. A single dam is capable of supplying all the benefits and safety of two at less cost.

Encroaching urban development has been noted downstream of the proposed dam. For this reason, the Maximum Probable Flood has been used as the spillway design flood. The dam has been designed to be as impervious as possible using materials from the immediate vicinity.

The dam will not have major flood control benefits. There will be unregulated flood discharges over the free overflow spillway for flood flows in excess of the 10-year event. In addition, there will be frequent small discharges from the culvert outlets into many of the small channels in the alluvial fan. Encroachment of the wadi fan by municipal or industrial development should therefore be controlled in order to maintain the primary purpose of the recharge facility.

The core zones of the embankment will be of the finest in size material close to the dam in the reservoir area. Since sudden drawdown conditions will prevail and because both faces will be saturated in the overflow sections, side slopes have been limited to 1 on 3. Riprap cover for the upstream face has been designed to resist wave action from 50 miles per hour (mph) winds with the reservoir full.

Gabion mattress linings to the spillway chutes have been specified in preference to the reinforcement fabric and tie rod concept proposed in the preliminary engineering design report. Mesh size for the steel reinforcing fabric would necessitate use of stone comparable in size to the riprap specified for the upstream face of the dam. Such large

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size stone would permit high flow velocities through the stone which would not be well restrained against downslope migration. It is believed that the individual cells and finer mesh openings of a gabion mattress will permit use of smaller stone and provide safety against downslope loss of spillway lining.

We have given considerable attention to the possibility of damage to the dam which would follow should a gabion basket rupture during passage of a large flood over the spillway. Flow velocity will be such that there would then be a danger of the contents of this chamber of a ruptured gabion being washed out thus exposing the gabion bedding to erosion. We propose that the first layer of gabion bedding material will be sized so that it will not pass through the gabion mesh into the (possibly empty) gabion basket above. This outer layer of gabion bedding will therefore remain stable and will protect the dam from further erosion damage.

The head differential across a dam induces permeable flow through the dam foundation. When the dam is built on highly pervious material such as is common in Oman and when the distance of flow beneath the dam is small, the rate of permeable flow beneath the dam may be sufficiently rapid to carry foundation material with it. Speedy collapse of the dam follows. It is necessary to add key trenches beneath the earth embankments because even with the generous side slopes of an earthen embankment, this dam needs additional protection against piping failure.

Small stilling basins lined with gabion mattress have been designed for the downstream toe of overflow sections. This is to transform the fast velocity flow down the slope of the spillway chute into slow velocity by forcing a hydraulic jump within the lined stilling basin. Downstream erosion of the natural wadi materials will therefore be minimized.

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3.4 Outlet Structures

Dispersed outflow for floods insufficient to cause spillway discharge will be accomplished by culverts. Six 48-inch culverts provide a maximum discharge capacity of a little in excess of 50 cubic meters per second. This is expected to be within the natural infiltration capacity of downstream channels as revealed by current meter discharge measurements at Al Khawd and the Coast Road and makes use of the greater infiltration capacity demonstrated by the infiltration tests [Trow-Oman Subsoil Survey (8)].

It is difficult to predict the exact infiltration capacity of the revised system. On the one hand, flow will be prolonged and this is expected to diminish infiltration rate with time. On the other hand, more channels than usual will receive low flows and this will tend to increase infiltration. Judgment indicates that the designed capacity will not overload downstream infiltration capacity. Culvert discharge is not expected to reach the Coast Road. If this judgment should prove to be wrong, stoplog grooves in the upstream culvert head walls may be used to attach restricted orifice openings by which outflow may be throttled. This will lengthen reservoir emptying time and give rise to a small additional evaporation loss.

At the instigation of the Water Resources Council, five additional 48-inch culverts have been provided. It is intended that these outlets will initially be closed but that they will provide a degree of flexibility of distribution as well as discharge magnitude from the retarding structure. Culvert closure will be by stoplogs in the case of three of these additional culverts while two will be provided with sluice gates. These two gated outlets will enable drainage of the reservoir should the six normal outlets become blocked with debris during a flood. They will also provide a means of controlling flow to any experimental downstream recharge devices which might be planned in the future.

Considerable apprehension has been voiced by some members of the Water Resources Council regarding the possibility of blockage of the six active culvert outlets by debris. We have considered this to be unlikely

but must defer to local experience. All culvert outlets will therefore be provided with debris deflectors in addition to the trash racks originally planned. Failure of operation of the culverts would in no way endanger the dam since the additional spillway discharge has been provided for in the design.

The two gated culverts will be kept "normally closed." Consequently, debris will not be drawn to them by rapid flow as the reservoir fills. These may be used to drain the reservoir, should this ever be desired.

Provision of debris deflectors for all culverts and control gates for two culverts has been introduced as a contract option. It is expected that this option will be required by the Oman government.

Culvert head walls have been designed more with the objective of maintaining rapid approach velocity than high hydraulic efficiency. The intention is to keep flows moving as fast as possible in order to move bed load sediment through the dam. An unpredictable result of this will be erosion of the culvert walls themselves by abrasive action, and also, possible abrasion of the gabion baskets downstream of culvert outlets. Should the steel mesh gabion baskets show signs of wear, the gabion fill should be stabilized with cement mortar grout at these locations. This should be monitored by regular annual inspection.

Culverts will all be at the same elevation so that they will discharge equally and maintain uniform distribution of outflow over the active alluvial fan. By maintaining flow in this manner it is expected that recharge will remain distributed much as it has been in the past except that it will be increased in intensity. Adjustment of distribution will be possible by the adjustment of stoplogs. Established groundwater users near the coast will not be adversely affected as they might have been occasionally in the past, when the wadi chose to prefer channels on one side or the other of the alluvial fan.

Culverts closely resemble normal highway culverts in design except for addition of seepage collars. These are added to prevent piping along the outside wall of the culvert when water levels are sustained upstream of the embankment for several days.

Culvert operation will be entirely automatic. No mechanical devices will be necessary and no operating staff required other than for maintenance purposes for a few days each year.

Reliance is placed mainly upon natural streambed infiltration capacity but undoubtedly additional recharge will take place through the reservoir floor. It is expected, however, that fine suspended sediment will be deposited within the reservoir. It will be necessary to regularly remove sediment deposits from the reservoir under the maintenance program described below. Because of uncertainty over the rapidity of sealing of the reservoir floor, it has been assumed that streambed infiltration must be sufficient to provide for all recharge, even though reservoir infiltration will be periodically improved by the maintenance program.

Recognizing a need for research of best methods of recharge, spillway length has been maintained as originally planned for the dam even though it is believed dispersed spillway discharge is a relatively inefficient method of groundwater recharge in this instance. A small cost savings might have been realized had spillway length been shortened achieving the same discharge capacity by constructing a higher dam. It would concentrate flood flows unnaturally over the downstream alluvial fan, might result in the formation of a large eroded downstream channel, and cause severe local flooding where it may not have been experienced before.

3.5 Seepage

Although not critical to success of the recharge system, some seepage is possible through and beneath the dam. Both could appear as surface flow if the sediments downstream of the dam should ever become saturated.

Sieve analyses of the natural surficial wadi materials show fairly uniform size gradation indicating permeability (hydraulic conductivity) of about 387 meters/day. Infiltration tests generally indicate vertical permeability less than this of 1.2 to 14.1 meters/day (8). This is to

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be expected because of horizontal layers of fine material associated with the way in which the alluvial materials were deposited.

Theoretical computation of flow through a dam assumes no loss of water vertically to the dam foundation during transit. Using this simplifying assumption, computed water loss through the dam, which will be constructed of selected natural wadi materials, amounts to about 18.5 cubic meters/second. Toe drains have been provided to pass this amount safely out of the dam.

Seepage of this magnitude is not expected. Initially, flow at the embankment will be primarily vertical since the water table is 24 meters below ground level. Little flow will reach the toe drains. Suspended sediment in reservoir storage waters is expected to rapidly seal both the embankments and reservoir floor. Consequently, the toe drains will continue to receive very little flow either through the dam or through the foundation.

Reservoir emptying times have been estimated on the assumption of zero seepage loss. Marked reduction of these times will indicate the presence of seepage which will either replenish groundwater storage or reappear as flow from the toe drains. Beginning with the reservoir full to spillway elevation and neglecting further inflow, estimated emptying times through six culvert outlets are given in Table 3.2. Data from Table 3.2 may be used in the future if it is desired to investigate the magnitude of seepage losses.

3.6 Wadi Bed Recharge Capacity

The use of the streambed is preferable to construction of special spreading areas for recharge purposes because the streambeds exhibit a capability of retaining their infiltration capacity despite the presence of sediment laden surface waters. It is suspected that the presence of silt in floodwaters would rapidly seal recharge basins. Stanley Consultants' experience (not in Oman), shows this can happen within a few hours.

Infiltration tests show that wadi beds are generally quite permeable and that high ground near wadis, although less permeable,

TABLE 3.2

ESTIMATED RESERVOIR EMPTYING TIMES
(ASSUMING ZERO INFLOW AND ZERO SEEPAGE
OR INFILTRATION LOSS)

Pool Level Below Spillway (m)	Time to Reach Pool Level (hours)
0	0.0
0.5	6.3
1.5	18.8
2.5	31.2
3.5	43.5
4.5	55.1
5.5	65.8
6.5	78.3

Source: Stanley Consultants

will still provide for some infiltration. Further downstream where velocities have been slowed by obstructions near the coast roads, generous silt deposits are to be seen. Silt remains in suspension when velocities are high; partially seals the alluvium when some infiltration takes place in less rapidly moving water; and may completely seal the alluvium when velocities are reduced by impoundment but has very little sealing effect in the wadi beds.

It is believed that a high rate of infiltration is maintained when velocity is high and that although silt may be drawn into the bed by infiltrating flow, the bed itself is in motion and that deposited sediment is thus resuspended. Two options are available to obtain the same result in spreading areas. Either flow must be sufficiently rapid to erode the surface of the recharge area or the surface must be periodically removed by artificial means. Both of these options involve continuing maintenance costs.

Construction of the retarding basin will change the nature of flow over downstream channels. Flow will be reduced in magnitude during small floods because of the restricted culvert outlets, but will also

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result in reduction in sediment load. A tendency toward erosion in the stream channels downstream of the retarding dam is expected. This will tend to keep fine sediment in motion and retain infiltration capability of the streambeds. (The special problem of erosion, and a solution which will not detract from streambed infiltration is described later in this report under Part 4 - Maintenance Procedures.)

3.7 Flood Control

Retention and reduction of flood peak outflow by the retarding dam carries with it flood control benefits. There is an accompanying danger which should be pointed out. This is well demonstrated by the results of flood routing presented in Table 3.3. The danger associated with flood control aspects of the recharge system is that many years may pass before a high flood occurs and people may be lulled into the false belief that construction of the dam has produced complete immunity from downstream flooding.

Table 3.3 shows that while protection will be excellent up to the 10-year level and fair up to the 25-year level, flood peak reduction at the 100-year level will be relatively minor.

Construction of the recharge dam will not remove the necessity for control of downstream development.

TABLE 3.3

FLOOD CONTROL
WADI AL KHAWD RECHARGE SYSTEM

Recurrence Interval (years)	Inflow to Reservoir	Outflow from Reservoir
10	525 (cms)[1]	54 (cms)[1]
25	643	180
50	739	430
100	828	620

[1]Cubic meters/second

Source: Stanley Consultants

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3.8 Recharge Monitoring and Development Planning

It is to be expected that increased surface water loss to groundwater recharge following construction of the recharge project will be strongly evidenced by marked reduction of flood volume from Wadi Al Khawd overflowing the Coast Road. If the outlet capacity of the dam culverts has been set a little too high, small flows will continue to be observed over the road and this may be adjusted, if desired, by throttling the culverts by inserting stoplogs in the inlet head walls.

Although use of wadi beds as the recharge medium is believed to be highly efficient, there will be a need to measure actual groundwater increase. This will be important both for design of a water supply system and for research purposes so that design of recharge systems elsewhere in Oman may be benefited. The actual recharge capability of the project is best determined by monitoring stream flows discharging into the reservoir at the former gaging station at Al Khawd and the water levels in the reservoir.

If the additional groundwater made available by the recharge system is not recovered, it will merely be lost to the sea. Design of the system, to productively utilize the water system and its subsequent operation, will be facilitated by advance groundwater monitoring. This should take the form not only of observation wells but also of profile samplers so that the effect upon quality changes may be determined. In addition to designing the recovery facilities, pumping tests will be required to measure hydraulic properties of the aquifer.

Groundwater modeling was not within the scope of this report. However, the Tetra Tech report (4) indicates that probable best location of any additional groundwater pumping capacity is a gallery midway between the dam and the Coastal Road. Consequently, observation wells, sampling wells, and future pumping tests should be planned with this future development in view.

The baseline data gathering function can be divided into three principal groups briefly mentioned in the foregoing. They are: 1) a system to measure recharge basin inflows and outflows in order to

determine actual recharge volumes, 2) a system to monitor groundwater levels and quality in the coastal plain north of the recharge basin to assess the impact of the recharge activities and 3) well pumping tests to obtain reliable aquifer characteristics to design future wells in the recharged area. It should be noted that these groups are not listed in order of priority or chronological sequence. They are grouped by function and objectives only. Methodology, manpower and equipment needs, and costs are briefly summarized in the following paragraphs to provide guidelines for an effective monitoring program.

Methodology for group one consists of reestablishing the stream flow gaging stations at Al Khawd and the Coastal Road and monitoring water levels in the reservoir. Stream flow volumes at a gaging station are obtained by continuous monitoring of water levels at a stable cross section. A stage-discharge relationship is developed over a number of years by current meter measurements of river discharge at various levels as they occur. This has been made difficult in the past because stations have been washed out in floods or personnel were not adequately trained (6). Efforts should continue to overcome these difficulties.

The discharge through culvert outlets of the dam and the flow over the spillway is uniquely related to the reservoir level. Thus, if levels are recorded, the discharge volume can be computed and recharge estimated as the difference between inflow and outflow volumes. Continuous recordings of stream and reservoir levels are required for accurate calculation of basin recharge volumes.

Instrumentation required for this type of monitoring consists of a float actuated recording device, a stilling well, and an instrument shelter for security. The simplest device is a horizontal drum recorder controlled by spring or weight driven clock or battery powered electric clock. Spring or weight driven clocks will operate remotely for up to one month, while electric clocks can run for six months or more. Flow actuated devices are often used in remote installations with intermittent flows. The instruments would be inspected weekly or monthly, charts changed, and clocks rewound or batteries replaced.

Chart records are manually read and converted to discharge volumes in the office. Over a period of years the data would not only help to establish the viability of the recharge basin concept, but would contribute to a better understanding of the hydrology and water resources of the area in general.

Group two consists of a recommended grid of up to twelve groundwater monitoring wells in the wadi fan north of the dam. The grid should be rectangular with wells uniformly spaced at least one kilometer apart. One row of perhaps three wells should be placed in the garden area north of the Coastal Road. Another row should be placed approximately 1 km south of the Coastal Road. The center well of these two rows should be a zoned monitoring well described below.

Ten of the twelve observation wells need only be 100 mm diameter (maximum), drilled to at least 50 meters depth, screened in the lower one to two meters. The steel or plastic well casings should be extended at least one meter aboveground, grouted at the surface to prevent surface water leakage to the well, and covered securely. The primary purpose of these wells is to monitor groundwater levels on a monthly basis or more frequently initially. The water levels obtained prior to completion of the recharge basin will help to establish baseline data for evaluating the impact of recharge on groundwater in the garden area. Data obtained subsequent to the project completion would track fluctuations in the groundwater supply available for existing and future development. Occasional grab samples could track long-term trends in any water quality parameter of interest.

The zoned wells are required if the impact of the project on salinity in the garden area is to be evaluated. A zoned well is essentially an ordinary well which has been backfilled alternately with samplers imbedded in sand at desired levels and separated by compacted bentonite clay seals. The two wells should be 200 mm diameter and driven to bedrock (approximately 100 m).

A patented system that would be recommended for this situation employs samplers that are driven by compressed gas (usually nitrogen) from the ground surface. Samplers should be placed at five meter

intervals from the bottom of the well to within five to ten meters of the lowest expected water level.

The equipment needed to collect a water sample is an adjustable wrench, a hand-carried tank of compressed gas, a pressure regulator, and sample bottles. All the samples could be collected from both wells in several hours. Water quality variation with depth can be determined with this data. Useful information regarding allowable well depths and pump capacities would be gained from this data. Frequency of sampling is recommended to be monthly during and immediately following the construction of the dam and less frequently thereafter. These wells can also be used to obtain groundwater levels.

Water levels in the observation wells can be obtained by several simple methods. One of the best and simplest methods is to drive compressed nitrogen gas through a 4 to 8 mm diameter copper tube attached to the casing and extending well below the water surface. The nitrogen line is attached to a pressure gage, and the compressed gas is bubbled into the water until a constant reading is obtained. The reading is the depth of water above the bottom of the gas supply tube, which extends to a known depth in the well. Hence, the water level can be computed. Lightweight, convenient battery operated wire reels with electrode and contact indicator as well as chalked measuring tapes are also commonly used.

The third group, the well tests, are recommended prior to developing a new well field for water supply. The probable best location for a line of wells, according to the Tetra Tech (par. 3.1.4) report is gallery B-B' (p. 26) located about 1 km south of the Coastal Road. Initially, at least one production well should be installed in this area. It should be similar to the recent wells installed in the Seeb well field. Two pairs of small (50 mm) diameter observation wells spaced at 25 and 50 m radially outward from the wells are used to establish drawdowns during an extended three-day pump test. The aquifer

hydraulic properties obtained from these tests are necessary for the proper design and placement of the well field.

Estimated costs for the monitoring systems are summarized below:

Group 1:

Drum recorders, stilling well, instrument shelter; 4 @ 1,100	=	4,400
Current meter	=	600
Miscellaneous survey equipment	=	100
Total Cost	=	5,100 R.O.*

Group 2:

100 mm monitoring wells to 50 m depth; complete with screen and casing; 10 @ 1,850	=	18,500
200 mm monitoring wells to 100 m depth; complete with screen and casing; 2 @ 2,450	=	4,900
Samplers; 30 @ 150	=	4,500
Install samplers in two wells @ 5 m spacing	=	1,900
Engineering	=	2,900
Contingencies	=	3,300
Total Cost	=	36,000 R.O.*

Group 3:

100 m of 400 mm dia. well, 30 m of 300 mm, dia. steel casing, 70 m of 300 mm stainless steel screen, gravel packed and grout seal	=	13,000
3 day well pump test	=	4,500
Engineering	=	1,500
Contingencies	=	
Total Cost	=	21,000 R.O.*

*All costs in Rials Omani (R.O.) 10/81

The primary manpower need for the recharge monitoring is one or two well-trained technicians and maintenance support personnel.

3.9 Summary

The preceding portions of this section of the report have discussed design of the structures to capture and use flood flows. Design computations are included in Appendix C.

3.10 References

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PART 4 - MAINTENANCE PROCEDURES

4.1 General

Hydraulic structures are not maintenance free. Attempts have been made to reduce estimated maintenance costs for the Wadi Al Khawd recharge system but it is important to understand that regular maintenance will be absolutely essential to successful continued operation.

Four areas will require constant attention. These are:

1. Removal of accumulated sediment deposits, particularly from within the reservoir area.
2. Removal of trash accumulations at culvert inlets.
3. Transfer of coarser material around the dam to compensate for expected downstream channel erosion.
4. Monitoring and repairing erosion damage. This may be experienced in the riprap, in the gabion spillway lining, in the stilling basin below culvert outlets, and inside the culverts.

Successful operation will require budgeting for regular attention and an adequate staff.

4.2 Sedimentation

It is believed that no sediment load measurements or estimates exist for Oman. In order to estimate sediment removal requirements, it has been necessary to develop projections based upon measurements in other parts of the world.

Measured sediment deposits in arid states of the USA were used to estimate conditions for the Al Khawd area. Use of data from Piute Reservoir in Utah, McMillan Reservoir and Elephant Butte Reservoir in New Mexico, Lake Mead in Nevada and Ericson Reservoir in Nebraska, result in a preliminary prediction of 53 cubic meters per year of sediment for each square kilometer of drainage area.

Thus, an estimated average sediment load of nearly 85,000 cubic meters per year can be expected at Al Khawd. Clearly, as water is lost

by infiltration into the alluvial fan, its sediment transport capability diminishes. Consequently, an estimated 76,400 cubic meters per year should arrive at the retarding basin. Sediment transfer through the dam will be aided by means of culverts but only an estimated 10 percent of the arriving sediment will pass through the system naturally. This means 90 percent of the sediment or 70,000 cubic meters/year to be removed by excavation.

Sediment will not be deposited at a uniform rate. Most years there will be little or no deposition but in years of large floods, substantial sediment deposits must be expected. If these are not removed during the following low flow period, there will be a steady loss of storage capacity.

Initial storage is adequate so that accumulation of a decade might have little effect on capacity. Ultimately, since success of the system depends upon regulation of flood outflow which requires that high flood inflows be stored, allowing sediment buildup will result in lessened recharge and hence somewhat reduced water supply.

4.3 Sediment Bypassing

Active channels in the alluvial fan should operate in a state of quasi equilibrium with regard to sediment transport. If the supply of sediment becomes reduced because of deposition in the upstream reservoir, spillway outflow of relatively sediment-free water will have the capability of transporting more sediment than it carries. Erosion is therefore to be expected at the downstream toe of the dam.

No special provision for erosion control at this point has been made in the dam design. It is anticipated that the depressed stilling basins will be able to absorb some downstream erosion. But if this is allowed to continue, the downstream end of the stilling basin will collapse thus destroying its effectiveness.

Large stones are expected to be deposited at the upstream end of the reservoir. These stones should be transported around the dam to provide continued maintenance of the stilling basin end sill. This is a matter requiring close attention for the first few years of operation until an adequate reserve of large sized stone has been placed downstream of the gabions.

Some smaller sized stones will be passed through the culverts. These may abrade the surface of the culverts and might damage the gabion baskets immediately below culvert outlets. Regular inspection both of the interiors of the culverts and also of gabions at culvert outlets is recommended.

The possibility of rupture of gabions on the spillway face is relatively remote but, should it occur, dam safety will be threatened. Close attention to the integrity of the gabion spillway lining, particularly near the spillway crest should be maintained and repairs made promptly.

Culvert inlets should be kept clear of trash and debris on a regular basis.

4.4 Engineering Inspection

Maintenance items other than those mentioned above should not require much attention. It is, however, necessary that the dam be thoroughly inspected on an annual basis by an engineer experienced in dam construction who will report on all items requiring attention and initiate maintenance work. This responsibility should be assigned to a senior dams engineer in the Ministry of Agriculture, Department of Water Resources. This effort will entail only a few days work each year.

4.5 Maintenance Equipment

The embankment dam has been designed similar to most highway embankments reinforced by the special features associated with impoundment and spillway overflow.

Normal highway construction equipment will suffice for maintenance purposes.

4.6 Summary

Design features and maintenance procedures have been discussed. Maintenance and operation manuals will be prepared during construction under the contract for construction supervision and control. The following section of the report will summarize project economic investigations.

PART 5 - ECONOMIC INVESTIGATIONS

5.1 Project Background

The desire to transform an economy and the availability of the resources to do so has led to a phenomenal development pace for Oman in the last decade. This accelerated pace forces compression of normal project evaluation-design-construct sequences and has necessitated significant decision-making prior to the availability of fundamental types of data. As a result, traditional modes of feasibility analysis are not generally used to guide the development process.

The rush to develop and the lack of hard data can lead to widely divergent expectations and justifications for a project. The Wadi Al Khawd aquifer recharge project is a classic example. Various reconnaissance level investigations have provided estimates ranging from 1.5 to 6.0 million cubic meters per year of water yield. Expected beneficial impacts have varied from heavy dominance of agricultural water use to concentration on urban water supply. Most Omani officials interviewed felt that the primary project purpose was to demonstrate the possibility of the recharge concept. The information value alone justified the needed expenditure.

This multitude of project perceptions and compressed analytic sequence critically influence the economic analysis of the project in several ways:

1. No firm or unanimous project purpose has been established. Thus, some basic policy assumptions regarding potential use of the water increment had to be made for purposes of the analysis. The approach used here is to allocate the water to its most beneficial (highest valued) use. Eventual Omani choices of the preferred water use may or may not conform to that assumption.

2. Final decisions regarding use of the water may be made by the Omanis after some years of monitoring establishes the actual yield. For that reason, it is inappropriate to develop the same level of design detail for well fields, pumping, treatment and transmission facilities as is being developed for the dam at this time. These ancillary facilities have been estimated on a conceptual basis for the purpose of evaluating the feasibility of urban water supply. An eventual decision to develop that supply will require additional design efforts based on proven yields.
3. Omani goals for the project clearly include both economic provision of needed water and the demonstration that aquifer recharge is technologically feasible. Economic optimization dictates project sizing to maximize net benefits. Technological demonstration, however, normally attempts to establish the maximum practical extraction achievable using a technique. The composite objective used in evaluation of this project is to seek the maximum recharge which can be economically justified. Smaller dams will be examined only if the maximum capture structure proves infeasible.

Given this project background, the economic analysis has been directed at the following fundamental questions:

1. What is the highest valued use of the water increment?
2. What are the economic benefits of that use?
3. What rate of return does the project yield when these benefits are compared to project costs?
4. What is the minimum acceptable rate of return?
5. Can the project be justified economically?

5.2 Use of the Water Increment

The hydrologic analysis, presented in Part 3, shows that this project is expected to capture an additional 3.0 to 5.0 million cubic meters (mcm) per year. To be conservative, the economic analysis assumes 4.0 mcm are available for use annually.

The two general purposes which have been considered for the water are:

1. To combat saline intrusion in the private wells which are used to irrigate the coastline gardens at Seeb.
2. As a new groundwater source to feed the urban distribution system which serves the capital area.

The economic analysis of these two uses differs substantially. For irrigation purposes, the economic benefit is the net income which can be derived through sale of the produce made possible by the availability of the water increment. For urban water supply, the economic benefits are the marginal costs of the alternative sources displaced by this project. For example, the variable operating costs of the desalination plant at Ghubra need not be incurred to the extent that this project substitutes for that plant. Those cost savings are then weighed against the recharge project costs to test feasibility.

The choice between uses has been based on the following analysis of the value of water for the two uses. The agricultural area in the alluvial fan where the recharged aquifer can augment well supply without added conveyance has been estimated to contain about 380 hectares. The crop distribution in this area is limited to fruits and vegetables. The hectarage distribution by crop has been taken as equal to that for the Batinah Coast. A very detailed study of the agricultural sector has recently been completed by HVA (Dutch Consultants) for the Ministry of Commerce and Industry. That study developed the best available estimates of the cost of production and possible sale price for each crop. The difference is net income. Combined with yield data, this information provides an estimate of net income per hectare for each crop. Weighing the net income figures, as shown in Table 5.1, by the crop distribution provides an average net income figure of 2,417 R.O. per hectare for the 380 hectares in the impact area. This figure is expressed in 1980 prices and assumes farming takes place in large units (vegetables - 65 hectares, fruit - 120 hectares) with experienced management to obtain optimal yields. Present yields are estimated at about one-third of the optimal level and are

TABLE 5.1

AGRICULTURAL BENEFIT DATA

	Batinah Hectares	Percent	Water Use (m^3/ha)	Yield Present	Yield Optimal	Net Income/ Hectare Optimal
Fruit						
Dates	10,226	62.1	18,670	3.7	10.0	928
Mango	2,604	15.8	12,130	1.1	25.0	8,873
Lime	1,323	8.0	14,200	10.0	15.0	1,560
Banana	1,470	8.9	22,500	16.0	45.0	1,897
Papaya	46	0.3	14,200	10.2	25.0	710
Other	76	0.5	14,200	18.0	25.0	710
Vegetables						
Onion	209	1.3	5,785	10.0	30.0	917
Watermelon	256	1.6	6,015	12.5	40.0	4,404
Tomato	94	0.6	3,790	12.5	50.0	4,670
Garlic	25	0.2	6,000	7.5	10.0	2,713
Sweet Potato	27	0.2	5,000	15.0	20.0	3,000
Other	106	0.6	5,000	10.0	20.0	3,000
Total	16,462	100.0				
Weighted Average			17,009	5.33	17.08	2,417

Source: HVA Study, Appendix II.F, P. II.185-II.186, and Stanley Consultants

expected to rise to no more than one-half the optimal level by 1985. Thus, actual net income per hectare expressed in current (1981) prices using 1985 yields can be estimated as:

2,417 R.O. x 1.10 x 0.5 = 1,329 R.O.

The weighted average of annual water requirements per hectare for the Batinah crop distribution is 17,000 cubic meters. Thus, 4.0 mcm would support about 235 hectares of crops in the impact area. This provides total net income of 235 x 1,329 R.O. = 312,315 R.O. or 78 baiza per cubic meter of water.

The Phase II Power and Urban Water Supply Study (April, 1979) provides estimates of the operating costs for the existing desalination plant which will be the marginal source. Table 5.2 shows the 1981 variable operating cost for the existing plant as estimated for the lowest cost alternatives for meeting the medium demand growth projection (schemes M4 and M5 in that report). Price levels used in that evaluation are for 1979. Updating the costs at 10.0 percent per year to 1981 yields an estimate of:

$$248 \times 1.10 \times 1.10 = 300 \text{ baiza/cubic meter}$$

Comparison of this benefit with the irrigation figure (78 baiza/cubic meter) demonstrates the clear economic preference for using the water for urban water supply. Admittedly, urban use will entail somewhat higher project costs to feed the water increment to the grid. Costs will not, however, be four times as high so the benefit-cost ratio for urban supply will undoubtedly be greater than for irrigation. On this basis, the economic analysis assumes the increment of 4.0 mcm is supplied to the urban grid.

TABLE 5.2

EXISTING DESALINATION PLANT
OPERATING COSTS

Total Production	6.21 mcm
Total Production Operating Cost (R.O. 000)	1,535
Fixed Operating Cost (R.O. 000)	262
Variable (R.O. 000)	1,273
Variable Treatment Cost (R.O. 000)	267
Total Variable Cost	1,540
Variable Cost per Cubic Meter	248 baiza

Source: Phase II Power and Urban Water Supply, Tables D.24 and E.20, and Stanley Consultants

The steps in evaluating the economic feasibility of urban water supply include:

1. Estimation of water demand in the areas served or to be served by the urban grid.
2. Identification of the supply alternatives and costs if the recharge project is not undertaken.
3. Estimation of project benefits which equal the cost savings from not operating the displaced marginal source.
4. Estimation of project costs including capital outlays and annual operating and maintenance costs.
5. Computation of internal rate of return.
6. Sensitivity of the rate of return to major assumptions.

5.3 Water Demand Projections

In recent years, nearly two-thirds of water use in the capital area was for domestic purposes. Commercial, industrial, and government/municipal entities use the remaining one-third. Thus, the key determinant of future demand is population growth.

The major factors influencing population growth in the capital area are well understood. Government adoption of an aggressive growth program has necessitated immigration of numerous expatriates to provide a sufficient labor force to implement projects at the desired rate. Second, the natural rate of increase of the native Omani population is quite high reflecting improved health care and a desire to expand the Omani population. Finally, the capital area has grown very rapidly as many Omanis have abandoned rural areas to obtain the higher incomes available in the urban center.

Firm quantification of this growth experience is impossible since no census has ever been published. The population growth forecasts which underlie the water, power, and housing supply planning efforts were accomplished by field counts of housing units and spot checks on the persons per housing unit.

The Phase I Power and Urban Water Supply Study provides the range of population in permanent housing projections shown in Table 5.3. The capital area runs from Muscat through Seeb and Bustan.

TABLE 5.3

CAPITAL AREA POPULATION PROJECTIONS[1]
1977-1990

Projection	1977	1980	1985	1990	1995
Low					
Number	98,600	106,700	138,300	169,200	206,600
Annual Rate		2.7	5.3	4.1	4.1
Annual Rate		----------	----------	4.5	----------
Medium					
Number	98,600	123,600	178,000	240,200	293,600
Annual Rate		7.8	7.6	6.2	4.1
Annual Rate		----------	----------	5.9	----------
High					
Number	98,600	143,300	232,500	313,300	401,600
Annual Rate		13.3	10.2	6.2	5.1
Annual Rate		----------	----------	7.1	----------

[1]Excludes those in labor camps and temporary accommodations.

Source: Power and Urban Water Supply Study, Phase I, Tables 3.1 to 3.3, and Stanley Consultants

The most recent population estimate for the capital area is based on a detailed housing inventory completed for the Ministry of Land Affairs and Municipalities in the spring of 1981. Key findings from that survey include:

1. Total population is considerably larger than has generally been recognized and is estimated at 225,000 without the military installations.
2. The incidence of temporary and substandard housing is higher than was expected.
3. The estimated population growth rate for 1981-86 being used for planning housing is about 7.5 percent per annum. Housing

would have to grow faster than that to reduce the number of substandard or temporary units. It is thought that about 3,000 to 3,500 new housing units per year are needed.

4. Persons per housing unit range from 5.5 to 8.1 with an overall average of 7.45.

Planners who conducted the 1981 survey indicate that this evidence corroborates the medium to high projections of population in permanent housing. The 1980-1985 growth rate in the medium projection of 7.6 percent conforms nicely with the 7.5 percent being used for housing planning from 1981-1986. The biggest problem indicated by the recent evidence is in deciding on appropriate service expansion to those currently in temporary or substandard housing. Basic government objectives include "Omanization" and stabilization of the rural sector. Success in these programs suggest that some of the temporary housing in the capital area will not be replaced by permanent dwellings.

Based on a World Bank mission review in 1978, the low and medium demand projections were selected for use in water supply planning. The recent survey suggests that even with Omanization and rural stabilization programs, the medium forecast is more plausible. Hence, the medium demand forecast as shown in Table 5.4, is adopted for this study. Low and high projections are also shown for comparison.

Water demand growth from 1980 to 1995 is more rapid than population in permanent housing growth because:

1. System expansion eliminates suppressed demand.
2. Some service may be provided to the large segment of temporary housing.
3. Per capita usage rises with income expansion.

Domestic consumption per capita per day is estimated at 263 liters by 1995 which is about 70 percent of the figure normally used for planning in the United States.

Selection of the appropriate demand projection is not critical in determining the feasibility of the Al Khawd recharge project since the project's water increment is small compared to total demand growth. For

TABLE 5.4

CAPITAL AREA WATER DEMAND PROJECTIONS

Forecast	1980[2]	1985	1990	1995
Low				
Cubic Meters/Day[1]	23,170	40,880	48,380	64,540
Growth Rate (%)	12.0	3.4	5.9	
Growth Rate (%)	7.1			
Medium				
Cubic Meters/Day[1]	25,730	58,930	74,920	108,110
Growth Rate (%)	18.0	4.9	6.2	
Growth Rate (%)	10.0			
High				
Cubic Meters/Day[1]	27,580	75,200	104,950	166,220
Growth Rate (%)	22.2	6.9	9.6	
Growth Rate (%)	12.7			

[1]Represents average daily demand.

[2]Reflects suppressed use due to system limitations.

Source: Power and Urban Water Supply Study, 1979, Table A.3, and Stanley Consultants

example, the 11,000 cubic meters per day provided by the recharge project is equal to about 2.0 years of demand growth beyond 1980 in the medium forecast. Even in the low forecast, it would only satisfy about 3.5 years of growth beyond 1980.

In consideration of capacity expansion needs, peak demands are relevant rather than average demands. Based on historic usage patterns, peak demand is estimated at 125 percent of average demand.

5.4 Supply Alternatives

To meet the rapid demand growth, water supply planning has focused on the following objectives:

1. To bring sufficient groundwater capacity on-line to allow the needed annual maintenance shutdowns of the existing desalination plant at Ghubrah.
2. To identify major new sources which will provide base load capacity to accommodate expected growth.
3. To identify desirable extensions of storage, transmission, and distribution systems.
4. To identify the optimal additions and use of desalination capacity at Ghubrah.

The major sources of new supply which have been evaluated are summarized in Table 5.5. Wadi Mayh is a small resource close to the capital area. A new well field there would be required to supply the urban grid. The most proximate major groundwater source on the Batinah Coast where water balances would allow significant extraction for urban supply is between Khabura and Sohar, a distance of 150 to 180 kilometers from the capital area. The Wadi Dayqah resource would be captured by a high dam and large reservoir approximately 90 kilometers from the capital area. A wide range of desalination plants has been examined as indicated in Table 5.5.

It is educational, though not precise, to compare the capital cost and annual cost per cubic meter of water for these various resources. Table 5.6 provides the comparison. Capital costs per unit for major new groundwater sources at distant locations are comparable to those of new desalination plants. Desalination, however, is clearly more costly on an annual basis because of heat and fuel requirements.

TABLE 5.5

POTENTIAL NEW WATER SUPPLY RESOURCES FOR CAPITAL AREA

Resource	Base Yield (m^3/day)	Peak Yield (m^3/day)	Annual Yield[1] (mcm)	Capital Cost[2] (R.O. 000)	Annual Cost[2] (R.O. 000)	Distance to Grid (km)
Wadi Mayh	5,500	7,000	2.1	1,910	74	5
Batinah	25,000	34,000	10.0	26,080	698	150
Wadi Dayqah	36,000	45,000	14.0	32,190	942	90

Desalination Plants	Rating (m^3/day)	Optimal Load Factor	Annual Yield (mcm)	Capital Cost[2] (R.O. 000)	Annual Cost[3] (R.O. 000)	Location
1	14,000	75	3.8	8,394	1,197	Ghubrah
2	18,750	75	5.1	9,337	1,568	Ghubrah
3	24,000	75	6.6	13,017	1,830	Ghubrah

(1) Based on use of base load nine months per year and at peak capacity three months per year.

(2) Based on January, 1979, price levels.

(3) Based on 1995 operating cost estimates for the smaller units and 1994 for the larger unit. They are still based on 1979 price levels but incorporate some real price increase in power costs.

Source: Power and Urban Water Supply Study, 1979, P. 15-2, 15-3, Tables E-19, E-20, and Stanley Consultants

TABLE 5.6

COMPARATIVE COSTS FOR AVAILABLE RESOURCES

Resource	Annual Yield (mcm)	Capital Cost (Bz/m3)	Annual Cost (Bz/m3)
Wadi Mayh	2.1	910	35
Batinah	10.0	2,608	70
Wadi Dayqah	14.0	2,299	67
Desalination Unit			
1 - (14,000 m^3/day)	3.8	2,209	315
2 - (18,750 m^3/day)	5.1	1,831	307
3 - (24,000 m^3/day)	6.6	1,972	277

Source: Table 5.5 and Stanley Consultants

The supply plans which are appropriate to test the Al Khawd recharge project against are various combinations and sequences of introducing the resources shown in Table 5.6. Table 5.7 summarizes the plans evaluated in the Phase II Water Supply Study. Although some schedule slippage has occurred, the government is proceeding to add a desalination plant at Ghubrah and has set aside 30 million R.O. for the Wadi Dayqah project. They are pursuing plan M5 although new discoveries in Wadi Adai could alter these plans in later years. Despite the possible changes, scheme M5 remains the lowest cost plan which has been analyzed in sufficient detail to assure the possibility of implementation. As such, plan M5 has been taken for the base case against which the recharge project is tested.

5.5 Project Benefits

Given the disparity in size between Al Khawd recharge and the other projects and the perceived uncertainty of supply from the recharge plan, it appears unlikely that the other projects would be delayed or cancelled in deference to Al Khawd. Rather, Al Khawd is likely to be considered as a backup source which must "prove" its potential yield. If possible, monitoring wells and gauging might be

TABLE 5.7

NEW RESOURCE SCHEDULE FOR VARIOUS SUPPLY PLANS

Year On-Line	Plan				
	M1	M2	M3	M4	M5
1981	Wadi Mayh	Wadi Mayh	Wadi Mayh	Wadi Mayh	Wadi Mayh
1982	Desal 18750	Desal 18750	Desal 18750	Desal 18750	Desal 18750
1983	Batinah	Wadi Dayqah	Desal 24000	Batinah	Wadi Dayqah
1984					
1985			Desal 24000		
1986	Desal 16000			Wadi Dayqah	
1987					
1988					Batinah
1989		Desal 16000	Desal 24000		
1990	Desal 16000				
1991					
1992	Desal 16000	Desal 16000			
1993			Desal 24000		
1994	Desal 16000	Desal 16000		Desal 14000	Desal 14000
1995					
Total Unit Cost of Water ($Baiza/m^3$) (1)	480	460	549	448	442

(1) Based on 1979 prices and present values of all costs for each alternative at 10%.

Source: Phase II Power and Urban Water Supply Study, 1979, Tables D-10-D-4, Tables E-12-E-20

used to establish what water increment has been captured and what can safely be extracted. Since it may not be regarded initially as a firm supply source, the primary economic benefits to the recharge project consist of the variable operating costs which can be avoided if Al Khawd provides 4.0 mcm per year to the urban grid. No capital or maintenance costs for other sources are treated as avoidable in determining project benefits.

Two other benefits to the project can be claimed although no adequate data are available to quantify them. Flood damage reduction attributable to the dam is expected to be significant. The May, 1981, flood accounted for substantial damage. A partial tabulation of damage at Seeb includes:

- 6 lives
- 600 houses
- 20 shops
- 10 workshops
- 5 boats
- unspecified road damage

Loss of the gauging equipment in the flood makes it difficult to determine the return frequency of the May, 1981, flood so no average annual figure can be reasonably constructed. The second unquantified benefit will accrue to well users along the coastline. Extraction is not expected to fully deplete the increment of water added to the aquifer. Any net increment should beneficially impact wells that have experienced saltwater intrusion. Hectarages and wells to be benefited cannot be identified with any precision so benefits are not quantifiable.

It is additionally important to note that all the water supply plans evaluated have included abandonment of Wadi Samail as an urban water supply source by 1983. Such abandonment would leave another 2.0 mcm which is currently being extracted from Samail well fields to combat the saltwater intrusion in coastline wells. This 2.0 mcm contribution is not a benefit attributable to the recharge project but is consistent with the benefits estimated below.

The estimation of benefits from cost avoidance is based on the following assumptions:

1. Economic life of the dam is 30 years which is believed to be a conservative estimate.
2. Cost savings are analyzed for the period 1983 through 1997. Projections beyond that time frame are highly variable and discounting procedures diminish the importance of remote benefits. Construction should be completed in October, 1983. Yield for 1983 is thus estimated to be 4.0/4 = 1.0 mcm.
3. A benefit equal to the residual value of the project in 1998 is claimed assuming straight line depreciation over 30 years.
4. Consistent with the supply plans, economic load scheduling calls for displacement of desal production first since it is the most expensive source. Output from the existing desal plant would be eliminated first, followed by the other desal units.
5. Cost savings are based on the Water Supply Planning Study and are thus computed using 1979 price levels. Annual escalation of 10.0 percent has been used to adjust the savings to a 1982 price level. No further escalation is included.
6. Fixed operating and maintenance costs are equal to the annual costs shown when the unit produces no output. All other annual costs are considered variable.
7. Post-distillate treatment cost is reduced by the proportion of desalination eliminated.
8. If total desalination plant production in a future year does not equal 4.0 mcm, the highest cost groundwater or reservoir source will also be displaced to make up the 4.0 mcm that Al Khawd is expected to provide.

9. For 1996 and 1997 production scheduling of the desalination plants are assumed equal to 1995 scheduling. This could only be changed by adding new resources to the plan in those years.

Benefit estimates are derived in Tables 5.8 through 5.10 which correspond to the three alternate sources which would be displaced by the Al Khawd recharge project if plan M-5 is implemented. Since this plan was the lowest cost solution, comparison with any other plan would yield higher benefits. Benefit estimates are combined in Table 5.10 to obtain total benefits based on 1979 price levels. These benefits are then adjusted to a current price base for comparison with project costs. The benefits shown in Table 5.11 will be used in conjunction with project costs to derive an internal rate of return in Section 5.7.

5.6 Project Costs

A complete discussion and breakout of the aquifer recharge project cost estimate is provided in Part 9 of this report. This section presents conceptual cost estimates for ancillary facilities needed to pump the water increment to the grid, identifies annual operation and maintenance costs, and constructs the expenditure schedule through 1997.

Table 5.12 presents the estimated project costs for:

1. A new well gallery downstream of the recharge dam.
2. Piping to collect the water.
3. A surface storage reservoir.
4. A diesel generating station.
5. A connecting main to the 600 millimeter main along the dual carriageway.

Costing has been prepared on a consistent basis with the alternative plans using prices expected for January, 1982. All costs in this section include debris deflectors and slide gate assemblies which are treated as additive bid items in PART 9. No distinction has been made between cash costs and economic costs since the two are

TABLE 5.8

BENEFITS FROM COST SAVINGS COMPARED TO PLAN M5
FIRST MARGINAL UNIT

Year	Production (mcm)	Production % of Desal	Annual Cost Fixed	Annual Cost Variable	Annual Cost Total	Treatment Cost Total	Treatment Cost This Unit	Variable Cost Total	Variable Cost Bz/m^3	Production Displaced (mcm)	Benefit
1983	3.61	63	262	708	970	287	181	889	246	1.00	246
1984	0.00	0	262	0	262	196	0	0	0	0.00	0
1985	2.75	40	262	689	951	309	124	813	296	2.75	813
1986	3.47	44	262	847	1,109	345	152	999	288	3.47	999
1987	3.89	43	262	964	1,226	383	165	1,129	290	3.89	1,129
1988	5.06	50	262	1,255	1,517	423	212	1,467	290	4.00	1,160
1989	0.00	0	262	0	262	0	0	0	0	0.00	0
1990	0.00	0	262	0	262	0	0	0	0	0.00	0
1991	0.00	0	262	0	262	0	0	0	0	0.00	0
1992	2.91	41	262	927	1,189	0	0	927	319	2.91	928
1993	4.30	46	262	1,331	1,593	157	72	1,403	326	4.00	1,304
1994	3.65	30	262	1,220	1,482	290	87	1,307	358	3.65	1,307
1995	5.84	39	262	1,929	2,191	385	150	2,079	356	4.00	1,424
1996	5.84	39	262	1,929	2,191	385	150	2,079	356	4.00	1,424
1997	5.84	39	262	1,929	2,191	385	150	2,079	356	4.00	1,424

NOTES:

Marginal Unit: Existing desal plant upgraded to 23,000 cmd.

1. All costs and benefits expressed in R.O. 000 unless specified.
2. Distillate post treatment costs are prorated to units based on desal production.
3. Total variable cost equals variable annual cost plus unit's share of treatment cost.
4. Benefit equals displaced production times variable cost per unit.
5. Costs and benefits have been based on January, 1979, price levels.
6. Production scheduling of the marginal unit based on plan M5 in Water Supply Study.

Source: Power and Urban Water Supply Study, 1979, Tables D-24 and E-20, and Stanley Consultants

TABLE 5.9

BENEFITS FROM COST SAVINGS COMPARED TO PLAN M5
SECOND MARGINAL UNIT

Year	Production (mcm)	Production % of Desal	Annual Cost Fixed	Annual Cost Variable	Annual Cost Total	Treatment Cost Total	Treatment Cost This Unit	Variable Cost Total	Variable Cost Bz/m³	Production Displaced (mcm)	Benefit
1983	4.79	37	149	559	708	287	106	665	139	0	0
1984	3.57	100	149	750	899	196	196	946	265	3.57	946
1985	4.11	60	149	868	1,017	309	185	1,053	256	1.25	320
1986	4.45	56	149	950	1,099	345	193	1,143	257	.53	136
1987	5.13	57	149	1,105	1,254	383	218	1,323	258	.11	28
1988	5.13	50	149	1,137	1,286	423	212	1,349	263	0.00	0
1989	1.71	100	149	478	627	0	0	478	280	1.71	478
1990	2.70	100	149	690	839	0	0	690	256	2.70	690
1991	4.78	100	149	1,169	1,318	0	0	1,169	245	4.00	978
1992	4.11	59	149	1,053	1,202	0	0	1,053	256	1.09	279
1993	5.13	54	149	1,325	1,474	157	85	1,410	275	0.00	0
1994	4.79	40	149	1,289	1,438	290	116	1,405	293	.35	103
1995	5.13	35	149	1,419	1,568	385	135	1,554	303	0.00	0
1996	5.13	35	149	1,419	1,568	385	135	1,554	303	0.00	0
1997	5.13	35	149	1,419	1,568	385	135	1,554	303	0.00	0

NOTES:

Marginal Unit: New Desal Plant (18750)
1. All costs and benefits expressed in R.O. 000 unless specified.
2. Distillate post treatment costs are prorated to units based on desal production.
3. Total variable cost equals variable annual cost plus unit's share of treatment cost.
4. Benefit equals displaced production times variable cost per unit.
5. Costs and benefits have been based on January, 1979, price levels.

Source: Power and Urban Water Supply, 1979, Tables D-24 and E-20, and Stanley Consultants

TABLE 5.10

BENEFITS FROM COST SAVINGS COMPARED TO PLAN M5
THIRD MARGINAL UNIT

Item	Total Annual Cost (R.O.)	Estimated % Variable	Variable Cost (R.O.)
Pumping	401,000	100	401,000
Maintenance	186,000	0	0
Staffing	201,000	0	0
Chemicals	109,000	100	109,000
Other	45,000	50	22,500
Total	942,000	57	532,500

So variable cost in all years is:

$$532{,}500 \text{ R.O.}/14 \text{ mcm} = 38 \text{ Bz}/m^3$$

Benefits are then estimated as:

Year	Production Displaced (mcm)	Cost (Bz/m^3)	Benefit (R.O. 000)
1984	0.43	38	16
.	.	.	.
.	.	.	.
.	.	.	.
1989	2.29	38	87
1990	1.30	38	49

NOTES:

Marginal Unit: Wadi Dayqah

Source: Power and Urban Water Supply Study, P. 9-29, and Stanley Consultants

TABLE 5.11

BENEFIT SUMMARY COMPARED TO PLAN 5

Year	Cost Savings (R.O. 000)				
	Existing Desal	New Desal	Wadi Dayqah	Total 1979 Prices	Total 1982 Prices
1983	246	0	0	246	327
1984	0	946	16	962	1,279
1985	813	320	0	1,133	1,507
1986	999	136	0	1,135	1,510
1987	1,129	28	0	1,157	1,539
1988	1,160	0	0	1,160	1,543
1989	0	478	81	565	751
1990	0	690	49	739	983
1991	0	978	0	978	1,301
1992	928	279	0	1,207	1,605
1993	1,304	0	0	1,304	1,734
1994	1,307	103	0	1,410	1,875
1995	2,079	0	0	1,424	1,894
1996	2,079	0	0	1,424	1,894
1997	2,079	0	0	1,424	1,894
1998					2,905

The 1998 benefit for residual value is estimated as:

Construction Costs (as of 6/82)

Dam	4,038,800 R.O.
Well fields, piping, etc. (Project cost less engineering)	1,770,815 R.O.
Total	5,809,615 R.O.

Residual Value in 1998 based on 30 year life:

15/30 x 5,809,615 = 2,904,808 R.O.

Say 2,905,000 R.O.

Source: Tables 5.7, 5.8, 5.9, and Stanley Consultants

nearly equivalent in Oman. The absence of major distortion from trade barriers, misvalued foreign exchange, taxation policies or excessive unemployment suggests that Oman's shadow prices and market prices are approximately equal. A laissez-faire emphasis is clearly evident in much of Oman's commercial policy.

TABLE 5.12

NEW WELL FIELD CAPITAL COST

	R.O.
Well Field Development	
11 wells (1,100 m^3/day capacity) @ 28,730	316,030
Pipeline From Wells to Lift Station	165,500
150 mm ductile iron - 1,000 m @ R.O. 18	(18,000)
200 mm ductile iron - 2,000 m @ R.O. 31	(62,000)
250 mm ductile iron - 2,000 m @ R.O. 42	(84,000)
Connections	(1,500)
Diesel Generating Station and Transmission Line	263,300
Surface Storage Reservoir and Lift Station (2,000 m^3 capacity, chlorination station and pumping station)	800,000
Main From Lift Station to Coastal Road Main	
400 mm asbestos cement pipe - 1,000 m @ R.O. 40	40,000
Recommendations for Personnel	25,000
Subtotal	1,609,830
Contingencies, 10%	160,985
Engineering, 6%	96,590
Total Project Cost	1,867,405

NOTES:

Prices are for June, 1982.

Source: Stanley Consultants

The total project cost for the dam and well field is 6,396,639 R.O. or 1,599 baiza per annual cubic meter produced. At 1979 prices this equates to 1,202 baiza per cubic meter which compares favorably with the costs shown in Table 5.6 for alternative resources.

Annual operating and maintenance costs, as shown in Table 5.13, also compare favorably with the alternative sources shown in Table 5.6. Not surprisingly, operating and maintenance costs per cubic meter of water are very similar to those estimated for Wadi Mayh. The sediment removed annually may provide an additional benefit. Some officials interviewed felt that the sediment may be useful for agriculture or in other construction projects. No benefit has been claimed for this potential use. No major rework of the dam is anticipated in the 15-year period of analysis.

The complete schedule of expenditures required to install, operate, and maintain the dam and the well field is shown in Table 5.14. In 1982, 60 percent of total project costs are expended. In 1983, the remaining 40 percent of project costs are combined with one-fourth of the annual operating and maintenance cost estimate since the project will only be operational in the final three months of that year. Subsequent costs are limited to the annual operation and maintenance charges.

5.7 Internal Rate of Return and Sensitivity Analysis

The base case internal rate of return combines the benefits shown in Table 5.11 with costs from Table 5.14 to determine the discount rate at which the present worth of benefits and costs are equal. Comparison of that rate with a minimum acceptable rate of return yields a conclusion regarding the economic feasibility of the project. Sensitivity analysis then is used to test the strength of that conclusion against changes in key assumptions in the analysis.

The base case evaluates the savings which will accrue if the Al Khawd aquifer recharge project is available to provide 4.0 mcm per

TABLE 5.13

ANNUAL OPERATING AND MAINTENANCE COSTS[1]

	R.O.
Well Field	
Power Costs $\frac{876,000 \text{ kwh} \times 26 \text{ Bz/kwh}}{1,000}$ =	22,776
Maintenance Costs	19,661
Mechanical equipment (2% of capital cost)	(13,766)
Civil works (0.5% of capital cost)	(5,895)
Staffing	46,600
Other (vehicles, communications, chemicals, etc.)	27,400
Subtotal	116,437
Dams	
Sediment Removal (70,000 m^3/year)	27,810
Miscellaneous	3,015
Subtotal	30,825
Total Annual Cost	147,262
Annual Cost per Cubic Meter[2]	37 baiza

(1) Based on June, 1982, prices.

(2) 26 Baiza at 1979 prices.

Source: Stanley Consultants

year to the urban grid beginning [illegible] 1983. Savings are estimated assuming plan M-5 is also implemented on schedule. In the near future, that plan calls for a new desal unit on-line in 1982 and Wadi Dayqah on-line in 1983. Delay of Wadi Dayqah would increase reliance on desal units and increase the rate of return for the recharge project. Table 5.15 details the computation for the base case rate of return which is shown to be 18.3 percent for the Al Khawd project.

The minimum acceptable rate of return on a project is legitimately a policy decision to be made by the Omani Government and other potential funding agencies. Full account must be taken of unquantified benefits, social and environmental impacts, and other developmental objectives in weighing project desirability. The current criterion rate of return cited by the Development Council for approval of projects is 10.0 percent. In light of the relative abundance of capital in Oman, this rate is a reasonable approximation of the opportunity cost of capital. Thus, the 10.0 percent criterion is adopted for use in this study. Given the 18.3 percent base case return, the choice is not critical except for sensitivity analysis.

TABLE 5.14

EXPENDITURE SCHEDULE[1]
1982-1997

Year	Outlay (R.O. 000)
1982	3,734
1983	2,526
1984	147
1985	147
1986	147
1987	147
1988	147
1989	147
1990	147
1991	147
1992	147
1993	147
1994	147
1995	147
1996	147
1997	147

[1]Based on June, 1982, price levels.

Source: Stanley Consultants

TABLE 5.15

BASE CASE INTERNAL RATE OF RETURN

		15%					20%		
Year	Period	Factor	Costs	Present Worth	Benefits	Present Worth	Factor	Present Worth Costs	Present Worth Benefits
1982	0	1.000	3,734	3,734	--	--	1.000	3,734	--
1983	1	.870	2,526	2,198	327	284	.833	2,104	272
1984	2	.756	147	111	1,279	967	.694	102	888
1985	3	.658	147	97	1,507	992	.579	85	872
1986	4	.572	147	84	1,510	864	.482	71	728
1987	5	.479	147	74	1,539	765	.402	59	618
1988	6	.432	147	63	1,543	667	.335	49	517
1989	7	.376	147	56	751	282	.279	41	210
1990	8	.327	147	48	983	321	.233	34	229
1991	9	.284	147	42	1,301	369	.194	29	252
1992	10	.247	147	37	1,605	396	.162	24	259
1993	11	.215	147	32	1,734	373	.135	20	233
1994	12	.187	147	27	1,875	351	.112	17	210
1995	13	.163	147	24	1,894	309	.093	14	176
1996	14	.141	147	21	1,894	267	.078	12	148
1997	15	.123	147	18	1,894	233	.065	9	123
1998	16	.107			2,905	311	.054	--	157
Total (R.O. 000)			8,318	6,662	24,541	7,751		6,393	5,892
Net Cost or Benefit						1,089		499	
						499			

Interpolation:

1,588 ÷ .05 = 31,760

1,089 ÷ 31,760 = .034

Internal Rate of Return

.15 + .034 = 18.3

Source: Stanley Consultants

The major features of the base case which influence the rate of return include:

1. Use of the water for urban water supply.
2. Production scheduling for alternative sources as developed in plan M-5.
3. Demand growth as forecast in the medium projection.
4. Availability of 4.0 mcm annually from the recharge project beginning in 1983.
5. Escalation of benefits from 1979 to 1982 at a ten percent annual rate for comparison with estimated construction costs based on 1982 price levels.

Since the base case rate of return is reasonably high, sensitivity analysis is best used to examine variations in assumptions which would lower rather than raise the rate. For example, no analysis has been made of other supply plans since M-5 is the lowest cost option. Substitution against plans which rely more heavily on desalination would raise benefits significantly.

Rates of return for five scenarios which lower benefits are shown in Table 5.16. The first case acknowledges the uncertainty inherent in the hydrologic evaluation. The long-term average of 4.0 mcm annual yield is sound but also consistent with periods of a few years where recharge would be substantially less. A fairly severe variant assumes that no water increment at all is captured in the first three years of the project and then 4.0 mcm is available annually for the remaining 12 years which have been analyzed. The rate of return for this case is 12.8 percent. Another test of sensitivity to yield was conducted by solving for the yield required to achieve the 10 percent criteria rate of return. This analysis shows that an average annual yield of 2.3 mcm would just meet the return criterion.

TABLE 5.16

SENSITIVITY ANALYSIS

Assumptions	Internal Rate of Return (%)
1. Base case.	18.3
2. No water increment for first three years.	12.8
3. No benefit escalation beyond 1979 price level savings.	12.8
4. Agriculture benefits only.	5.9
5. Low demand-least cost plan (L2), based on high discount rates (10%, 16%).	14.4
6. Cost including monitoring equipment	18.2

Source: Stanley Consultants

A second case tests the influence of the escalation assumptions used to update cost savings from the 1979 estimates to the 1982 base for comparison with construction costs. Overall escalation of 10.0 percent per year increases savings by 33.0 percent. More refined analysis of escalation of particular cost items might lead to a different choice of updating factors. Another severe test assumes benefits are not escalated at all. Thus benefits based on 1979 prices are compared with costs based on 1982 prices. This case has a return rate of 12.8 percent.

Economic analysis in this report is founded on the highest valued use of the water which was established to be urban water supply. An indication of the feasibility of using the water for irrigation is provided by a preliminary analysis based on the following assumptions:

1. The 4.0 mcm available from the project supports 235 hectares of agriculture that would be eliminated by saltwater intrusion if the project is not built.
2. Net incomes from production are based on 40 percent increases in yield above present levels.
3. Price levels are assumed to be 10 percent higher than those used for 1981 in the HVA study.

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4. Benefits are thus 10 percent higher than those shown in Section 5.2 or 343,000 R.O. per year.
5. Costs are limited to the initial outlay for the dams of 4,355,200 R.O. plus 31,000 R.O. for annual maintenance.
6. No flood protection benefits or sediment sale benefits are included.

The rate of return for this case is 5.9 percent.

Although recent evidence supports medium to high growth projections of demand, it is useful to consider the implications of lower growth. A very aggressive displacement of expatriates with Omanis, a stabilization of the rural sector which stems the inflow to urban areas, and/or a slackened pace of government development projects could lower growth in demand. To test viability against low growth scenarios, substitution against a low growth plan was analyzed. Plan L2 is the lowest cost plan at discount rates of 10 and 16 percent. Plan L2 brings Wadi Dayqah on-line in 1983 and two small desal plants (9,000 cubic meters/day) on-line in 1992 and 1994. This plan does not include the additional desal unit which is currently planned for Ghubrah in 1982. Inclusion of that unit would raise the rate of return shown. Against plan L2, the Al Khawd project return is 14.4 percent. A final sensitivity test adds the first costs for monitoring to determine if the rate of return remains acceptable with monitoring. The first costs for the three part monitoring program total 62,100 R.O. Assuming 60 percent of these costs are incurred in 1982 and 40 percent in 1983, the internal rate of return falls to 18.2 percent.

Use of a 10.0 percent criterion rate of return for this project indicates that the project remains economically viable under fairly severe relaxation of assumptions. Only a complete change of water use or a dramatic demand reduction and cancellation of the capacity addition at Ghubrah would compromise this finding.

5.8 Summary and Conclusions

The Al Khawd aquifer recharge project as analyzed in this section consists of the recharge dam and ancillary development to provide water to the urban grid. The total project cost in 1982 prices is estimated to be 6,222,605 R.O. including 4,355,200 R.O. for the dam alone. Without the project, it has been assumed that the medium water demand forecast will be met by additional desalination capacity, early development of the dam and reservoir in Wadi Dayqah and later development of a major new groundwater supply 150 kilometers up the Batinah Coast. The 4.0 mcm yield from the Al Khawd project eliminates variable operating costs of the marginal sources. These cost savings are the project's primary benefits. Comparison of these benefits with project costs yields an estimated internal rate of return of 18.3 percent.

Additional project benefits not included in that return rate include:

1. Substantial reduction in flood flows and related reduction in flood damages. It will be important, however, to prohibit floodplain development since the area is still susceptible to flooding from major flood events.
2. Possible sale of sediment removed annually in the dam maintenance program.
3. An unquantified increment of water in the aquifer which is not pumped to the urban grid. This increment should assist in combating the saline intrusion in coastal wells.

Sensitivity analysis indicates that the project remains viable against a 10.0 percent criterion under relaxation of the most critical assumptions of the analysis.

The economic analysis provides a firm basis for recommending project implementation.

PART 6 - SOCIOCULTURAL, SOCIOECONOMIC FEASIBILITY

6.1 General

The Wadi Al Khawd aquifer recharge project is expected to make available an additional 4.0 MCM/year of low cost water resources for use in the capital area. The capital area (Muscat to Seeb) is presently experiencing rapid population growth and investment in the urban infrastructure. Water supply to support expected growth will be provided from desalination or remote ground and surface water sources whether or not the recharge project is implemented. Thus, it appears that the sociocultural and socioeconomic impacts of continued urbanization of the capital area are independent of the Wadi Al Khawd recharge project. Urbanization impacts will occur with or without the project.

This section reviews the compatibility of the proposed recharge project with the sociocultural environment of the capital area and focuses on the distribution of project benefits among the population.

6.2 Project Beneficiaries

It has been demonstrated in Part 5 that the most economically effective use of the water increment from this project will be to supplement the existing urban water supply. The 11,000 daily cubic meters of incremental water will supply approximately 26 percent of 1983 total urban demand and 14 percent of 1990 demand. Residential, commercial, industrial, and governmental users will benefit. Residential users currently account for about 65 percent of total water demand. It is estimated that there are presently over 10,000 residential connections to the system.

In addition to increases in recoverable water supplies, the Wadi Al Khawd project will retard the rate of surface water runoff, thereby reducing the downstream flood damage potential. While average damage reductions cannot be estimated from available data, it is expected that floods as large as the May, 1981, event would be substantially

reduced in impact. That flood claimed six lives and damaged 600 homes and 30 businesses in Seeb. However, this project is not intended for flood control and a substantial threat of flood damage remains. Therefore, it is important that the downstream area be set aside as a flood hazard area and development limited.

6.3 Minimum Participator Profiles

The project will directly benefit all residents of the capital area because of the additional potable water made available for distribution. These benefits are unlikely to be recognized by water system users because:

1. Water from the project will make up a small proportion of the overall system's supply.
2. It is unlikely that domestic water rates will directly or immediately reflect the economic benefits procured by the exploitation of the Wadi Al Khawd resource.
3. If the project is not implemented or the available resource is not used in the grid, other sources are likely to be provided to meet distribution system requirements.

For example, total cost savings in 1985 are estimated at 1,507,000 R.O. The domestic share of savings based on portion of use and existing rates would be approximately 60 percent or 904,200 R.O. This is about 48 R.O. per residential hook-up for the year. This 48 R.O. per month saving compares with average total household expenditure of more than 300 R.O. per month for middle income government employees in the capital area in 1977. By 1985, average expenditure would be 640 R.O. at 10 percent annual rate of increase. Thus, savings would equal about 0.6 percent of expenditure.

Thus, while of considerable total benefit to the capital area population, the cause-effect relationship between project implementation and social well-being is not likely to be sharply perceived by individual system users.

Residents of Seeb who presently draw supplies from individual wells may benefit from improvements in water quality if the aquifer

recharge is successful in reducing salinity. This impact would be perceivable to residents.

Distribution system extensions to Seeb are presently under design and will soon be implemented with or without the recharge project. Since the availability of hook-ups in Seeb will probably parallel recharge project implementation, residents are likely to associate water system extension with the new supply.

The village of Al Khawd, about 8 kilometers above the project site, currently obtains water from local wells. Publicity surrounding the recharge project may kindle interest in obtaining water from the urban grid. Such supply has not been assumed in the analysis because it would be expensive to provide conveyance to this village. A decision on the desirability of extending the grid to Al Khawd should be made independently from assessment of recharge project feasibility. More cost effective options may exist for needed supply in Al Khawd.

6.4 Obstacles

Discussions with staff of the Public Authority for Water Resources, Development Council on the Environment, Ministry for Electricity and Water, and Ministry for Labor and Social Affairs indicated an absence of religious, social, or cultural barriers to project construction or use of the recharge water. The project site is uninhabited and owned by the Omani government. Future land use plans indicate that the project area is to continue as an undeveloped government reserve, most likely because of its status as a floodplain.

Water rights to this property are also held by the government and groundwater is presently extracted by the government for use by PD(O) and by the capital area municipal system. Therefore, the question of water rights should pose no barriers for use of the improved aquifer.

6.5 Communication Strategy

While analysis to date justifies continued optimism toward success of the project, care must be taken to avoid inflated expectations. Thus far, excessive optimism has been avoided by labeling the project as "experimental." Such a practice should continue, even after construction is completed.

All publicity, project area signs, releases, and interagency discussions should stress the purpose and the experimental status of the project for the following reasons:

1. Other water resource projects for the capital area should not be halted or delayed in deference to this project.
2. The experimental status of the Wadi Al Khawd project should assist in avoiding premature conjecture on the applicability of this concept in other parts of Oman. Until detailed studies of individual wadis and their runoff/recharge characteristics are completed, duplication elsewhere is speculative.
3. Variations in yield are expected. The experimental status of the project will help to avoid its premature labeling as a failure if benefits are not immediately gained.
4. The project is not intended for flood control. A substantial threat of flood damage remains and development downstream of the dam should be limited.

Once certain of actual long-term capabilities, the experimental status could be removed and the recharge project regarded as a part of the capital area's firm water supply.

6.6 Applicability in Other Areas

Successful experimentation with the Al Khawd recharge project would certainly lead to consideration of the concept in other areas of Oman. Interviews indicated particular interest in use of recharge dams for irrigation in agricultural areas. Further detailed, site specific analysis would ultimately be required for such applications, but general concerns can be identified.

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The major social concept to address is the falaj water system, which is said to prevail in many parts of the country outside the capital area. Used in traditional communities for potable water and irrigation, the falaj represents the basis for a portion of a community's social structure and customs. The high maintenance requirements of a falaj and the complex priorities given to the use of its fluctuating water yields are integral to the social and economic organization of a traditional village.

Establishment of a modern water system utilizing an upgraded aquifer, pumps, and pipes will likely assure a larger quantity of firm water and significantly reduced maintenance requirements to villages. This in turn will remove existing time and economic limitations to the village and contribute to the community's transition. Obviously, other factors besides water are involved in the modernization of traditional communities. These factors include access to roads, availability of outside communications, economic base, development of educational facilities, and rural electrification.

Approximately 4,000 operational falaj systems exist, and concern has been expressed that the number is continuing to decline due to the increasing cost and difficulty of maintenance. As people migrate to the urban areas (primarily the capital area) less labor is available to maintain the falaj. Then, as the falaj systems fail, pressure to migrate is enhanced. In order to break this apparent cycle, the Ministry of Agriculture and Fisheries has made commitments of funds to restore and maintain falaj systems.

Application of the recharge concept to other areas of Oman must be coordinated with other infrastructure development plans within the Sultanate. The remaining falaj systems in Oman are valued highly from an intrinsic standpoint, and replacement of a functioning falaj with a modern water system may conflict with preservation objectives. Placement of recharge and pumping systems in portions of Oman in new unsettled lands or where falaj are beyond repair or lost, however, could substantially improve the social and economic conditions of an area and could reduce urban migration pressures.

6.7 Summary

The Al Khawd aquifer recharge project is not expected to alter the rate of urbanization as capital area growth moves toward Seeb and Barka. Benefits of the project are summarized in Table 6.1.

TABLE 6.1

SOCIOECONOMIC BENEFIT SUMMARY

Beneficiaries	Benefit
Capital area water users	Small decrease in potable water cost per consumer relative to total expenditure per consumer.
Flood-prone properties at Seeb	Reduction in potential flood damage.
Coastline well users at Seeb	Some reduction in salinity.

Source: Stanley Consultants

Villages most directly aware of the project would be Al Khawd and Seeb. Seeb should share significantly in project benefits. Al Khawd's interest in improved water supply may be heightened by this project but the most economic system for supply to that village will probably be independent of this project.

A careful communication of project potential will be required to avoid excessive expectations. Application of this same system elsewhere in Oman to be successful will require careful study to evaluate the social impacts of complementing or displacing falaj systems of irrigation.

PART 7 - PHYSICAL ENVIRONMENT

7.1 Environmental Evaluation

This portion of the report presents an assessment of the environmental consequences of constructing a rock/gravel fill dam on the Wadi Al Khawd for the purpose of aquifer recharge. This evaluation is prepared in accordance with Sections 118(b) and 611 of the Foreign Assistance Act of 1961, as amended, to ensure that environmental factors and values are incorporated into the AID decision-making process.

7.2 Fauna

The following description of faunal species potentially inhabiting the project area was prepared using three major literature sources. Those sources are:

1. "The Mammals of Arabia," by David L. Harrison, 1864.
2. "The Scientific Results of the Oman Flora and Fauna Survey, 1975," edited by the Ministry of Information and Culture, The Sultanate of Oman.
3. "The Draft Environmental and Natural Resources Profile on the Sultanate of Oman," prepared by the Arid Lands Information Center, Office of Arid Lands Studies, University of Arizona, 1981.

Other references include "The Birds of Oman," by M. D. Gallagher and Woodcock (1980) and "Arid-land Ecosystems: Structure, Functioning, and Management," (Vol. 1), edited by D. W. Goodall and R. A. Perry (1979).

Existing literature does not provide specific estimates of wildlife populations for Oman or regions within Oman. However, information on relative abundance of some species is available and is noted in the species list found in Appendix A.

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The arid condition of the project area is a limiting factor controlling plant and animal populations. Sparse vegetative cover and forage results in relatively low faunal populations.

7.3 Mammals

A list of mammals potentially inhabiting or visiting the project area is presented in Table 1, Appendix A. This list is comprehensive, including those species of northern Oman and the Batinah that may frequent the project area. The general distribution and characteristics of species are also indicated in Table 1.

Literature cites three species as being common to the lowland plains of the Batinah. They are the Ethiopian hedgehog (*Paraechinus aethiopicus dorsalis*), house mouse (*Mus musculus gentilulus*), and the common red fox (*Vulpes vulpes arabica*). Other species in the Orders Rodentia and Chiroptera could be abundant. The Oman hare (*Lepus capensis omanensis*) could also be relatively common.

7.4 Avifauna

Approximately 31 species are common breeding birds and migrants in the Wadi Al Khawd project area. A listing of these birds is provided in Table 2, Appendix A. This table also includes information on each bird's status and other characteristics. Species listed primarily inhabit hillsides, rocky wadis, and the coastal plains of the Batinah.

Common species of the Batinah plain are the grey francolin (*Francolinus pondicerianus*) and little green bee-eater (*Merops orientalis*). Gallagher (1975) indicates the Indian roller (*Coracias benghalensis*), collared dove (*Streptopelia decaocto*), and graceful warbler (*Prinia gracilis*) are common on the Batinah.

7.5 Herpetofauna

Of the 48 species of reptiles known to occur in Oman, eighteen potentially inhabit the project area. The Family Gekkonidae (Geckoes) contains the largest number of species expected to be found in the project vicinity. Both species of amphibians found in Oman may occur

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in the Wadi Al Khawd. Table 3 in Appendix A lists those species expected to inhabit the project area.

7.6 Endangered Species - Faunal

The International Union for Conservation of Nature and Natural Resources (IUCN) and the U.S. Fish and Wildlife Service list seven faunal species as endangered in Oman. None of these species inhabit or are expected to visit the project area.

7.7 Flora

The Wadi Al Khawd project area is located on a dry, stony, and sparsely vegetated coastal plain. The dominant species are *Acacia tortilis* and *Euphorbia larica*. Acacia trees are three to six meters in height and are conspicuous on the landscape. Euphorbia shrubs of 0.5 to 2 meters in height appear to be randomly distributed in some gravel wadi bottoms while absent from other similar areas.

Other plant species expected to occur in the project area include *Acacia ehrenbergiana*, *Ziziphus spina-christi*, *Prosopis specigern*, *Salvadora persica*, *Lycium sp.*, *Pterophyrum scoparium*, and *Tamarix arabica*. *Ziziphus spina-christi* and *Prosopis spicigera* are common trees found along wadi banks. Small sandy areas in the wadi are capable of supporting *Calligonium sp.*, *Leptadenia sp.*, *Ponicum sp.*, and *Heliotropium sp.* Herbs, such as *Asphodelus sp.*, *Centaurea sp.*, *Indigofera sp.*, and *Plantago sp.*, may also occur in sandy soils.

7.8 Endangered Species - Flora

The IUCN Plant Red Book (1978) identifies two plants as rare or endangered in Oman. Both of these plants occur in the northern mountains and are not found in the project area.

7.9 Environmental Impacts

7.9.1 Primary Impacts - Construction of a water retention dam will necessitate alteration of the terrestrial environment through earthwork in the vicinity of the dam. The dam will be 4,668 meters in length with an average width at the base of 45 meters. Material to construct the dam will be composed of a fine bedding, coarse

bedding, drain stone, riprap, and stone for a gabion mattress and will come from the Wadi adjacent to the dam.

The primary impact of project implementation will be a loss of natural vegetation and wildlife habitat in those areas disturbed during construction. A small reduction in wildlife populations can be expected based upon the extent of habitat disturbance and relatively small population base. Current inhabitants of the disturbed area may be displaced to suitable adjacent habitat, but unless this habitat is below carrying capacity the net results will be reduction of the species population.

An estimated 310 hectares will be disturbed as a result of grading and dam construction.

7.9.2 Secondary Impacts - Use of the water from the Wadi Al Khawd aquifer following construction of the retention dam should not result in significant impacts to the biological environment. Any urbanization brought about from additional water supplies will occur adjacent to the dual carriageway between Seeb and Muscat. In this region, vegetation is scarse and little wildlife habitat exists. Environmental impacts would be minor with the possible displacement of individuals from species populations.

Expansion of agricultural lands would increase the habitat and carrying capacity for coastal wildlife species, especially birds. However, since agricultural expansion is expected to be quite limited, insignificant population changes are anticipated.

7.10 Environmental Consequences

The general character and habitat of the Wadi Al Khawd vicinity will remain essentially unchanged. Flora will be lost in those areas where grading and dam construction occur.

Construction of the water retention dam will result in an insignificant reduction in flora such as *Acacia sp.* and *Euphorbia sp.* Faunal species using this habitat for forage and cover will be displaced and populations reduced.

The potential exists for flora and fauna to become re-established adjacent to the dam. It is expected that populations would return to levels existing prior to construction.

Wildlife populations may decrease slightly during silt removal due to the destruction of habitat.

Again, population of flora and fauna will become re-established over a period of time.

7.11 Irreversible or Irretrievable Commitment of Resources

Fuel and labor are the major resources that will be irretrievably committed during construction and for operation and maintenance.

7.12 Mitigation of Adverse Environmental Impacts

Existing vegetation, particularly larger trees and shrubs, should be preserved adjacent to the construction area. This will provide some refuge for displaced animal species and promote the return of habitat in disturbed areas.

7.13 Guidelines for Ecological Soundness

1. Inform the Office of the Advisor for the Conservation of the Environment, Division of H. M. for Protocol of the location and schedule of project construction.
2. Inform this agency of any caves potentially inhabited by bats prior to destruction. Such action is necessary because a number of bat species may inhabit Oman that are not presently identified.

PART 8 - CONSTRUCTION PROCEDURES AND SCHEDULES

8.1 General

The aquifer recharge project appears to be feasible from an engineering, economic, and environmental standpoint. Construction of this project requires the establishment of procedures and a schedule to complete work. This section of the report discusses management of the construction effort.

8.2 Construction Plan

The following is a "Sequence of Operations" for construction of this project:

1. Bid and award.
2. Mobilize.
   a. Equipment.
   b. Material.
   c. Personnel.
3. Construct diversion channel.
4. Clear and strip borrow areas.
5. Clear and grub.
6. Excavate foundation.
7. Excavate key way.
8. Place and compact key way.
9. Place and compact core material until it reaches culvert elevation as shown on plans.
10. Excavate for seepage collars.
11. Place bedding and RCP.
12. Form and pour seepage collars.
13. Form and pour upstream and downstream headwalls and aprons.
14. Build toe drain.

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15. Continue placing and compacting core material until it reaches an elevation adequate to support construction equipment.
16. Begin placing riprap bedding and riprap.
17. Begin placing gabion bedding and gabions.
18. Continue core placement.
19. Continue bedding and riprap.
20. Construct temporary diversion dam.
21. Excavate remaining key way.
22. Complete placing and compacting key way.
23. Complete placing and compacting core material.
24. Complete placing riprap and gabion bedding.
25. Complete placing riprap and gabions.
26. Remove temporary diversion dam.
27. Final dress-out and clean-up.
28. Demobilize.

During mobilization, concrete culvert pipe and gabions should be ordered as they are not available in Oman. Ample time is allowed in the construction schedule for procurement and delivery of these items.

It is assumed that water will flow in the wadi at times during the construction period (455 calendar days). A diversion channel is provided to control this water.

Excavation of the diversion channel is a portion of the clearing and grubbing activity at the site. At the same time, several crews can begin the stockpile of core zone material and select fill near the dam site.

The terrain is easily accessible and borrow material is abundantly available; therefore, no haul roads are required.

Bulldozers, scrapers, earthmovers, and trucks will be used to move the material. Select fill requires screening; core zone material does not. The fine material remaining after screening of the select fill can be used for core zone material.

The key way is excavated, filled, and compacted; and the core zone material is placed up to the grade required for the concrete culvert pipes. Following this operation, excavation for the seepage collars, laying of concrete culvert pipe, and forming and pouring of the concrete seepage collars and pipe headwalls is to be done.

Consideration should be given by the construction contractor to development of specialty crews to construct head walls, collars, and lay pipe. Three different types of crews will be needed for the operation. This will enable three culvert sites to be worked on in stages with a pipe laying crew first, followed by a formwork crew, then a concrete crew. Utilization of the same crews throughout the job improves quality control.

Caution must be exercised upon backfilling around culverts. It is necessary to compact layers parallel to pipe until core material reaches an elevation sufficient to support construction equipment. Once desired elevation is reached, the toe drain and the bulk of core zone will be formed. Toe drain elevation having been reached permits riprap and gabion bedding placement. Alternate placing of beddings and riprap will occur until dike is topped out. Once most embankment has been formed, gabions can be placed and filled.

Gabion delivery should pose no problem. Due to the sequential need, there is ample time to have them delivered. The last gabion should be placed before the cofferdam is contructed. A cost effective procedure has been developed and incorporated into the construction schedule and cost estimate. Manpower requirements shown on the schedule adequately reflect amount and scope of work required.

8.3 Construction Supervision and Control

An independent engineering firm should be engaged to serve as the supervisory engineer to the Government of Oman. The engineer should be retained for the full period of construction and final acceptance of the works. The engineer shall furnish complete engineering and advisory services needed to complete work. The engineer shall make

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available and use his best efforts to provide in a workmanlike and professional manner such engineering, advisory, management, and all other relevant services as may be necessary to obtain the contract objectives.

8.3.1 General Responsibilities

1. Establish the requisite project organization under the supervision of a resident project manager at project site (Seeb area).
2. Provide necessary consultation, cooperation, and coordination with Omani Government, U.S. AID, and prime general contractor, and the other project donors to effect the earliest completion of the project.
3. Provide direction and management in connection with all the technical aspects of the project.
4. Prepare a standard procedures manual for Omani Government approval containing implementation and administrative procedures to be used throughout the remainder of the project.
5. Prepare documentation for claims against contractors and material suppliers, review claims prepared by others, and prepare documentation for defense by Omani Government, as required for any and all legal actions related to award of contracts, administration or execution of bulk supply contracts, and supervision of construction. Render assistance and advice to Omani Government in connection with any claims arising from the procurement program. Prepare and execute claims for insurance purposes.
6. Advise Omani Government, as required, concerning right-of-ways, easements, and compensation.
7. Prepare timely and meaningful monthly progress reports. Present them to Omani Government and AID not later than 15 days from the end of the month reported upon.

8.3.2 Supervision of Construction - The engineer shall supervise construction, acting as the Omani Government's representative with regard to approvals, certifications, and changes. Supervising services shall include the following:

1. Review, comment as necessary, and approve the prime construction contractor's monthly schedule with special attention given to coordination of this construction schedule with delivery dates of equipment and materials.
2. Check and certify all drawings, including shop and erection drawings submitted by the construction contractor, including the review and certification of laboratory, shop, and mill test reports on the materials and equipment for compliance with specifications.
3. Monitor, coordinate, and provide engineering supervision and inspection of construction work to ensure continuous, efficient progress. Supervision of construction activities shall include, but not be limited to, witnessing and approval of soil tests, screening of select materials, and cast-in-place concrete, as described in technical portion of specifications.
4. Interpret drawings and specifications and consult with Omani Government, the construction contractor, and subcontractor, if any, to ensure compliance with said documents and to ensure timely progress of construction work.
5. Review and approve any changes which might be required and issue necessary contract modifications. Without prior notification to the Omani Government, the contractor can authorize additional expenditures per individual variation order, and up to a cumulative total amount for a number of variation orders, in monetary amounts to be determined by negotiation with the Omani Government and definition in a letter of understanding.
6. Maintain location records of culverts, collars, headwalls as installed, and including such field changes as are

necessary, and record such locations and dimensions on one set of construction drawings reserved for this purpose. (This set of drawings will be turned over to Omani Government at the conclusion of the work, and shall constitute the "as-built" drawings and plans of the project.)

7. Monitor and determine actual construction progress, certify the progress cited in progress payment invoices submitted to Omani Government by the construction contractor.
8. At the satisfactory completion of all construction work, the engineer shall certify the invoice for the construction contractor's final payment.
9. Carry out the responsibilities of representing the owner in accordance with the construction contract.

8.3.3 Soil Testing and Analysis - Review soil test results furnished by the independent construction contractor (testing laboratory) for the project to insure that design parameters are met.

8.4 Contracting Procedures

8.4.1 Advertising - Solicitation of potential contractors should be effected through widely disseminated advertising by publishing a notice of the availability of prequalification questionnaires and invitations for bids in the Commerce Business Daily of the U.S. Department of Commerce and other appropriate local, regional, or international journals and newspapers. The notice for the Commerce Business Daily should contain such information as:

1. A brief description of the project, the services required, and anticipated bid date and construction schedule.
2. The name of the contracting agency.
3. The address at which interested firms may obtain prequalification questionnaires.
4. The deadline for receipt of prequalification information.
5. A statement regarding the eligible nationality of the contractor.
6. Approximate dates of the availability of invitations for bids and bid opening.

7. A statement that it is anticipated that AID will participate in the financing of the project.

8.4.2 Prequalification - The responsible party should furnish to all interested firms responding to the prequalification notice and to all other firms which it wishes to solicit an outline for presenting their general and specialized qualifications, applicable experience, job capacity, etc. The "prequalification questionnaire for construction contractors" contained in AID Handbook 11 should be used. The prequalification information should be submitted within a reasonable time following the publication of the notice in the Commerce Business Daily but within not less than 30 days after publication. Following evaluation of the information submitted by interested firms responding to the prequalification questionnaire, the owner should be promptly advised of the names and addresses of those firms qualified for his review and approval.

8.4.3 Solicitation of Bids - Following the owner's approval of the prequalified list of bidders, an Invitation for Bids (IFB) should be prepared. The IFB must be approved by AID prior to issuance to the prequalified bidders. The IFB should include the following:

1. A cover letter to each of the firms invited to bid identifying the IFB by number and listing the parts of the IFB.
2. Instructions to bidders which will include all the information the bidder will require to submit the bid in the proper form. It should also include the appropriate name and address for mailing the bid; the date and hour of the bid opening; a brief description of the proposed construction and all other pertinent information necessary to allow the bidder to submit an appropriate bid.

8.4.4 Contract Award - Sealed bids received prior to the closing time should be opened and read publicly at the time and place specified in the IFB. At a minimum, the name of the bidder and total bid price should be announced. The bids should be recorded on a Bid Tabulation Sheet identifying the IFB number, date, name of bidder, amount(s), whether a bid bond was included, and other appropriate information.

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The owner and engineer should then jointly conduct an evaluation of the bids. The bids should be "responsive" and be reasonable in price as compared to the detailed cost estimate. The owner should reject any bid which is nonresponsive. AID approval of the proposed contract award is required. AID will approve the contract prior to signature by the government of Oman. The following documentation is required for submittal to AID:

1. The final contract (unsigned) with a statement indicating that it is identical to the contract included in the previously approved IFB, or showing that the contract included with the IFB has been changed. If the contract included in the IFB is not used, a draft of the proposed contract must be submitted to AID for its approval.
2. A certified copy of the bid tabulation.
3. A statement that the selected bidder is responsible and submitted the lowest responsive bid, is otherwise eligible for the contract, and that the bid price is reasonable.

If AID agrees with the proposed award after reviewing the documentation submitted, it will inform the owner of its approval. Upon receipt of AID's approval of the proposed award, the successful bidder should be informed that it has been awarded the contract, and usually in the same communication should be advised of a proposed date and place for signing the contract. The contract should be signed by the owner by using the Form of Agreement. Copies of the signed contract should be distributed accordingly. Unsuccessful bidders should then be notified by letter of the award and the bid bonds returned.

8.4.5 Method of Payment - The method of payment to the contractor will be chosen in consultation with AID and will be described in the IFB. The three basic methods for payment are:

1. Direct reimbursement to the borrower/grantee.
2. Direct letter of commitment to the contractor.
3. Letter of commitment to a U.S. bank supplemented by the borrower/grantee's letter of credit.

Regardless of which method is chosen, payments will be made on the basis of verification of performance and the submission of required documents. The IFB and contract should also include the procedure for local currency payments.

8.4.6 Bonding Requirements -

1. Bid Bond - The contractor should be required to submit a bid bond in the amount of 5 to 10 percent of the bid price or $5,000,000, whichever is less.
2. Performance Bonds - These are issued by sureties to insure completion of contract requirements. In the event a contractor defaults, the surety will remedy the default, take over performance itself, arrange for another contractor to complete the work, or pay the additional costs incurred by the owner for completing the contract, up to the amount of the bond. The amount of the performance bond should be 100 percent of the contract price.

8.5 Construction Schedule

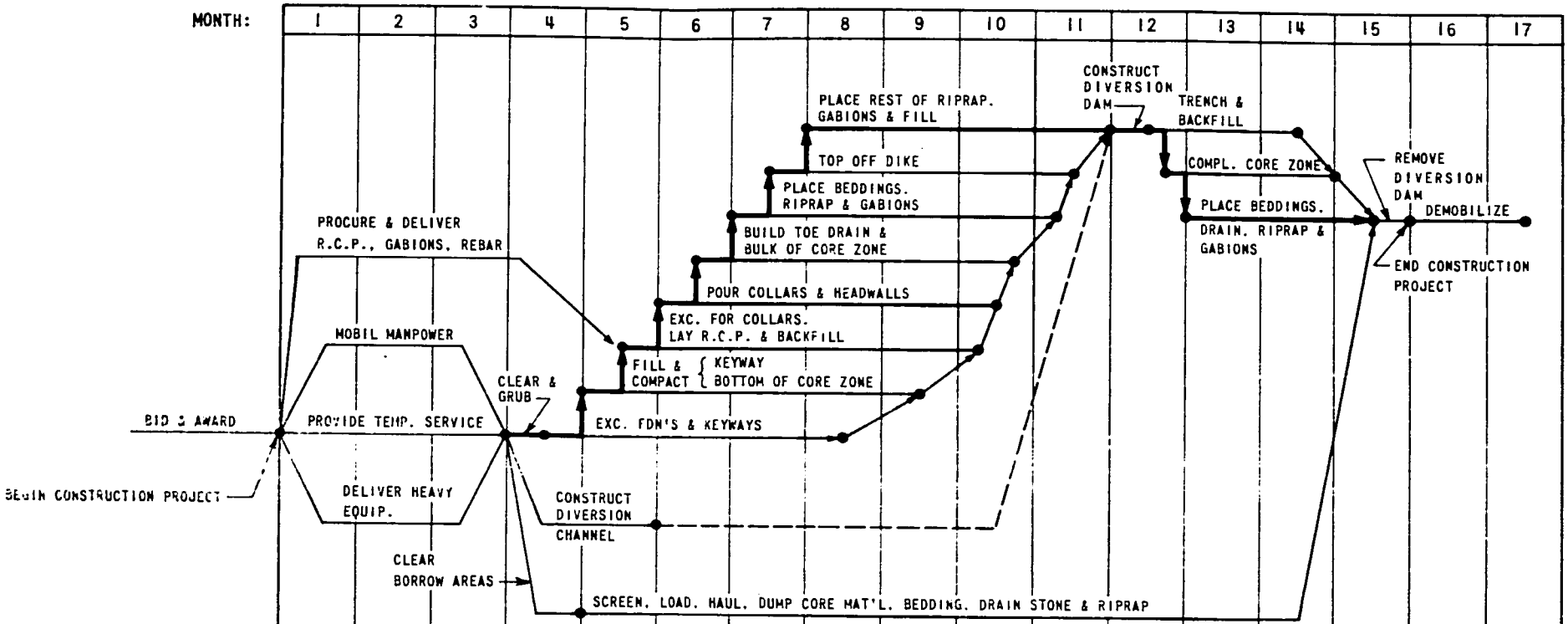
The construction schedule as shown on Figure 8.1 is based on 455 calendar days for construction of the project.

Work force requirements are noted as are quantities of earthwork on the lower portion of the sheet.

These representative figures serve to confirm the basis of unit costs used in the estimate.

Delivery of lead time items does not become a major consideration. The schedule verifies sufficient time being available for the delivery.

The schedule is not a tight schedule and we would expect that the construction contractor's schedule will reflect something less than the 455 days assumed. A 10-hour day has been used in the development of the CPM, and it is conceivable that a contractor may elect to work longer hours thereby reducing total days worked but not the total manpower, services, etc., required.



MANPOWER REQUIRED: NUMBER OF MEN PER MONTH

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
MOBILIZATION AND DEMOBILIZATION	10	20	30												10	20	20
CONSTRUCTION CREW		10	20	90	100	110	120	130	130	130	115	90	80	80	40	40	
TOTAL WORK FORCE	10	30	50	90	100	110	120	130	130	130	115	90	80	80	50	60	20

CONSTRUCTION OPERATIONS: QUANTITIES MEASURED IN THOUSAND M^3/MONTH

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
CORE ZONE MATERIAL				10	40	110	125	136	120	60	40	25	15	10			
SELECT FILL							20	20	20	20	10	5	5				
RIPRAP							10	10	10	10	5	5	5	5			

MONTH: 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17

ENGINEER'S PROJECTED CONSTRUCTION SCHEDULE
FIGURE 8.1

PART 9 - PROJECT COST ESTIMATES

9.1 Construction Cost Estimate

The construction cost estimate (see Table 9.1) represents the contract costs of completing all construction work including mobilization, overhead, and profit.

It is assumed that the majority of the work will be performed by a prime general contractor. It has been assumed that 60 percent of this construction firm will be foreign ownership and 40 percent Omani ownership.

Omani's will be employed when available, but the majority of craft labor will be Pakistani and Indian. Labor rates for these crafts vary from $1.05 per hour for laborers to $5.50 per hour for heavy equipment operators. These unit rates include all fringe benefits and home leave where applicable. Normal working day is ten hours.

This estimate assumes that sufficient construction equipment is available within Oman. Therefore, no allowance has been made for mobilization of this equipment.

Mobilization and demobilization includes the cost to recruit and transport to Oman 12 English speaking people and one hundred Third World Nationals. Also included in mobilization and demobilization is the cost of the site office, workshops, toilets, and miscellaneous structures. Items included as overhead are the cost for nonproductive personnel such as accounting, clerk-typists, surveyors, quality control inspectors, watchmen, guards, etc. Also included in overhead are the housing required for foreign supervisors. Overhead also includes the cost of insurance, money costs, and home office expenses.

TABLE 9.1

CONSTRUCTION COST ESTIMATES

Description	Quantity	U/M	Foreign Material Unit	Foreign Material Total	Foreign Labor Unit	Foreign Labor Total	Foreign Equipment Unit	Foreign Equipment Total	Local Material Unit	Local Material Total	Local Labor Unit	Local Labor Total	Total Project
Diversion and Care of Water		LS		20,000		2,000		2,500				2,500	27,000
Clearing and Grubbing		LS				500						500	1,000
Foundation Preparation	147,505	M3			0.050	7,375	0.210	30,976			0.030	4,425	42,776
Embankment Fill:													
Core Zone	691,345	M3			0.260	179,750	0.600	414,807			0.040	27,654	622,211
Toe Drain and Gabion Bedding	64,715	M3			0.290	18,767	0.770	49,831			0.040	2,589	71,187
Riprap Bedding	27,635	M3			0.290	8,014	0.770	21,279			0.040	1,105	30,398
Riprap	59,875	M3			1.460	87,418	2.420	144,898	2.50	149,688	0.340	20,358	402,362
Erosion Protection	7,800	M3			0.290	2,262	0.770	6,006			0.040	312	8,580
Reinforced Concrete Culvert Pipe: 48" Diameter R.C.P.	482	LM	114.900	55,382	5.470	2,637	9.130	4,401			1.370	660	63,080
Reinforced Concrete	309	M3	40.000	12,360	8.550	2,642	20.450	6,319	2.740	847	1.710	528	22,696
Gabions 12"	136,965	M2	3.250	445,136	0.750	102,724	1.400	191,751	2.310	316,389	0.280	38,350	1,094,350
Adjust Existing Wells	45	LM	10.000	450	2.000	90	2.500	113					653
Waterline Relocation		LS		10,000		2,150		5,000				400	17,550
Mobilization and Demobilization				68,750		33,446							102,196
Supervision						272,232							272,232
Overhead				147,516		67,060				79,680		37,558	331,808
Subtotal				759,588		789,067		877,881		546,604		136,939	3,110,079
Profit	10%												311,008
Escalation (To June, 1982 Bid Date)	7%												239,476
Contingency	10%												366,437
Total Base Bid in R.O. (Rials Oman)									Dollars		$11,776,000		4,027,000R.O.
Additive Bid Items													
Slide Gate Assembly	2	Ea	925	1,850	300	600	400	800	325	650	100	200	4,100R.O.
Debris Deflectors	11	Ea			30	330	60	660	500	5,500	110	1,210	7,700R.O.

Source: Stanley Consultants

Profit is included as 10 percent of all direct costs, mobilization, supervision, and overhead. Profit is split between foreign and local in the relationship of the ownership (60 percent foreign - 40 percent local).

The unit prices used in this estimate include escalation for the period of time from the start of construction to the finish; escalation from the date of preparation to the start of construction (June, 1982) is listed as a separate item on the cost estimate.

This estimate is divided by local and foreign costs. Foreign costs include depreciation of equipment, material shipped into Oman, foreign personnel, and overhead and profit on the foreign portion of the general construction contractor. The majority of craft labor which will be Pakistani or Indian are considered foreign personnel. Local costs include locally purchased materials, Omani labor, and taxes. The unit costs for reinforced concrete have been split between foreign and local components. This is to take into consideration and fact that cement, reinforcing steel, and fuel are imported, and sand, aggregate, and water are local components.

Various cost sources have been used in preparing cost estimates:

1. Past experience in work in Oman.
2. Mid-East Division of the Corps of Engineers' Unit Price Book for Oman.
3. Costs gathered by Stanley Consultants' estimating personnel while in Oman.

9.2 Engineering and Supervision Costs

The engineering design fee is a predetermined and negotiated item and is not considered a part of the total project cost.

Construction management services (engineering firm independent of contractor) to assure compliance with plans and specifications is considered as a part of the total project cost. These costs are summarized below:

Staff: Three expatriate personnel will be required on the project for sixteen months. These are a construction manager,

assistant construction manager, and a field inspector.

Assume an average of three people per family.

All costs have been escalated to February, 1983.

Travel: (Assume one round trip per person to the U.S. and one local leave trip)

18 round trips x $2,500 = $45,000

Housing and Utilities:

3 facilities @ $3,500/month x 16 months = $168,000

Direct Cost: Salary based on hourly fees (including benefits) for overseas personnel.

Construction Manager	16 mo. x $13,600/mo. = $217,600
Field Inspectors (2)	32 mo. x $8,600/mo. = $275,200
Local Hire:	
(Secretary, driver, etc.)	16 mo. x $5,000/mo. = $80,000

Miscellaneous Expense: Includes education, post allowance, relocation, insurance, passports, visas, and field office expenses.

Lump Sump	$138,000

Total Estimated Cost of Engineering Services for Construction Management

$923,800 Dollars

316,400 R.O.

9.3 Total Project Cost

Total project cost is the comprehensive cost of the project. This cost includes the construction cost and the cost for construction management services.

Construction Cost	4,027,000 R.O.
Construction Management Services	316,400 R.O.
Total Project Cost	4,343,400 Rials Omani
Total Project Cost	12,682,000 Dollars

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9.4 Operating and Maintenance Costs

Itemized below is an estimate of costs for the maintenance procedures as outlined in Part 4 of this report. Operating and maintenance cost is an additional cost and is not included in the total project cost.

Assuming 70,000 cubic meters of sediment to be excavated per year, maintenance cost would include:

1 - Front end loader
1 - Equipment Operator
1 - Oiler
2 - Dump truck 16.5 C.Y. capacity
2 - Drivers

Based on this crew, an annual cost for excavating is:

	Material and Labor	
Mobilization and Demobilization	Lump Sum	$ 5,000
Excavation	70,000 m^3 x $0.40	$28,000
Hauling	70,000 m^3 x $0.76	$53,200
		$36,200
		use $90,000/year
		or 30,825 R.O./year

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PART 10 - CONCLUSIONS AND RECOMMENDATIONS

10.1 Conclusions

This study examines the design and feasibility of recharging the aquifer beneath Wadi Al Khawd with storm water by infiltration. The preliminary recharge scheme was presented in a report by the Public Authority for Water Resources titled "Preliminary Engineering Design for Wadi Al Khawd Recharge Scheme."

The following summarizes our conclusions with regard to this project:

1. Our independent evaluation of the potential of the Wadi Al Khawd project, based upon review of existing hydrologic measurements, field infiltration tests, and soils analyses agrees substantially with previous estimates. It is estimated that the average additional groundwater recharge produced by this project will range between three and five million cubic meters per year.
2. Three design schemes were analyzed. They are:
   a. The preliminary design by the Public Authority for Water Resources which consisted of two permeable dams in series. The dams spread the water by sheet flow over wide leveled spreading basins to increase infiltration.
   b. Two impermeable dams with outlets that release the water into the existing wadi channels within their measured natural infiltration rate.
   c. One impermeable dam with outlets that release the water into the existing wadi channels within their measured natural infiltration rate.

Our analysis concluded that the one dam approach (c) is the most effective method of aquifer recharge, the least expensive and provides an equivalent factor of safety.

3. The construction cost of the project is estimated to be 4,027,000 Rials Omani (11,758,200 dollars). Construction management services is estimated to be 316,400 Rials Omani (923,800 dollars).
4. The construction period is 16 1/2 months. This includes the time required for mobilization and demobilization.
5. Two general purposes have been considered for the additional water. They are:
   a. For irrigation of agricultural crops.
   b. As a new groundwater source to feed the urban distribution system which serves the capital area.
6. The highest valued use of water appears to be urban water supply.
7. For economic analysis, it is assumed that 4.0 million cubic meters per year are pumped to the urban grid.
8. The primary benefit of the project is the cost savings in supplying this needed increment to the grid.
9. A project cost of 6,222,605 Rials Oman (18,168,190 dollars) is used for analyzing the economic feasibility of this project for water supply to the capital area. This estimate includes 1,867,405 Rials Omani (5,460,250 dollars) for ancillary facilities needed to pump the water increment to the water supply grid, and 11,800 Rials Omani (34,450 dollars) for additive bid items.
10. Comparing project benefits to project costs yields an estimated internal rate of return of 18.3 percent. Additional project benefits not included in the rate of return include:
    a. Experience gained and data collected for design, operation and management of future recharge projects elsewhere in Oman.
    b. Reduction in flood flows and related reduction in flood damages.
    c. Possible sale of sediment removed in the dam maintenance program.

11. A sensitivity analysis indicates that the project remains viable against a 10.0 percent criterion under relaxation of the most critical assumptions of the analysis.

12. The capital area (Muscat to Seeb) is presently experiencing rapid population growth and investment in urban infrastructure. Urbanization impacts will occur with or without this project. At this particular site there are no religious, social, or cultural barriers to the construction of the structure or use of the recharge water. Proper communication strategies will be required to prevent misunderstanding of project purposes and potentials.

13. The general character and wildlife habitat of the Wadi Al Khawd vicinity will remain essentially unchanged. There will be insignificant reduction in flora. Therefore, the adverse environmental impacts of this project will be insignificant.

10.2 Recommendations

The need for water in arid Oman is substantial. Aquifer recharge can be an important source of water supply. This project is not only economically, sociologically, and environmentally feasible, it is an important demonstration project for possible application elsewhere in Oman. We recommend its construction and development.

Respectfully submitted,
STANLEY CONSULTANTS, INC.

By G. F. Tavener
G. F. Tavener
Hydrology/Hydraulic Engineer

By Duane T. Kexel
Duane T. Kexel
Economist

By Stephen R. Yankee
Stephen R. Yankee
Technical Manager

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Table 1 - Provisional List of Mammals of the Wadi Al Khawd Vicinity

Species	English Name	Distribution	Characteristics
Order INSECTIVORA			
Paraechinus aethiopicus dorsalis	Ethiopian Hedgehog	Throughout the arid desert and steppe of the Arabian Peninsula.	Common species of the low land plains of the Batinah Coast. Nocturnal in activity.
Order CHIROPTERA			
Rhinopoma muscatellum	Oman Mouse-tailed Bat	Northern Oman mountains and foothills widespread, inhabits small dry caverns.	
Taphazous nudiventris zayidi	Zayid's Sheath-tailed Bat	Northern Oman.	Crevice dwelling species; usually found in large colonies.
Taphazous perforatus	Tomb Bat	Northern Oman. Possible inhabitant of project area.	
Asellia tridens tridens	Trident Bat	Throughout Oman. Possible inhabitant of project area.	
Triaenops persicus macdonaldi	Persian Leaf-nosed Bat	Northern Oman. Possible inhabitant of project area.	
Myotis emarginatus desertorum	Notch-eared Bat	Seaward foothills of northern mountains. Possible inhabitants of project area.	

Table 1 - Provisional List of Mammals of the Wadi Al Khawd Vicinity

Species	English Name	Distribution	Characteristics
Eptesicus nasutas batinensis	Sind Serotine Bat	Northern Oman. Possible inhabitant of project area.	
Pipistrellus bodenheimeri	Bodenheim's Pipistrelle	Seaward foothills of northern mountains. Possible inhabitants of project area.	
Pipistrellus Kuhli ikhanius	Kuhl's Pipistrelle	Northern Oman. Possible inhabitants of project area.	
Order CARNIVORA			
Canis lupus arabs	Wolf	Throughout Oman. Unlikely visitor in project area.	
Vulpes vulpes arabica	Common Red Fox	The most abundant carnivore in Oman. Widely distributed throughout the country. Principally nocturnal.	
Ichneumia albicauda albicauda	White-tailed Mongoose	Batinah; seaward foothills. Inhabitant of vegetated areas adjacent to the Gulf of Oman.	
Hyaena hyaena sultana	Stripe Hyaena	Batinah; unlikely visitor to project area.	
Felis silvestris gordoni	Gordon's Wildcat	Northern Batinah and seaward foothills. Marginal in project area.	

Table 1 - Provisional List of Mammals of the Wadi Al Khawd Vicinity

Species	English Name	Distribution	Characteristics
Order ARTIODACTYLA			
Capra ibex nubiana	Nubian Ibex	Throughout Oman.	
Ovis ammon	Asiatic Mouflon; Argali; Red Sheep	Northern Oman.	
Gazella gazella muscatensis	Muscat Gazelle	Northern Oman.	
Order LAGOMORPHA			
Lepus capensis omanensis	Oman Hare	Northern Oman. One of the most versatile Arabian mammals.	
Order RODENTIA			
Jaculus jaculus vocatar	Three-toed Jerboa	Occurs throughout the greater part of the desert and steppe regions of the Batinah.	
Rattus rattus rattus	Black Rat; House Rat	Occurs throughout northern Oman. A versatile, adaptable rodent.	
Mus musculus gentilulus	House Mouse	Abundant in Oman both as a commensal and as a free-living form.	
Acomys dimidiatus homericus	Spiny Mouse	A widespread species favoring rocky habitat. Predominantly nocturnal.	

Table 1 - Provisional List of Mammals of the Wadi Al Khawd Vicinity

Species	English Name	Distribution	Characteristics
Gerbillus nanus setonbrownei	Seton-Browne's Gerbil	Occurs on the Batinah Coast.	
Mertones crassus crassus	Sundevall's Jird	Occurs in sandy hillocks, especially those surrounded by thorny bushes. Predominantly nocturnal.	

Sources: Mammals

The Mammals of Arabia; Volumes I, II, III; David L. Harrison; 1964; Ernest Benn Limited; London.

The Scientific Results of the Oman Flora and Fauna Survey 1975; Ministry of Information and Culture; Sultanate of Oman.

Environment and Natural Resources Profile on the Sultanate of Oman; Arid Lands Information Center Office of Arid Lands Studies; University of Arizona; 1981.

Personal Communication with M. D. Gallagher, Assistant Advisor (Field Studies); Office of the Adviser for the Conservation of the Environment, Division of HM for Protocol.

Table 2 - Provisional List of Birds of the Wadi Al Khawd Vicinity

Species	English Name	Distribution	Characteristics
Family ACCIPITRIDAE			
Neophron percnopterus percnopterus	Egyptian Vulture	Resident; nesting in isolated pairs. Concentrates at food.	
Family FALCONIDAE			
Falco tinnunculus	Eurasian Kestrel	Passage migrant, winter visitor and partial resident, breeding locally above 600 m.	
Family PHASIANIDAE			
Ammoperdix heyi intermedia	Sand Partridge	Breeding resident about 600 m.	
Francolinus pondicerianus pondicerianus	Grey Francolin	Common breeding resident on Batinah, less common near foothills.	
Family CHARADRIIDAE			
Hoplopterus indicus	Red-wattled Lapwing	Resident, possibly breeding in project area.	
Family SCOLOPACIDAE			
Tringa ochropus	Green Sandpiper	Passage migrant and winter visitor.	
Tringa glareola	Wood Sandpiper	Passage migrant and winter visitor.	

Table 2 - Provisional List of Birds of the Wadi Al Khawd Vicinity

Species	English Name	Distribution	Characteristics
Family PTEROCLIDIDAE			
Pterocles lichtensteinii lichtensteinii	Lichtenstein's Sandgrouse	Locally common resident with seasonal movements.	
Pterocles exustus exustus	Chestnut-bellied Sand-grouse	Resident in nearby area, may occur in project area.	
Family COLUMBIDAE			
Columba livia palaestinae	Rock Dove	Resident above 600 m. in singles, pairs, and small flocks. May occur in project area.	
Streptopelia decaocto	Collared Dove	Local breeding resident and visitor; numbers vary annually.	
Streptopelia senegalensis cambayensis	Palm Dove	Breeding resident, also a passage migrant.	
Family STRIGIDAE			
Otus brucei	Bruce's Scops Owl	Migrant, possible breeding resident. May occur in project area.	
Family APODIDAE			
Apus pallidus pallidus	Pallid Swift	Resident and migratory.	

Table 2 - Provisional List of Birds of the Wadi Al Khawd Vicinity

Species	English Name	Distribution	Characteristics
Family CORACIDAE			
Coracias benghalensis	Indian Roller	Local breeding resident with seasonal movements; also a winter visitor and passage migrant.	
Family MEROPIDAE			
Merops orientalis muscatensis	Little Green Bee-eater	Common breeding resident with seasonal movements.	
Family ALAUDIDAE			
Ammomanes deserti taimuri	Desert Lark	Very local breeding resident in or near hills, often near wadis.	
Galerida cristata	Crested Lark	Breeding resident with seasonal movements; possibly also a winter visitor.	
Family HIRUNDINIDAE			
Ptyonoptogne fidigula obsoleta	Pale Crag Martin	Breeding resident with seasonal movements.	
Family MOTACILLIDAE			
Motacilla emerea	Grey Wagtail	Passage migrant and winter visitor.	

Table 2 - Provisional List of Birds of the Wadi Al Khawd Vicinity

Species	English Name	Distribution	Characteristics
Motacilla alba	White Wagtail	Passage migrant and winter visitor, fairly common and widespread.	
Family PYCNONOTIDAE			
Pycnonotus barbatus xanthopygos	Yellow-vented Bulvul	Breeding resident, widespread.	
Family SYLVIIDAE			
Prinia gracilis	Graceful Warbler	Breeding resident in or near cultivation.	
Family TIMALIIDAE			
Turdoides squamiceps muscatensis	Arabian Babbler	Breeding resident.	
Family NECTARINIIDAE			
Nectarinia asiatica brevirostins	Purple Sunbird	Breeding resident with local movements; partial migrant.	
Family LANIIDAE			
Lanius excubitor aucheri	Great Grey Shrike	Resident.	
Family CORVIDAE			
Corrus splendens	House Crow	Resident; may occur in project area.	

Table 2 - Provisional List of Birds of the Wadi Al Khawd Vicinity

Species	English Name	Distribution	Characteristics
Corrus ruficollis ruficollis	Brown-necked Raven	Breeding resident.	
Family PASSERIDAE			
Passer domesticus hufafae	House Sparrow	Breeding resident near habitation.	
Petronia xanthocollis transfuga	Yellow-throated Sparrow	Breeding visitor.	
Family ESTRIIDIDAE			
Euodice malabarica	Indian Silverbill	Resident, may occur in project area.	
Family EMBERIZIDAE			
Emberiza striolata striolata	House Bunting	Local breeding resident in mountains and foot-hills. Almost to sea level.	

Sources: Birds

The Scientific Results of the Oman Flora and Fauna Survey 1975; Ministry of Information and Culture; Sultanate of Oman.

Environment and Natural Resources Profile on the Sultanate of Oman; Arid Lands Information Center Office of Arid Lands Studies; University of Arizona; 1981.

Personal Communication with M. D. Gallagher, Assistant Advisor (Field Studies); Office of the Adviser for the Conservation of the Environment, Division of HM for Protocol.

Table 3 - Provisional List of Reptiles and Amphibians of the Wadi Al Khawd Vicinity

Species	English Name	Distribution	Characteristics
Family GEKKONIDAE (GECKOS)			
Bunopus tuberculatus		Most of Arabia; low lands and deserts.	
Syrtodactylus scaber		Coastal areas.	
Heridactylus turcicus		Foothills, deserts near coastal areas.	
Hemidactylus flaviviridis		Coastal areas.	
Pristurus rupestris		Coastal mountains and foothills.	
Ptyodactylus hasselquistii		Mountains and foothills; rocky places.	
Family AGAMIDAE (AGAMID LIZARDS)			
Agama sinaita		Rocky wadis, cliffs, rock outcrops.	
Uromastyx thomasi		Foothills, desert fringe; may occur in project area.	
Uromastyx microlepis		Desert.	
Family CHAMAELEONIDAE (CHAMAELEONS)			
Chamelec chamaeleon		Foothills; desert fringe; may occur in project area.	

Table 3 - Provisional List of Reptiles and Amphibians of the Wadi Al Khawd Vicinity

Species	English Name	Distribution	Characteristics
Family SCINCIDAE (SKINKS)			
Chalcides ocellatus		Coastal areas; may occur in project area.	
Family LACERTIDAE (LACERTIDAE LIZARDS)		Coastal areas; may occur in project area.	
Eremias adramitana		Abundant on sandy and gravelly plains near mountains.	
Family LEPTOTYPHLOPIDAE (BLIND SNAKES)			
Leptotyphlops macrorhynchus		Coastal areas; may occur in project area.	
Family COLUBRIDAE (TYPICAL SNAKES)			
Spalerosphis diadema		Coastal areas; may occur in project area.	
Telescopus dhara		Coastal areas; may occur in project area.	
Family VIPERIDAE			
Echis coloratus		Widespread in rocky areas; frequently encountered in and around wadis with some surface water.	
Echis carinatus		Foothills; desert fringe.	

Table 3 - Provisional List of Reptiles and Amphibians of the Wadi Al Khawd Vicinity

Species	English Name	Distribution	Characteristics
Family CURONIDAE (TYPICAL TOADS)			
Bufo orientalis		Numerous in wadis with some surface water, occurring on gravel areas and rock exposures.	
Bufo dhufarensis		Dry, flattish, often gravelly areas.	

Sources: Reptiles and Amphibians

The Scientific Results of the Oman Flora and Fauna Survey 1975; Ministry of Information and Culture; Sultanate of Oman.

Draft Environment and Natural Resources Profile on the Sultanate of Oman; Arid Lands Information Center Office of Arid Lands Studies; University of Arizona; 1981.

Personal Communication with M. D. Gallagher, Assistant Advisor (Field Studies); Office of the Adviser for the Conservation of the Environment, Division of HM for Protocol.

Computed by J.A. Hollatz Date 7-8-81
Checked by P. Petitti Date 7-9-81
Reviewed by J. Hollatz Date 10-15-81
Approved by [illegible] Date 12-17-81

Job No. 8069-01-397 Page No.
Subject Wadi Recharge
Cost Estimate
USAID
Sheet No. 1 of 5

ESCALATION OF CONSTRUCTION COSTS IN OMAN

THE FOLLOWING ESCALATION PROJECTIONS ARE BASED ON DISCUSSION WITH CONTRACTORS IN THE OMAN AREA AND ACTUAL BID INFORMATION FROM THE MIDEAST DIVISION OF THE CORPS OF ENGINEERS.

INFORMATION OBTAINED IN FEB 1981.

JAN 1978

14 %

JAN 1979

12 %

JAN 1980

12 %

JAN 1981

10 %

JAN 1982

JAN 1979 TO JAN 1981 = 1.12 x 1.12 = 1.25 = 25 %

ESCALATION TO JUNE 1982 BID DATE = 7 %

3004

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Computed by J.A. Hollatz Date 6-26-81
Checked by P. Petitti Date 7-7-81
Reviewed by J. Hollatz Date 10-15-81
Approved by [illegible] Date 12-17-81

Job No. 8069-01-397 Page No.
Subject Cost Estimate
USAID
Wadi Al Khawd
Sheet No. 2 of 5

Mobilization & Demobilization

Personnel (Recruiting + Travel to Oman)

English	12 men @ $2,000	$24,000
Pakistani	100 men @ $750	75,000

Equipment (Available in Oman)

Camp Facilities Assume personnel will live in capital area and take bus to the job site. Contractor must provide:

Site Offices	500 m2 @ $50 =	25,000
Workshop	600 m2 @ $35 =	21,000
Toilets	12 ea @ $300 =	3,600
		$148,600
		50,821 R

Contractor Supplied Cars, Eng. Office, and Operation 17,929

68,750 R

3004

110

Computed by P. PETITTI Date 7-15-81
Checked by JA HOLLATZ Date 10-15-81
Reviewed by JA HOLLATZ Date 10-15-81
Approved by [illegible] Date 12-17-81

Job No. 8069-01 Page No.
Subject COST ESTIMATE
VSAID
WADI AL KHAWD
Sheet No. 3 of 5

SUPERVISION

HOME OFFICE

PROJECT MANAGER	½ TIME	x 14 MO @ 6,000 =	$ 42,000
BOOK KEEPER/CLERK		x 14 MO @ 2,000 =	28,000

ENGLISH SUPERVISION

GENERAL SUPERINTENDANT	2 EA x 12 MO @ 5,000	=	120,000
CIVIL SUPERINTENDANT	2 EA x 12 MO @ 4,500	=	108,000
QUALITY CONTROL CHIEF	2 EA x 12 MO @ 4,500	=	108,000
CONTRACT ADMINISTRATOR/COMPTROLLER	2 EA x 12 MO @ 3,500	=	84,000

SOILS TESTING SERVICE

CIVIL SUPERINTENDANT	12 MO @ 4,500	=	54,000
TEST PERSONNEL	3 EA x 12 MO @ 4,000	=	144,000

THIRD WORLD SUPERVISION

IN OMAN THERE IS AN ADDITIONAL GRADE OF SUPERVISION BETWEEN THE DISCIPLINE SUPERINTENDANT AND THE CRAFT FOREMAN

	6 EA x 12 MO @ 1,500	=	108,000
			$ 796,000
			272,232 R.O.

3004

Computed by J.A. Hollatz Date 6-26-81
Checked by P. Petitti Date 7-7-81
Reviewed by J. Hollatz Date 10-15-81
Approved by [illegible] Date [illegible]

Job No. 8069-01-397 Page No.
Subject Cost Estimate
USAID
Wadi Al Khwad
Sheet No. 4 of 5

		Foreign	Local
Overhead			
Non-Productive Overhead Personnel			
Accountant	12 mo @ $900=	10,800	
Clerk/Typists	12 mo @ $750=	9,000	
Surveyors and Quality Control Inspectors	8 men @ 12 mo @ $1,000=	96,000	
Watchman & Guards	3 men @ 12 mo @ $400=		4,80
Maintenance & Motor Pool			
Supervisor	12 mo @ $1,500	18,000	
Helper	2 men @ 12 mo @ $700		16,4
Misc. Laborers	4 men @ 12 mo @ $500		24,00
Housing & Food			
Housing			
English	12 men @ 12 mo @ $400		57,6
Third World	100 men @ 12 mo @ $200		240,0
Food			
English	12 men @ 12 mo @ $475	68,400	
Third World	100 men @ 12 mo @ $225	270,000	

3004

Computed by: J.A. Hollatz Date: 6-26-81
Checked by: P. Petitti Date: 7-7-81
Reviewed by: J. Hollatz Date: 10-15-81
Approved by: [illegible] Date: 12-17-81

Job No. 8069-01-397 Page No.
Subject: Cost Estimate
USAID
Wadi Al-Khawd
Sheet No. 5 of 5

	Foreign	Local
Overhead (cont.)		
Insurance		
Builders Risk, Liability, Etc		
1% of Direct Costs	40,000	
Money Costs		
Interest Costs for Financing		
2% of Direct Costs	80,000	
Bank Letters		
0.4% of Direct Costs	16,000	
Home Office Costs		
Travel 4 trips @ $3,000	12,000	
General Administration		
12 mo @ $600 =	7,200	
	$ 627,400	$ 342,800
	R.O. 214,570	117,238

3004

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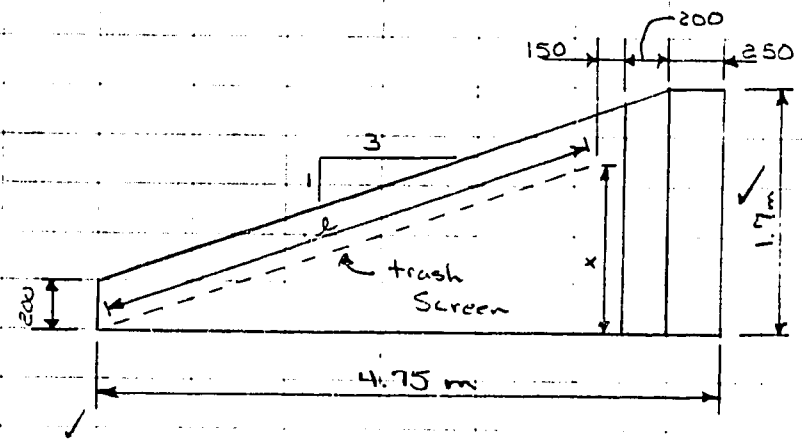
STRUCTURAL COMPUTATIONS

Job No. 8069 Page No.

Computed by D.G. Rogers Date 10-9-81
Checked by W. TYNER Date 10/14/81
Reviewed by Date
Approved by J.R. Varner Date 12/14/81

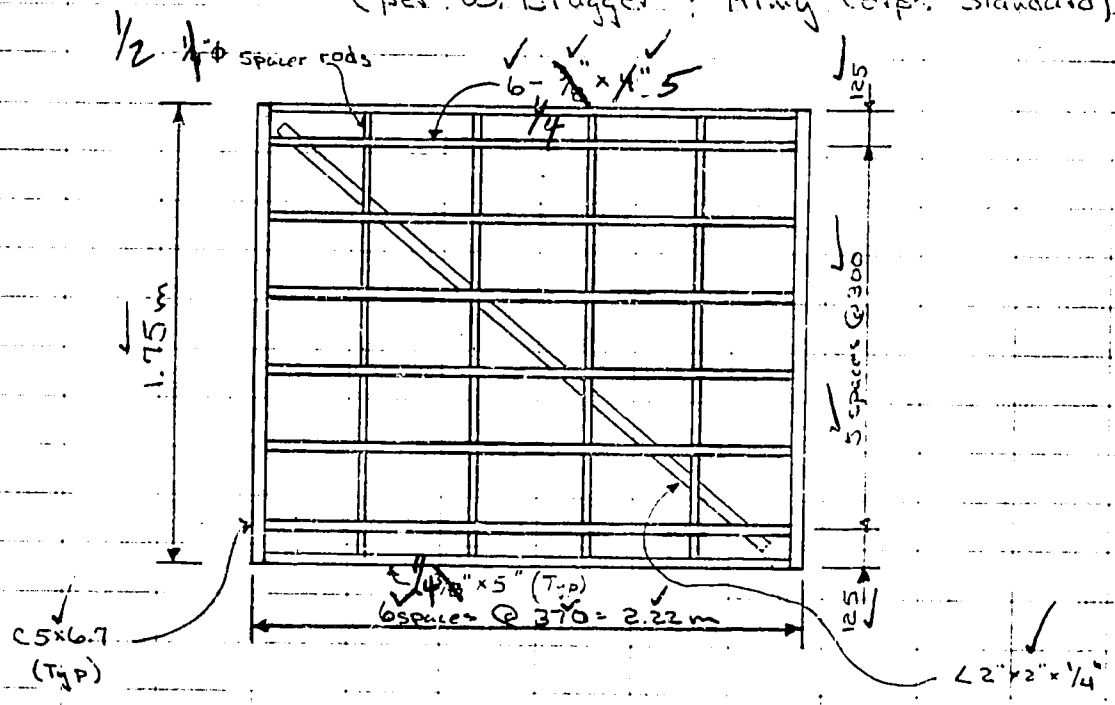
Subject Structural
Wadi Al Khawd
Trash Screen

Sheet No. 1 of 5



$$\frac{x}{4.15\,m} = \frac{1.7\,m}{4.5\,m} \qquad \underline{x = 1.567\,m} \qquad \therefore\ \ell = \sqrt{(1.567)^2 + (4.15)^2} = \underline{4.44\,m}$$

Divide screen into two sections (per S. Yankee)
Assume a 5' head difference and screen plugged
(per W. Brugger & Army Corps Standard)



Top Plan (2 Sections Req'd)

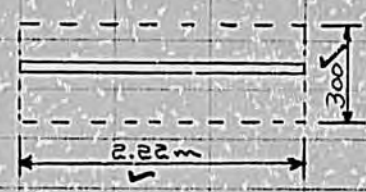
Galvanize after fabrication

.004

Computed by D.G. Rogers Date 10-12-81
Checked by W. TYNER Date 10/14/81
Reviewed by Date
Approved by Date 12/14/81

Job No. 8069 Page No.
Subject Structural
Wadi Al Khawd
Trash Screen
Sheet No. 2 of 5

Assume screen plugged



$w = 5'(.0624\ kcf)(1') = .312\ k/'$

$M = \frac{(.312\ k/')(2.22m \times 3.28)^2}{8} = 2.09\ k\text{-}'$

try 5" x 1/4" bar

$r_x = \sqrt{\frac{I_x}{A}} = \sqrt{\frac{1/4(5")^3/12}{1/4(5)}} = 1.44\ in$ $r_y = \sqrt{\frac{5(1/4)^3/12}{1/4(5)}} = .072\ in$

$kl/r \leq 200$ $\therefore \frac{(1)l}{.072} \leq 200$ $l = 14.4"$

for 1/4" x 5" bar $S = \frac{1/4(5)^2}{6} = 1.04\ in^3$

$f_b = \frac{2.09(12)}{1.04in^3} = 24\ ksi < 27\ ksi$ use 1/2" ϕ spacer rods @ 370

$\Delta = \frac{5(.312 \times \frac{1}{12})(2.22 \times 3.28 \times 12)^4}{384(29{,}000)\left(\frac{1/4(5)^3}{12}\right)} = .26" \approx l/336$ O.K.

End Channels:

$w = 1/2(.312\ k/')(2.22 \times 3.28) = 1.14\ k/'$

$M = \frac{1.14(1.75 \times 3.28)^2}{8} = 4.68\ k\text{-}'$

Try C5x6.7

$f_b = \frac{4.68\ k\text{-}'(12)}{3in^3} = 18.72\ ksi < 22\ ksi$ O.K.

3001

Computed by D.G. Rogers Date 10-14-81
Checked by W. TYNER Date 10/14/81
Reviewed by ..... Date .....
Approved by ..... Date 12-14-81

Job No. 8069 Page No. .....
Subject Structural
Wadi Al Khawd
Trash Screen
Sheet No. 3 of 5

Wt. of Section:

$8 \times [1/4" \times 5" \times (2.22m \times 3.28)] (1/144) (490\#/ft) = 248\#$

$2 \times [1.75 \times 3.28] (6.7\#/ft) = 77\#$

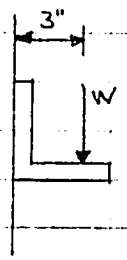
$5 \times [1.67\#/ft] (1.75 \times 3.28) = 48\#$

$1 \times [3.19\#/ft] (2.6 \times 3.28) = \cancel{54\#}\ 27$

$\cancel{427\#}$

400

Support Angles



$W = 5'(.0624\ kcf)(1/2 \times 1.75m \times 3.23)$
$+ 1/2(.427^k)(1/(3.22 \times 3.28))$
$= .92\ k/1$

for a 1' strip $S = \frac{12(t)^2}{6}$ $F_b = 22\ ksi$

$M = .92^k(3") = 2.78^{k-"}$

$22\ ksi = \frac{2.78}{12(t^2)/6}$ $t = .251"$

Use $\angle\ 3" \times 3" \times 3/8"$ connect to wall

with 3/8" Φ Exp Anchors @ 1'-6"

Check bearing:

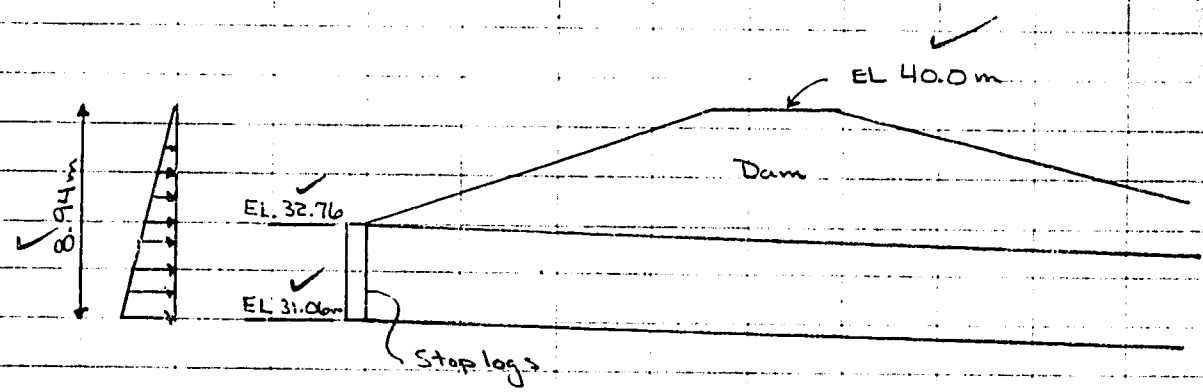
$.92^k/1\ (1.5') = 1.38^k$

$f_b = \frac{1.38^k}{3/8" \times 3/8"} = 9.81\ ksi < 33\ ksi$ O.K.

3004

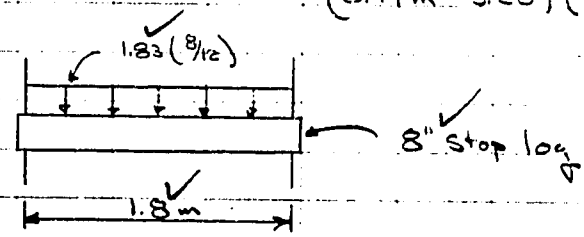
Computed by D.G. Rogers Date 10-9-81
Checked by W. TYNER Date 10/14/81
Reviewed by Date
Approved by Date 12-14-81

Job No. 8069 Page No.
Subject Structural
Wadi Al Khawd
Stop Logs
Sheet No. 4 of 5



Assume load for design is:

$(8.94\,m \times 3.28)(.0624\,kcf) = \underline{1.83\,kcf}$



$$M = \frac{1.83(8/12)(1.8 \times 3.28)^2}{8} = \underline{5.32^{k-1}}$$

for an 8" x 8" log : $S = \frac{8(8)^2}{6} = 85.33\,in^3$

$$f_b = \frac{5.32(12)}{85.33} = .748\,ksi < 1.2\,ksi$$

Use 8" x 8" Douglas fir $f_b = 1200$ psi Creosote logs

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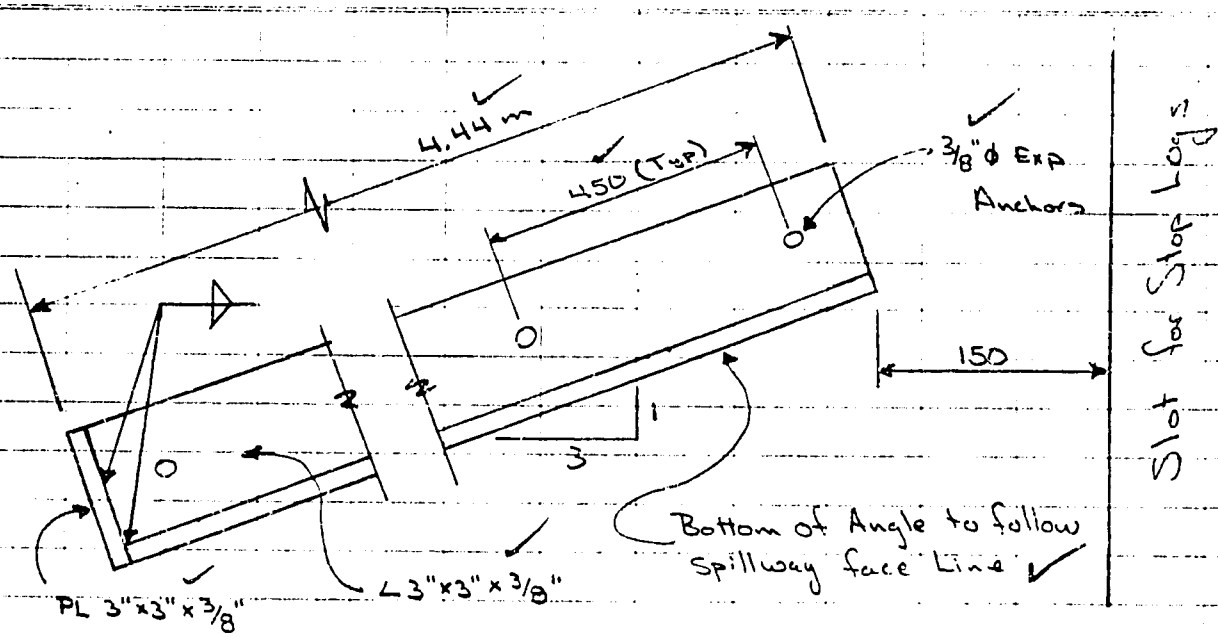
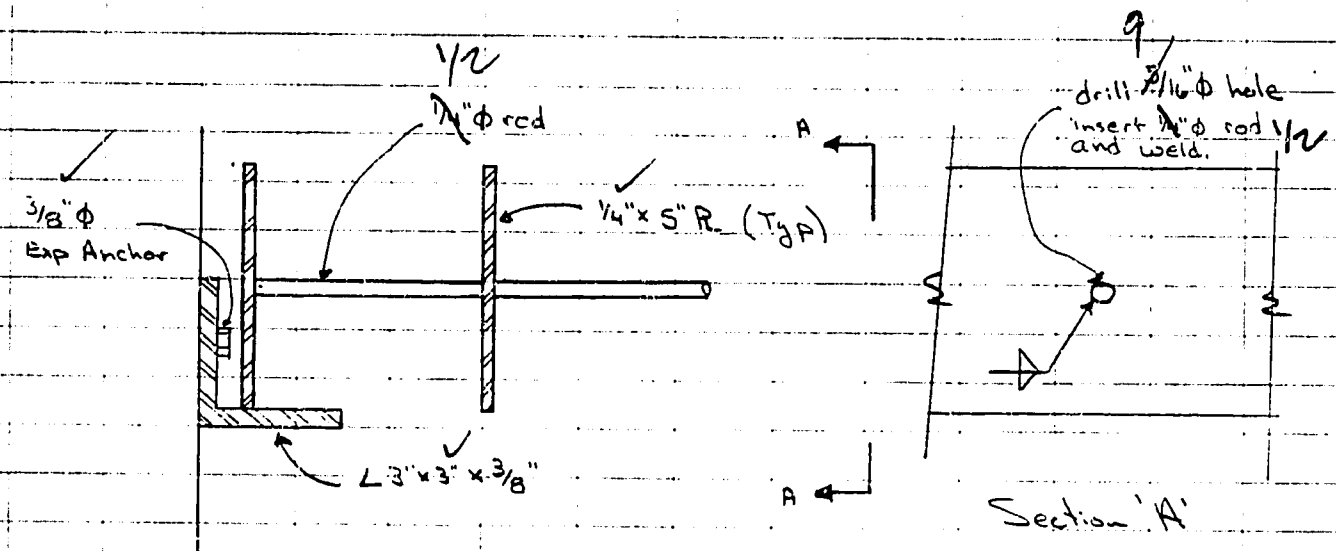
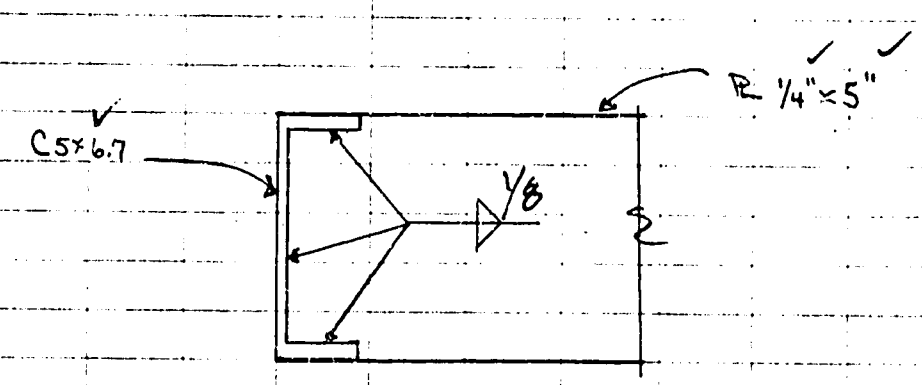
Computed by D.G. Rogers Date 10-14-81
Checked by W. TYNER Date 10/14/81
Reviewed by Date
Approved by Date 12-14-81

Job No. 8069 Page No.
Subject Structural
Wadi Al Khawd
Trash Screen
Sheet No. 5 of 5





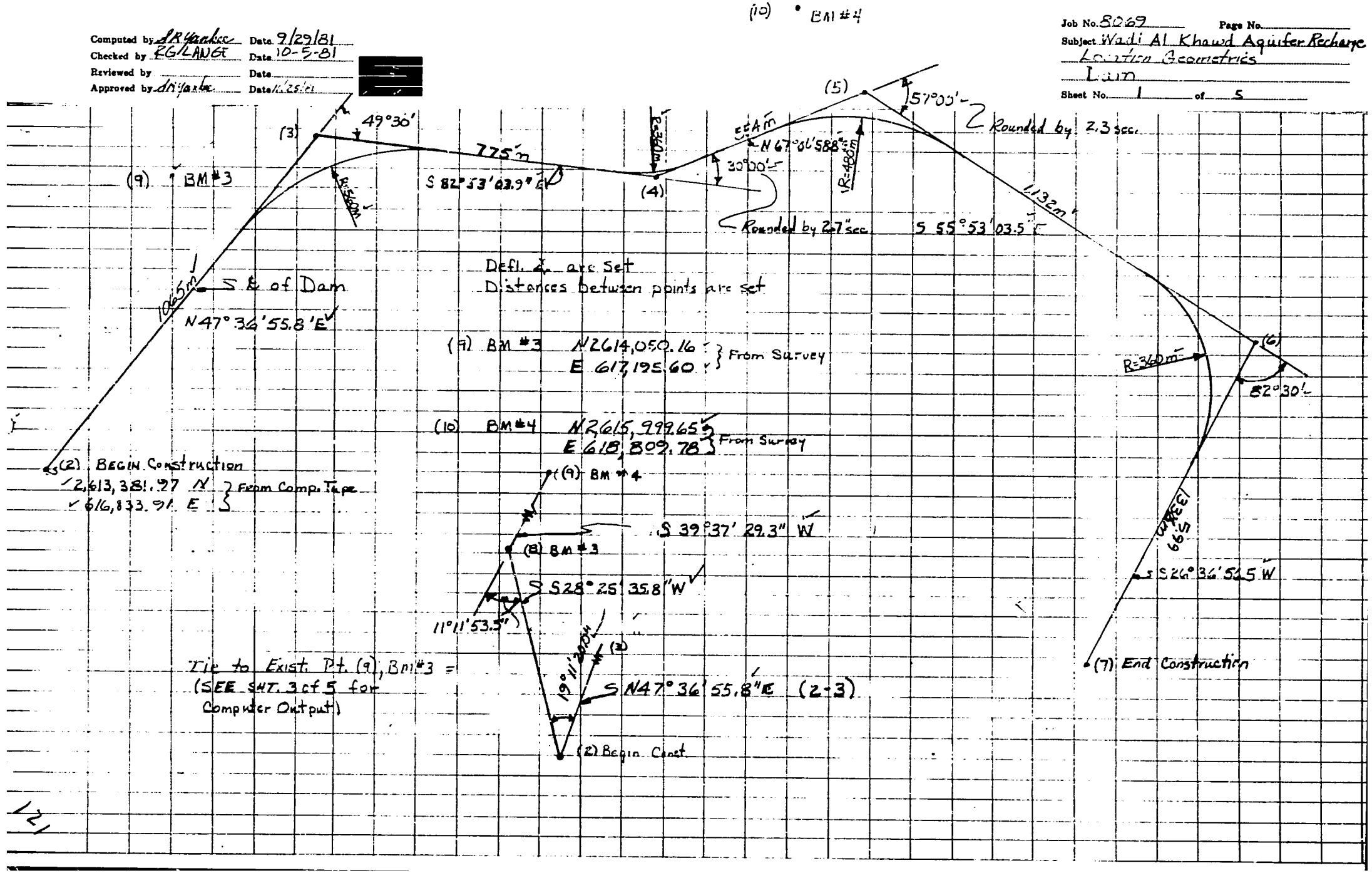


.3004

LOCATION COMPUTATIONS

Computed by A.R. Yankee Date 9/29/81
Checked by R.G. LANGE Date 10-5-81
Reviewed by _____ Date _____
Approved by [illegible] Date [illegible]

Job No. 8069 Page No. _____
Subject Wadi Al Khawd Aquifer Recharge
Location Geometrics
Lain
Sheet No. 1 of 5



121

Computed by: DR Yonker Date: 11/18/81
Checked by: RG LANGE Date: 11-18-81
Reviewed by: Date:
Approved by: DR Yonker Date: 11/25/81

Job No. 8069 Page No.
Subject: Wadi Al Khaud Aquifer Recharge
Location Geometrics
Dam - Curve Data
Sheet No. 2 of 5

```
CMDD 1

STARTING FILE 15

E*A
PT                2
N.     13381.97000
E.     15833.91000
----------------
E*A
PT                3
N.     14099.89000
E.     17620.56000
----------------
E*A
PT                4
N.     14003.89000
E.     18389.59000
----------------
E*A
PT                5
N.     14219.32000
E.     18899.99000
----------------
E*A
PT                6
N.     13584.42000
E.     19837.18000
----------------
E*A
PT                7
N.     12390.00000
E.     19238.65000
----------------
E*A
PT                8
N.     14050.16000
E.     17195.60000
----------------
E*A
PT                9
N.     15999.65000
E.     18809.78000
----------------
INV
S*W       39.37293
DIST    2531.02515

PT                8
N.     14050.16000
E.     17195.60000
----------------
INV
S*W       28.25358
DIST     759.80098

PT                2
N.     13381.97000
E.     15833.91000
----------------
INV
N*E       47.36558
DIST    1065.00110

PT                3
N.     14099.89000
E.     17620.56000
----------------
INV
S*E       82.53039
DIST     774.99880

PT                4
N.     14003.89000
E.     18389.59000
----------------
INV
N*E       67.06588
DIST     554.00203

PT                5
N.     14219.32000
E.     18899.99000
----------------
INV
S*E       55.50035
DIST    1131.99961

PT                6
N.     13584.42000
E.     19837.18000
----------------
INV
S*W       26.06565
DIST    1335.99200

PT                7
N.     12390.00000
E.     19238.65000
----------------
```

Note: All N coordinates all 100,000
All E Coordinates all 200,000

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Computed by [illegible] Date [illegible]
Checked by RG LANGE Date 10-5-81
Reviewed by Date
Approved by [illegible] Date 11/25/81

Job No. [illegible] Page No.
Subject [illegible]
Sheet No. 3 of 5

POINT	N	E
1	—	—
[illegible]	2,613,381.97	616,833.91
[illegible]	2,614,099.89	617,620.56
[illegible]	2,614,003.89	618,389.59
[illegible]	2,614,219.32	618,899.99
[illegible]	2,613,584.42	619,837.15
7	2,612,390.00	619,238.65
(8) BM [illegible]	2,614,050.16	617,195.60
(9) BM [illegible] 1	2,615,999.65	618,809.78

3004

123

Computed by S.H. Yarbee Date 9/30/81
Checked by R.G. LANGE Date 10-5-81
Reviewed by Date
Approved by Date

Job No. 8069 Page No.
Subject Wadi Al Khawd Aquifer Recharge
Location Geometrics
Dam - Curve Data
Sheet No. 4 of 5

Eq. Used
$T = R \tan \Delta/2$
$L = \Delta/Dm(10)$
$E = T(\tan \Delta/4) =$
$Dm = \frac{572.958}{Rm}$

Curve #1

$\Delta = 49°30'$ RT.
$R = 560m$
$T = 258.16$
$L = 483.81$
$E = 56.64$

Curve #2

$\Delta = 30°00'$ LT.
$R = 360m$
$T = 96.46$
$L = 188.50$
$E = 12.70$

Curve #3

$\Delta = 57°00'$ RT.
$R = 480m$
$T = 260.62$
$L = 477.52$
$E = 66.19$

Curve #4

$\Delta = 82°30'$ RT.
$R = 360m$
$T = 315.71$
$L = 518.36$
$E = 118.82$

3004

12.4

Computed by S R Yankee Date 10/29/81
Checked by RGLANGE Date 11-18-81
Reviewed by Date
Approved by Date

Job No. 8069 Page No.
Subject Wadi Al Khawd
Aquifer Recharge
Dam - Stationing
Sheet No. 5 of 5

(2)	10+000.00	Begin Construction
	1+065.00	
(3)	11+065.00	P.I. #1
	-258.16	
	10+806.84	P.C.
	483.81	
	11+290.65	P.T.
	775.00	
	12+065.65	
	- 258.16	
(4)	11+807.49	P.I. #2
	-96.46	
	11+711.03	P.C.
	188.50	
	11+899.53	P.T.
	554.00	
	12+453.53	
	-96.46	
(5)	12+357.07	PI #3
	-260.62	
	12+096.45	P.C.
	477.52	
	12+573.97	P.T.
	1+132.00	
	13+705.97	
	-260.62	
(6)	13+445.35	P.I. #4
	-315.71	
	13+129.64	P.C.
	518.36	
	13+648.00	P.T.
	1+335.99	
	14+983.99	
	- 315.71	
(7)	14+668.28	End Construction

3004

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Job 8069
Wadi Al Khawd Aquifer Rech.
Location Geometrics
Field Bench Marks
Sheet 1 of 1

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AIR SURVEY & DESIGN INC.
AIR AND GROUND SURVEYORS
305 SPRING STREET
HERNDON, VIRGINIA 22070

TO Stanley Consultants
Stanley Building
Muscatine, Iowa 52761
Attention: Mr. S. Yankee

DATE October 14, 1981
SUBJECT Wadi Al Khawd

Dear Sir,

We list below the revised elevation for the Bench marks on the above named project.

BM 1	E 622431-96	N 2614585-08	Height	16•155
BM 2	E 620960-62	N 2613579-16	"	26•871
BM 3	E 617195-60	N 2614050-16	"	30•014
BM 4	E 618809-78	N 2615999-65	"	18•412

Sincerely,

SIGNED

FORM 5425-NA — MATTICK FORMS — WHEELING, IL 60090

All Bench Marks are steel pins set in Concrete

126

HYDRAULIC COMPUTATIONS

Job No. 8069-01 Page No.

Computed by G. Davison Date Nov. 1981
Checked by Date
Reviewed by Date
Approved by Date

Subject WADI AL KHAWD RECHARGE (SINGLE DAM CONCEPT) HYDROLOGIC & HYDRAULIC COMPS

Sheet No. of

SUBJECT	PAGES
FLOOD PEAK ~ FREQUENCY ESTIMATES	1, 2
NAT. CHANNEL INFILTRATION RATES (1977)	3
SINGLE EVENT FLOOD INFILTRATION RELATIONSHIP (VOL. AT CONST ROAD FROM VOL. AT AL KHAWD)	4
DERIVED VOL. PASSING CONST ROAD 1975 thru 1978	5
TOTAL ANNUAL DISCHARGE / FREQUENCY AT RETARDING DAM	6
FLOOD VOLUME FREQUENCY AND AVE. ANNUAL (COMP.)	7, 8
RECHARGE (INFILTRATION) D/S RETARDING DAM	9
PARTIAL DURATION SERIES FLOOD VOL. ABOVE BASE	10, 11
Typical wadi channel rating curves	12, 13
RATING CURVE, SINGLE 48-IN CULVERT	14, 15
RESERVOIR EMPTYING TIME (SINGLE DAM CONCEPT)	16
RESERVOIR - STORAGE - OUTFLOW (FOR RESERVOIR ROUTING)	17, 18
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AVE. ANNUAL LOSS OF SURFACE WATER TO SEA	27, 28
CHUTE FLOW PROFILE (MAX. PROB. FLOOD)	29, 30, 31, 32
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DAM U/S SLOPE PROTECTION	37, 38
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TYPICAL DAM CONSTRUCTION MATERIALS	40 - 42
DAM D/S SLOPE STABILITY	43 → 56
DAM BREAK ANALYSIS	57 - 59
FILTER ZONE MATERIAL SIZING	60
CULVERT QUANTITIES	61, 62

3004

[redacted]

Computed by G. Ravenau Date July 1981
Checked by [illegible] Date 12/81
Reviewed by ..... Date .....
Approved by [illegible] Date [illegible]

Job No. 8069-01 Page No. 1 (REVISE)
Subject Wadi Al Khawd Recharge
Flood Peak ~ Frequency
Estimated from discharge Measurem
Sheet No. 1 of 2

Note Early Float measurements were not timed to catch peak flood discharges and so cannot be used.
The Water Resource Department (with technical advice from The Food Agriculture Organization of United Nations) have begun to maintain more continuous measurements.
The gaging station at Al Khawd is now being maintained as a first order station and will have the measurement of the highest accuracy so far achieved in Oman.
Autographic stage recording began in April 1975

Taking a "water year" beginning at March 1 each year to include as many complete 12-month periods as possible and using data supplied in "Runoff Measurements in Oman" (FAO FIELD DOCUMENT No. 7, MAY 1978)

Year (beginning March 1)	Peak flood m³/sec	Plotting Position Weibull	Hazen	Used.	mag.
1974	340	2.50	2.67	2.59	2
1975	137	1.25	1.14	1.20	4
1976	520	5.00	8.00	6.50	1
1977	230	1.67	1.60	1.64	3

Mean Annual: 307 Std. dev.: 164.6

Plotting the above data (see sheet 2/2) and taking note that there is a possibility of zero flood in any year and also that The University of Arizona in its "Environment and Natural Resources Profile on The Sultanate of Oman", June '81 gave the estimated maximum probable flood as 1900 m³/sec, The plotted curve yields the following estimates

MPF	1900		m³/sec
100-year	823	1400	m³/sec
50-Year	734	~~1100~~	m³/sec
25-Year	643	~~950~~	m³/sec
10-Year	521	~~700~~	m³/sec
5-Year	425	~~500~~	m³/sec
2-Year	280	~~250~~	m³/sec

Note These estimates exceed ~~are almost double~~ the estimates given by PAWR May 1980, believed to have been arrived at from channel cross section width measurem
It is believed these higher values are more accurate as well as more conserv

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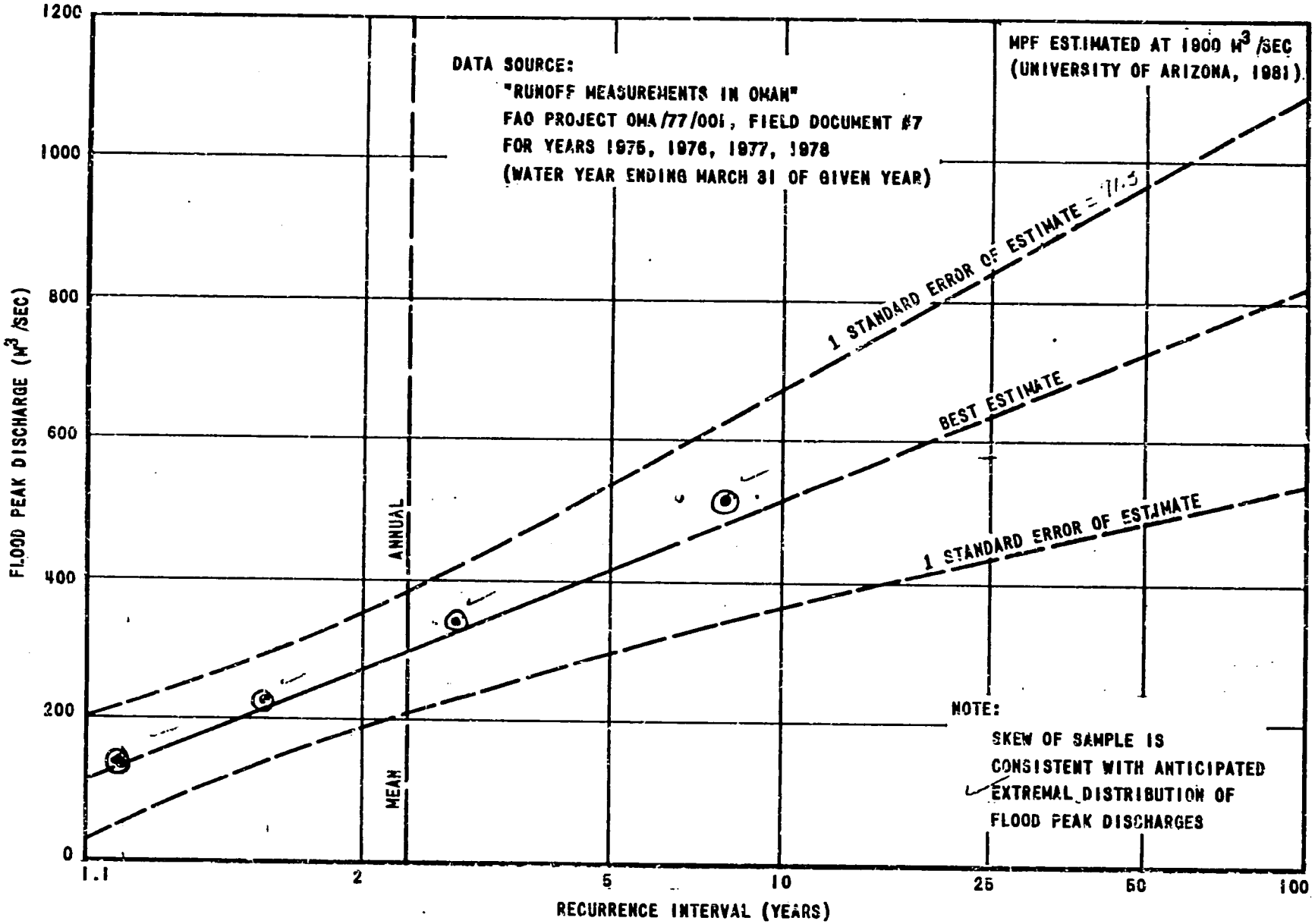

Note Hazen plotting positions fit theoretical curve best above.

ANNUAL PEAK DISCHARGE-FREQUENCY AT AL KHAWD
WADI AL KHAWD
FIGURE 3.1

P.2

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Job No. 8069-01 Page No. 3

Computed by G. Reuben Date June '81
Checked by R. Payne Date 12/81
Reviewed by Date
Approved by Date

Subject Wadi Al Khawd Recharge
Channel Recharge Rate D/S Retarding Structure
Sheet No. of

Estimates of the capability of the natural wadi channels d/s Al Khawd to absorb flow as recharge to the groundwater supply may be derived as follows:—

From "Preliminary Engineering Design for Wadi Al Khawd Recharge Scheme" (Prepared by Public Authority for Water Resources May 1980)

p. 1. "Usually, once every 4 or 5 years heavy rainfall generate storm runoff of large enough magnitude to allow Wadi Al Khawd to flow to the sea"

plate 4. of the same report gives.

2 year flood peak discharge = 130 ~~cfs~~ m^3/sec
5 year flood peak discharge = 250 ~~cfs~~ m^3/sec } Our est is higher

Consequently the natural wadi channel between Al Khawd and the sea must be capable of absorbing between 130 to 250 ~~cfs~~ m^3/sec as groundwater recharge.

This may be too high ✓

This estimate is confirmed by data contained in FAO Project OMA/77/001, Field Document No. 7 "Runoff Measurement in Oman"

Here corresponding flood peak discharges at Al Khawd and the coast road, may be directly compared. (See Table 4:4 sheet 2 and Table 4:5) of the above report

Date of Flood	Peak at Al Khawd	Peak at Coast Rd	Absorb
24/25. 5. 77	73 m^3/sec	< 20 m^3/sec	> 53
17/18. 11. 77	230 "	30 "	200
20/21. 11. 77	20 "	Tr "	20
25/26. 11. 77	105 "	34 "	71
8/10. 3. 78	170 "	51 "	119

+ Note this flood was right "on top" of a flood of two days previous the Tr may have been drainage from the first event

Average recharge capability of the above 5 events = 92.6 m^3/sec

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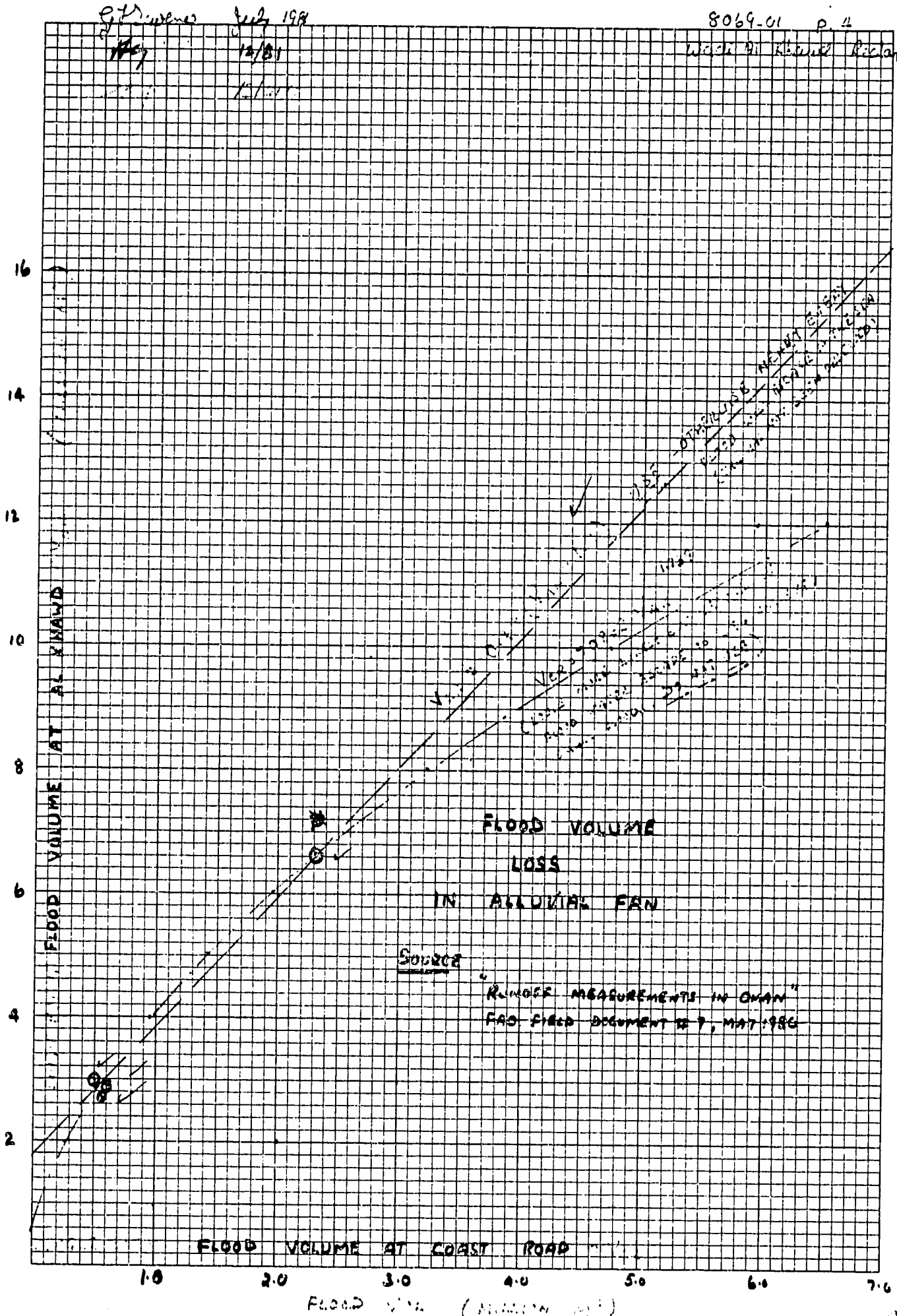

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Approved by: Date:

Job No. 8064-01 Page No. 5
Subject: Wadi Al Khawd Recharge
Lost Surface Water to Sea
(Existing Condition)
Sheet No. of

Assume Volume of flood water arriving at the coast road is related to the volume passing Al Khawd by the relationship:-

$$V_{CR} = 0.48(V_{AK} - 1.8) \text{ million } M^3.$$

And All flow crossing the coast road is lost to the sea

Where V_{CR} = Vol. of flood flow in any flood event crossing to

V_{AK} = Corresponding flood Vol. passing Al Khawd.

From 4 years data at Al Khawd – (Table 4.4 "Runoff Measurements in

Date	Flood Vol at Al Khawd	Flood Vol at coast road	($M^3 \times 10^6$)
3/4.10.74	.27	0.00	
9/11.2.75	4.73 [6]	1.41	
1/2.1.76	0.26	0.00	
29/30.1.76	0.43	0.00	Ave. flood Flow (Wadi Al Kha
2/6.2.76	4.07 [7]	1.06	16.81 × 10^6 m^3/year.
7/10.2.76	1.53 [8]	0.00	
12/13.3.76	0.55	0.00	
20/22.3.76	1.03	0.00	Ave. Spillage to sea
25/27.3.76	5.91 [5]	1.97	(natural condition)
5/10.4.76	16.36 [1]	6.99	4.83 × 10^6 m^3/year.
14.4.76	0.34	~~0.00~~	
22/23.4.76	0.73	0.00	
3/4.8.76	0.91	0.00	Note No. of floods
13/14.8.76	0.63	0.00	to sea in 4 years
15/18.9.76	0.78	0.00	= 10
10/13.10.76	0.82	0.00	Ave 2.5 events/year
21/24.1.77	6.00 [3]	2.02	This may have been more
24/26.2.77	6.00 [4]	2.02	than average since
23/27.5.77	2.90 [6]	0.53	"Once every 4 or 5 years
12/14.10.77	0.10	0.00	is regular
17/19.11.77	3.05 [9]	0.58	Std. Dev. of Annual Spillage
20/21.11.77	0.32	0.00	Volume
24/28.11.77	2.70 [10]	0.43	
11/13.2.78	0.13	0.00	
26/27.2.78	0.15	0.00	
8/11.3.78	6.60 [2]	2.30	
	67.25	19.31	

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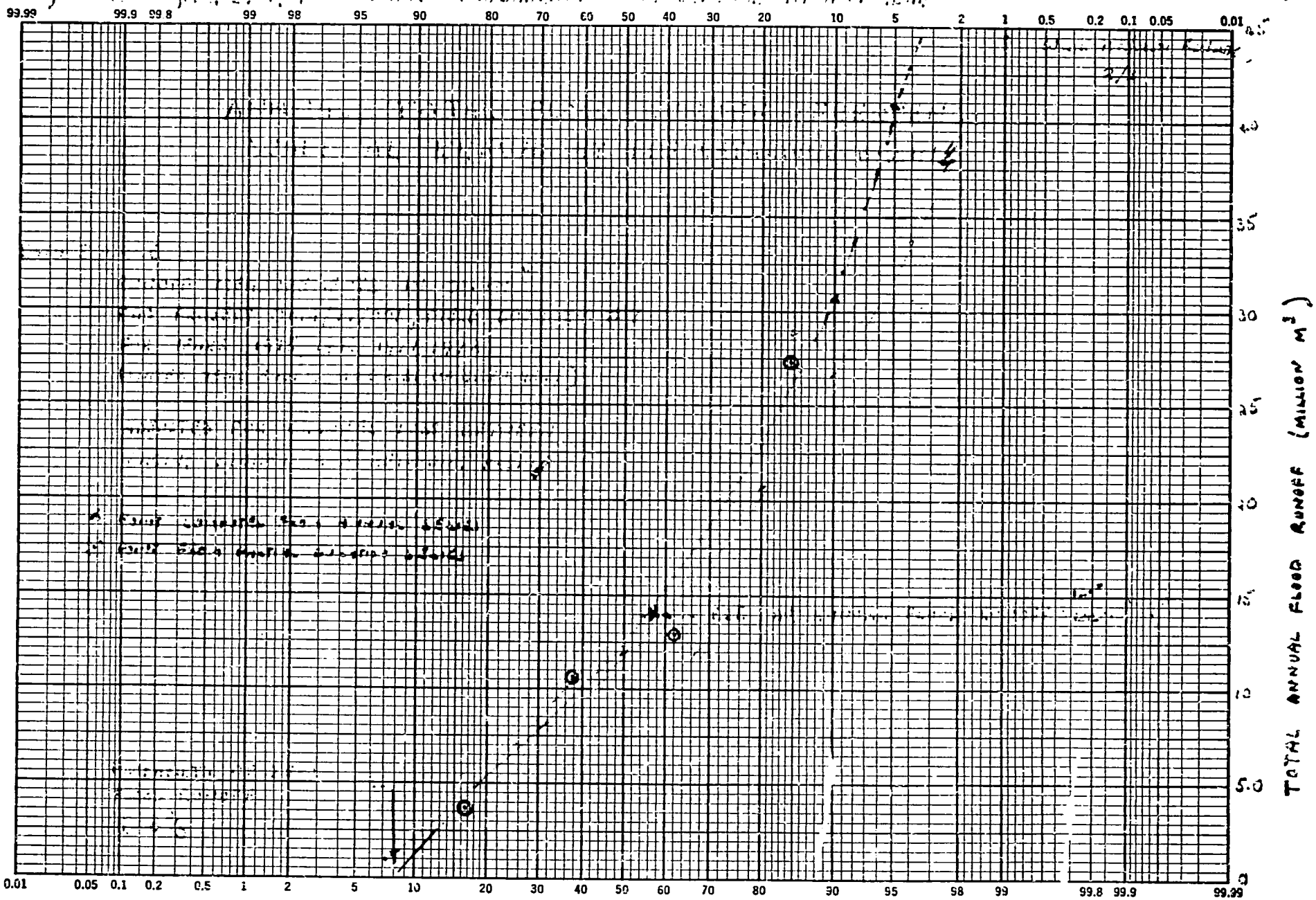

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Job No. 8069-01 Page No. 7

Computed by G. Rawson Date Nov. 1981
Checked by Hymne Date 12/81
Reviewed by Date
Approved by Date

Subject Wadi Al Khawd Recharge
EST. FLOOD VOL - FREQUENCY
ARRIVING AT SINGLE DAM
Sheet No. of

Assume 25% of total infiltration between Al Khawd and Coast Road gaging stations occurs between Al Khawd and the single dam.

FOR WATER YEARS BEGINNING MARCH 12

FLOOD VOLUME (M. M³)

WATER YEAR	SINGLE EVENT (Ann. Max.)			ANNUAL TOTAL		
	AL KHAWD	COAST RD.[1]	DAM	AL KHAWD	COAST RD.[1]	DAM
'74-75	4.73	1.41	3.90	5.00	1.41	4.10
'75-76	4.07	1.06	3.32	6.29	1.06	4.98
'76-77	16.36	6.99	14.02	40.06	13.00	33.30
'77-79	6.60	2.30	5.53	15.90	3.94	12.89
Average	7.94	2.94	6.69	16.81	4.83	13.82
Std. dev.	5.71		4.97	16.24	5.59	13.58
Std. error[2] of Average	2.96		2.48	3.12	2.30	6.79

[1] CONDITION BEFORE CONSTRUCTION OF RETARDING DAM.

ASSUMING EXTREMAL DISTRIBUTION:

Rec. Intvl.	SINGLE EVENT (MAX. FLOW)			ANNUAL TOTAL FLOW		
10-Year	15.4		13.2 *	38.1		31.6
25-Year	19.7		16.9 *	50.1		41.7
50-year	22.8		19.6 *	59.1		49.2
100-year	25.9		22.3 *	67.9		56.6

* NOTE These values are in reasonable agreement with values obtained previously for the 2-dam concept upon which flood hydrographs for the above frequencies were developed. Reservoir inflow hydrographs will therefore not be changed for the single dam concept.

[2] NOTE Because of the large Standard Deviation of annual values and the small number of years of record (4) Standard Error of Estimate of average annual values is relatively large. The arithmetic averages computed above remain the BEST ESTIMATES, but the possibility of 37% in max. annual flood volume and 49% in total annual flood runoff is acknowledged, for average annual values.

3004

Job No. 6069-01 Page No. 8

Computed by G. Pavlou Date June 1981
Checked by Date
Reviewed by Date
Approved by Date

Subject Wadi Al Khawd Recharge
Annual Total flood Runoff
Arriving at Retarding Basins
Sheet No. 3 of 3

Source of Data. "Runoff Measurements in Oman" FAO PROJECT OMN/77/001, FIELD DOCUMENT 151 Adjusted for channel losses to Retarding Basins
And "ANNUAL TOTAL FLOOD RUNOFF - FREQUENCY" (Previous Computations).

Percent of YEARS	TOTAL FLOOD VOLUME	AVE FLOOD VOL	FRACTION OF TOTAL TIME	incremental Volume
0 - 8	0	0	.08	.000
10	1.2	0.6	.02	.012
15	3.5	2.35	.05	.118
20	5.4	4.45	.05	.223
25	7.0	6.20	.05	.310
30	8.5	7.75	.05	.388
35	9.8	9.15	.05	.458
40	11.0	10.40	.05	.520
45	12.2	11.60	.05	.580
50	13.5	12.85	.05	.642
55	14.7	14.10	.05	.705
60	16.0	15.35	.05	.767
65	17.2	16.60	.05	.830
70	18.5	17.85	.05	.893
75	20.0	19.25	.05	.963
80	21.6	20.80	.05	1.040
85	23.6	22.60	.05	1.130
90	26.0	24.8	.05	1.240
95	29.5	27.75	.05	1.388
99.9	44.0	36.75	.049	1.836
	Average Annual Total			14.045.

Superseded but a reasonable check

Cf 13.82

Note Nearly all this flow will be retained for slow release. Only the small volume that escapes over the spillway will overload downstream channel flow and give rise to loss to the sea.

3004

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Job No. 8064-01 Page No. 9

Computed by G. Reverne Date June '81
Checked by Reverne Date 12/81
Reviewed by Date
Approved by Date

Subject Wadi Al Khawd Recharge
Recharge capacity D/S
retarding Basins
Sheet No. 1 of

Assuming existing wadi channels only are utilized for recharge purposes
The natural channels between Al Khawd and The Coast Road have been shown by actual measurement (Prev. comp.) to be able to recharge 90 m³/sec as the average with a possible minimum of 20 m³/sec

The channels d/s the retarding basins ~~will be also an~~ estimated to be capable of receiving 0.825 of this
0.825 x 90 = 74 m³/sec (allow 50 m³/sec)

If releases from the retarding basins do not exceed a conservative fraction of this =
NO FLOOD WATER WILL BE LOST TO THE SEA

OR Max. possible recharge will be obtained.

Note In point of fact it will be physically impossible to confine releases from the retarding basins to 16 → 74 m³/sec because, during very large floods the spillway will be overtopped and large outflows will occur
Some of this outflow will be spread between natural wadi channels and will be partially absorbed as recharge. To be conservative, in examining economics of recharge, assume no benefit from spillway overflow (which will occur once in every ~~20~~ 10 year or so — say.

Note If existing channels are used for recharge, freshwater ground-water ~~distribution~~ will be intensified but very little change in distribution will occur. If a portion of the recharge gain is required for Muscat's water supply, a special additional recharge basin might be planned.

3004

Job No. 8069-01 Page No. 2.10

Computed by G Ravenna Date June '81

Checked by [illegible] Date 12/81

Reviewed by Date

Approved by Date

Subject Wadi AL Khawr Recharge
FLOOD Vol. - frequency
(Partial Duration Series)

Sheet No. of

Single Event Floods.

Series 1974 - 1978 data for Al Khawd
Reduced estimate for Retarding Basins.

Date	Flood Volume[2] at Al Khawd	Flood Volume[2] at Basins	M Mag	Est. Recurr.[1] Int. (Year)	Prob. %
5/10-4-76	16.36	14.76	1	6.5	
8/11-3-78	6.60	5.78	2	2.6	
21/24-1-77	6.00	5.23	3	1.6	
24/26-2-77	6.00	5.23	4	1.2	
9/11-2-75	4.73	5.14	5	0.94	
25/27-3-76	5.91	3.66	6	0.78	
2/4-2-76	4.07	3.45	7	0.66	
17/19-11-77	3.00	2.47	8	0.58	
23/27-5-77	2.90	2.37	9	0.51	
24/28-11-77	2.70	2.19	10	0.46	
7/10-2-78	1.53	1.11	11	0.42	
20/22-3-76	1.03	0.65	12	0.38	

SINGLE EVENT Probabilities

Note Ave 4 flood/year. $N = 4$

Average of Weibull & Hazen plotting position used. $\left(T = \frac{N\left(\frac{2M-1}{M-1/2}\right)+1}{2N}\right)$

2. Computed from the equation $V_{RD} = 0.92\,(V_{AK} - 0.32) \times 10^6\ m^3$

V_{RD} = flood Vol. at retarding dam

V_{AK} = flood Vol. at Al Khawr ($10^6\ m^3$)

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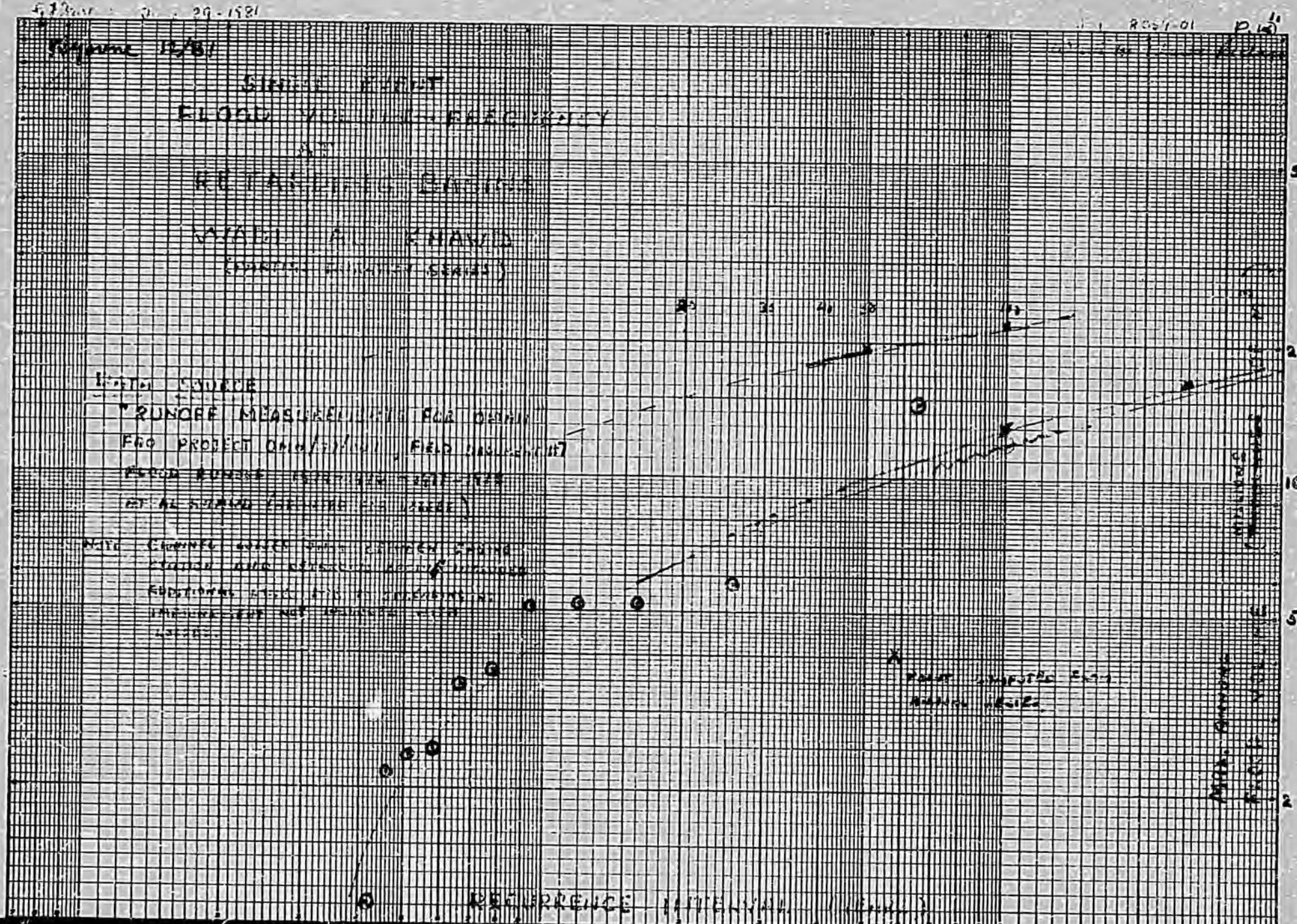

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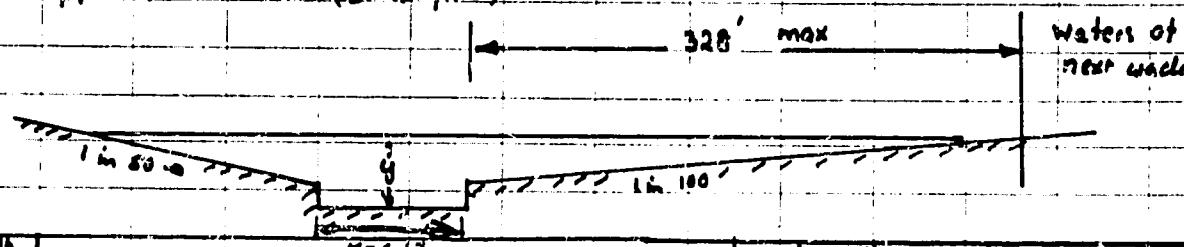
Job No. ..... Page No. 13

Computed by G. Flaveny Date Oct 1981
Checked by W. Byrne Date 12/81
Reviewed by ..... Date .....
Approved by ..... Date .....

Subject: Wadi Al Khawd Recharge
Estimated minor wadi rating curve
d/s Culvert #1 (Single Dam Concept)
Sheet No. ..... of .....

Wadi bed slope = 1m in 330m = .00303 ✓
Wadi roughness — Mannings' n = .028 (both channel & O'bank)
Approx. Section (simplified)

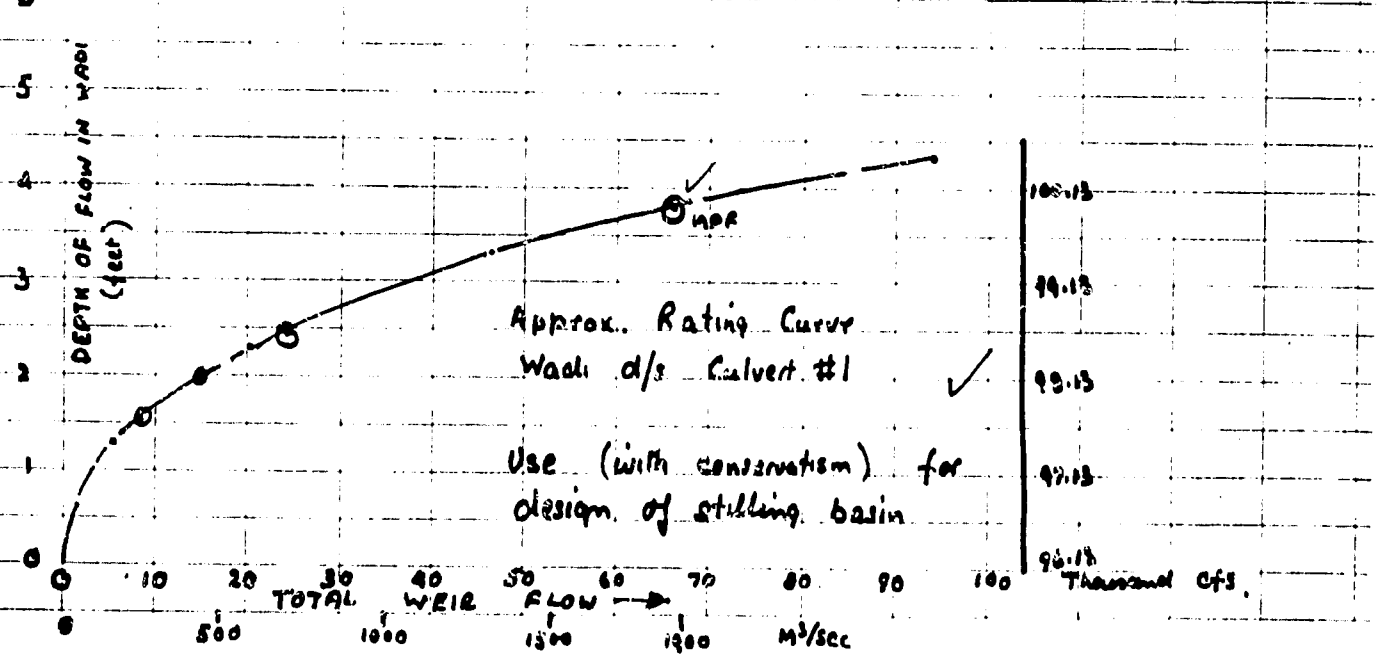


Depth (ft)	Channel			Overbank			Combined	Discharge (cfs)	
y	A_c	R_c	K_c	A_o	R_o	K_o	K	$Q_{(wadi)}$	$Q_{(weir)}$
.33	24.75	.327	623 ✓	—	-	623	623	34	394
1.33	99.75	1.294	6,255	75	.5	2507	8,762	482	5,565
2.33	174.75	2.194	15,658	300	1.0	15,921	31,579	1,738	20,057
3.33	249.75	3.058	27,928	675	1.5	46,942	74,870	4,121	47,553
4.33	324.75	3.882	42,571	1174	2.224	106,149	148,720	8,186	94,458
5.33	399.75	4.661	59,249						

$$K_a = \frac{1.49}{n} A R^{2/3}$$

Note This wadi accepts overflow from about 260 meters of weir. ✓
Total weir discharge will therefore be 3000/260 = 11.54 × wadi discharge
in the immediate vicinity of culvert #1

Note culvert discharge alone (up to 234 cfs before weir discharge ✓
occurs, will cause flow depth of ± 0.9 feet (0.27 m)
in the wadi



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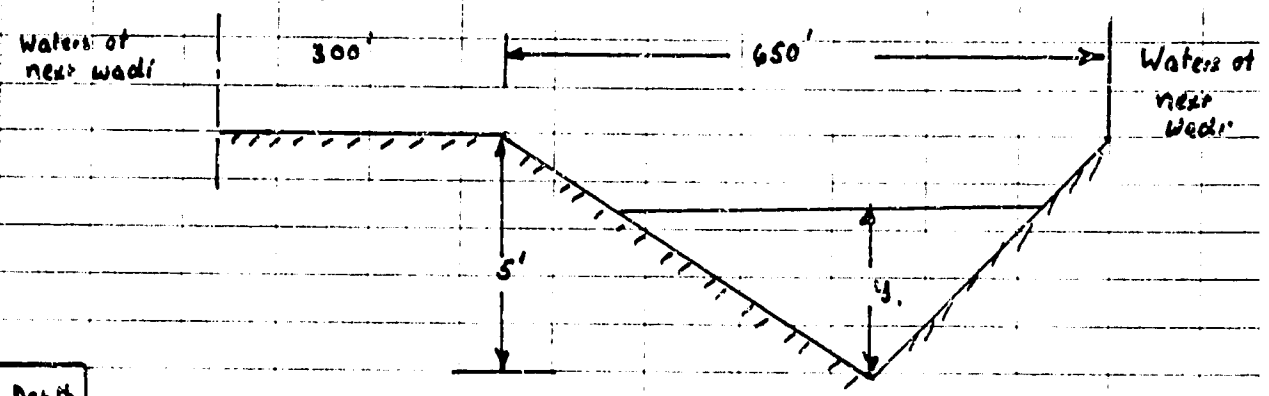
Computed by G. Mavrus Date Oct 1981
Checked by [illegible] Date 12/81
Reviewed by Date
Approved by Date

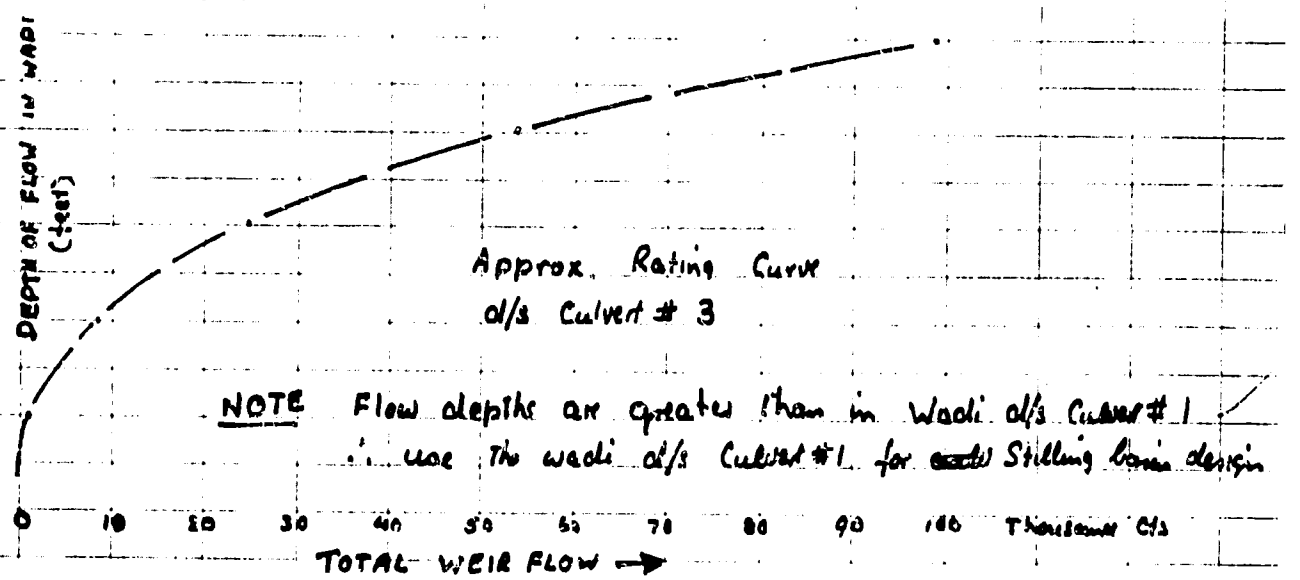
Job No. 8069-01 Page No. 13
Subject Wadi Al Khawr Recharge
Estimated minor wadi rating curve
d/s Culvert # 3 (Single Dam Control)
Sheet No. of

Wadi bed slope 1 in 275 = .00364
Manning's n = .028 (assumed)
Approx. Average Section (Simplified)



Depth (feet) y	Ac	Rc	Kc	Ao	Ro	Ko	K Total	Q(wadi) cfs	Q(weir) cfs
	← Channel		→	← Overbank		→			
1	65	0.5	2,173 ✓	—	—	—	2,173 ✓	131	1,354
2	260	1.0	13,799	—	—	—	13,799	833	8,613
3	585	1.5	40,683	—	—	—	40,683	2,455	25,385
4	1040	2.0	87,617	—	—	—	87,617	5,286	54,657
5	1625	2.5	158,862	—	—	—	158,862	9,585	99,109

This minor wadi accepts weir flow from about 290 m. of weir
Consequently, total weir flow = wadi flow × 3000/290 = wadi flow × 10.34 ✓
at any given instow



Approx. Rating Curve
d/s Culvert # 3

NOTE Flow depths are greater than in Wadi d/s Culvert # 1 ✓
∴ use the wadi d/s Culvert #1 for ~~each~~ Stilling basin design

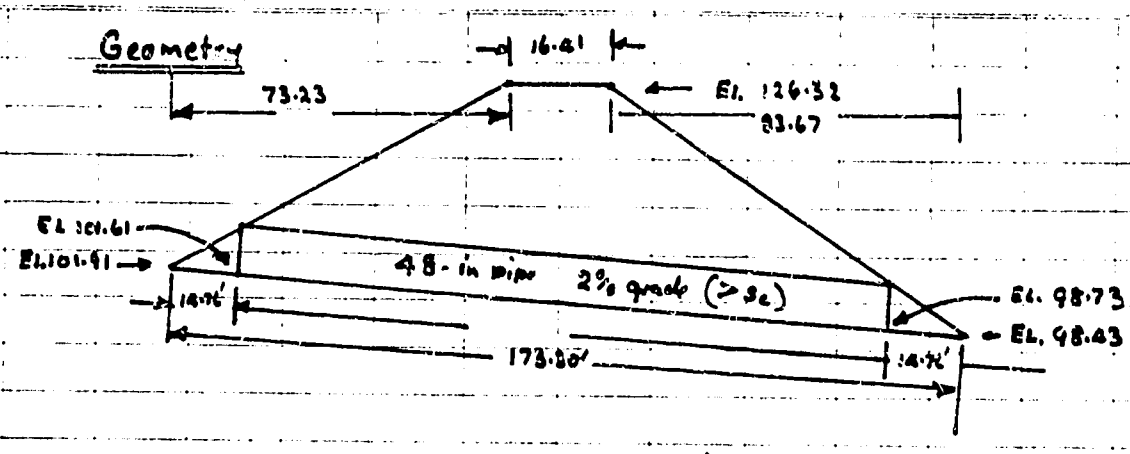
3004

[redacted]

Computed by G. Marcus Date Oct 1981
Checked by V. Yarne Date 12/81
Reviewed by Date
Approved by Date

Job No. 8069-01 Page No. 14
Subject Wadi Al Khawd Recharge
Single Dam Concept
Rating Curve, Single 48" culvert
Sheet No. 1 of 2

Geometry



3 modes of flow will be considered:

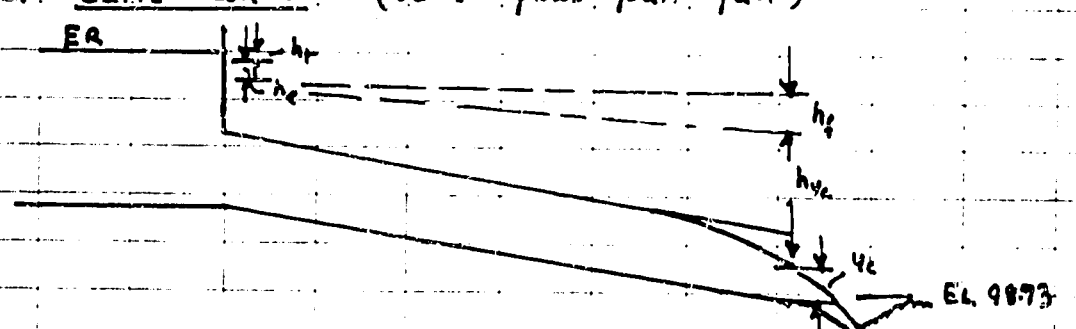
(1) Inlet control (critical depth inside pipe entrance)
(2) Outlet control - pipe flows part full at outlet (critical depth at outlet)
(3) Outlet control - pipe flows full at outlet (Long Culvert -)

(1) Inlet Control (assume effective to 1.35 D or H = 5.4') (±)

Trash rack loss = $1.4 \times V_w^2/2g$ where V_w = Velocity in headwalls
Entrance loss = $0.5\ V_c^2/2g$ where V_c = Critical velocity at entrance

y_c/D	y_c	Q_c	H_{vc}	$1.5 \times H_{vc}$	V_w	$1.4 H_{vw}$	H	ER (feet)
.2	0.8	8	0.28	0.42	1.11	0.03	1.25	102.86
.4	1.6	29	0.60	0.90	1.96	0.08	2.58	104.19
.6	2.4	63	1.00	1.50	2.73	0.16	4.06	105.67
.75	3.0	98	1.46	2.19	3.20	0.22	5.41	107.02

(2) Outlet Control (Outlet flows part full)



$$E_R - 98.73 = h_t + h_e + h_f + H_{vc} + y_c = \frac{V^2}{2g}\left(0.1 + 0.5 + \frac{185.2\,n^2 L}{D^{1.33}}\right) + y_c + h_{vc}$$

$$Q = 88.15 \times \sqrt{(E_R - 98.73 - H_0)}$$ where $H_0 = y_c + H_{vc}$ which is a function of Q.

3004

[redacted]

Computed by G. Hassane Date Oct-1981
Checked by Pryorne Date 12/81
Reviewed by Date
Approved by Date

Job No. 8069-01 Page No. 15
Subject Wadi Al Khawed Recharge
Single Dam Concept
Rating Curve - Single 48-in Culvert
Sheet No. 2 of 2

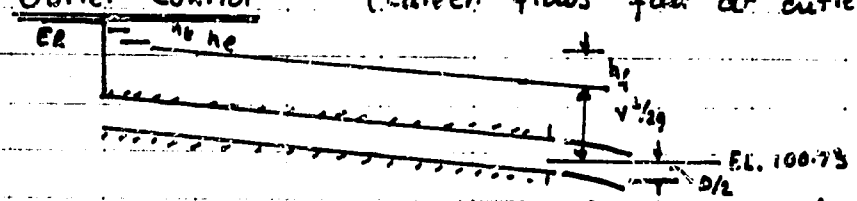
(2) Outlet Control (outflow from part full pipe at critical depth)

y_c/D	=	.90	.91	.92	.93	.94	.95	.96	.97	.98	.99
Q	=	151	156	162	167	177	186	198	213	237	282
H_{ce}/D	=	.62	.66	.70	.75	.81	.88	.99	1.14	1.40	1.97
H_o	=	6.08	6.28	6.48	6.72	7.00	7.32	7.80	8.44	9.52	11.84

Assume Outflow flow full above 282 cfs
$E_R >= 120$

E_R (METERS)	E_R (FEET)	H_o (by trial)	Q (cfs)
33.0	108.27	6.34	158
34.0	111.55	7.80	197
35.0	114.84	9.23	231
36.0	118.12	10.68	260

(3) Outlet Control (Culvert flows full at outlet)



$E_R - 100.73 = h_t + h_e + h_f + V^2/2g = V^2/2g\,(.1 + .5 + .7088 + 1) = 2.3088\,V^2/2g$

$Q = 66.37\sqrt{(E_R - 100.73)}$

E_R (METERS)	E_R (Feet)	Q (cfs)
37.0	121.40	301
38.0	124.68	325
38.50	126.33	336
38.65	126.82	339
38.81	127.32	342
39.00	127.96	346
40.00	131.24	367

SPILLWAY ELEVATION Q = 9.51 m^3/sec

No. of culverts to pass 50 m^3/sec at spillway el = 5.25 ?

USE 6 — 48-inch Culverts

ADD 5 48-inch culverts to double capacity but block with Stoplogs.

3004

[redacted]

Computed by G. Flowers, Date Oct 1981
Checked by Myerne Date 12/81
Reviewed by Date
Approved by Date

Job No. 8069-01 Page No. 16
Subject Wadi Al Khawr Recharge (Single Dam Concept)
Reservoir emptying time with zero inflow
Sheet No. of

With 6 - 48 inch Culverts discharging freely

Res. Elevation m.	Res. Elevation feet	Storage (A-ft)	Storage (cfs-hrs)	Outflow (cfs)	Emptying Time Incremental (hours)	Emptying Time Accumulating (hours)
38.5	126.32	9,283	112,324	2,016	—	0
38.0	124.68	8,246	99,777	1,950	6.33	6.33
37.0	121.40	6,310	76,351	1,806	12.47	18.80
36.0	118.12	4,582	55,442	1,560	12.42	31.22
35.0	114.84	3,081	37,377	1386	12.26	43.48
34.0	111.55	1,857	22,470	1182	~~10.71~~ 11.61	55.09
33.0	108.27	914	11,059	948	10.71	65.80
32.0	104.99	284	3,436	276	12.46	78.26
31.53	103.46	104	1,258	99	11.62	89.88
31.45	103.19		1039	76	2.50	92.38
31.37	102.92	Interpolate	820	50	3.48	95.86
31.26	102.56		538	~~25~~ 28	7.49	103.35
31.16	102.24		268	12	13.00	116.35

Time to empty last 10 cm. of depth through culverts is theoretically infinite since the outflow rate tends to zero but there will be a small pool over upstream.

116.35 hours = 116.35/24 = 4.85 days

Note Last two elevations given above are 10 and 20 cms above the sill of the inlet headwall. At these elevations, control is still at the 48-inch culvert inlet.

Note these emptying times are for ZERO INFLOW from upstream. This is unlikely but the computation serves to show that reservoir retention times — and HENCE EVAPORATION LOSS will be small.

3004

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Computed by G. Nauven Date Oct.
Checked by Wyoure Date 12/81
Reviewed by Date
Approved by Date

Job No. 8069-01 Page No. 17
Subject Wadi Al Khawd Recharge
Reservoir Storage Area - Discharge
Revised for 48-inch outlet Culverts
Sheet No. of

Note 6 culverts are assumed to run free with the remaining 5 temporarily blocked with stoplogs (This is the design method of operation

Area scale 1 sq. inch on planimeter ≡ 41,210 M^2 by grid lines of ma

	E_R (METERS)	E_R (feet)	Area Sq. ins (map)	Area M^2	Storage M^3	Storage A-ft	Outflow (cfs)
Dead Storage	31.0		3.60	148,356			
	31.06	101.91	Interpolated	173,725	0	0	
	31.53	103.45		372,448	138,351	104.1	9
	32	104.99	13.86	571,171	350,101	283.9	27
	33	108.27	23.86	983,271	1,827,322	914.1	948
	34	111.55	32.60	1,343,446	2,290,681	1,857.4	1,18
	35	114.84	41.12	1,694,555	3,809,681	3,089	1,38
	36	118.12	48.26	1,988,795	5,651,356	4,582	1,56
	37	121.40	55.18	2,273,968	7,782,738	6,310	1,80
	38	124.68	60.17	2,500,211	10,169,827	8,246	1,95
H.	38.5	126.32		2,613,333	11,448,213	9,283	2,01
0.5	38.65	126.82	Interpolated.	2,647,629	11,842,785	9,603	12,48
1.0	38.81	127.32		2,683,468	12,269,273	9,948	~~31,613~~ ~~49,40~~
1.67	39.00	127.99	66.16	2,724,454	12,783,216	10,365	~~65,913~~ ~~68,182~~
4.92	40.0	131.24	71.87	2,961,763	15,627,324	12,671	~~326,173~~ ~~324,338~~

+ Graphically interpolated outflow.

Spillway Discharge = $(29529 + 32H) \times H^{1.5}$ cfs.

MPF discharge = 1900 m^3/sec = 67,108 cfs Less Culvert discharge = 65,029 cfs

Reservoir Level to pass MPF = 128.01 - - - (H = 1.691) feet
= 39.02 Meters ✓

Discharge Capacity with Zero Freeboard (E_R = 40.0 meters) = 323,970 + 6 × 367 = 326,173 ✓ = 9234 m^3

(4.86 times estimated mpf!)

3004

Computed by G Rawlins Date Oct 1981
Checked by [illegible] Date 12/81
Reviewed by Date
Approved by Date

Job No. 8069-01 Page No. 18
Subject Wadi Al Khawd Reclamation Project
Single Dam Concept
Reservoir Routing Input table
Sheet No. of

WSEL	Storage A-FT	Storage cfs-HRS	Outflow cfs	
101.91	—	—	—	0
103.45	104.1	1,260	86.2	99
104.99	283.9	3,435	310	276
108.27	914.1	11,061	985	948
111.55	1957.4	22,475	1330	1182
114.84	3089.0	~~25,277~~ 37,327	1536	1386
118.12	4582.0	55,442	1719	1560
121.40	6310.0	76,351	1885	1806
124.68	8246	99,777	2038	1950
126.32	9,283	112,324	2,110	2016
126.42	9,347	113,099	3,048	2956 } Interpolation
126.62	9,475	114,648	6,975	6876 }
126.82	9,603	116,196	12,571	12480
127.32	9948	120,371	31,681	31613
127.96	10365	125,417	64,196	65,913
131.24	12671	153,319	324,565	326,173

37
38

2022

~~42-inch culvert outlet~~ SUPERSEDED → 48-inch culverts used

Note Computer is programmed for English units hence reservoir storage outflow, inflow hydrographs and computer outflow hydrographs are all in sec-feet which

3004

[redacted]

Job No. 8069-01 Page No. 19

Computed by G. Rawson Date July 28 '81
Checked by Hyome Date 12/81
Reviewed by Date
Approved by Date

Subject Wadi Al Khawd Recharge
Synthetic 100-year Inflow hydrograph

Sheet No. of

Prev. Comp. give 100-year flood peak = 823 cfs
100-year Vol. peak = 21.8×10^6 m^3
= 6056 cms-hours

hour	(first Estimate) cms	Adjusted (cms)	Adjusted (cms)	cms	cfs
0	823	823	823	0	0
1	730	725	735	10	35
2	615	610	620	20	70
3	540	530	550	35	1,23
4	455	440	470	50	1,76
5	420	405	435	70	2,47
6	345	430	360	90	3,17
7	300	285	315	120	4,23
8	265	250	280	168	5,93
9	230	220	240	240	8,47
10	200	195	205	315	11,12
11	165	165	168	435	15,36
12	140	140	140	550	19,42
13	120	120	120	735	25,96
14	95	95	95	823	29,06
15	90	90	90	620	21,89
16	80	80	80	470	16,60
17	70	70	70	360	12,71
18	60	60	60	280	9,89
19	50	50	50	205	7,24
20	45	45	45	140	4,94
21	35	35	35	95	3,35
22	25	25	25	80	2,82
23	20	20	20	60	2,119
24	15	15	15	45	1,589
25	16	10	10	25	88
26	5	5	5	15	530
27	0	0	0	5	177
	5948 cms × hours		6061 cms-hr	0	0

3004

8069-01 P. 20

Wadi Al Khawd Recharge
Single Dam Concept
Revised for 6 - 48-inch culvert outlet
100-Year flood routing

BASIN FLOOD SYNTHESIS
PROGRAM HYD01
WRITTEN FOR KEYBOARD CONTROL

Latest Revision APR 1979

Routing interval, (Hours) = .250
~~Unit Graph duration (Hrs) = .250~~ Not used.

Hyoung 12/81

100 Year Inflow to Reach No. 1
TOTAL DRAINAGE AREA = 100.00 Sq.M } Not used
Distributed storm Area = ~~100.00~~ Sq.M

.0	88.0	177.0	265.0	353.0	425.0	507.0	601.0
706.0	839.0	971.0	1,104.0	1,236.0	1,352.0	1,479.0	1,617.0
1,766.0	1,942.0	2,119.0	2,295.0	2,472.0	2,616.0	2,782.0	2,969.0
3,179.0	3,384.0	3,629.0	3,914.0	4,238.0	4,583.0	4,980.0	5,431.0
5,934.0	6,560.0	7,192.0	7,831.0	8,477.0	8,990.0	9,603.0	10,315.0
11,126.0	12,202.0	13,267.0	14,321.0	15,364.0	16,148.0	17,086.0	18,179.0
19,426.0	21,381.0	23,121.0	24,648.0	25,960.0	27,701.0	28,799.0	29,255.0 ⇨
29,068.0	27,100.0	25,249.0	23,515.0	21,898.0	20,441.0	19,072.0	17,792.0
16,600.0	15,529.0	14,525.0	13,587.0	12,715.0	11,992.0	11,281.0	10,580.0
9,890.0	9,195.0	8,521.0	7,870.0	7,241.0	6,601.0	6,005.0	5,453.0
4,945.0	4,448.0	4,017.0	3,653.0				

Peak inflow

100-Year outflow from reservoir No. 1
TOTAL DRAINAGE AREA (Sq.M) = ~~100.00~~ not used.
Storm Distribution area = 100.00 Sq. M.
Peak Outflow = 21,984.6 (cfs)
Time to Peak = 15.000 hours

.0	.9	3.4	7.6	13.5	20.8	29.5	39.7
51.6	65.6	82.0	100.6	122.1	145.7	171.3	199.0
229.1	261.8	299.2	340.8	385.3	432.3	481.7	533.9
589.2	647.9	710.2	776.9	848.8	926.4	962.6	984.3
1,008.3	1,035.1	1,065.0	1,097.9	1,134.0	1,172.9	1,203.6	1,233.6
1,266.0	1,301.5	1,340.6	1,383.1	1,416.3	1,450.8	1,487.3	1,526.1
1,569.4	1,624.7	1,685.3	1,750.5	1,813.1	1,851.6	1,892.1	1,933.8
1,971.9	2,006.2	9,980.8	20,048.4	21,984.6	21,390.9	20,200.4	18,912.3
17,662.1	16,498.4	15,426.6	14,428.2	13,497.9	12,664.3	11,997.2	11,332.6
10,648.7	9,959.4	9,273.2	8,601.7	7,949.8	7,308.8	6,726.4	6,247.0
5,743.3	5,240.2	4,755.9	4,313.3				

Peak reservoir outflow = 21,984.6 cfs
= 620 m^3/sec

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Job No. 8069-01 Page No. ~~30~~

Computed by G. Rousseau Date July 28 1981
Checked by V. Boone Date 12/81
Reviewed by Date
Approved by Date

Subject Wadi Al Khawd Recharge
Synthetic 50-year Inflow Hydrograph

Sheet No. of

Prev. Comps. give 50-year flood peak = 734 cfs cms.
50-year flood Vol = 19.0 x 10^6 m^3 = 5279 cms x hours

This slightly exceeds the 100-year flood of "Preliminary Design

hours	cms	adjusted cms	Re-arranged cms	cfs		Δt: 0.5 hours
0	734	734	0	0	0	19,249
1	655	650	8	283	142	16,777
2	555	545	16	565	283	14,305
3	495	480	30	1,060	424	12,539
4	425	405	45	1,589	565	10,773
5	395	375	65	2,296	813	9,537
6	325	305	80	2,826	1060	8,300
7	285	265	105	3,709	1325	7,246
8	255	235	145	5,121	1589	6,181
9	225	205	205	7,241	1943	5,298
10	195	175	265	9,360	2296	4,415
11	165	145	375	13,245	2561	3,709
12	145	125	480	16,954	2826	3,002
13	125	105	650	22,958	3268	2,737
14	105	85	734	25,925	3709	2,472
15	95	80	545	19,249	4415	2208
16	80	70	405	14,305	5121	1943
17	70	65	305	10,773	6181	1678
18	55	55	235	8,300	7241	1413
19	45	45	175	6,181	8301	1060
20	40	40	125	4,415	9360	706
21	30	30	85	3,002	11303	565
22	20	20	70	2,472	13245	424
23	16	16	55	1,943	15100	283
24	12	12	40	1,413	16954	141
25	8	8	20	706	19956	71
26	4	4	12	424	22,958	0
27	0	0	4	141	24,497	
28	5,559	5279	0	0	25925	
29	cms/hours	cfs x hr			22,587	

3004

G Hawa Oct 1981

8069-01 P 22.

Wadi Al. Khawir Recharge
Single Dam Concept
Revised for 6 - 48 inch culvert outlet
50 - Year Flood Routing

$\Delta t = .25$ hour

50 Year Inflow to Reach No. 1
TOTAL DRAINAGE AREA = ~~100.00~~ Sq.M
Distributed storm Area = ~~100.00~~ Sq.M Not used

.0	71.0	142.0	212.0	283.0	334.0	397.0	475.0
565.0	686.0	808.0	933.0	1,060.0	1,176.0	1,302.0	1,440.0
1,589.0	1,782.0	1,965.0	2,136.0	2,296.0	2,395.0	2,517.0	2,660.0
2,826.0	2,997.0	3,201.0	3,439.0	3,709.0	3,996.0	4,327.0	4,707.0
5,121.0	5,651.0	6,181.0	6,711.0	7,241.0	7,605.0	8,080.0	8,665.0
9,360.0	10,348.0	11,325.0	12,290.0	13,245.0	13,957.0	14,813.0	15,812.0
16,954.0	18,740.0	20,336.0	21,742.0	22,958.0	24,604.0	25,647.0	26,087.0
25,925.0	24,094.0	22,371.0	20,756.0	19,249.0	17,881.0	16,601.0	15,409.0
14,305.0	13,323.0	12,407.0	11,557.0	10,773.0	10,122.0	9,492.0	8,885.0
8,300.0	7,737.0	7,196.0	6,678.0	6,181.0	5,706.0	5,254.0	4,823.0
4,415.0	3,979.0	3,598.0	3,272.0				

Peak inflow = cfs

50-Year outflow from reservoir No. 1
TOTAL DRAINAGE AREA (Sq.M) = 100.00
Storm Distribution area = ~~100.00~~ Sq. M. Not used
Peak Outflow = 15,075.7 (cfs)
Time to Peak = 15.750 hours

.0	.7	2.7	6.1	10.8	16.6	23.4	31.4
40.9	52.3	65.8	81.5	99.3	119.8	142.3	167.1
194.2	224.3	257.5	295.0	336.9	380.7	425.9	473.0
522.5	574.5	629.5	688.1	751.0	818.6	891.5	953.3
973.5	996.1	1,021.2	1,049.0	1,079.3	1,111.7	1,146.1	1,182.7
1,209.4	1,239.0	1,271.8	1,307.8	1,346.9	1,386.0	1,419.2	1,452.6
1,488.5	1,527.9	1,573.7	1,630.8	1,691.7	1,756.6	1,816.0	1,853.0
1,890.1	1,925.6	1,957.1	1,982.9	2,006.5	5,738.2	12,583.0	15,075.7
14,916.4	14,113.4	13,204.0	12,337.9	11,607.1	10,884.6	10,213.2	9,574.7
8,962.7	8,374.4	7,808.7	7,265.6	6,774.7	6,375.2	5,944.9	5,509.3
5,081.4	4,656.3	4,239.2	3,852.7				

Peak Outflow = 15,075.7 cfs
= **430 m^3/sec**

Job No. 8069-01 Page No. ~~28 (revised)~~

Computed by G. Dussenne Date July 28 1981
Checked by M. Moore Date 12/81
Reviewed by Date
Approved by Date

Subject Wadi Al Khawir Recharge
Synthetic 25-year Inflow Hydrograph.

Sheet No. of

Prev. Comps give 25-year peak discharge = 643 cms
25-year flood vol. = 15.6×10^6 m^3
= 4333 cms x hrs

Vol. is intermediate between 50-yr & 100 yr of "Preliminary Engineering Design"

1st shot

hours	m³/sec	(m³/sec)	(cfs)
		0	0
0	643	6	212
1	550	18	636
2	455	32	1,130
3	390	50	1,766
4	330	70	2,472
5	290	95	3,355
6	260	120	4,238
7	230	165	5,828
8	195	230	8,124
9	165	290	10,243
10	140	390	13,775
11	120	550	19,426
12	110	643	22,711
13	95	445	15,717
14	80	330	11,656
15	70	260	9,183
16	60	195	6,887
17	50	140	4,945
18	40	110	3,885
19	32	80	2,826
20	24	60	2,119
21	18	40	1,413
22	12	24	848
23	6	12	424
24	0	0	0
	4365 cms/hour		
	0.12		

3004

G Travers Oct 1981

8069-01 P 24

Wadi Al Khawd Recharge
Single Dam Concept
Revision for 6-48-inch Culvert outlet.

25-Year Flood Routing

Δt: 0.25 hour.

25 Year Inflow to Reach No. 1

TOTAL DRAINAGE AREA = ~~100.00~~ Sq.M
Distributed storm Area = ~~100.00~~ Sq.M } NOT USED { INFLOW HYDROGRAPH ENTERED FROM KEYBOARD -(NOT GENERATED)

.0	33.0	80.0	139.0	212.0	311.0	415.0	523.0
636.0	746.0	865.0	993.0	1,130.0	1,282.0	1,439.0	1,600.0
1,766.0	1,936.0	2,110.0	2,289.0	2,472.0	2,676.0	2,891.0	3,118.0
3,355.0	3,576.0	3,797.0	4,017.0	4,238.0	4,569.0	4,945.0	5,364.0
5,828.0	6,336.0	6,888.0	7,848.0	8,124.0	8,670.0	9,206.0	9,730.0
10,243.0	10,994.0	11,832.0	12,760.0	13,775.0	14,989.0	16,336.0	17,815.0
19,426.0	20,469.0	21,364.0	22,112.0	22,711.0*	20,688.0	18,847.0	17,191.0
15,717.0	14,553.0	13,488.0	12,522.0	11,656.0	11,021.0	10,397.0	9,785.0
9,183.0	8,576.0	7,991.0	7,428.0	6,887.0	6,319.0	5,806.0	5,348.0
4,945.0	4,680.0	4,415.0	4,150.0	3,885.0	3,587.0	3,312.0	3,058.0
2,826.0	2,649.0	2,472.0	2,296.0				

Peak inflow

25-Year outflow from reservoir No. 1

TOTAL DRAINAGE AREA (Sq.M) = 100.00
Storm Distribution area = ~~100.00~~ Sq. M.
Peak Outflow = 6,463.2 (cfs)
Time to Peak = 17.250 hours

.0	.3	1.4	3.5	6.9	11.8	18.6	27.4
38.1	50.8	65.5	82.3	101.5	123.7	148.6	176.2
206.6	239.7	275.6	317.5	362.4	410.6	462.3	517.7
577.0	639.9	706.3	776.0	849.1	926.5	962.5	984.0
1,007.5	1,033.5	1,062.0	1,094.2	1,129.5	1,166.6	1,198.3	1,226.5
1,256.5	1,288.4	1,323.0	1,360.5	1,396.7	1,427.9	1,462.2	1,499.7
1,540.9	1,590.7	1,647.5	1,706.5	1,767.3	1,816.4	1,843.9	1,868.8
1,891.2	1,911.5	1,930.1	1,947.1	1,960.9	1,973.2	1,984.7	1,995.3
2,005.2	2,014.2	3,593.0	5,571.5	6,333.8	6,463.2	6,270.6	5,937.2
5,557.2	5,199.3	4,886.0	4,595.9	4,317.9	4,038.2	3,755.3	3,481.2
3,222.0	2,989.2	2,861.3	2,735.6				

Peak outflow 6463 cfs

= 180 m^3/sec ✓

Note output deleted above was entered merely to enable the program to proceed. It is normally required when hydrographs are being generated from rainfall data. The values given did not enter the computations in this instance

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Job No. 8061-01 Page No. ~~26 previous~~

Computed by G. Reuven Date July 29 '8

Checked by [illegible] Date 12/81

Reviewed by Date

Approved by Date

Subject Wadi Al Khawd Recharge Synthetic 10-year inflow hydrograph

Sheet No. of

Prev. Comps Give

10-year peak inflow = 522 m^3/sec = 18,437 cfs.

10-year peak Vol. = 12×10^6 M^3 = 117,733 cfs × hours

= 3333 cms × hours

Using 50-year Discharge ~ Frequency hydrograph from "Preliminary Engineering Design for Wadi Al Khawd" (for volume)

hour	m^3/sec			m^3/sec		cfs
					0	0
0	522	(adjusted to fit computed peak)		522	10	353
1	450	-5		445	15	530
2	375	-10		365	25	883
3	330	-15		315	40	1,413
4	300	-20		280	50	1,766
5	260	-25	75	235	90	3,179
6	240	-30		210	120	4,238
7	220	-30		190	190	6,711
8	180	-30		150	235	8,300
9	150	-30		120	315	11,126
10	130	-30		100	445	15,717
11	120	-30		90	522	18,437
12	90	-30		60	365	12,892
13	80	-30		50	280	9,890
14	70	-30	210 345	40	210	7,147
15	60	-25		40	150	5,298
16	45	-15		30	100	3,532
17	35	-10		25	60	2,119
18	30	10		20	40	1,413
19	20	5		15	30	1,060
20	15			15	20	706
21	10			10	15	530
22	5			5	5	177
23	0			0	0	0

117,417 cfs × hrs

3004

G Stevens Oct 1981

8069-01 P. 26

Wadi Al Khawd Recharge
Single Dam Concept
Revised for 6 - 48 inch Culvert
Outlet.

10-Year Flood Routing

Δt: 0.25 hours.

```
 10            Year Inflow to Reach No.      1
TOTAL DRAINAGE AREA =          100.00        Sq.M
Distributed storm Area =       100.00        Sq.M
```

Not USED – 10 yr. flood entered from keyboard

.0	105.0	199.0	281.0	353.0	381.0	420.0	469.0
530.0	602.0	684.0	778.0	883.0	1,032.0	1,170.0	1,297.0
1,413.0	1,518.0	1,612.0	1,694.0	1,766.0	2,152.0	2,517.0	2,859.0
3,179.0	3,311.0	3,532.0	3,841.0	4,238.0	4,939.0	5,585.0	6,176.0
6,711.0	6,992.0	7,351.0	7,787.0	8,300.0	8,841.0	9,492.0	10,254.0
11,126.0	12,449.0	13,655.0	14,745.0	15,717.0	17,172.0	18,110.0	18,532.0
18,437.0	16,812.0	15,347.0	14,040.0	12,892.0	12,117.0	11,359.0	10,616.0
9,890.0	9,120.0	8,407.0	7,749.0	7,147.0	6,677.0	6,212.0	5,752.0
5,298.0	4,823.0	4,371.0	3,940.0	3,532.0	3,112.0	2,737.0	2,406.0
2,119.0	1,909.0	1,722.0	1,556.0	1,413.0	1,325.0	1,237.0	1,148.0
1,060.0	955.0	861.0	778.0				

```
  10-Year outflow from reservoir No.     1
TOTAL DRAINAGE AREA (Sq.M) =                 100.00
Storm Distribution area = 100.00 Sq. M.  NOT USED
Peak Outflow =                1,916.0      (cfs)
Time to Peak =               18.250        hours
```

.0	1.0	4.0	8.5	14.5	21.4	28.8	36.9
45.9	56.0	67.4	80.3	94.9	112.1	132.0	154.2
178.4	204.3	231.7	260.4	291.1	327.4	371.2	421.7
478.3	538.5	601.4	668.6	742.0	825.9	922.5	967.4
995.4	1,025.3	1,056.7	1,090.0	1,125.6	1,163.6	1,197.1	1,226.7
1,259.0	1,295.0	1,335.2	1,379.1	1,414.5	1,450.6	1,489.6	1,530.0
1,573.2	1,620.4	1,662.8	1,701.1	1,735.6	1,767.3	1,796.6	1,815.2
1,828.1	1,839.9	1,850.6	1,860.1	1,868.7	1,876.4	1,883.5	1,889.7
1,895.3	1,900.2	1,904.3	1,907.8	1,910.6	1,912.8	1,914.3	1,915.3
1,915.9	1,916.0	1,915.9	1,915.4	1,914.8	1,913.9	1,913.0	1,911.9
1,910.6	1,909.2	1,907.7	1,906.0				

Peak outflow = 1916 cfs

= **50 m^3/sec** (54 to be precise)

Note This outflow is carried completely by the culverts. There will be no spillway overflow at the 10-year level. ✓

Peak reservoir stage = **37.76 meters** (for spillway at EL 38.5)

(Pool contains 10-year flood comfortably)

Job No. 8069-01 Page No. 27

Computed by G. Reeves Date Nov. 1991

Checked by [illegible] Date 12/81

Reviewed by Date

Approved by Date

Subject Wadi Al Khawd Recharge Estimated Loss of surface water over single dam spillway

Sheet No. 1 of 2

Spillway overflow Hydrographs ($\Delta t = 0.25$ hours) (cfs units)

FLOOD EVENT				
10-YEAR	25-YEAR	50-YEAR	100-YEAR	
0	0	0	0	
	1,577	3,722	7,965	
	3,555	10,567	18,032	
	4,318	13,060	19,969	
	4,447	12,900	19,375	
	4,255	12,097	18,184	
	3,921	11,188	16,996	
	3,541	10,322	15,646	
	3,183	9,591	14,482	
	2,870	8,869	13,411	
	2,580	8,197	12,412	
	2,302	7,559	11,432	
	2,022	6,947	10,648	
	1,739	6,358	9,981	
	1,465	5,793	9,317	
	1,206	5,250	8,633	
	974	4,759	7,943	
	845	5,359	7,257	
	720	3,929	6,586	
	595	3,493	5,934	
	470	3,065	5,293	
	345	2,640	4,710	
	220	2,223	4,231	
	95	1,837	3,727	
	0	1,451	3,224	
		1,065	2,740	
		679	2,297	
		293	1,854	
		0	1,411	
			968	
			525	
			82	
M. M³ of flow	0.00	1.20	4.13	6.76
Annual probability	0.1	0.04	0.02	0.01

(Overflow from spillway when tail water levels exceed 2016)

Spillway overflow = Q = Hydrograph - 2016

Vol. of Overflow

$= \Sigma Q \times 900$ cubic ft

$= \Sigma Q \times 25.48$ M³.

Average annual spillway overflow

= 0.21 Million M³

(See next sheet)

Previous Average Annual loss to sea (prev. comps) = 4.83 M. M³

This amount, less spillway overflow (0.21 M. M³) assumed lost to sea less groundwater loss to sea - which will depend upon the efficiency of the well system used to extract ground water. (Allow 0.62)

4.83 - .21 - .62 = 4.00 M.M

EST. Av. Ann. Recharge

= 4.00 M. M³ (±) 2.0 M. M³.

3004

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NO. 340R-10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH

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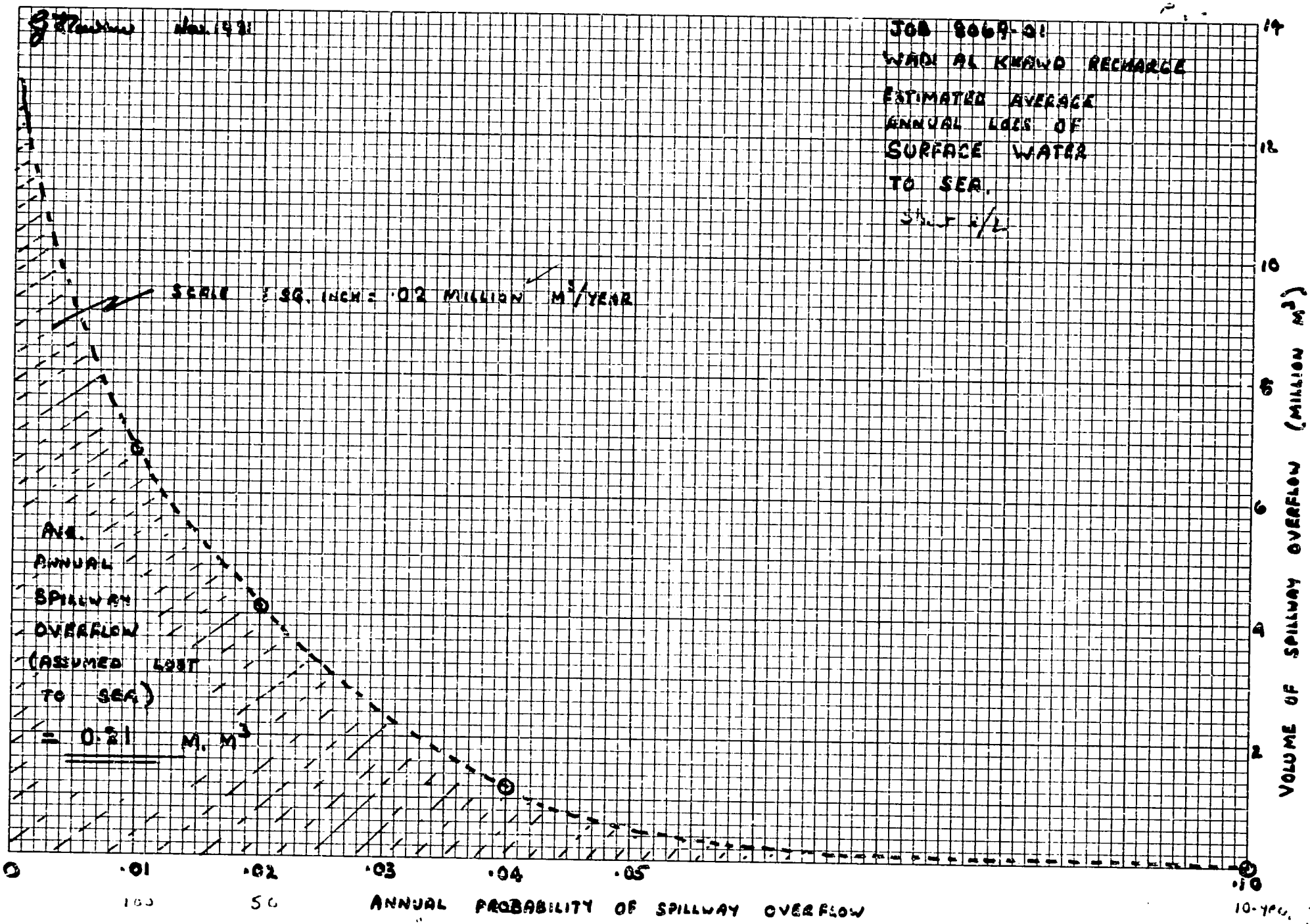
MADE IN U.S.A.



G Reviewed
Oct 1981

Job 9069-01 P. 29
Wadi Al Khawd Recharge
(Single Dam Concept)
Stilling basin design
Safety against sweep-out
Sheet 1/2

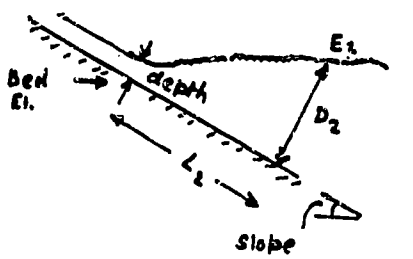
CHUTE FLOW PROFILE
(FLOW DEPTHS GIVEN NORMAL TO SLOPE)

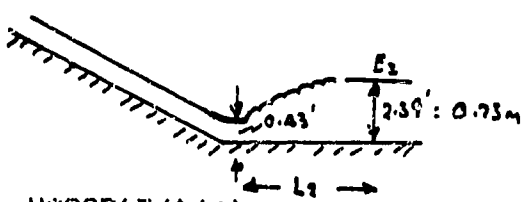
BED WIDTH OF CHUTE = 9843 (feet)
MANNING'S n ON CHUTE = .03 * (for shear estimate)
CHUTE SIDE SLOPE = 1 ON 13.33

DISCHARGE = 67100 (cfs) (MPF)

RGovne 12/81

DIST.	BED EL.	SLOPE	DEPTH	F	D2	E2	L2
.0	126.32	.0000	1.13	1.00	1.13	127.45	7.
1.6	125.82	.3333	.70	2.05	3.92	126.17	11.
3.2	125.32	.3333	.61	2.51	4.66	125.73	13.
6.3	124.32	.3333	.53	3.15	5.48	124.81	16.
9.5	123.32	.3333	.49	3.54	5.90	123.85	17.
15.8	121.32	.3333	.45	3.98	6.33	121.88	18.
22.1	119.32	.3333	.44	4.16	6.48	119.90	19.
28.5	117.32	.3333	.43	4.21	6.53	117.90	19.
41.1	113.32	.3333	.43	4.27	6.58	113.91	19.
53.8	109.32	.3333	.43	4.26	6.57	109.90	19.
66.4	105.32	.3333	.43	4.27	6.58	105.91	19.
79.1	101.32	.3333	.43	4.26	6.57	101.90	19.
87.1	98.76	.0000	.43	4.27	2.39	101.15	15.





F = FROUDE NUMBER = V*SQR(T/A/B)
D2 = CONJUGATE DEPTH
SEE FIG.15-20 OF' OPEN CHANNEL
HYDRAULICS' BY VEN TE CHOW
E2 = CONJUGATE WATER ELEVN.
IF JUMP FORMS AT THIS POINT
LENGTH OF JUMP = L 2

```
READY
EDIT\
EDIT SETOP=0
BS ERROR
```

Note

Actual stilling basin sized deeper and wider than required by above computation. This is based upon judgment because the stilling basin will also serve to transfer water laterally to wadi outlets.
Channels subject to heavy inflow from 1 side tend to "slosh". Hence greater depth of flow is advisable. Also, additional depth of flow is required for lateral transfer
Hydraulic Jump at a Froude # of 4.27 will tend to oscillate - greater depth highly desirable
+ No reduction in n made for energy because of small depths of flow. This will provide a realistic estimate of sliding shear on gabions

+ re-run with n = .62×.03
= .0186
for energy.
SEE SHEET 2/2

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G Thomas
04/1981
Ag Jone (2/81

Job 8069-01 P.30
Wadi Al Khawd Recharge
Stilling Basin Design
Safety against Sweep-out
Sheet 2/2

CHUTE FLOW PROFILE
(FLOW DEPTHS GIVEN NORMAL TO SLOPE)

BED WIDTH OF CHUTE = 9843 (feet)
(NING'S n ON CHUTE = .0186 ‡
CHUTE SIDE SLOPE = 1 ON 13.33

DISCHARGE = 67100 (cfs)

DIST.	BED EL.	SLOPE	DEPTH	F	D2	E2	L2	
.0	126.32	.0000	1.13	1.00	1.13	127.45	7.	Bed El. 38.5m.
1.6	125.82	.3333	.69	2.10	4.00	126.18	12.	
3.2	125.32	.3333	.59	2.63	4.83	125.75	14.	
6.3	124.32	.3333	.49	3.46	5.82	124.84	17.	
9.5	123.32	.3333	.44	4.11	6.44	123.89	19.	
15.8	121.32	.3333	.39	5.02	7.17	121.96	21.	Not used, applies to jumps forming on 1 on 3 slope
22.1	119.32	.3333	.36	5.61	7.58	120.00	22.	
28.5	117.32	.3333	.34	5.97	7.82	118.02	23.	
41.1	113.32	.3333	.33	6.35	8.05	114.04	23.	
53.8	109.32	.3333	.32	6.49	8.13	110.04	24.	
66.4	105.32	.3333	.32	6.53	8.16	106.05	24.	
79.1	101.32	.3333	.32	6.55	8.17	102.05	24.	
88.2	98.43	.0000	.32	6.54	2.82	101.25	17.	Bed El. 30.0m (Jump on level floor.)

F = FROUDE NUMBER = V*SQR(T/A/S)
D2 = CONJUGATE DEPTH
(SEE FIG.15-20 OF' OPEN CHANNEL HYDRAULICS' BY VEN TE CHOW)
E2 = CONJUGATE WATER ELEVN.
(IF JUMP FORMS AT THIS POINT)
L2 = LENGTH OF JUMP

‡ 62% estimated Mannings' n for gabion lining used for energy computation (See U.S. Bureau of Reclamation "Design of Small Dams" - Spillways)

<u>Note</u> Flow reaches "uniform" depth on spillway face. Hence Hydraulic jump remains the same wherever formed below El. 105.32' (32.1 m)

Conjugate depth on level stilling basin floor = 2.82' = 0.86 m

Use a depth of 0.90 m for the stilling basin because flow may not be longitudinal over the end sill but lateral to a low point at a wadi

Min. length for Jumps 17' = 5.2 m. Use 7.5 m because of lateral flow being present.

G Nassen Oct 1987

Job 8069-01 P. 31
Wadi Al Khawa Recharge
BASIC computer program listing for chute design

LIST

```
 10 PRINT TAB(23);"CHUTE FLOW PROFILE"
 15 PRINT TAB(15);"(FLOW DEPTHS GIVEN NORMAL TO SLOPE)": PRINT : PRINT
 20 IF F9>0 THEN 110
 30 INPUT "BED WIDTH OF CHUTE (FT) = ",W
 40 INPUT "MANNING'S n ON CHUTE    = ",N
 50 INPUT "COTANGENT OF CHANNEL SIDE-SLOPE = ",Z
 60 INPUT "LONG. SLOPE AT CREST (tan) = ",S2: LET R2=1/SQR(S2*S2+1)
 70 INPUT "WILL YOU USE THE PRINTER ? ",F9
 80 PRINT : INPUT "CHUTE DISCHARGE (CFS) = ",Q: LET L9=0
 90 IF F9>0 THEN  SET OP=2: GOTO 10
100 GOTO 150
110 PRINT "BED WIDTH OF CHUTE = ";W;" (feet)"
120 PRINT "MANNING'S n ON CHUTE = ";N
130 PRINT "CHUTE SIDE SLOPE = 1 ON ";Z: PRINT
135 PRINT TAB(20);"DISCHARGE = ";Q;" (cfs)": PRINT : SET OP=0
140 PRINT : INPUT "CREST ELEVATION (ft) = ",Z2
150 INPUT "DO YOU REQUIRE CRITICAL DEPTH START ? ",F
160 IF F>0 THEN 240
170 PRINT : INPUT "DEPTH AT CREST = ",Y2
180 LET A2=W*Y2+Z*Y2*Y2: LET P2=W+2*Y2*SQR(1+Z*Z): LET T=W+2*Z*Y2
190 LET V2=Q/A2: LET F=V2*SQR(T/A2/32.2): GOTO 280
240 LET Y2=(Q/W*Q/W/32.2)↑.3333
250 LET A2=Y2*W+Y2*Y2*Z: LET P2=W+2*Y2*SQR(1+Z*Z): LET T=W+2*Z*Y2: LET V2=Q/A2
260 LET F=V2*SQR(T/A2/32.2): LET E=F-1: IF ABS(E)<.01 THEN 280
270 LET Y2=E/F/(1.5*T/A2-Z/T): GOTO 250
280 LET D2=((1.395+5.41*S2+10.64*S2*S2)*(F-1)+1)*Y2
290 LET L2=(6.2-21.47*S2+34.67*S2*S2)*D2
300 LET E2=Z2-S2*L2+D2/R2
350 IF F9>0 THEN  SET OP=2
360 IF L9>0 THEN 380
370 PRINT " DIST. BED EL. SLOPE DEPTH     F     D2     E2     L2"
380 PRINT %6F1;L9;%8F2;Z2;%6F4;S2;%6F2;Y2;F;D2;" ";E2;%6F0;L2
390 SET OP=0: LET Z1=Z2: LET V1=V2: LET Y1=Y2: LET R1=R2: LET A1=A2: LET P1=P2
400 PRINT : INPUT "SLOPE DIST. TO NEXT SECTION = ",L: LET L9=L9+L
410 INPUT "BED EL.AT NEXT SECTION ",Z2
420 INPUT "BED SLOPE AT NEXT SECT. (TAN) = ",S2: LET R2=1/SQR(1+S2*S2)
430 LET U=L/2*(N/1.486)↑2*(P1/A1)↑1.333
440 LET M=Z1+R1*Y1+V1*V1*(1/64.4-U)-Z2
450 LET V2=SQR(50*(Z1-Z2)+V1*V1): LET Y2=Q/V2*Y2/A2
460 LET A2=Y2*W+Y2*Y2*Z: LET E=A2-Q/V2: LET Y0=E/(W+2*Z*Y2)
470 IF ABS(E)>.005 THEN  LET Y2=Y2-Y0: GOTO 460
480 LET P2=W+2*Y2*SQR(1+Z*Z): LET T=W+2*Y2*Z: LET U=L/2*(N/1.486)↑2*(P2/A2)↑1.3
33
490 LET V=SQR((M-Y2*R2)/(1/64.4+U)): IF ABS(V-V2)<.01 THEN 510
500 LET V2=V: LET Y2=Q/V2*Y2/A2: GOTO 460
510 LET F=V2*SQR(T/A2/32.2): GOTO 280
```

0.62 n for energy
1.38 n for depths

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G Reuben 05/1981
Wyoma 12/81

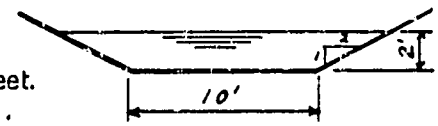
Job 8069-01 P.32
Wadi Al Khawd Recharge
Manning's n for
Gabion Mattress

Example of solution of Manning's Formula:

GIVEN:
Trapezoidal channel having bottom width = 10 feet, side slopes 1 on 2, depth of flow = 2 feet, slope = 0.5%.
The channel is lined with gabions filled with stone having a mean size of 4".
Find the velocity and discharge in the channel.

SOLUTION:

Flow area = $\frac{(10 + 18)}{2} \times 2 = 28$ square feet

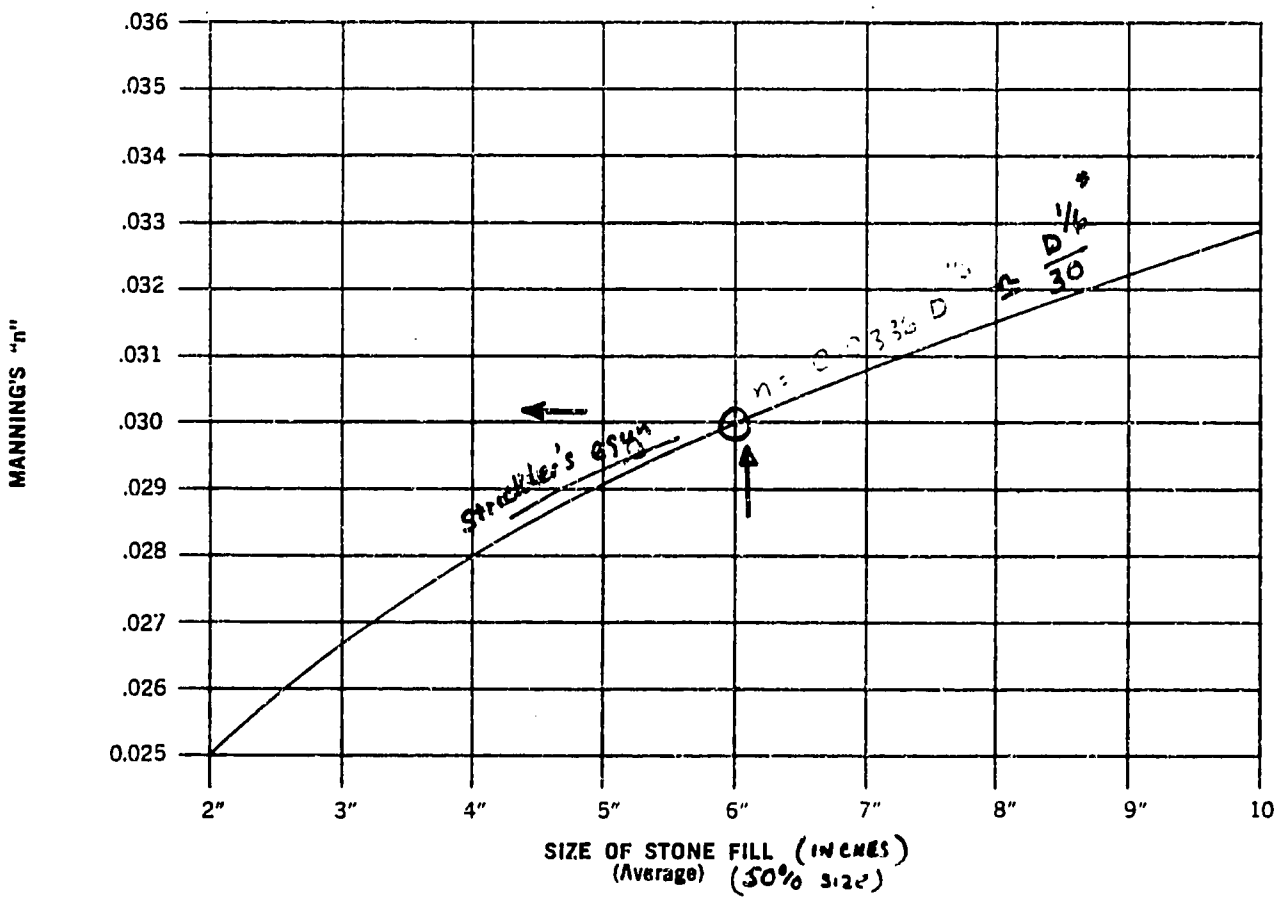


Wetted Perimeter = $10 + 2\ (\sqrt{2^2 + 4^2}) = 19$ feet.
Hydraulic Radius = A/WP = 28/19 = 1.5 feet.
Since hydraulic radius is less than 5 feet, the roughness factor of the gabions is obtained from the graph as 0.028

From the Chart:
1) Locate the intersection of R = 1.5 and S = 0.5%
2) Follow the diagonal line to the intersection of n = .028
3) Proceed vertically to top of chart to find V = 4.9 fps

Discharge, Q = AV = 28 × (4.9) = 137 cfs

VARIATION OF MANNING'S "n" WITH SIZE OF STONE FILL



NOTE: When hydraulic radius is greater than about 5 feet, a constant value of 0.025 for "n" may be used if the stone fill is between 2" and 10" in size.

* where D = stone size in feet

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[redacted]

Computed by G. Newton Date Oct 1981
Checked by Byone Date 12/81
Reviewed by Date
Approved by Date

Job No. 8069.01 Page No. P 33
Subject Wadi Al Khawd Recharge (Single dam Concept)
Safety against piping failure
Sheet No. 1 of 2

1. Spillway Section

Assume reservoir pool may be maintained at spillway level sufficiently to ~~prevent~~ provide danger of piping failure. Apply the "weighted line of creep" design method for dams on pervious foundation (See "Handbook of Applied Hydraulics" (first edition) by Davis

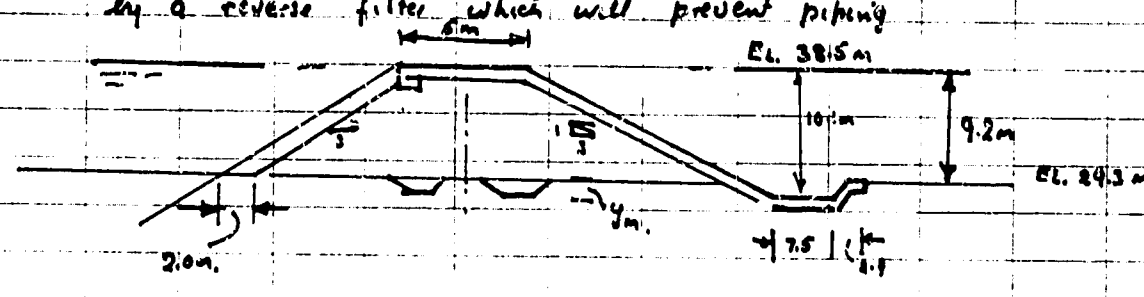
Safe weighted creep ratio for dam on sand gravel foundation for dams between 17 and 50' high

= 2.78 for gravelly sand → 2.97 for medium gravel → 3.22 for coarse sand.

because the deposits at Wadi Al Khawd are slightly cemented the above figures may be conservative but use 3.3 to 3.4 for design

Max. sustained head, Spillway section = 38.5 − 29.3 = 9.2 m.

Horizontal creep extends from the inside of riprap bedding on the upstream face to the end of the gabions in the stilling basin on the downstream face (because the gabions are backed by a reverse filter which will prevent piping



Horizontal Creep distance = −2.0 + 3×9.2 + 5 + 3×10.1 + 7.5 + 2.9 = 71.3 m

Vertical Creep = 4 y meters, (assuming twin key trenches, y_m de

Effective Creep ratio = $\dfrac{71.3/3 + 4y}{9.2}$ and should exceed 3.3 → 3.4

or $y >= (3.3 \times 9.2 - 71.3/3)/4 >= 1.65$
or $(3.4 \times 9.2 - 71.3/3)/4 >= 1.86$ } USE 1.90

Make y = ~~4.0~~ 1.90 meters in the spillway section

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Job No. 8069-01 Page No. ~~23-8~~

Computed by G. Dawson Date Aug 81
Checked by [illegible] Date 12/81
Reviewed by Date
Approved by Date

Subject Wadi Al Khawd Recharge
Flow Net - 6.25 m dam
Computer Input
Sheet No. 1 of 3

Note Computer input assumes alluvial fan saturated to ground surface for at least 93 m. d/s of the dam and no ground-water flow beyond this point. This is an improbable model designed to provide an estimate of leakage rate beneath the dam and the exit hydraulic gradient at the dam's d/s. toe only.
(Actual leakage will probably be far less)

Computer Input. (See diagram on sheet 2/3)

No. of Rows = 10
No. of Columns = 15
Default Conductance between nodes in row = 83 m^3/day/m.
Default Conductance between column nodes = 46 m^3/day/m
Default Piezometric Head at all nodes = 100.00 m

Change -

Row 1, Columns	1-9	10	11	12-15
Vert. Cond =	88	66	44	0
Fixed Head =	106.25	106.25	106.25	0

Row 2, Column	1-9	10	11-12	13	14	15
Hor Cond	84	172	172	0	0	0
Vert Cond.	44	33	22	22	0	0
Fixed Head				0	100.00	

Row 3 Column	1-9	10	11-14	15
Hor. Cond	85	169	169	-
Vert Cond	46	35	23	-
Fixed Hd.			100.00	

Row 4-10 Columns	~~1-9~~	10	11-14	15
Hor. Cond (Same)			166	-
Vert Cond		35	23	-
Fixed Hd.			100.00	

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G Mavens Aug 1981

K_H = 350 m/day (based on sieve analysis)

K_V = 10 m/day (based on infiltration tests)

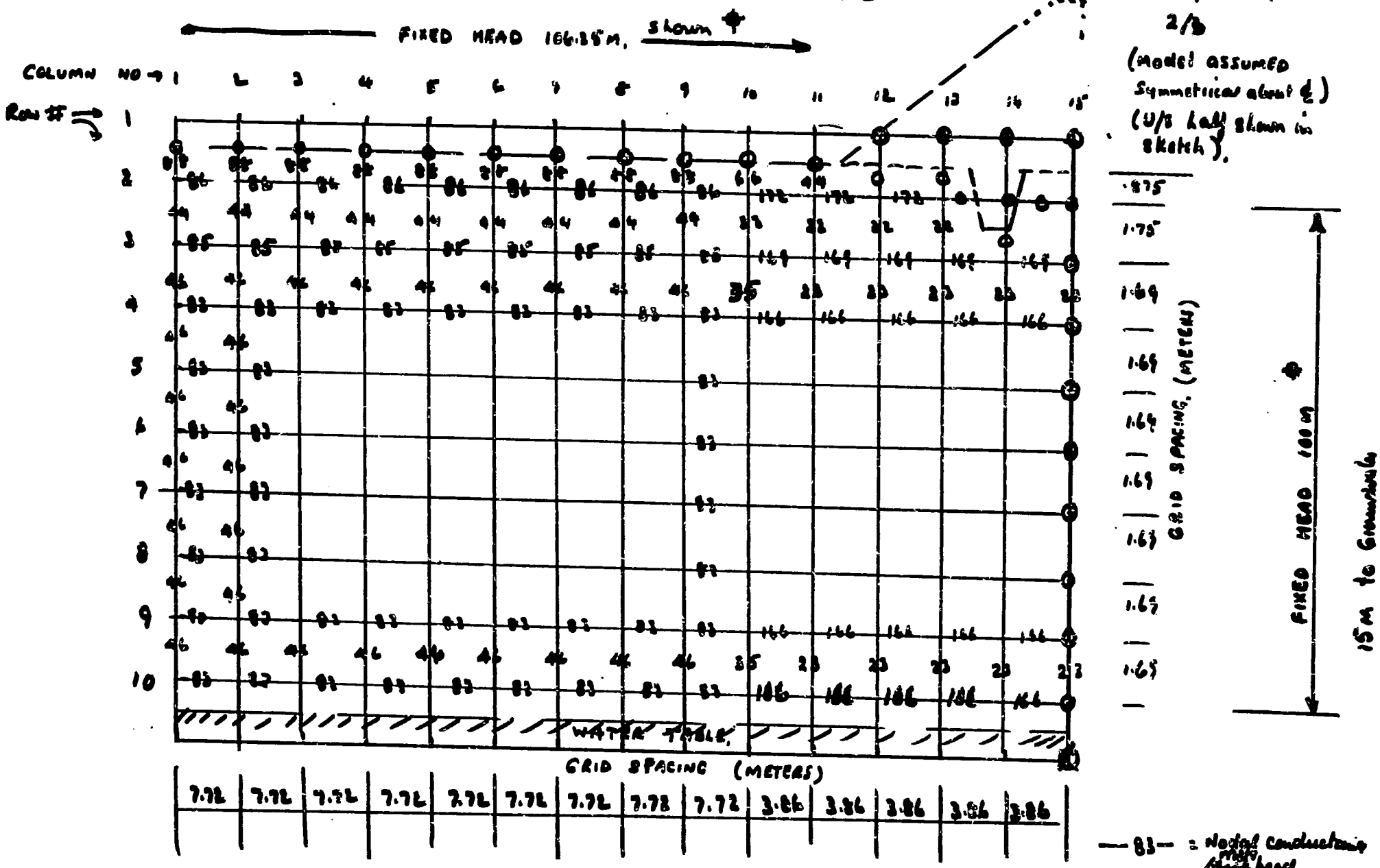
Model. 6.25 m. dam

Conductivity in row = $\frac{K_H \times \Delta Y}{\Delta X}$; eg. at point (Row = 4, Column = 4), C = $\frac{350 \times 1.64}{7.72}$ = 83

8060 P. 35

Flow Net Model (6.25 m. dam) Computer input 2/3

(Model assumed symmetrical about ℄) (U/S half shown in sketch).



— 83 — = Nodal conductance (m²/day) head

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[redacted]

Computed by: G Rostow Date: Aug 1981
Checked by: [illegible] Date: 12/81
Reviewed by: Date:
Approved by: Date:

Job No. 8069-01 Page No. 36
Subject: Wadi Al Khawd Recharge
6.25 m High Dam
Equilibrium Flow Net
Sheet No. 3 of 3

PRINT-OUT when last correction = 0.01 meters

PIEZOMETRIC HEADS AT EQUILIBRIUM (HEAD LOSS DOUBLED IN MODEL)

Computer Program "G-4 Model" (5-hour running time on Apple IIC + Running Applesoft)

Row \ Column	1	2	3	4	5	6	7	8	9	10
1	106.25	106.25	106.25	106.25	106.25	106.25	106.25	106.25	106.25	106.25
2	105.96	105.96	105.95	105.93	105.89	105.85	105.78	105.67	105.48	105.12
3	105.41	105.39	105.35	105.30	105.21	105.09	104.92	104.66	104.27	103.65
4	104.91	104.88	104.83	104.75	104.63	104.47	104.25	103.96	103.56	103.04
5	104.46	104.43	104.37	104.28	104.14	103.96	103.73	103.44	103.08	102.65
6	104.08	104.05	103.98	103.88	103.74	103.55	103.32	103.05	102.72	102.34
7	103.77	103.74	103.67	103.56	103.42	103.24	103.02	102.75	102.45	102.10
8	103.53	103.50	103.43	103.33	103.19	103.01	102.80	102.54	102.26	101.94
9	103.38	103.34	103.28	103.17	103.03	102.86	102.65	102.41	102.14	101.83
10	103.30	103.26	103.20	103.09	102.96	102.79	102.58	102.34	102.08	101.78

Continued

Row \ Column	11	12	13	14	15	16
1	106.25	—	—	—	—	
2	104.79	104.29	104.01	—	100.00	—
3	103.17	102.57	101.84	100.91	100.00	99.09
4	102.49	101.92	101.32	100.67	100.00	99.33
5	102.17	101.67	101.13	100.57	100.00	99.43
6	101.92	101.47	101.06	100.50	100.00	99.50
7	101.73	101.32	100.89	100.45	100.00	99.55
8	101.59	101.21	100.82	100.41	100.00	99.59
9	101.50	101.15	100.77	100.39	100.00	99.61
10	101.46	101.11	100.75	100.38	100.00	99.62

(etc.)

℄ of "symmetrical" model (except that losses continue from left to right for columns 16 – 29

Note ℄ head set at 100.00 in stead of 103.125 thus doubling head losses and discharges in model.

(Effective head = 2(106.25 – 100.00) = 12.5 m in model.

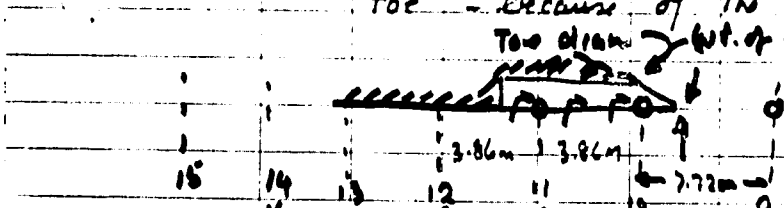
(Actual D/S head – points 1 through 11 of mirror image of the above, would be 100 – 6.25 = 93.75 making a head loss of 106.25 – 93.75 = 12.5 m in model vs. 6.25 m in prototype.

Outflow from reservoir $\frac{1}{2}\left(169 \times (0.91) + \frac{1}{2}(703.37 - 700) \times 166\right)$

= 356.6 m³/day / m of dam

(= 357 × 3500 / 24 / 3600 = 14.5 m³/sec for entire dam – insignificant)

Max exit hydraulic gradient at toe of dam (note this corresponds to point Row 2, Column 9, at upstream toe – because of the D/S toe drain)



Exit hyd. grad = $\frac{(106.25 - 105.48)}{.875 \times 2} = \frac{0.86}{2} = 0.43$

This is O.K. (material equals at G-1 = 1.25) (Note bottom layer material might move at a lower grade)

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[redacted]

Computed by G. Newsome Date July 1981
Checked by [illegible] Date 12/81
Reviewed by Date
Approved by Date

Job No. 8065-07 Page No. 37
Subject Wadi Al Khawd Recharge Design for wave action within Recharge Reservoirs
Sheet No. 1 of 2

For 1980, the following max. 10 minute wind velocities were recorded at Seeb Airfield:-

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug.	Sept	Oct	Nov	Dec
Peak 10 min Vel Kts.	20	26	30	22	21	16	22	18	16	16	14	20
Vel. mph.	23	30	35	25	24		25					23
Rec. Int. (months)	2.2	6.5	13	4.3	2.6		3.3					1.9

Graphical 5-year peak 10 min gust = 50 mph. (see sheet 2)

From U.S. Army Engineer "Shore Protection Planning & Design" Technical report # 4, For a depth of 15 feet, effective fetch of 7000' and wind speed of 50 mph., wave height = 2.8' allow 3.0 3' wave for design.

Assume local rock weighs 165 pounds/ft³ and has a specific gravity of 2.644. The weight of rock required on a 1 on 3 slope to withstand 3' waves, using rounded stone, 2 stones thick (K_D = 2.6)

$$W_r = \frac{165}{2.6 \times 3}\left(\frac{3}{2.644-1}\right)^3 = 128.5 \text{ say } 130 \text{ lb } (60 \text{ kg})$$

Individual stones will be about 300 mm, 0.3 m and riprap thickness will be 600 mm (allow 750 mm)

Thickness of Riprap

130 lb stone is about 0.92' (Square dimension)

(factor for rounding)

For 2 stones thickness, allow 1.13 × 0.92 × 2/3.281 = 0.64 Meters

Allow 0.65 Meters ✓

3004

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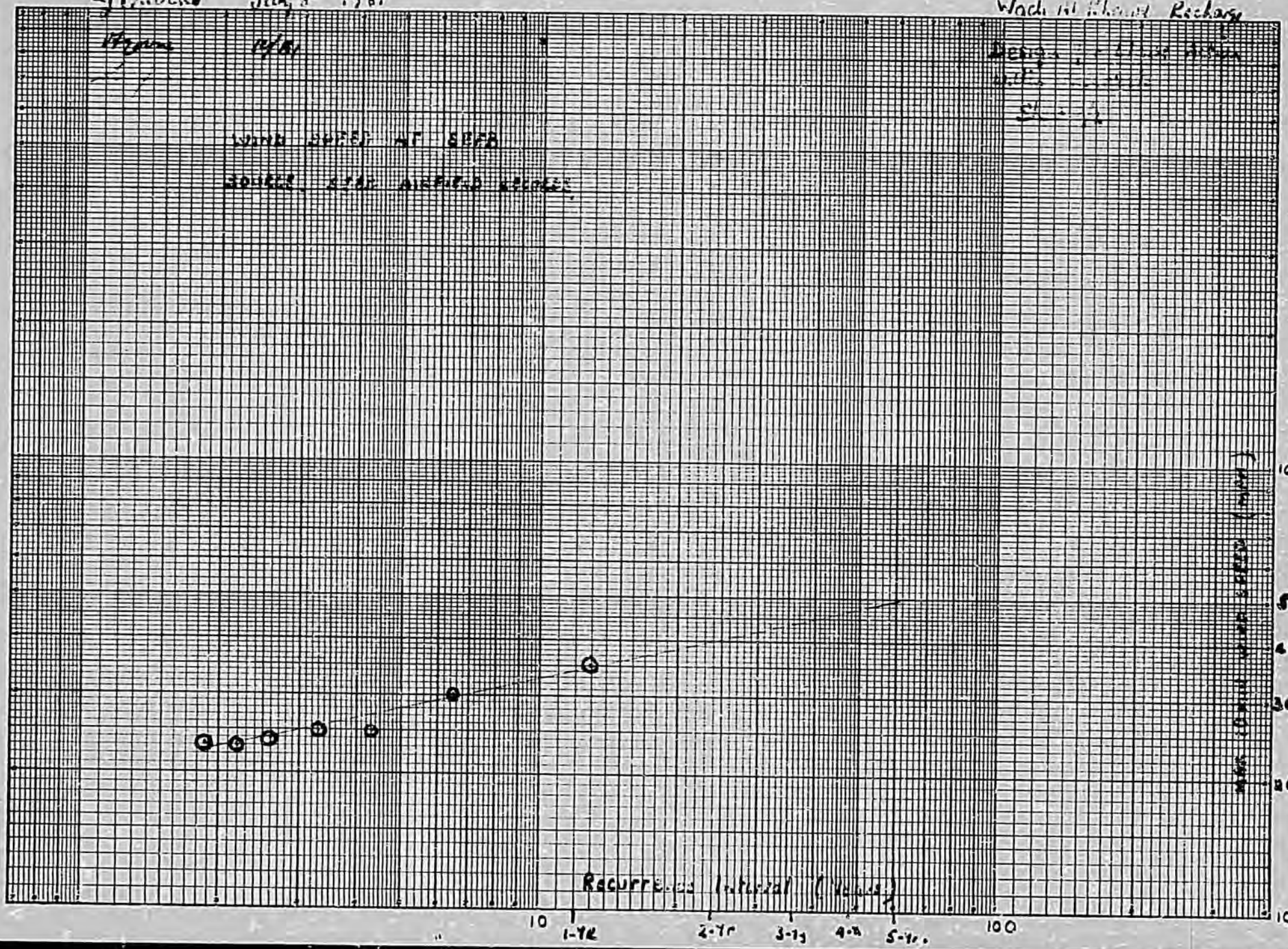
NO. 3123-R. LOGARITHMIC: 2¾ BY TWO 3¾-INCH CYCLES. codex GRAPH PAPER® IN STOCK DIRECT FROM CODEX BOOK CO. INC NORWOOD MASS. 02062 PRINTED IN U.S.A.

8069 P. 38

Wash in Ground Recharge

Wash sweep at SEFB

Double star airfield recharge



[redacted]

Job No. 8069-01 Page No. 39

Computed by G. Hartman Date July '81
Checked by [illegible] Date 2/81
Reviewed by Date
Approved by Date

Subject Wadi Al Khawir Recharge
Ave. Annual Sediment Load
at Recharge Reservoir
Sheet No. of

Note We have been able to trace no records of sediment load measurements, or even sediment load estimates for Oman.
We have attempted to adjust measured, long term sediment load measurements for arid region in USA to still more arid region in Oman. (Highly tentative computation)

USA data +

Reservoir	Location	Drainage Sq. Km.	Reservoir Sedimentation $m^3/Sq.Km/Yr.$	Annual Precip mm.
Piute Reservoir	Utah	6,309	32.7	
Mc. Millan Reservoir	N. Mexico	32,630	34.6	
Elephant Butte Res.	"	44,230	93.2	
Lake Mead	Nevada	432,490	206.5	
Ericson Reservoir	Nebraska	110	278.0	
		515,769	183	

Drainage-area-weighted Sed. Yield (Arid USA) = 183 $m^3/sqKm/year$.

Average stream discharge (Arid USA) = .00145 $m^3/sec/Km^2$

Ave discharge, Wadi Al Khawir = .00049 $m^3/sec/Km^2$

Assuming sediment load concentration varies as $Q^{1.15}$? ("Mobile Bed Hydraulics" T. Blench)

Est. ave. annual sediment load, Wadi Al Khawir : $183 \times \left(\frac{.00049}{.00145}\right)^{1.15}$

: 52.6 $m^3/sq.Km/year$

Total ave. annual sediment load at Wadi Al Khawir = 52.6 × 1615
: 84,874 $m^3/year$.

Estimated fraction of sediment load reaching retarding dam = 90% : 76,390 $m^3/year$

Estimated fraction retained in reservoir = 90% : 68,700 m^3/yr

Allow 70,000 m^3/yr.

+ Source: "The Water Encyclopedia" by David K. Todd. (Units changed)

3004

Job No. 8069-01 Page No. ~~32~~ 40

Computed by G. Flaxlone Date July 7 '82
Checked by W. Gorsse Date 7/8/
Reviewed by Date
Approved by Date

Subject Wadi Al Khawd Recharge
Seive Analysis of random sample
from wadi bed (surface)
Sheet No. of

Total Sample

Seive Size (mm)	Held on Seive grams	Weight Passing Seive	Percent Passing	Seive No.	Percent Passing No. 8 Seive
	136.5				
9.423		172.0	55.8	–	———
	95.5				
2.380		76.5	24.8	8	100.0
	19.5				
1.168		57.0	18.5	16	74.5
	7.5				
0.840		49.5	16.0	20	64.7
	19.0				
0.297		30.5	9.9	50	39.9
	16.5				
0.177		14.0	4.5	80	18.3
	5.5				
0.149		8.5	2.8	100	~~3.7~~ 11.1
	7.5				
0.074		1.0	0.3	– 200	1.3
	1.0				
0.000	———				
TOTAL	308.5				

Sample taken by Steve Yanker in Planters' peanut can.

T

Specific Gravity = 2.7

$\tan \phi = \pm 1.00$ USE 0.85

(Difficult to ascertain from small sample because of large stones which roll down slope when angle of repose is being formed.)

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Computed by G. Pavone Date Oct 1981
Checked by [illegible] Date 12/81
Reviewed by Date
Approved by Date

Job No. 8059-01 Page No. 41
Subject Wadi Al Khawd Recharge
Single Dam Concept
Design Shear Values
Sheet No. of

Note: In response to an unexpected request for embankment slope stability [illegible] weight was used using U.S. Bureau of Reclamation Small dam design criteria from which we assumed a conservatively low friction angle of 33.8° ($\tan \phi = 0.67$)

Samples of Wadi Al Khawd sand-gravel were then given a quick shear test and found to have $\tan \phi$ = 0.91 on the average and 0.85 minimum FOR UNCOMPACTED MATERIAL

The minimum value of 0.85 was used for design although the value is clearly safe because
1) Uncompacted soil was tested.
2) Alluvial fan deposits rapidly become cemented and have cohesion.

From typical borrow materials cited in the Trow-Dames soils report the following estimated unit weights were derived

Sample	Classification	Dry Density: Max. Proctor Density	Dry Density: Design* Density	Dry Density: Opt. Moisture
SS5	GP	132.3	125.7	4.3
SS6	GP	133.5	126.8	2.9
SS7	GP	136.4	129.6	3.4
SS8	GP	132.5	131.6	4.6
		Average [illegible]	128.4	3.8

* Design density = 95% Modified Proctor

Voids ratio: [illegible] / 128.4 = .28 → 0.26

e = [illegible]

Min. Moist weight for design 133.3 pcf - allow 136 pounds/ft³.
Dry weight for design = 131.2 pcf (e = .26)
Saturated weight for design = 131.2 + 62.4 × .26/1.26 = 144 pcf. use 143.5 pcf
Submerged weight = 143.5 - 62.4 = 81.1 pcf.

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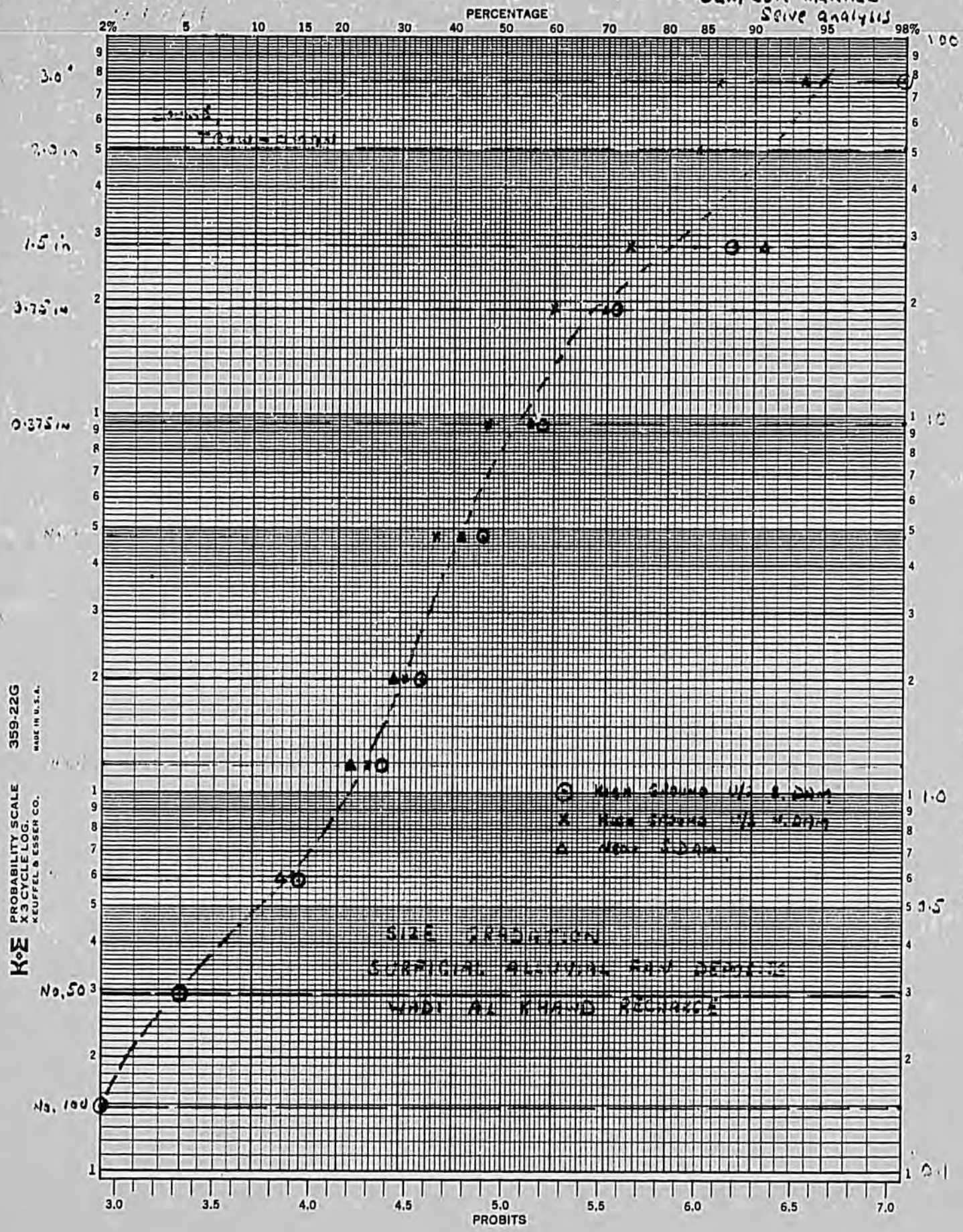
P Mallem
Approved 12/81

8069-01 Page ~~22~~ 22 (Revised)
Wadi Al Khawd Recharge
Dam Core materials
Sieve analysis



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Job No. 8064-01 Page No. 43

Computed by G Parsons Date Oct. 1981

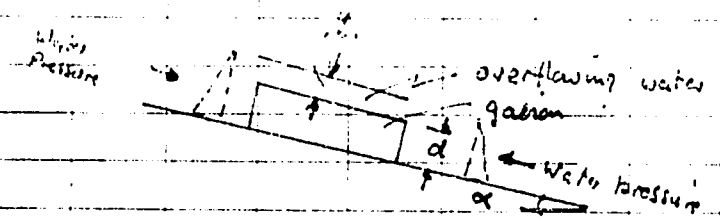
Checked by [illegible] Date 12/81

Reviewed by Date

Approved by Date

Subject Wadi Al Khawd Recharge
Single Dam Concept
Stability of Gabion Mattress

Sheet No. of



Consider 1 square foot of surface area of gabion mattress.

Angle of spillway to horizontal $= \alpha$

Angle of friction between gabion & embankment $= \phi$

Thickness of gabion mattress $= d$ feet.

Voids ratio of gabion fill stone $= e$

Specific gravity of gabion fill stone $= S$

Depth of flow of overflowing spillway discharge $= y$ feet

Dry weight of gabion fill $= \dfrac{62.4S}{1+e}$ pounds/ft^3.

Submerged weight of gabion fill $= \dfrac{62.4}{1+e} \times (S-1)$ pounds/ft^3.

Weight of water in voids of gabion $= \dfrac{62.4\,e}{1+e}$ pounds/ft^3.

Friction force resisting sliding $= \dfrac{62.4\,(S-1)\,d \cos\alpha \tan\phi}{1+e}$ pounds/ft^2

Sliding force due to gabion fill $= \dfrac{62.4\,(S-1)\,d \sin\alpha}{1+e}$

Sliding force due to water in voids $= \dfrac{62.4\,e\,d}{1+e} \sin\alpha$ { Note this is either dead weight if gabion slips or seepage force if gabion does not slip

Sliding force due to overflowing water $= 62.4\,y \sin\alpha$ (assuming y_n = normal depth

$$\text{Factor of safety} = \frac{(S-1)\tan\phi \cot\alpha}{(S-1) + e + \frac{y}{d}(1+e)}$$

When $S = 2.65$, $\tan\phi = .85$, $\cot\alpha = 3$, $y/d = 0.43$ (for MFF)

Factor of safety $= 1.54$

3004

1-11

PROGRAM DES3

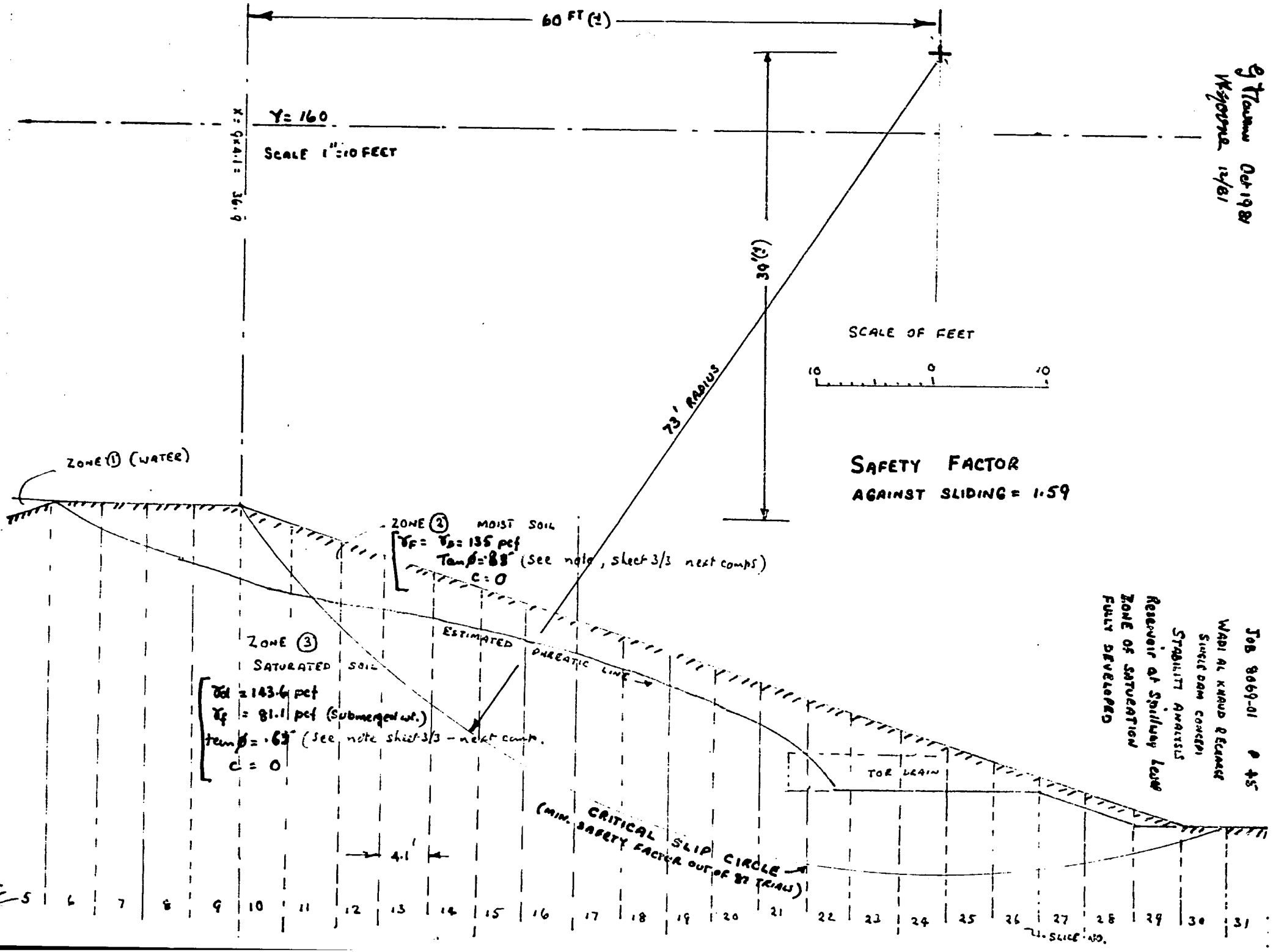
SLOPE STABILITY BY STANDARD METHOD OF SLICES
WITH OPTIONAL INCREASED DRIVING MOMENT
FROM FRICTION DUE TO OVERFLOW

LIST

```
   10 PRINT TAB(20);"SLOPE STABILITY ANALYSIS"
   20 PRINT TAB(17);"SLIP CIRCLE,  METHOD OF SLICES"
   25 IF Q=0 THEN 40
   30 PRINT TAB(8);"INCLUDING SURFACE SHEAR FROM EMBANKMENT OVERFLOW": PRINT : PR
INT
   40 IF F9>0 THEN  SET OP=0: GOTO 100
   50 DIM S(50,5),Z(4,4)
   60 PRINT : INPUT "EMBANKMENT OVERFLOW (cfs/ft) = ",Q
   70 INPUT "MANNING'S n ON SLOPE = ",NO: LET U=(Q*NO/1.486)↑2*62.4
   80 PRINT : INPUT "WILL YOU USE THE PRINTER ? ",F9
   90 IF F9>0 THEN  SET OP=2: GOTO 10
  100 PRINT : PRINT "DATA FOR UP TO 50 SLICES MAY BE ENTERED": PRINT
  110 INPUT "# OF SLICES TO BE ENTERED = ",N
  120 INPUT "WIDTH OF EACH SLICE (ft) = ",W
  130 FOR I=1 TO N
  140   PRINT : PRINT "FOR SLICE # ";I
  150   INPUT "      ELEVN. OF TOP OF ZONE 1 = ",S(I,1)
  160   INPUT "      ELEVN. OF TOP OF ZONE 2 = ",S(I,2)
  170   INPUT "      ELEVN. OF TOP OF ZONE 3 = ",S(I,3)
  180   INPUT "      ELEVN. OF TOP OF ZONE 4 = ",S(I,4)
  190   IF Q>0 THEN  INPUT "      EMBANKMENT SLOPE (rads) = ",S(I,5)
  200 NEXT
  210 PRINT : PRINT "NOW ENTER DATA FOR FOUR EMBANKMENT ZONES": PRINT
  220 FOR J=1 TO 4
  230   PRINT : PRINT "FOR ZONE # ";J
  240   INPUT "      UNIT WEIGHT FOR DRIVE COMP. = ",Z(J,1)
  250   INPUT "      UNIT WEIGHT FOR FRICTION    = ",Z(J,2)
  260   INPUT "      TAN. ANGLE OF FRICTION      = ",Z(J,3)
  270 INPUT "      COHESION (psf)              = ",Z(J,4): NEXT
  280 PRINT : LET N1=1: LET K=1: IF Q>0 THEN  LET K=2
  290 PRINT : INPUT "START SLIP AT LHS SLICE # ",E1
  300 INPUT "END SLIP AT RHS SLICE # ",E2
  310 LET Y8=S(E1,K)-S(E2,K): LET X8=W*(E2-E1+1)
  320 LET A=SQR(X8*X8+Y8*Y8)/2: PRINT "MIN. RADIUS = ";%9F2;A: PRINT
  330 INPUT "TRIAL RADIUS (ft) = ",R
  340 LET B=SQR(R*R-A*A): LET Z=B/A: LET X0=W*E2-(X8-Z*Y8)/2: LET Y0=S(E2,K)+(Y8+
Z*X8)/2
  350 LET C0=0: LET F0=0: LET M0=0
  360 FOR I=E1 TO E2
  370   LET X=(I-.5)*W: LET D=SQR(R*R-(X0-X)*(X0-X)): LET E=Y0-D
  380   LET H1=0: LET H2=0: LET H3=0: LET H4=0: LET C=0: LET F=0: LET M=0: LET G=
0
  390   IF E>S(I,K) THEN  LET I=E2: GOTO 520
  400   IF S(I,2)<E THEN  LET J=1: LET H1=S(I,1)-E: GOTO 460
  410   LET H1=S(I,1)-S(I,2)
  420   IF S(I,3)<E THEN  LET J=2: LET H2=S(I,2)-E: GOTO 460
  430   LET H2=S(I,2)-S(I,3)
  440   IF S(I,4)<E THEN  LET J=3: LET H3=S(I,3)-E: GOTO 460
  450   LET J=4: LET H3=S(I,3)-S(I,4): LET H4=S(I,4)-E
  460   LET C=Z(J,4)*W/D*R*R: LET M=W*(X0-X)*(H1*Z(1,1)+H2*Z(2,1)+H3*Z(3,1)+H4*Z(
4,1))
  470   LET F=W*D*Z(J,3)*(H1*Z(1,2)+H2*Z(2,2)+H3*Z(3,2)+H4*Z(4,2))
  480   IF Q=0 THEN 520
  490   LET Y=H1*COS(S(I,5)): LET G=U/Y↑2.333*W*((X0-X)*TAN(S(I,5))+Y0-S(I,K))
  520 LET C0=C0+C: LET F0=F0+F: LET M0=M0+M+G: NEXT
  530 LET F=(C0+F0)/M0
  540 IF F9>0 THEN  SET OP=2
```

Program DES 3 (Continued)

```
550 IF N1>1 THEN 600
560 IF Q=0 THEN 590
570 PRINT "EMBANKMENT OVERFLOW = ";Q;" (cfs)"
580 PRINT "MANNING'S n ON SURFACE = ";N0: PRINT
590 PRINT " E1 E2    R    X0    Y0          C0          F0          M0     F"
600 PRINT %3I;E1;E2;%5F0;R;%6F0;X0;Y0;%11F0;C0;F0;M0;%6F2;F
610 LET N1=N1+1: SET OP=0: PRINT
620 INPUT "WILL YOU CHANGE RADIUS ? ",F2
630 IF F2>0 THEN 330
640 GOTO 290
650 STOP : END
690 SET OP=2
700 PRINT TAB(20);"E1 = SLIP BEGINS AT LHS OF THIS SLICE"
710 PRINT TAB(20);"E2 = SLIP ENDS AT RHS OF THIS SLICE"
720 PRINT TAB(20);"R = RADIUS OF SLIP CIRCLE (FEET)"
730 PRINT TAB(20);"X0 = X CO.ORDINATE OF CENTER"
740 PRINT TAB(20);"Y0 = Y CO.ORDINATE OF CENTER"
750 PRINT TAB(20);"C0 = MOMENT OF FULLY UTILIZED COHESION"
760 PRINT TAB(20);"F0 = MOMENT OF FULLY UTILIZED FRICTION"
770 PRINT TAB(20);"M0 = DRIVING MOMENT"
780 PRINT TAB(20);"F  = COMPUTED SAFETY ANGINST SHEAR FAILURE"
790 FOR I=1 TO 4
800   PRINT : PRINT TAB(20);"ZONE # ";I
810   PRINT TAB(20);"UNIT WEIGHT FOR DRIVE = ";Z(I,1)
820   PRINT TAB(20);"UNIT WEIGHT FOR FRICTION = ";Z(I,2)
830   PRINT TAB(20);"TAN. ANGLE OF FRICTION = ";Z(I,3)
840 PRINT TAB(20);"COHESION (psf)          = ";Z(I,4): NEXT
850 SET OP=0: STOP
```


60 FT (?)
Y = 160
X = 64.1 = 36.9
SCALE 1" = 10 FEET
30' (?)
73' RADIUS
SCALE OF FEET
10 0 10
SAFETY FACTOR
AGAINST SLIDING = 1.59
ZONE ① (WATER)
ZONE ② MOIST SOIL
γF = γD = 135 pcf
Tan φ = .88 (See note, sheet 3/3 next comps)
c = 0
ESTIMATED PHREATIC LINE
ZONE ③
SATURATED SOIL
γd = 143.6 pcf
γf = 81.1 pcf (Submerged wt.)
tan φ = .62 (See note sheet 3/3 - next comp.
c = 0
TOR DRAIN
CRITICAL SLIP CIRCLE
(MIN. SAFETY FACTOR OUT OF 82 TRIALS)
4.1'
5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31
SLICE NO.
JOB 8069-01 P 45
WADI AL KHAUD RECLAMAGE
SINGLE DAM CONCEPT
STABILITY ANALYSIS
RESERVOIR AT SPILLWAY LEVEL
ZONE OF SATURATION
FULLY DEVELOPED
G Thomas Oct 1981
Wisborne 12/81

[redacted]

Computed by: G. Rowlands Date: Oct 1981
Checked by: [illegible] Date: 12/81
Reviewed by: Date:
Approved by: Date:

Job No. 8069-01 Page No. 46
Subject: Wadi Al Khawd Recharge (Single Dam)
Slope stability with reservoir at spillway / compaction impact
Sheet No. of

	Slice	Top El. Zone 1	Top El. Zone 2	Top El. Zone 3	Top El. Zone 4			
	1	126.32	120.17	120.17	0			Zone (1)
	2	126.32	121.54	121.54	0			Water
	3	126.32	122.90	122.90	0			γ_d = 62.4 pcf
	4	126.32	124.27	124.27	0			γ_f = 0 pcf
	5	126.32	125.64	125.64	0			$\tan\phi$ = 0
								C = 0 psf.
Spillway Crest.	6	126.32	126.32	124.80	0			
	7	126.32	126.32	123.00	0			Zone (2)
	8	126.32	126.32	121.40	0			Sand-Gravel above
	9	126.32	126.32	120.00	0			Zone of Saturation
	10	125.64	125.64	119.20	0			γ_D = 135 lb/ft³
	11	124.27	124.27	118.10	0			γ_f = 135 lb/ft³
	12	122.90	122.90	117.20	0			$\tan\phi$ = 0.85
	13	121.54	121.54	116.20	0			C = 0 psf
	14	120.17	120.17	115.30	0			
	15	118.80	118.80	114.40	0			Zone (3)
	16	117.44	117.44	113.40	0			Sand-Gravel be
	17	116.07	116.07	112.20	0			Zone of Saturation
Spillway Slope	18	114.70	114.70	110.80	0			γ_d = 143.5 pcf
	19	113.34	113.34	109.30	0			γ_f = 81.1 pcf
	20	111.97	111.97	107.90	0			$\tan\phi$ = 0.85
	21	110.60	110.60	106.60	0			C = 0
	22	109.24	109.24	101.50	0			
	23	107.97	107.87	101.50	0			Zone (4)
	24	106.50	106.50	101.50	0			Not used
	25	105.14	105.14	101.50	0			(Zone 3 data extr
	26	103.77	103.77	101.50	0			
	27	102.40	102.40	101.00	0			
	28	101.04	101.04	98.76	0			
	29	99.67	99.67	98.76	0			
	30	98.76	98.76	98.76	0			
	31	98.76	98.76	98.76	0			
Stilling Basin	32	98.76	98.76	98.76	0			
	33	98.76	98.76	98.76	0			
	34	98.76	98.76	98.76	0			

3004

8069-01 P47
Wadi AL Khawd Recharge
Slope stability of D/S face
Case 1 with reservoir full
to spillway level.

SLOPE STABILITY ANALYSIS
SLIP CIRCLE, METHOD OF SLICES

E2	R	XO	YO	CO	FO	MO	F
29	80.	92.	175.	0.	6655026.	3225472.	2.06
8 29	85.	94.	181.	0.	6704025.	3239425.	2.07
8 29	70.	88.	163.	0.	6619191.	3196124.	2.07
10 29	60.	91.	153.	0.	4847677.	2365318.	2.05
10 29	65.	93.	159.	0.	4800842.	2358574.	2.04
10 29	70.	95.	165.	0.	4788460.	2352278.	2.04
10 29	75.	96.	171.	0.	4793289.	2347210.	2.04
10 29	80.	98.	177.	0.	4811136.	2342514.	2.05
10 30	60.	92.	150.	0.	5502158.	2689785.	2.05
10 30	70.	96.	163.	0.	5354523.	2665703.	2.01
10 30	80.	100.	175.	0.	5335819.	2645695.	2.02
10 30	73.	97.	167.	0.	5342022.	2659136.	2.01
10 31	60.	93.	148.	0.	6081215.	2941258.	2.07
10 31	70.	97.	162.	0.	5794896.	2883953.	2.01
10 31	80.	100.	174.	0.	5696854.	2834761.	2.01
10 31	90.	104.	186.	0.	5667774.	2790168.	2.03
10 31	76.	99.	169.	0.	5725786.	2853417.	2.01
10 31	77.	99.	171.	0.	5717939.	2848571.	2.01
10 31	78.	100.	172.	0.	5710073.	2843939.	2.01
31	79.	100.	173.	0.	5703071.	2839338.	2.01
10 32	70.	98.	160.	0.	6321149.	3119362.	2.03
10 32	80.	101.	173.	0.	6130910.	3041493.	2.02
10 32	90.	105.	185.	0.	6033157.	2971524.	2.03

Min. Safety factor = 2.01

E1 = SLIP BEGINS AT LHS OF THIS SLICE
E2 = SLIP ENDS AT RHS OF THIS SLICE
R = RADIUS OF SLIP CIRCLE (FEET)
XO = X CO.ORDINATE OF CENTER
YO = Y CO.ORDINATE OF CENTER
CO = MOMENT OF FULLY UTILIZED COHESION
FO = MOMENT OF FULLY UTILIZED FRICTION
MO = DRIVING MOMENT
F = COMPUTED SAFETY ANGINST SHEAR FAILURE

Safety Factor Exceeds 2.01

ZONE # 1
EFFECTIVE UNIT WEIGHT FOR DRIVE = 62.4
EFFECTIVE UNIT WEIGHT FOR FRICTION = 0
TANGENT OF FRICTION ANGLE = 0
COHESION (psf) = 0

ZONE # 2 (Gabions + Moist soil underlayer)
EFFECTIVE UNIT WEIGHT FOR DRIVE = 135
EFFECTIVE UNIT WEIGHT FOR FRICTION = 135
TANGENT OF FRICTION ANGLE = .85
COHESION (psf) = 0 (Effective cohesion due to wire mesh ignored)

ZONE # 3 (Saturated soil zone)
EFFECTIVE UNIT WEIGHT FOR DRIVE = 143.5
EFFECTIVE UNIT WEIGHT FOR FRICTION = 81.1
TANGENT OF FRICTION ANGLE = .85
COHESION (psf) = 0

ZONE # 4 (not penetrated by slip circle)
EFFECTIVE UNIT WEIGHT FOR DRIVE = 143.5
EFFECTIVE UNIT WEIGHT FOR FRICTION = 81.1
TANGENT OF FRICTION ANGLE = .85
COHESION (psf) = 0

G F Taucner Oct 1981

Figure 17/81

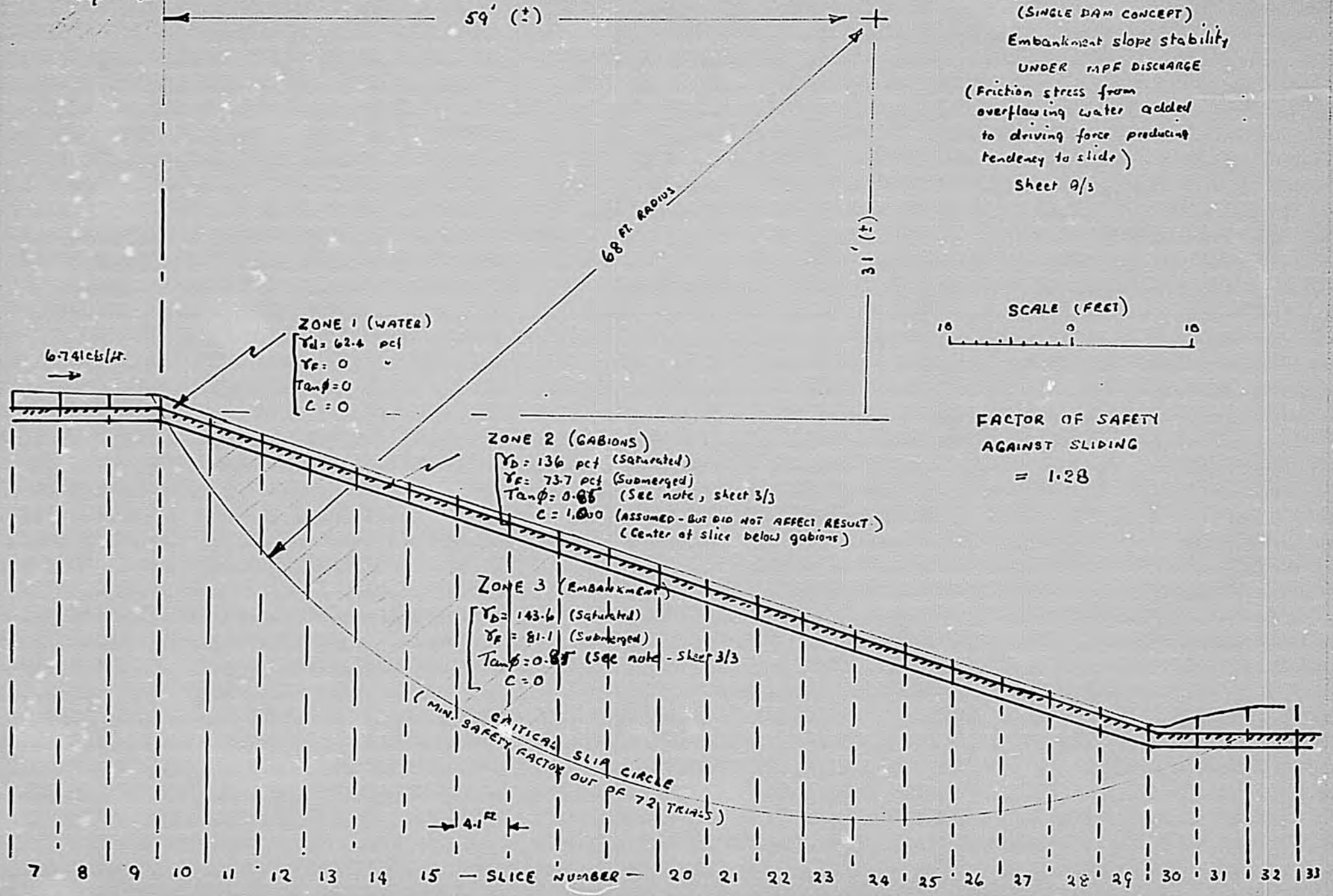
Job 8069-01

WADI AL KHAWD RECHARGE
(SINGLE DAM CONCEPT)

Embankment slope stability
UNDER MPF DISCHARGE

(Friction stress from overflowing water added to driving force producing tendency to slide)

Sheet 2/3



FACTOR OF SAFETY AGAINST SLIDING = 1.28

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Job No. 8069-01 Page No. 49

Computed by G. Reuben Date Oct 1981

Checked by R. Youme Date 12/81

Reviewed by Date

Approved by Date

Subject Wadi Al Khawd Recharge (Single Dam)
Slope Stability D/S face passing
MPF / Computer Input

Sheet No. of

	Slice	EL. TOP OF ZONE 1	EL. TOP OF ZONE 2	EL. TOP OF ZONE 3	EL TOP OF ZONE 4	Embankment Slope (radians)	Embankment Properties
	1	123.13	123.17	120.17	0	-.32175	
	2	128.13	121.54	121.54	0	-.32175	Manning's n on
	3	128.13	122.90	122.90	0	-.32175	slope = 0.03
	4	128.13	124.27	124.27	0	-.32175	
	5	128.00	125.64	125.64	0	-.32175	ZONE 1
Spillway Crest	6	127.78	126.32	123.32	0	.00000	(Water)
	7	127.73	126.32	125.32	0	.00000	γ_d = 62.4 pcf
	8	127.68	126.32	125.32	0	.00000	γ_s = 0 pcf
	9	127.55	126.32	125.32	0	.00000	tan ϕ = 0
	10	126.34	125.64	124.59	0	.32175	C = 0 psf
	11	124.82	124.27	123.22	0	.32175	
	12	123.80	122.90	121.85	0	.32175	Zone 2
	13	122.02	121.54	120.49	0	.32175	(Gravel) (Saturated)
	14	120.64	120.17	119.12	0	.32175	γ_d = 136.1 pcf
	15	119.25	118.80	117.75	0	.32175	γ_s = 73.7 pcf
	16	117.89	117.44	116.39	0	.32175	tan ϕ = 0.85
Spillway Face	17	116.52	116.07	115.02	0	.32175	C = 1000 psf
	18	115.15	114.70	113.65	0	.32175	(effective)
	19	113.79	113.34	112.29	0	.32175	
	20	112.42	111.97	110.92	0	.32175	Zone 3
	21	111.05	110.60	109.55	0	.32175	Sand Gravel
	22	109.69	109.24	108.19	0	.32175	(Saturated)
	23	108.32	107.87	106.82	0	.32175	γ_d = 143.5
	24	106.95	106.50	105.45	0	.32175	γ_s = 81.1
	25	105.59	105.14	104.09	0	.32175	tan ϕ = .85
	26	104.22	103.77	102.72	0	.32175	C = 0
	27	102.85	102.40	101.35	0	.32175	
	28	101.49	101.04	99.99	0	.32175	Zone 4
	29	100.12	99.67	98.62	0	.32175	(Same as Zone 3)
	30	99.60	98.76	97.76	0	.00000	
	31	100.4	98.76	97.76	0	.00000	
	32	101.0	98.76	97.76	0	.00000	
Stilling Basin	33	101.0	98.76	97.76	0	.00000	
	34	101.0	98.76	97.76	0	.00000	

30.04

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J F Tavener Oct 1981
V Byorne 12/81

Job 8069-01 P 50
Wadi Al Khawd Recharge (Single Dam)
CASE 2 WITH MPF PASSING SPILLWAY
ENTIRE DAM SECTION SATURATED
D/S SLOPE STABILITY
Sheet 1/2

SLOPE STABILITY ANALYSIS
SLIP CIRCLE, METHOD OF SLICES
INCLUDING SURFACE SHEAR FROM EMBANKMENT OVERFLOW

EMBANKMENT OVERFLOW = 6.741 (cfs)
MANNING'S n ON SURFACE = .03

E1	E2	R	X0	Y0	C0	F0	M0	F		
8	29	60.	84.	149.	0.	5823974.	3289288.	1.77		
8	29	70.	88.	163.	313914.	5497573.	3354451.	1.73		
8	29	80.	92.	175.	344866.	5361173.	3413291.	1.67		
8	29	90.	96.	187.	379797.	5300850.	3468775.	1.64		
8	29	100.	99.	198.	416842.	5278728.	3522188.	1.62		
8	29	120.	105.	219.	494383.	5291424.	3625255.	1.60		
8	29	140.	111.	239.	574474.	5341345.	3725357.	1.59		
8	29	150.	114.	250.	615099.	5373681.	3774712.	1.59		
8	29	160.	117.	260.	656001.	5409181.	3823722.	1.59		
8	29	180.	123.	280.	6.5x1000x180 0.	5489508.	3914180.	1.40	1.54	hand comp.
8	29	190.	126.	290.	780890.	5532241.	3962036.	1.59		
8	29	170.	120.	270.	6.5x1000x170 0.	5448237.	3866229.	1.41	1.69	Hand Comp.
8	29	175.	122.	275.	6.5x1000x175 0.	5468669.	3890216.	1.41	1.70	

Note 1000 psf effective "cohesion" for gabions included but missed in the computations when the center of a slice fails to intersect the slip circle inside Zone 2 (the gabion)

10	31	100.	107.	197.	416693.	4290142.	2951703.	1.59
10	31	140.	120.	239.	1148451.	3940757.	2880229.	1.77
10	31	80.	100.	174.	344663.	4583118.	2991003.	1.65
10	31	110.	110.	207.	908147.	4186156.	2933199.	1.74
10	31	90.	104.	186.	379621.	4417102.	2970805.	1.61
10	31	97.	106.	193.	405409.	4325344.	2957347.	1.60
10	31	102.	108.	199.	845975.	4267802.	2947966.	1.73
10	31	99.	107.	196.	412918.	4301642.	2953577.	1.60

10	30	80.	100.	175.	340293.	4222078.	2806864.	1.63
10	30	90.	103.	187.	376427.	4118614.	2812386.	1.60
10	30	100.	107.	197.	414319.	4044789.	2817915.	1.58
10	30	110.	110.	208.	453314.	3988117.	2823413.	1.57
10	30	120.	113.	218.	493059.	3942203.	2828885.	1.57
10	30	130.	116.	229.	533343.	3903415.	2834331.	1.57
10	30	140.	119.	239.	1148132.	3869990.	2839759.	1.77

Safety Factor exceeds 1.5

10	29	70.	95.	165.	302736.	3814144.	2486089.	1.66		
10	29	80.	98.	177.	337224.	3702075.	2500194.	1.62		
10	29	90.	102.	188.	374291.	3631751.	2513925.	1.59		
10	29	100.	105.	199.	412832.	3583435.	2527371.	1.58		
10	29	110.	108.	209.	452315.	3548041.	2540609.	1.57		
10	29	120.	112.	219.	492449.	3520810.	2553691.	1.57		
10	29	130.	115.	230.	533058.	3499025.	2566655.	1.57		
10	29	140.	118.	240.	5[illegible] 0.	3481153.	2574531.	1.35	1.57	
10	29	150.	121.	250.	619439 0.	3467531.	2586690.	1.34	1.58	
10	29	160.	124.	260.	657755.	3455966.	2598930.	1.58		

Min. Safety factor 1.5

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G Hawwin
Aqaba 12/81

Job 8069-01 P51
Wadi Al Khawd Recha
Case 2 with MPF passi
Spillway
Slope Stability of D/S S
Sheet 2/2.

E1 = SLIP BEGINS AT LHS OF THIS SLICE
E2 = SLIP ENDS AT RHS OF THIS SLICE
R = RADIUS OF SLIP CIRCLE (FEET)
XO = X CO.ORDINATE OF CENTER
YO = Y CO.ORDINATE OF CENTER
CO = MOMENT OF FULLY UTILIZED COHESION
FO = MOMENT OF FULLY UTILIZED FRICTION
MO = DRIVING MOMENT
F = COMPUTED SAFETY ANGINST SHEAR FAILURE

ZONE # 1
UNIT WEIGHT FOR DRIVE = 62.4
UNIT WEIGHT FOR FRICTION = 0
TAN. ANGLE OF FRICTION = 0
COHESION (psf) = 0

ZONE # 2
UNIT WEIGHT FOR DRIVE = 136.1
UNIT WEIGHT FOR FRICTION = 73.7
TAN. ANGLE OF FRICTION = .85
COHESION (psf) = 1000 ← Allowed because surface is not otherwise stable and stones would move down-slope under action of spillway flow

ZONE # 3
UNIT WEIGHT FOR DRIVE = 143.5

UNIT WEIGHT FOR FRICTION = 81.1
TAN. ANGLE OF FRICTION = .85 ← Conservatively low.
COHESION (psf) = 0

ZONE # 4
UNIT WEIGHT FOR DRIVE = 143.5
UNIT WEIGHT FOR FRICTION = 81.1
TAN. ANGLE OF FRICTION = .85
COHESION (psf) = 0

Program DES 3A

(Very conservative – Seepage force added to saturated weight to yield driving moment. Vertical component of seepage force added to submerged weight for frictional resistance)

```
 10 PRINT TAB(20);"SLOPE STABILITY ANALYSIS"
 20 PRINT TAB(17);"SLIP CIRCLE,  METHOD OF SLICES"
 22 PRINT TAB(15);"INCLUDING EFFECT OF SEEPING WATER"
 25 IF Q=0 THEN 40
 30 PRINT TAB(8);"INCLUDING SURFACE SHEAR FROM EMBANKMENT OVERFLOW": PRINT : PR
INT
 40 IF F9>0 THEN  SET OP=0: GOTO 100
 50 DIM S(50,6),Z(4,5)
 60 PRINT : INPUT "EMBANKMENT OVERFLOW (cfs/ft) = ",Q
 70 INPUT "MANNING'S n ON SLOPE = ",NO: LET U=(Q*NO/1.486)↑2*62.4
 80 PRINT : INPUT "WILL YOU USE THE PRINTER ? ",F9
 90 IF F9>0 THEN  SET OP=2: GOTO 10
100 PRINT : PRINT "DATA FOR UP TO 50 SLICES MAY BE ENTERED": PRINT
110 INPUT "# OF SLICES TO BE ENTERED = ",N
120 INPUT "WIDTH OF EACH SLICE (ft) = ",W
130 FOR I=1 TO N
140   PRINT : PRINT "FOR SLICE # ";I
150   INPUT "      ELEVN. OF TOP OF ZONE 1 = ",S(I,1)
160   INPUT "      ELEVN. OF TOP OF ZONE 2 = ",S(I,2)
170   INPUT "      ELEVN. OF TOP OF ZONE 3 = ",S(I,3)
180   INPUT "      ELEVN. OF TOP OF ZONE 4 = ",S(I,4)
190   IF Q>0 THEN  INPUT "      EMBANKMENT SLOPE (rads) = ",S(I,5)
195   INPUT "      SLOPE OF PHREATIC LINE (rads) = ",S(I,6)
200 NEXT
210 PRINT : PRINT "NOW ENTER DATA FOR FOUR EMBANKMENT ZONES": PRINT
220 FOR J=1 TO 4
230   PRINT : PRINT "FOR ZONE # ";J
240   INPUT "      UNIT WEIGHT FOR DRIVE COMP. = ",Z(J,1)
250   INPUT "      UNIT WEIGHT FOR FRICTION    = ",Z(J,2)
255   INPUT "      WT. GRAVITATIONAL WATER (pcf) = ",Z(J,5)
260   INPUT "      TAN. ANGLE OF FRICTION      = ",Z(J,3)
270 INPUT "      COHESION (psf)              = ",Z(J,4): NEXT
280 PRINT : LET N1=1: LET K=1: IF Q>0 THEN  LET K=2
290 PRINT : INPUT "START SLIP AT LHS SLICE # ",E1
300 INPUT "END SLIP AT RHS SLICE # ",E2
310 LET Y8=S(E1,K)-S(E2,K): LET X8=W*(E2-E1+1)
320 LET A=SQR(X8*X8+Y8*Y8)/2: PRINT "MIN. RADIUS = ";%9F2;A: PRINT
330 INPUT "TRIAL RADIUS (ft) = ",R
340 LET B=SQR(R*R-A*A): LET Z=B/A: LET X0=W*E2-(X8-Z*Y8)/2: LET Y0=S(E2,K)+(Y8+
Z*X8)/2
350 LET C0=0: LET F0=0: LET M0=0
360 FOR I=E1 TO E2
370   LET X=(I-.5)*W: LET D=SQR(R*R-(X0-X)*(X0-X)): LET E=Y0-D
380   LET H1=0: LET H2=0: LET H3=0: LET H4=0: LET C=0: LET F=0: LET M=0: LET G=
0
390   IF E>S(I,K) THEN  LET I=E2: GOTO 520
400   IF S(I,2)<E THEN  LET J=1: LET H1=S(I,1)-E: GOTO 460
410   LET H1=S(I,1)-S(I,2)
420   IF S(I,3)<E THEN  LET J=2: LET H2=S(I,2)-E: GOTO 460
 30   LET H2=S(I,2)-S(I,3)
 40   IF S(I,4)<E THEN  LET J=3: LET H3=S(I,3)-E: GOTO 460
450   LET J=4: LET H3=S(I,3)-S(I,4): LET H4=S(I,4)-E
460   LET C=Z(J,4)*W/D*R*R: LET M=W*(X0-X)*(H1*Z(1,1)+H2*Z(2,1)+H3*Z(3,1)+H4*Z(
4,1))
470   LET F=W*D*Z(J,3)*(H1*Z(1,2)+H2*Z(2,2)+H3*Z(3,2)+H4*Z(4,2))
471   LET G0=(H1*Z(1,5)+H2*Z(2,5)+H3*Z(3,5)+H4*Z(4,5))*W*SIN(S(I,6))
472   LET F=F+G0*SIN(S(I,6))*D*Z(J,3): LET M=M+G0*SIN(S(I,6))*(X0-X)
```

```
473   LET H0=Y0-S(I,1)+H1/2: LET G1=H1*Z(1,5)*H0
 74   LET H0=H0+H1/2+H2/2: LET G1=G1+H2*Z(2,5)*H0
475   LET H0=H0+H2/2+H3/2: LET G1=G1+H3*Z(3,5)*H0
476   LET H0=H0+H3/2+H4/2: LET G1=G1+H4*Z(4,5)*H0: LET M=M+G1*W*COS(S(I,6))
480   IF Q=0 THEN 520
490   LET Y=H1*COS(S(I,5)): LET G=U/Y↑2.333*W*((X0-X)*TAN(S(I,5))+Y0-S(I,K))
520 LET C0=C0+C: LET F0=F0+F: LET M0=M0+M+G: NEXT
530 LET F=(C0+F0)/M0
540 IF F9>0 THEN  SET OP=2
550 IF N1>1 THEN 600
560 IF Q=0 THEN 590
570 PRINT "EMBANKMENT OVERFLOW = ";Q;" (cfs)"
580 PRINT "MANNING'S n ON SURFACE = ";N0: PRINT
590 PRINT " E1 E2    R    X0    Y0          C0          F0          M0     F"
600 PRINT %3I;E1;E2;%5F0;R;%6F0;X0;Y0;%11F0;C0;F0;M0;%6F2;F
610 LET N1=N1+1: SET OP=0: PRINT
620 INPUT "WILL YOU CHANGE RADIUS ? ",F2
630 IF F2>0 THEN 330
640 GOTO 290
650 STOP : END
690 SET OP=2
700 PRINT TAB(20);"E1 = SLIP BEGINS AT LHS OF THIS SLICE"
710 PRINT TAB(20);"E2 = SLIP ENDS AT RHS OF THIS SLICE"
720 PRINT TAB(20);"R = RADIUS OF SLIP CIRCLE (FEET)"
730 PRINT TAB(20);"X0 = X CO.ORDINATE OF CENTER"
740 PRINT TAB(20);"Y0 = Y CO.ORDINATE OF CENTER"
 50 PRINT TAB(20);"C0 = MOMENT OF FULLY UTILIZED COHESION"
/60 PRINT TAB(20);"F0 = MOMENT OF FULLY UTILIZED FRICTION"
770 PRINT TAB(20);"M0 = DRIVING MOMENT"
780 PRINT TAB(20);"F  = COMPUTED SAFETY ANGINST SHEAR FAILURE"
790 SET OP=0: STOP
```

Computed by: G Yavuz Date: Oct 1981
Checked by: M Jorne Date: 12/81
Reviewed by: Date:
Approved by: Date:

Job No. 8069-01 Page No. P54
Subject: Wadi Al Khawer (Single Dam Concept
D/S Slope stability prior to slip
Sheet No. of

Slice	El. Top of Zone 1	El. Top of Zone 2	El. Top of Zone 3	El. Top of Zone 4	Embankment Slope (rads)	Piezometric Slope (rads)	
1	129.13	122.17	122.17	—	-.32 175	0	Zone (1)
2	128.13	121.54	121.54	—	-.32 175	0	(Water)
3	128.13	122.90	122.90	—	-.32 175	0	γ_d = 62.4
4	128.13	124.27	124.27	—	-.32 175	0	γ_f = 0
5	128.00	125.64	125.64	—	-.32 175	0	γ_w = 0
6	127.78	126.32	123.32	—	0	.01 3	tan ϕ = 0
7	127.73	126.32	125.32	—	0	.01 3	C = 0
8	127.68	126.32	125.32	—	0	.02 2	
9	127.55	126.32	125.32	—	0	.04 1	Zone (2)
10	126.34	125.64	124.59	—	.32 175	.25 0	(Gravel)
11	124.82	124.27	123.22	—	.32 175	.32 175	γ_d = 73.7
12	123.80	122.90	121.85	—	.32 175	.32 175	γ_f = 73.7
13	122.02	121.54	120.49	—	"	"	γ_w = 17.5
14	120.64	120.17	119.12	—	"	"	tan ϕ = .85
15	119.25	118.80	117.75	—	"	"	C = 1000
16	117.89	117.44	116.39	—	"	"	(effective)
17	116.52	116.07	115.02	—	"	"	
18	115.15	114.70	113.65	—	"	"	Zone (3)
19	113.79	113.34	112.29	—	"	"	Sat. Sand-Gravel
20	112.42	111.97	110.92	—	"	"	γ_d = 81.1
21	111.05	110.60	109.55	—	"	"	γ_f = 81.1
22	109.69	109.24	108.19	—	"	"	γ_w = 12.9
23	108.32	107.87	106.82	—	"	"	tan ϕ = .85
24	106.95	106.50	105.45	—	"	"	C = 0
25	105.59	105.14	104.09	—	"	"	
26	104.22	103.77	102.72	—	"	"	Zone (4)
27	102.85	102.40	101.35	—	"	"	
28	101.49	101.04	99.99	—	"	"	(Same as 3)
29	100.12	99.67	98.62	—	.32 175	.32 175	
30	99.60	98.76	97.76	—	.00 000	.00 000	
31	100.4	98.76	97.76	—	.00 000	.00 000	
32	101	98.76	97.76	—	.00 000	.00 000	
33	101	98.76	97.76	—	.00 000	.00 000	
34	101	98.76	97.76	—	.00 000	.00 000	

3004

G Flowers Oct 1981
Wyoma 1/81

Program DES 3A

8069-01 P55
Wadi Al Khawa Recharge
D/S slope stability
Passing MPF over Spillway
Safety prior to slippage
examined

SLOPE STABILITY ANALYSIS
SLIP CIRCLE, METHOD OF SLICES
INCLUDING EFFECT OF SEEPING WATER
INCLUDING SURFACE SHEAR FROM EMBANKMENT OVERFLOW

EMBANKMENT OVERFLOW = 6.741 (cfs)
MANNING'S n ON SURFACE = .03

E1	E2	R	X0	Y0	C0	F0	M0	F
10	30	120.	113.	218.	493059.	4009235.	2415498.	1.86
10	30	140.	119.	239.	1148132.	3936677.	2436280.	2.09
10	30	100.	107.	197.	414319.	4112610.	2393970.	1.89
10	30	110.	110.	208.	453314.	4055460.	2404806.	1.87
10	30	112.	111.	210.	461211.	4045491.	2406956.	1.87
10	30	118.	112.	216.	485061.	4017814.	2413372.	1.87
10	30	122.	114.	220.	501078.	4000948.	2417623.	1.86
10	30	124.	114.	222.	509117.	3992926.	2419744.	1.86
10	30	126.	115.	225.	517175.	3985148.	2421860.	1.86
10	30	128.	116.	227.	525251.	3977596.	2423973.	1.86
10	30	130.	116.	229.	533343.	3970251.	2426077.	1.86
10	30	132.	117.	231.	541452.	3963103.	2428180.	1.86
10	30	134.	118.	233.	1098984.	3956257.	2430212.	2.08
10	31	120.	114.	218.	987229.	4165338.	2491198.	2.07
10	31	130.	117.	228.	1067425.	4083698.	2481589.	2.08
10	31	110.	110.	207.	908147.	4256265.	2501052.	2.06
10	31	100.	107.	197.	416693.	4361398.	2511134.	1.90
10	31	90.	104.	186.	379621.	4489855.	2521048.	1.93
10	31	95.	106.	191.	397958.	4422013.	2516030.	1.92
10	31	98.	107.	195.	409157.	4384894.	2513078.	1.91
10	31	102.	108.	199.	845975.	4338807.	2509175.	2.07
10	31	99.	107.	196.	412918.	4373030.	2512105.	1.91
10	31	101.	108.	198.	420480.	4349980.	2510164.	1.90
10	29	80.	98.	177.	337224.	3763719.	2114135.	1.94
10	29	90.	102.	188.	374291.	3692638.	2132022.	1.91
10	29	100.	105.	199.	412832.	3643913.	2149423.	1.89
10	29	110.	108.	209.	452315.	3608316.	2166489.	1.87
10	29	120.	112.	219.	492449.	3581010.	2183315.	1.87
10	29	130.	115.	230.	533058.	3559235.	2199962.	1.86
10	29	140.	118.	240.	0. *	3541437.	2211511.	1.60
10	29	150.	121.	250.	0. *	3527967.	2227665.	1.58
10	29	160.	124.	260.	657755.	3516584.	2243782.	1.86
10	29	170.	128.	269.	699659.	3507076.	2259665.	1.86
10	29	180.	131.	279.	741686.	3498753.	2275551.	1.86
10	29	200.	137.	299.	826030.	3484696.	2307329.	1.87

Slip circle must intersect gradation and hence cohesion should be present - but not detected by coarse grid of slice method. Moment of cohesive resistance computed graphically enabling computation of safety factors added in ink.

Safety factor 1.5, exceeds

G Flowers Oct 1981
Rzovne 12/81

Program DES 3A

8069-01 P56
Wadi Al. Khawa Recharge
D/S slope stability
Prior to slippage
Sheet 2/2

SLOPE STABILITY ANALYSIS
SLIP CIRCLE, METHOD OF SLICES
INCLUDING EFFECT OF SEEPING WATER
INCLUDING SURFACE SHEAR FROM EMBANKMENT OVERFLOW

Stability prior to slippage

Only that portion of weight of water transferred to soil particles because of friction counted in the analysis
Buoyant weight of saturated soil particles used for both drive and friction

```
E1 = SLIP BEGINS AT LHS OF THIS SLICE
E2 = SLIP ENDS AT RHS OF THIS SLICE
R = RADIUS OF SLIP CIRCLE (FEET)
XO = X CO.ORDINATE OF CENTER
YO = Y CO.ORDINATE OF CENTER
CO = MOMENT OF FULLY UTILIZED COHESION
FO = MOMENT OF FULLY UTILIZED FRICTION
MO = DRIVING MOMENT
F  = COMPUTED SAFETY ANGINST SHEAR FAILURE

ZONE #  1
UNIT WEIGHT FOR DRIVE =  62.4
UNIT WEIGHT FOR FRICTION =  0
UNIT WEIGHT INTERSTICIAL WATER =  0
COHESION  (psf)        =  0

ZONE #  2
UNIT WEIGHT FOR DRIVE =  73.7
UNIT WEIGHT FOR FRICTION =  73.7
UNIT WEIGHT INTERSTICIAL WATER =  17.5
COHESION  (psf)        =  1000

ZONE #  3
UNIT WEIGHT FOR DRIVE =  81.1
UNIT WEIGHT FOR FRICTION =  81.1
UNIT WEIGHT INTERSTICIAL WATER =  12.9
COHESION  (psf)        =  0

ZONE #  4
UNIT WEIGHT FOR DRIVE =  81.1
UNIT WEIGHT FOR FRICTION =  81.1
UNIT WEIGHT INTERSTICIAL WATER =  12.9
COHESION  (psf)        =  0

ZONE #  5
UNIT WEIGHT FOR DRIVE =
OB ERROR IN LINE   810
```

Entire weight of Zone 1 applied for friction of overflowing wa

Submerge weight

wt. of water/cubic ft of s

Submerged weight

wt. of water/cubic foot of s soil

No zone 5.

(No zone 5 in computations)

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Computed by G. Nassar Date Oct 1981
Checked by R. Yorke Date 12/81
Reviewed by Date
Approved by Date

Job No. 8069-01 Page No. P 57
Subject Wadi Al Khawd Recharge
Single Dam Concept
Approximate Analysis of dam break
Sheet No. 1 of

Sources

Negative surge within reservoir "Open Channel Hydraulics" Vente Ch
also Journal of Hydraulics division ASCE Nov. 1970
Positive wave attenuation D/S Dam "Technical Release No. 66
Simplified Dam-Breach Routing Procedure" SCS 1979.

Assume rectangular section with diverging flow downstream of dam (due to spreading out over cone of alluvial fan)
1 foot width of flow at dam spreads to ~~2.75~~ 2.50 feet width of flow at the coast road
Assume uniform flat slope of 1 in 155 with Mannings n = .024
Depth of water behind dam at instantaneous break = 26 feet
Inflow is negligible at moment of break
Volume behind 1 foot of dam break = $26 \times 26 \times 155/2 = \underline{\underline{52390\ ft^3}}$.

Max. discharge at instant of dam break $= \left(\frac{2}{\alpha+2}\right)^3 \times y_R \times \sqrt{\alpha g y_R}$

where α is a coefficient depending on valley shape = 1.0 for rectangular
y_R = instantaneous depth of water in reservoir = 26' at moment of brea

Hence Q_{max} at dam $= \left(\frac{2}{3}\right)^3 \times 26 \sqrt{32.2 \times 26} = \underline{\underline{222.9 \text{ cts / foot width of break}}}$

At Section 0 (the dam) (width of flow considered = 1.00 feet)

$$Q = \frac{1.486}{.024} y^{1.667} \left(\frac{1}{155}\right)^{1/2} = 4.973 y^{1.667} \quad ; \quad \text{Area} = y = \left(Q/4.973\right)^{0.6}$$

At Section 1 (2871' D/S. dam) (Width of flow = 1.375'

$$Q = 1.375 \times 4.973 y^{1.667} = 6.838 y^{1.667} \quad ; \quad \text{Area} = 1.375 y = \left(Q/4.021\right)^{0.6}$$

Average area between Section 0 & 1 $= \left(Q/4.458\right)^{.6}$; Storage $= \left(130{,}437\, Q\right)^{.6}$

Putting ~~S = K~~ $Q = K S^m$; $K = \overline{130{,}437}$ and $m = 1.667$

$$K_0^* = \frac{222.9 \times 130{,}437}{52390^{1.667}} = .395 \quad ; \quad Q_0^* = .672$$

$$K^* = .395 (1 - .672)^0 = .395 \quad ; \quad Q^* = .672$$

Job No. 8069-01 Page No. 58

Computed by G. Ravenes Date Oct 1981
Checked by M. Gowne Date 12/81
Reviewed by Date
Approved by Date

Subject Wadi Al Khawd Recharge
Single Dam Concept
Approximate Analysis of Dam Break
Sheet No. 2 of

Max. Discharge at Section 1 (2871' D/S Dam) = .672 × 222.9 = 149.8 cfs

Max. Stage at Section 1 = $(149.8/6.838)^{.6}$ = 6.37'

Velocity 23.5 ft/sec.

At Section 2, 2 × 2871 = 5742' D/S Dam (Width of flow = 1.75 feet)

$Q = 1.75 \times 4.973\, y^{1.667} = 8.703\, y^{1.667}$; Area = $1.75y = \left(\frac{Q}{3.424}\right)^{.6}$

Average area between 1 & 2 = $\left(\left(\frac{Q}{4.021}\right)^{.6} + \left(\frac{Q}{3.424}\right)^{.6}\right)/2 = \left(\frac{Q}{3.704}\right)^{.6}$

Storage between 1 & 2 = $2871 \times \left(\frac{Q}{3.704}\right)^{.6} = (156{,}989\, Q)^{.6}$

Storage between Dam & 2 = $(130{,}437\, Q)^{.6} + (156{,}989\, Q)^{.6} = (456{,}668\, Q)^{.6}$

$K_0^x = \frac{222.9 \times 456{,}668}{52390^{1.667}} = 1.383$; $Q_0^x = 0.48$

and $K^x = K_0^x$; $Q^x = Q_0^x$

Hence max discharge at section 2 = 0.48 × 222.9 = 106.99 cfs

Max. depth of flow = $(Q/8.703)^{.6}$ = 4.5 feet

Vel. 13.59 ft/sec.

At Section 3, 3 × 2871 = 8613' D/S DAM (Width of flow = 2.125')

$Q = 2.125 \times 4.973\, y^{1.667} = 10.568\, y^{1.667}$; Area = $2.125y = \left(\frac{Q}{3.008}\right)^{.6}$

Average Area between 2 and 3 = $\frac{1}{2}\left(\left(\frac{Q}{3.424}\right)^{.6} + \left(\frac{Q}{3.008}\right)^{.6}\right) = \left(\frac{Q}{3.206}\right)^{.6}$

Storage between 2 & 3 = $2871 \times \left(\frac{Q}{3.206}\right)^{.6} = (181{,}375\, Q)^{.6}$

Storage between Dam & 3 = $(456{,}668\, Q)^{.6} + (181{,}375\, Q)^{.6} = (975{,}950\, Q)^{.6}$

$K^* = K_0^x = \frac{222.9 \times 975{,}950}{52390^{1.667}} = 2.955$; $Q_0^x = Q^x = 0.3$

Max. discharge at Section 3 / foot of break = .3 × 222.9 = 66.9 cfs

Max. depth of flow at 3 = $(66.9/10.568)^{.6}$ = 3.03 feet

Vel. 10.39 ft/sec.

188

Computed by: G. Kavanagh Date:
Checked by: V. Kyrone Date: 12/81
Reviewed by: Date:
Approved by: Date:

Job No. 8069-01 Page No. 59
Subject: Wadi Al Khawd Recharge
Single Dam Concept
Simplified Dam Break Analysis
Sheet No. of

At Section 4, $4 \times 2871' = 11{,}484'$ (3.5 Km) D/S Dam (width of flow = 2.5')

$Q = 2.5 \times 4.973\, y^{1.667} = 12.433\, y^{1.667}$; Area $= 2.5y = (Q/2.699)^{.6}$

Average area between Section 3 & 4 $= (Q/2.935)^{.6}$

Storage between Sections 3 & 4 $= 2871\,(Q/2.935)^{.6} = (198{,}122\,Q)^{.6}$

Storage between Dam & Section 4 $= (975{,}950\,Q)^{.6} + (198{,}122\,Q)^{.6}$
$= (1{,}682{,}592\,Q)^{.6}$

$K^* = K_Q^* = \dfrac{222.9 \times 1{,}682{,}592}{52390^{1.667}} = 5.095$; $Q_G^* : Q^* = .185$

Max Discharge at Section 4 (Coast Road) = 41.23 cfs/ft of brea

Max. ave depth of flow at coast road $= (41.23/12.433)^{.6} = 2.05'$

Velocity = 8.04 ft/sec.

Note with no reservoir inflow, reservoir drains practically dry in 2 minutes if the complete dam breaks.

Approx time to reach peak stage at coast road

$= \pm 5 \times T_0 = 5 \times 94.4 = 7.9$ minutes

Job No. 8069-01 Page No. 43 60

Computed by G. Mathew Date July 14-81 Subject Wadi Al Khawir Recharge Dam Design

Checked by M. Zaman Date 17/8/81 Toe Drain, Bedding Zone Material

Reviewed by Date

Approved by Date Sheet No. of

1. Bedding material below Riprap

Riprap size will be about 0.92' = 11.04" (P. 35 of Comp.)

To avoid migration of material through the riprap, bedding zone should be from 11.04/4 To 11.04/6 or 2 3/4" → 1 3/4"

This is larger than typical alluvial fan material. Hence a special bedding zone for the riprap is recommended.

In metric sizing, 2 3/4" = 75 mm
1 3/4" = 50 mm

∴ Use 50 mm to 75 mm stone for Riprap bedding. ✓

② Gabion Bedding

Gabions are filled with 4" → 8" material (Gabion Manuf. data)
(Specify 100 mm → 200 mm stone for Gabion fill)

To prevent migration ~~from~~ of dam material through the gabions, a filter zone of 0.8" → 1.6" stone will be required. This size is abundantly available but, in the sample taken about 80 % of natural material would migrate through the gabion if no filter zone were provided. This would cause some subsidence of the embankment
∴ Specify a filter zone of 0.8" → 1.6" stone

20 mm to 40 mm size ✓

③ Toe drain Material

Natural Wadi material must not migrate through the toe drain
The 50% size of dam material will be about 7 mm
∴ desirable toe drain stone should be 5×7 = 35 mm ✓
∴ USE SAME MATERIAL FOR TOE DRAIN AS FOR RIPRAP BEDDING

3004

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Computed by G. Pavens Date July 16 '81
Checked by [illegible] Date 12/81
Reviewed by Date
Approved by Date

Job No. 8069-01 Page No. ~~45~~ 61
Subject Wadi Al Khawd Recharge
Quantities in Culverts
42-inch Culverts
Sheet No. 2 of 2

(Note changes due to design development)

42-inch Culverts (North Dam)

Length $\cancel{6}\times3+5+\cancel{7.254}\ 6.95\times3 - 4.5 - \cancel{6}\ 4.5$ = ~~41.3~~ 34.14 LM

U/s. Headwall

$(1.5\times2.3 - .7854\times1.295^2)\times.25$	=	~~0.533~~ 1.913 m^3
+ 2 side walls	=	~~1.688~~
+ Base slab	=	3.278
+ Front cutoff	=	0.403
+ haunch	=	0.181
Total		~~6.0~~ 6.42 m^3 (say

All as for 48-inch pipe culvert

Seepage Collars

$5\times(2.9^2 - .7854\times1.295^2)\times.3$ = 10.64 m^3

D/s Headwall

End wall	0.533	
2 Side Walls	2.250	as before
base slab	4.313	Same as U/s.
front cut-off	0.403	
haunch	0.181	
Total	~~7.68~~	6.42 m^3

Superceded by headwalls to scale.

TOTAL QUANTITIES FOR 9 - 42-inch Culverts

42-inch culvert pipe = $9\times$ ~~41.3~~ 34.14 = ~~371.7~~ 307.3 LM

Reinforced concrete in headwalls and seepage collars

$9\times(\cancel{6.0}\ 6.42 + 10.64 + \cancel{7.68}\ 6.42)$ = ~~207.4~~ 211.4 ~~214.8~~ M^3

3004

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Job No. 8069-01 Page No. 62

Computed by G. Pereira Date July 16 '81
Checked by [illegible] Date 12/81
Reviewed by Date
Approved by Date

Subject Wadi Al Khawd Recharge
QUANTITIES IN CULVERTS
48-inch Culverts

Sheet No. 1 of 2

48-in culvert, South Dam (Change for 1 on 3 D/S Slope)

Length = 5×3 + 4 + 6.1×3 − 4.5 − 4.5 = 32.9 m of 48-inch (28.65)

U/S Headwall

1.5×0.25×2.3 − .7854×1.473²×.25 =	0.436 (.551)	m³	(endwall
4.5×1.8×0.25 =	1.688 (1.913)	m³	(2 side walls)
(4.5+.25)×(1.8+.25+.25)×0.30 =	3.278	m³	Base slab
0.7×0.25×(1.8+.25+.25) =	0.403	m³	(front cutoff)
(1.8+.25+.25)×.15×.525 =	0.181	m³	haunch
TOTAL	6.00	m³	
	6.326		

Seepage Collars

4×(3.07×3.07 − .7854×1.473²)×.30 = 9.27 m³ say 9.30 m³

D/S Headwall)

(1.5×2.3 − .7854×1.473²)×.25 =	0.436	m³	endwall
6×1.5×.25 =	2.250	m³	2 side walls
(6+.25)×2.3×.3 =	4.313	m³	base slab
0.7×2.3×.25 =	0.403	m³	front cut-off
2.3×.15×.525 =	0.181	m³	haunch
TOTAL	7.583		say 7.60 m³
Same as U/S =	6.326		

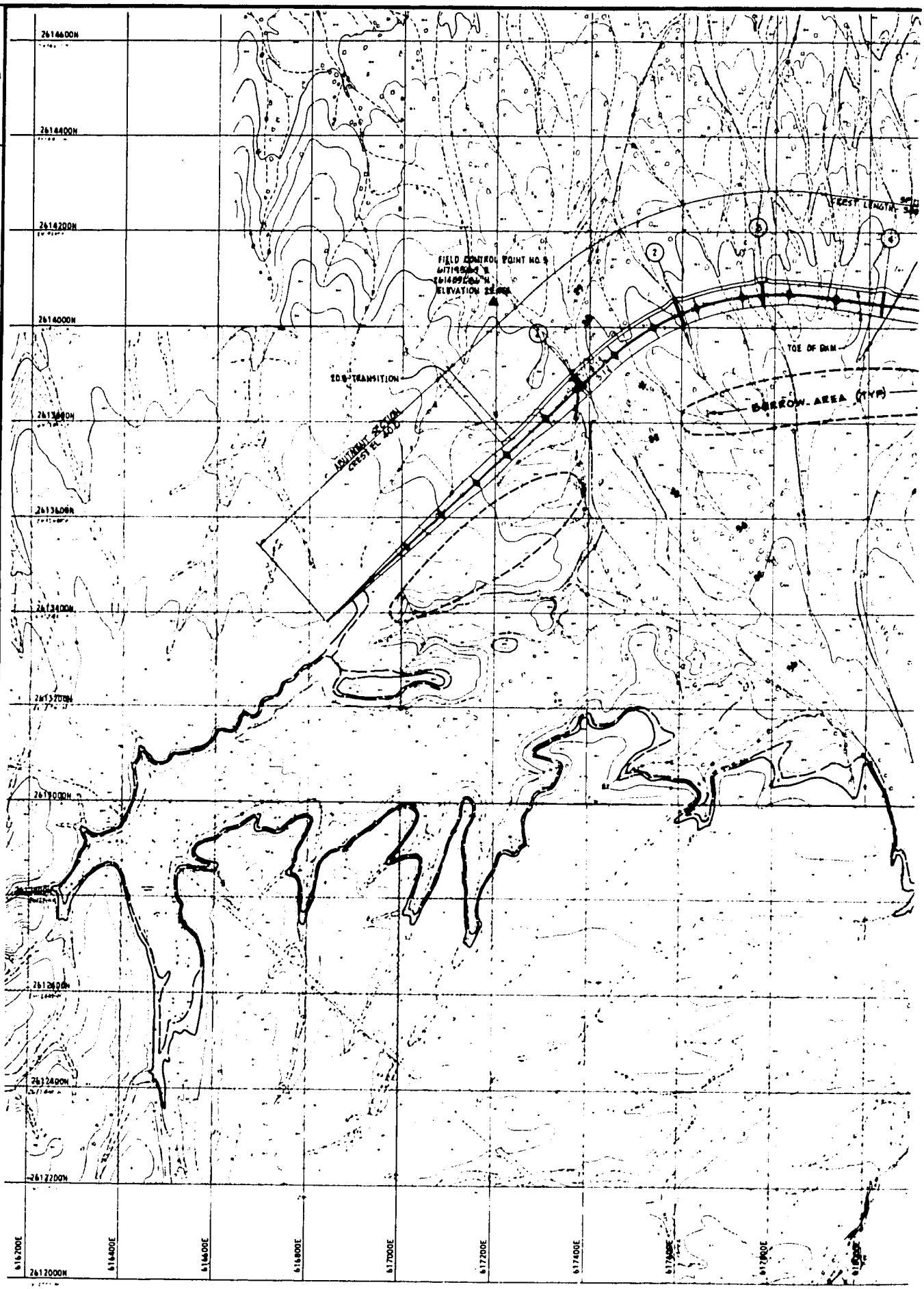
Total Quantities for 8 - 48-inch culverts

48-inch Culvert Pipe 8×32.9 (28.65) = 263.2 (229.2) L.M

Reinforced concrete in headwalls + seepage collars

8×(6.00 (6.33) + 9.30 + 7.60 (6.33)) = 183.2 (175.7) m3

3004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2614400N
2614200N
2614000N
2613800N
2613600N
2613400N
2613200N
2613000N
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2612600N
2612400N
2612200N
2612000N
616200E
616400E
616600E
616800E
617000E
617200E
617400E
617600E
617800E
FIELD CONTROL POINT NO. 9
TOE OF DAM
BORROW AREA (TYP)

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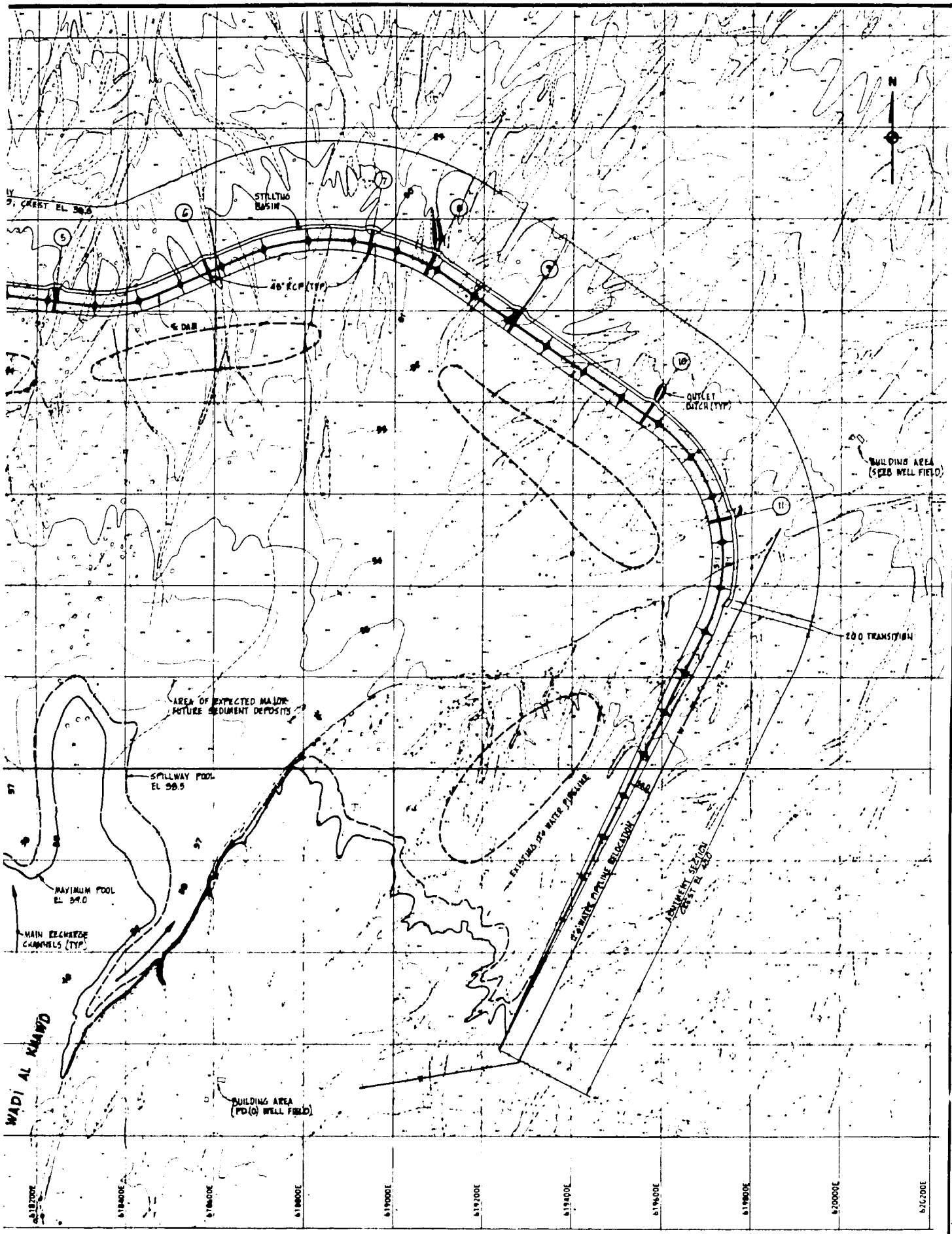


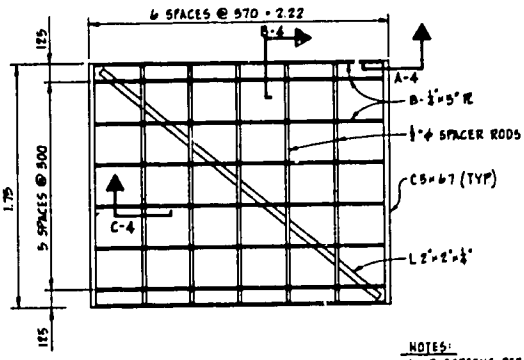
1 THRU 11 — COLVERT IDENTIFICATION FOR SCHEDULE & DETAILS SEE SHEETS 4 & 5

AT FULL SIZE

0 1 2 3 IN

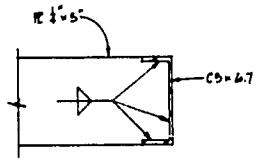
0 1 2 3 4 5 6 7 8 cm

STANLEY CONSULTANTS	WADI AL KHAWD AQUIFER RECHARGE PROJECT	PLAN RECHARGE DAM	SCALE 1:4000 JOB 8069-2 REV 0

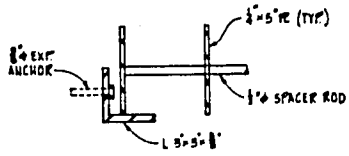


TRASH SCREEN
SCALE: 1:20

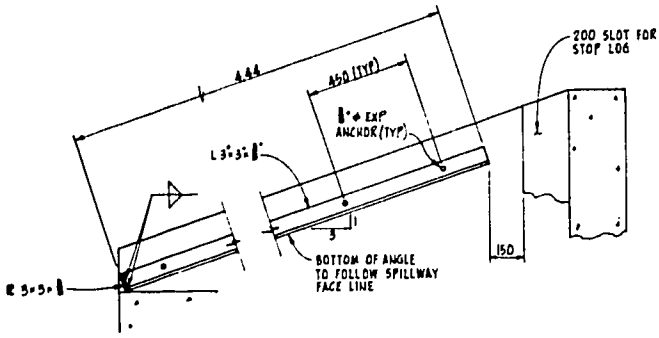
NOTES:
1. 2 SCREENS PER HEADWALL.
2. TRASH SCREENS REQD ON ALL INLET HEADWALLS.
3. GALVANIZE AFTER FABRICATION



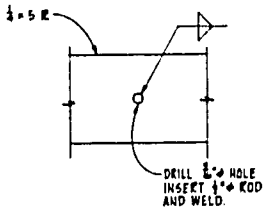
SECTION A-4/4
NO SCALE



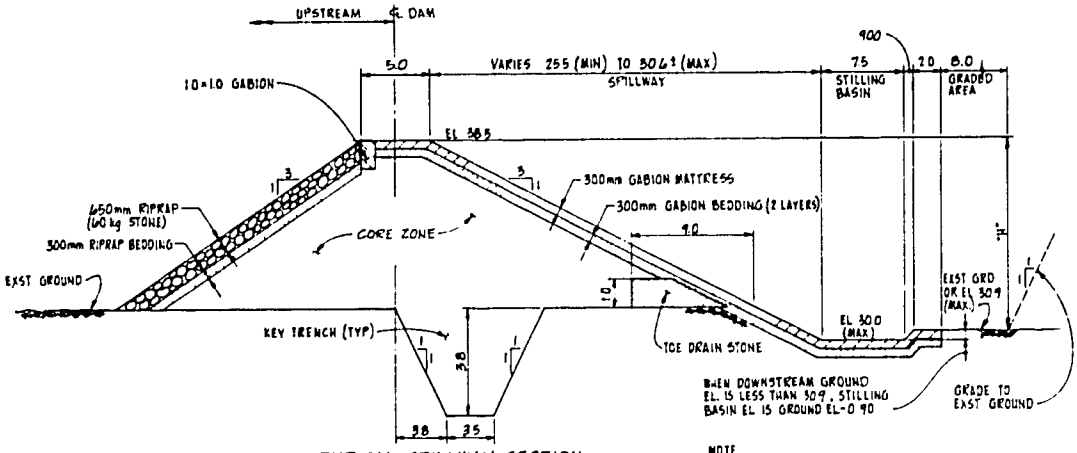
SECTION B-4/4
NO SCALE



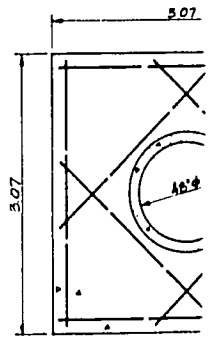
TRASH SCREEN SUPPORT
NO SCALE



SECTION C-4/4
NO SCALE

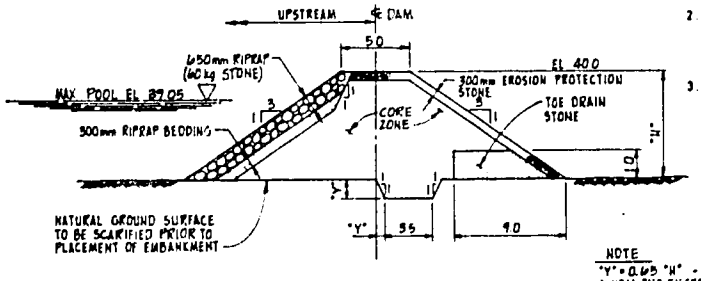


TYPICAL SPILLWAY SECTION
(NTS)



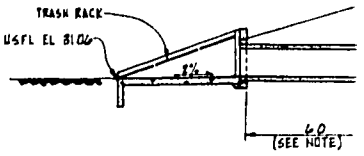
SEEPAGE
SCALE: 1:30

NOTE
1. WHEN OUTLET FLOWLINE OF CULVERT HDWL. IS LOWER THAN EXISTING GROUND, DITCH OUTLET WITH 3.5m WIDTH CHANNEL BED HAVING 1:1 SIDE SLOPES.
2. BORROW MATERIAL OF SPECIFIED GRAIN SIZE MAY BE EXCAVATED FROM THE UPSTREAM RESERVOIR AREA. BORROW EXCAVATION SHOULD NOT BE DEEPER THAN THE UPSTREAM TOE OF THE DAM.
3. SCARIFY NATURAL GROUND SURFACES PRIOR TO PLACING EMBANKMENT.



NOTE
"Y" = 0.65 "H" - 2.15
WHEN "H" EXCEEDS 3.30

TYPICAL ABUTMENT SECTION



NOTE:
MOVE SEEPAGE COLLAR BETWEEN PIPE JOINTS THE DIMENSIONS SHOWN

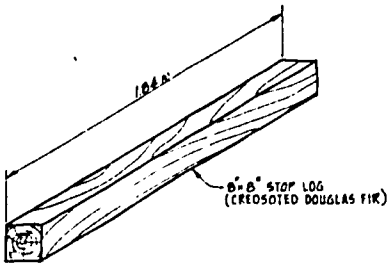
TYPICAL SECTIONS - DAM
NO SCALE

REVISION	BY	APPROVED	APPROVED	DATE	NO.	REVISION	BY	APPROVED	APPROVED	DATE	NO.	REVISION	BY	APPROVED	APPROVED	DATE

DESIGNED
DRAWN
CHECKED
APPROVED
APPROVED
DATE

195

195



TYPICAL STOP LOG

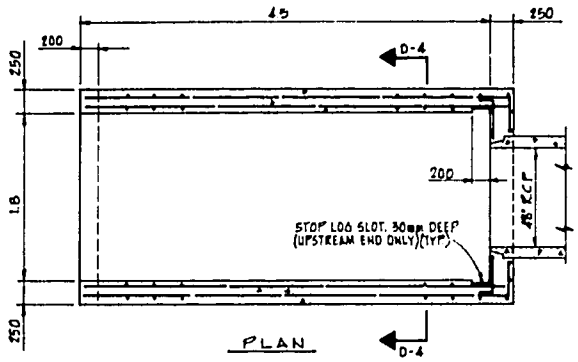
NO SCALE (TRIM OR WEDGE FOR TIGHT FIT IN SLOT)

CULVERT SCHEDULE

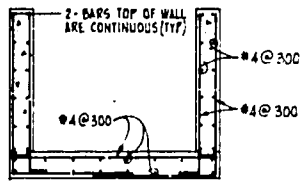
CULVERT NO	LOCATION **	DIAMETER	LENGTH	INLET ELEVATION	OUTLET ELEVATION	NO OF HOBLS	STOP LOGS/ GATED
1	KM 10+738	48" RCP	43.82	31.06	30.0	2	OPEN
2	KM 10+894	48" STEEL 48" RCP	16.35 26.12	31.06	30.0	2	GATED * (OPTIONAL)
3	KM 11+178	48" RCP	43.82	31.06	30.0	2	OPEN
4	KM 11+435	48" RCP	43.82	31.06	30.0	2	STOP LOGS
5	KM 11+647	48" RCP	43.82	31.06	30.0	2	OPEN
6	KM 12+010	48" RCP	43.82	31.06	30.0	2	STOP LOGS
7	KM 12+380	48" RCP	43.82	31.06	30.0	2	OPEN
8	KM 12+523	48" RCP	43.82	31.06	30.0	2	STOP LOGS
9	KM 12+750	48" RCP	43.82	31.06	30.0	2	OPEN
10	KM 13+105	48" STEEL 48" RCP	16.35 26.12	31.06	30.0	2	GATED * (OPTIONAL)
11	KM 13+383	48" RCP	43.82	31.06	30.0	2	OPEN

* IF THIS OPTION IS NOT REQUIRED, THIS CULVERT WILL BE CLOSED WITH STOPLOGS AND CONSTRUCTED WITHOUT STEEL PIPE, OR CONTROL TOWER, EXACTLY THE SAME AS THE OTHER CULVERTS.

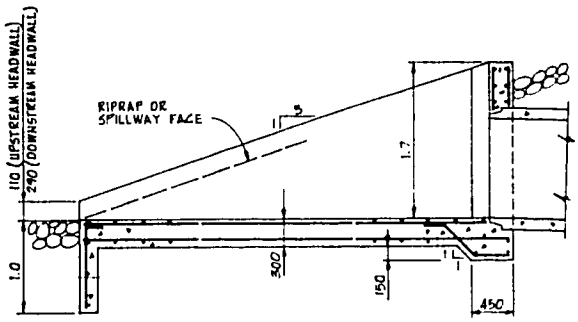
** CULVERT LOCATION MAY HAVE TO BE ADJUSTED SLIGHTLY TO CONFORM TO EXISTING CHANNELS.



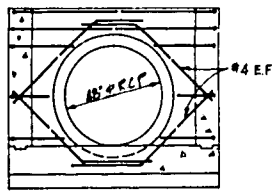
PLAN



SECTION $\frac{D-4}{4}$



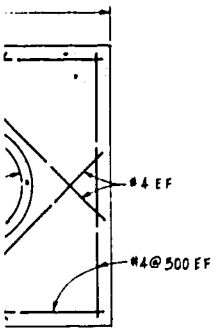
LONGITUDINAL SECTION
(SLOPE IN SAME DIRECTION AS CULVERT)



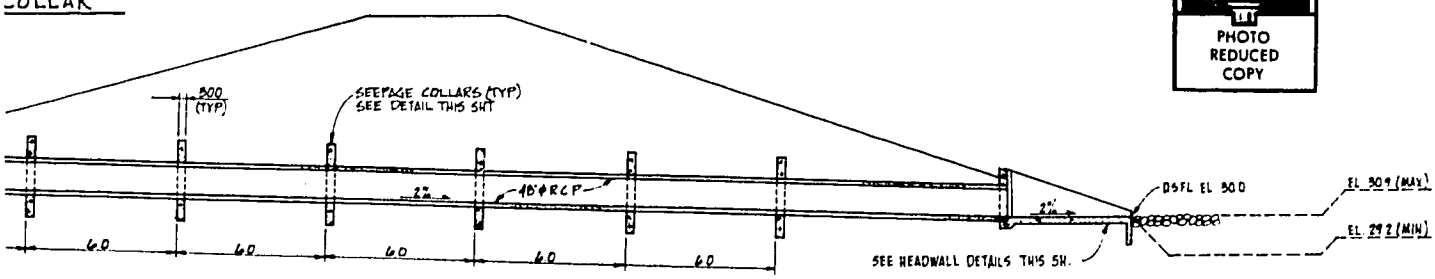
HEADWALL ELEVATION

HEADWALL DETAILS

SCALE: 1:30



COLLAR



) MID-POINT
AKEST TO
ABOVE.

TYPICAL LONGITUDINAL SECTION - 48"φ CULVERT

SCALE: 1:100

PHOTO REDUCED COPY

AT FULL SIZE

STANLEY CONSULTANTS

WADI AL KHABU
AQUIFER RECHARGE PROJECT

MISCELLANEOUS DETAILS
SHEET 1

SCALE AS NOTED

NO. 8069-4 REV 0

RECORD PRINT REVISION ____ IN FILES

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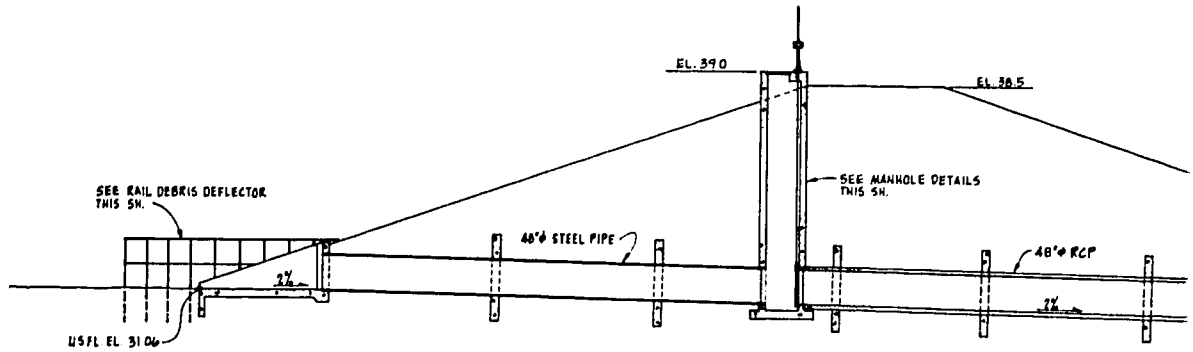
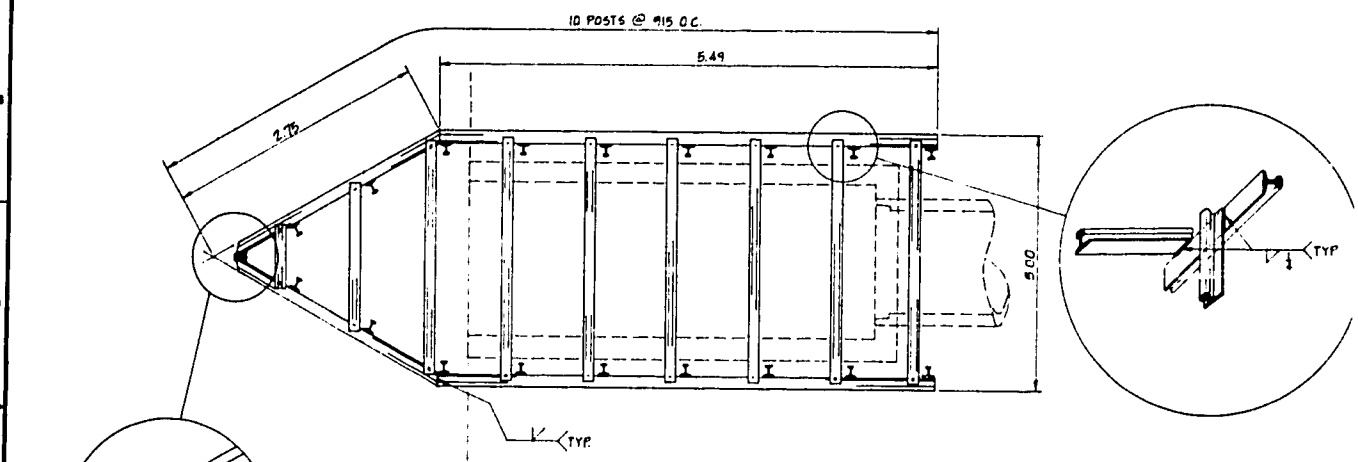
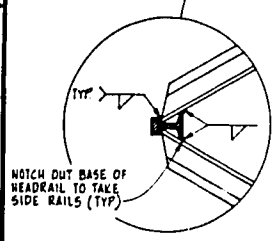
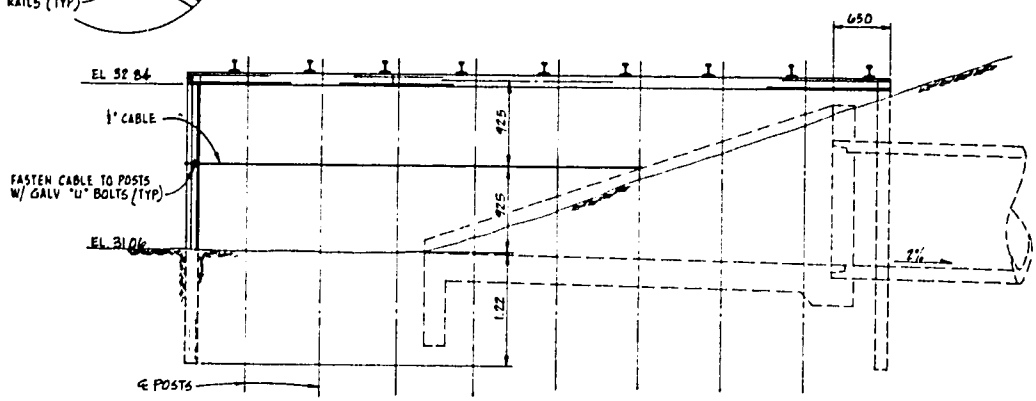


TYPICAL SECTION
ADDITIVE BID ITEM - SLUICE GATE CONTROL
SCALE: 1:100 (2-REQUIRED)



PLAN





ELEVATION

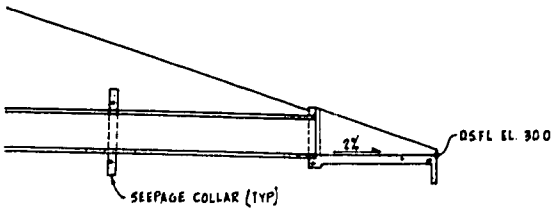
NOTES
1. DEFLECTOR TO BE CONSTRUCTED OF SALVAGED RAILROAD RAILS. (90# RAILS MIN.)
2. USE ⅜" FILLET WELDS EACH CONNECTION (UNO)
3. CENTER DEBRIS DEFLECTOR OVER HEADWALL
4. CONSTRUCT DEFLECTORS BEFORE PLACING RIPRAP

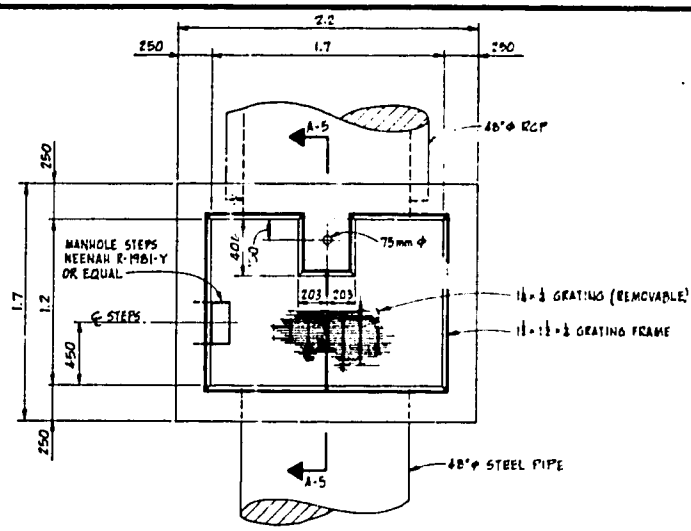
ADDITIVE BID ITEM
DEBRIS DEFLECTOR
NO SCALE

NO.	REVISIONS	DRAWN	APPROVED	APPROVED	DATE	NO.	REVISIONS	DRAWN	APPROVED	APPROVED	DATE	NO.	REVISIONS	DRAWN	APPROVED	APPROVED	DATE
1						2						3					

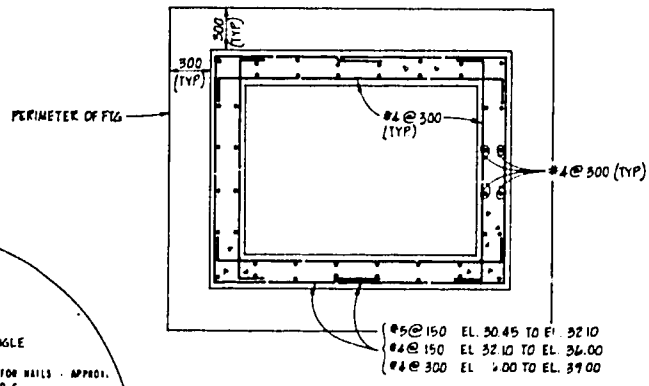
DESIGNED A. C. TAN
DRAWN J.D. PIERZ
CHECKED
APPROVED S.R. YAN
APPROVED
DATE DEC. 78

197

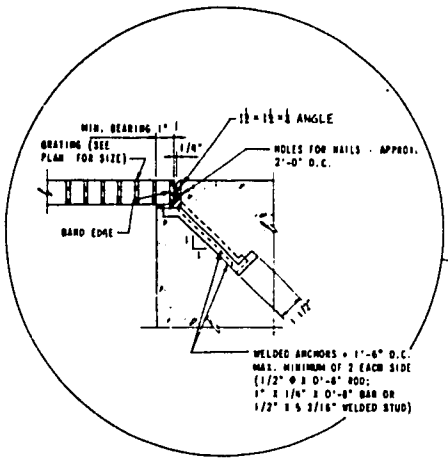


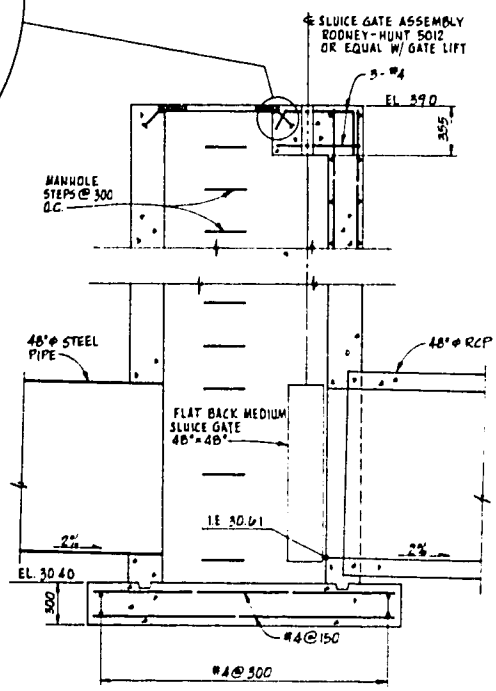


PLAN



PLAN SECTION - REINFORCING





SECTION $\frac{A-5}{5}$

PHOTO REDUCED COPY

TYPICAL CONTROL TOWER W/ SLUICE GATE
PLAN, SECTIONS AND DETAIL

SCALE: 1:20

AT FULL SIZE

0 1 2 3 IN

0 1 2 3 4 5 6 7 8 cm

STANLEY CONSULTANTS	WADI AL KHAWD AQUIFER RECHARGE PROJECT	MISCELLANEOUS DETAILS SHEET 2	SCALE AS NOTED; DWG 8069-5; REV 0

RECORD PRINT REVISION ____ TO FILES