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REPORT ON

SURVEY OF THE AWASH RIVER BASIN

VOLUME IV

UNITED NATIONS SPECIAL FUND
FOOD AND AGRICULTURE ORGANIZATION OF THE UNITED NATIONS

SURVEY OF THE AWASH RIVER BASIN

VOLUME IV

WATER STORAGE AND POWER DEVELOPMENT

FOOD AND AGRICULTURE ORGANIZATION OF THE UNITED NATIONS

SPECIAL FUND OF THE UNITED NATIONS

Rome 1965

PREFACE

The Report on the Survey of the Awash River Basin comprises the following volumes:-

Volume	I	General Report.
Volume	II	Soils and Agronomy.
Volume	III	Climatology and Hydrology.
Volume	IV	Water Storage and Power Development.
Volume	V	Irrigation and Water Planning.

These volumes are all issued on the authority of the Special Fund of the United Nations and the Food and Agriculture Organization.

Volume I is a comprehensive Report covering concisely all aspects of the Survey, and setting out the conclusions reached and the recommendations made. It is wholly prepared by FAO. It embraces and is based on the contents of the other volumes.

The other volumes were all drafted by the Sub-Contractors, S.O.G.R.E.A.H. of Grenoble, France, who carried out the main work of the Project. They have subsequently been edited by FAO. Each of these volumes in its more specialized field provides and analyses the relevant data, discusses the results, and sets out the conclusions to which they point. The discussions in one volume of course in various respects have reference to and depend on the discussions in other volumes.

The scope of Volume IV, as will be seen from the Table of Contents, covers the following:-

I	Water Control for Irrigation and the Production of Hydro-Electric Power - Potential Sites, and the scope of Studies Required.
II	The Potential Diversion of flows from the River Meki into Lake Galilea, with a preliminary estimate of costs.
III	The regulation of the flows of the River Kesem, with a preliminary estimate of costs.
IV	The regulation of the flows of the River Kebena, with a preliminary estimate of costs.
V	The regulation of flows of the River Awash at Tendaho, with a preliminary estimate of costs.

The information contained in Volume IV, and the conclusions reached, are used in the discussions contained in other volumes, notably Volume V.

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WATER STORAGE AND POWER DEVELOPMENT

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CHAPTER I. CONTROL OF WATER RESOURCES AND PRODUCTION OF HYDROELECTRIC POWER

I. WATER ECONOMY CONSIDERATIONS

The climate and hydrological investigations discussed in Volume III have established the basic patterns of water as an essential factor in the Awash River Basin. These patterns, though still partly governed by natural conditions, are strongly influenced by human agencies, especially in the Upper Basin. The trend towards the control of water resources is certainly expected to continue and, it is hoped, under a rational plan for the use of the water. Its economically efficient use depends on : (i) efficiently reducing losses ; (ii) advantageous timing of water use ; (iii) securing additional supplies during shortages ; and (iv) matching supplies to demand.

I - 1. Reducing Losses

The hydrological surveys show that the natural water losses are mainly due to evaporation from open water surfaces and marshland. Two forms of preventive action to be considered are :

(i) Limiting evaporation losses from artificial reservoirs by building the dam to such a height that the proportion of shallow water areas remains reasonable even at maximum storage level, and by so framing the operating rules that the duration of maximum storage periods is kept strictly to the minimum.

(ii) Reducing marshland and flood areas to a minimum by controlling the river flows (discussed in this Volume) and also by appropriate flood protection and drainage arrangements (discussed in Volume V).

'Artificial' losses in irrigation areas - expressed by 'irrigation efficiency' - will be reduced by the technical arrangements and operating standards discussed in Volume V.

I - 2. Selecting Advantageous Timing for Water Use

Water use areas - i.e., irrigation areas - will be selected and their boundaries defined according to the quality of land and the availability and proximity of water supplies as well as rural development prospects and facilities.

In particular, water consumption is expected to increase in the Upper Basin, where more supplies will be needed for towns, parks, private and market gardens and orchards, and for irrigation as a means of controlling farmland erosion. These aspects of water use in the Awash Basin are discussed in Volume V.

The need for an annual calendar of water use arises mainly because monthly requirements of irrigation water over the various parts of the basin are staggered in time, and so balance out to some extent. This is discussed at the end of Volume II. The 'calendar' must also allow for the highest possible guaranteed electricity production. The present volume relates this question to large artificial reservoir operation.

Attention has already been drawn to the possibility of diverting supplies from an adjacent catchment into the Awash Valley.

I - 3. Matching Supplies to Demand

The whole problem of hydraulic regulation by big dams hinges on this question. As the theme of this Volume, it is discussed under its two main aspects of :

(i) Satisfying the irrigation water requirements for agricultural development; and (ii) securing the maximum hydroelectric power production. The pattern is (i) a general review of existing plant and equipment in the basin; (ii) a determination of its effectiveness so as to establish new requirements; and (iii) a complete review of hydraulic regulation possibilities.

II. REVIEW OF PRESENT PLANT AND EQUIPMENT

This is limited to two exclusively power-production plants : Aba Samuel, a comparatively small plant which has existed for a long time; and the much bigger Koka plant now in operation, and its immediate downstream extensions under construction.

II - 1. Aba Samuel

The first rational use of the hydraulic potential of the Awash Basin followed the completion of the Aba Samuel dam on the river Akaki in June 1940. Its site, some 30 km from Adis Abeba, was eminently suitable for the capital's initial electricity requirements.

The dam, a masonry structure, has a height of 25 m and a volume of 40,000 m3. It supplies the power station in the Akaki gorge via a canal, a penstock head tank and a number of penstocks. The chief features of this plant, which is still in service, are :

River discharge	250 - 300 hm3 annually
Total storage capacity............	65 hm3
Net head..........................	95 m
Installed output..................	8,250 kVA
Guaranteed output.................	4,750 kV
Average productivity..............	23 GWh/year
Load factor.......................	55 %

Improvements under consideration for a fuller use of the hydrological and topographical features of the Akaki are : (i) raising the existing dam to increase its storage capacity to about 200 hm3; (ii) providing a power station at the foot of the dam; and (iii) using an additional 55 m natural fall immediately downstream from the present power plant.

By these combined improvements, the productivity of the Akaki plant could be increased by 60 GWh/year. They are not, however, priority items in the Ethiopian Electric Light and Power Authority's (EELPA) program, which shows a preference to concentrate on the Koka scheme.

II - 2. Koka

The commissioning of the Koka plant (Awash I) in 1960, after prolonged studies and design work, marked the first step in the control of the River Awash. The dam is a concrete gravity structure, except above the river bed, where it is of the buttress type with four gated spillways (sector gates). It measures 458 m along its crest and rises to a maximum height of 42 m. It features a bottom outlet and a water intake structure. The power plant is supplied via three 3.5 m dia. pen-stocks, each feeding a vertical-shaft Francis turbine. The Koka reservoir, known as 'Lake Gelilea', lies in a wide basin extending to the west beyond the Sidamo road.

Particulars for this first major Ethiopian hydroelectric scheme are listed for reference. Some of the data, especially hydrology, still await exact confirmation.

Catchment area size	11,000 km2
Mean river discharge	1,535 hm3/year
Min. and max. discharge recorded between 1943 and 1960..............	1.5 m3/s and 630 m3/s
Total storage.........................	1,840 hm3
Available storage.....................	1,660 hm3 (between 100.30 m and 110.30 m)
Reservoir surface area at maximum storage..........................	236 km2
Mean specific evaporation rate at Koka	1,600 mm/year
Mean possible turbine flow...........	42 m3/s
Nominal flood discharge..............	1,000 m3/s
Freeboard to absorb floods...........	2.5 m above max. 110.30 m level
Geometric head range.................	between 32 m and 42 m
Installed output.....................	54,000 kW
Guaranteed output....................	23,000 kW
Mean productivity....................	110 GWh/year (line losses deducted)
Load factor..........................	55 %

The power produced is conveyed to Adis Abeba (81 km) and Dire Dewa (337 km) by two 132 kV transmission lines. The overall cost of the schemes amounted to E$ 35,500,000 :

For the dam and its equipment	E$ 12,500,000
For the power station and its supply system	E$ 12,500,000
For the power transmission line	E$ 10,500,000

II - 3. Schemes in Progress Downstream from Koka

Flows discharging from the Koka dam are fully regulated for power production and are thus available for further use at the natural falls downstream where the Awash enters the Middle Valley. For this reason EELPA is now putting into effect two projects :

(i) Awash II, comprising a small dam with a water intake (reservoir capacity 6 hm3) immediately downstream from Bobe bridge (on the road from Nazret to Asela), a diversion channel for overflows from the small reservoir to the thalweg collecting drainage from the Wenji plantation, and a 1,870 m long tunnel to be driven in the right bank to Awash II power station.

(ii) Awash III, comprising a dam and a reservoir collecting the discharge from Awash II, and a 1,280 m long tunnel to Awash III power station.

Overall characteristics for these double power schemes are :

Total net head	119 m
Installed output.....................	64,000 kW
Productivity.........................	364 GWh/year

Based on estimates made in 1964, the combined cost of the Awash II and Awash III schemes is expected to be E$ 60,000,000. Awash II is expected to be ready in 1966, and Awash III in 1967. A third site of about the same size is to be developed between Awash III and the warm springs at Sodere at a later date (Awash IV).

III. NEW HYDRAULIC REGULATION REQUIREMENTS

The Koka scheme and those immediately downstream already favourably affect power production and the control of flows downstream.

III - 1. Power Production

Total electricity consumed in Ethiopia in 1964 amounted to roughly 125 GWh, and consumption from 1960 to 1964 has increased at an average rate of about 15 GWh/year. This is likely to increase in the next few years.

The Koka plant has amply catered for demand not covered by other Ethiopian power plants. Awash II, III and IV should be more than sufficient for the anticipated increase in demand over the next ten years, at least in their area of influence, which is basically the developing-economy region from the capital to Nazret.

III - 2. Irrigation and Flood Protection

Present knowledge of the natural flows in the Awash before the filling of the Koka reservoir is based on data measured from July 1953 onwards at the old level gauge on the road bridge by Awash station in the gorge. The practically linear correlation established between these data and those provided by the new Awash station gauge installed at a point 3 km upstream from the bridge in 1962 (which has been in regular use ever since) shows that :

(i) Before 1960, the lowest water levels occurred at flows of about 200 l/s, so that the river used to dry up completely downstream from the Middle Valley.

(ii) Since the Koka dam has been in regular service (1962), the lowest rates of flow recorded at Awash station have practically never fallen below 30 m3/s.

The Koka dam makes it possible to use run-of-the-river Awash flows to provide regular irrigation supplies for the Middle Valley plains. Without Koka, it would only have been possible to ensure regular irrigation for a very small amount of land, or, alternatively, to practice water-spreading during high-water periods, as is still being done in the Asayita delta.

Flood peaks formerly reaching about 700 m3/s at Awash Station are now attenuated. Maximum flows of about 300 m3/s are now recorded only when due to such major tributaries as the Arba Dima.

Certain riverside areas in the Middle Valley, and even the Wenji Basin before dyking, used to be regularly submerged before 1960, but are now safe from flooding. It is generally assumed that the several hundred meter wide band of forest along both banks of the Awash has suffered from this reduction in flooding frequency. Trees are dying in large numbers along its edges.

The Koka dam has helped to raise low-water levels in the Lower Plains, but the lower course of the Awash is still subject to pronounced flooding due to its left-bank tributaries downstream from Gewani marsh. A plant like Koka cannot suffice by itself to solve all the development problems in the Awash Basin. New requirements will have to be met, and further means of regulation found.

III - 3. Principles Involved in Further Control

Power production requires new plants, in order to (i) meet the additional demand expected to arise in the present consumption area after ten years; (i.e., as overall power consumption in Ethiopia approaches the 500 GWh/year mark); and (ii) supply regions of the Basin newly opened up for economic development, especially by developing their agricultural potential.

The first point can be met by making Koka more efficient through strict evaporation and seepage loss and silt control measures, and through diverting flows from an adjacent catchment area to improve average reservoir filling conditions; and by using the hydroelectric potential in the vicinity, especially between Awash IV and the point of entry of the river into the Middle Valley (Melka Sedi). The second point involves a review of all the possibilities for providing new hydroelectric plant suitable more specifically to supply the Middle Valley or the Lower Plains, and preferably for combining the plant with such regulation dams as are already necessary for irrigation and flood protection purposes.

The irregularity of daily and weekly electricity demand shows up in the turbine flows at Koka and the plants immediately downstream. The quantities of water returned to the Awash by these plants remain the same from month to month throughout the year, whereas irrigation demand depends on the climatic cycle. To render Koka more effective for agricultural development, a dam will be necessary to compensate for variations in these flows during the day and from one day to the next, and to adapt the monthly flows to future irrigation demand.

The regulating effect of the Koka dam diminishes downstream, as unregulated flows come in from tributaries. Additional dams will be needed to regulate these and ensure the best possible use of water supplies for irrigation. These additional dams may be located on tributaries, preferably those carrying the largest flows, or on the Awash itself, provided that this does not involve the sacrifice of too much valuable land.

Investigation subjects for flow regulation and power production schemes can be summed up as : (i) increasing the effectiveness of the Koka reservoir; (ii) developing the hydroelectric potential of the Awash immediately downstream from power schemes now in progress; and (iii) creating reserve storage in the middle and lower Awash and its major tributaries.

IV. INCREASING THE EFFECTIVENESS OF THE KOKA RESERVOIR

IV - 1. Prevention of Evaporation Losses

Reservoir levels (see Fig. 1) show that the initial filling of the reservoir was completed in October 1961, and that the annual patterns of variation in reservoir levels were fairly uniform during the next three years.

Though full use of the turbines has not so far been made, it is reasonable to consider that the average given by these three cycles represents the 'normal'

KOKA RESERVOIR OPERATING CHARACTERISTICS

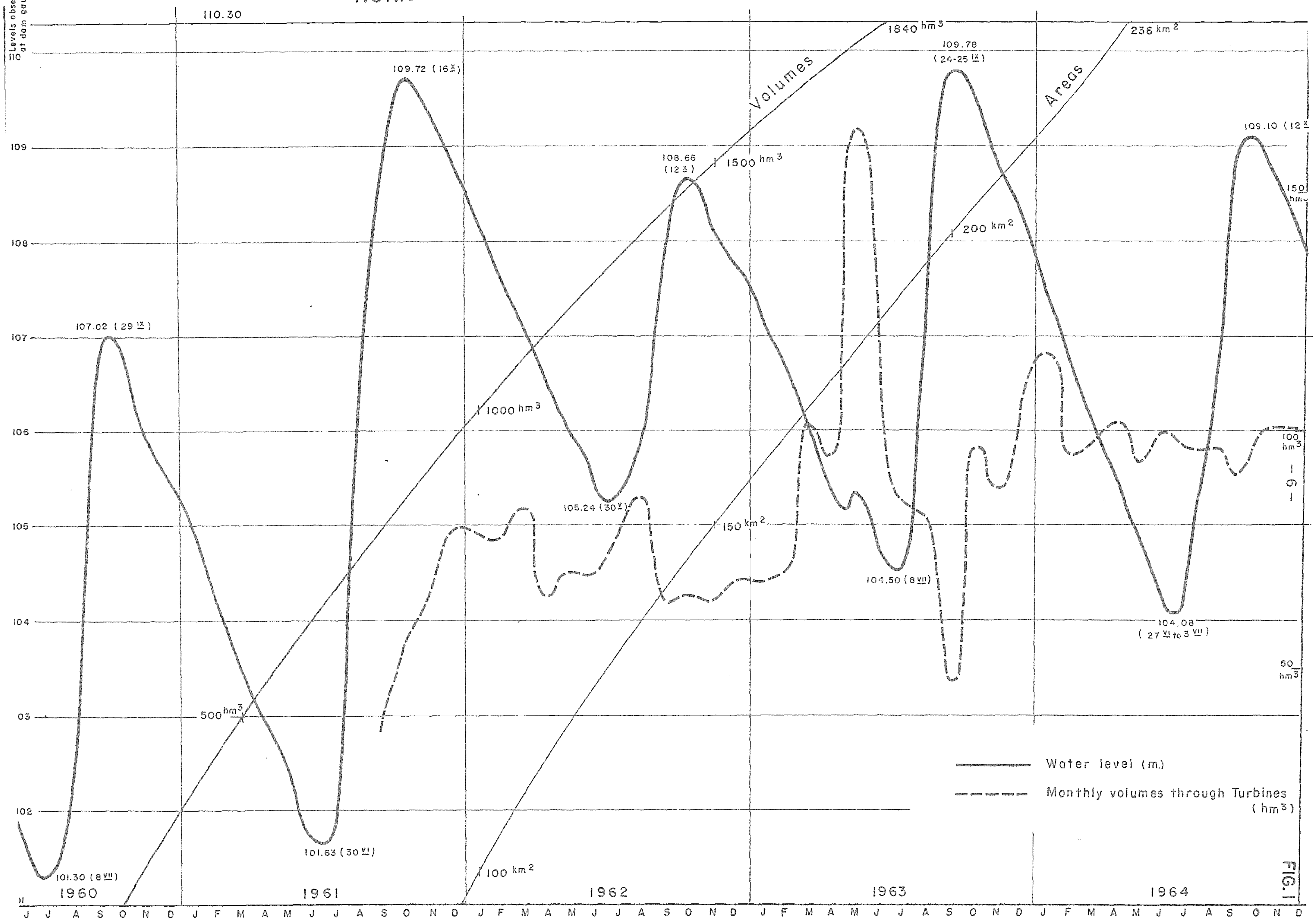


FIG. I

reservoir variation. This is given by the top line of figures in the table below; the second line gives the corresponding reservoir surface areas (taken from EELPA diagrams); the third one gives normal monthly evaporation from a large open expanse of water (from climate data observed at Koka), and the fourth gives normal monthly evaporation losses.

	O	N	D	J	F	M	A	M	J	J	A	S	Total
Dam gauge level reading	109.30	108.75	108.25	107.65	107.00	106.40	105.80	105.40	104.80	104.85	106.45	108.80	-
Water surface area (km^2)	221	212	203	193	182	172	162	156	147	148	173	213	-
Evaporation (mm)	158	126	130	109	171	194	156	175	149	103	128	152	1,751
Evaporation loss (hm^3)	34.9	26.7	26.4	21.1	31.1	33.4	25.3	27.3	21.9	15.0	22.2	32.4	317.7

The loss of an annual average of some 320 hm^3 of water by evaporation is considerable. It is equivalent to flow at a continuous rate of 10 m^3/s, or about 30 GWh of lost electricity production.

Unfortunately, no direct preventive measures against evaporation can yet be seriously considered. The development of suitable preventive methods is still at the *experimental stage* (*e.g., non-miscible, non-volatile surface films immune to wind*). The only possible action might be to reduce the total evaporation area of the reservoir (i.e., to increase mean reservoir depth) with the reservoir operating conditions remaining the same as before. As evaporation is certainly more active in the shallower fringe areas, which are less apt to receive fresh water supplies and, therefore, more liable to warm up, they could be profitably eliminated from the reservoir. This idea lines up with the one mentioned in the erosion study; i.e., sacrificing the shallowest marshy part of the reservoir upstream from the Sidamo road and reclaiming it by warping with sediment-laden water from the Awash and Mojo rivers. Before undertaking a systematic warping program, much could be gained by isolating these areas with comparatively impervious dykes, so as to stop any further regular inflows.

IV - 2. Prevention of Seepage Losses

Appreciable water losses observed at the initial filling of the Koka reservoir showed that the basin is not completely impervious. Some losses are obvious downstream from the dam in natural 'funnels' at the foot of the right-bank wing dyke.

In a depression along the right bank of the river about 2 km below the dam, there was a very hot spring (Bulbula) before the dam was built. Not only has its discharge increased since the filling of the reservoir and its temperature dropped, but several new springs have also appeared nearby.

As the areas in which these losses of water occur are scattered, direct measurements of the total leakage rate is a delicate matter. An indirect assessment is possible, by comparing the total inflows into the reservoir with its total outflows (turbine flow + evaporation loss). This is done in Volume III, and shows an annual leakage loss of about 400 hm3. Its exact value apparently depends on the mean water level in the reservoir. The mean leakage rate thus amounts to 13 m3/s, less than half of which reappears between Koka and Wenji. These estimates show how serious the problem is. The losses alone represent a loss in potential energy production of around 35 GWh/year for Awash I, and twice this amount for Awash II + Awash III.

Leakage observed at the foot of the wing dyke may be traced to a fault alongside the full length of the eastern reservoir edge. It may be due to erosion of more or less pulverulent volcanic ash, directly above the fault, of the type also observed in the escarpment rising above this particular area. Closer study of this problem may suggest a grout screen of some appropriate material as a possible remedy. This might not be a simple operation, as there do not appear to be any suitable impervious formations to which to join the bottom and sides of the screen. The springs further downstream are much more difficult to explain without a systematic survey.

The bottom of Lake Gelilea is a very thick clay deposit of lacustrine origin overlying a thick tuff blanket. A pervious formation, generally consisting of fissured lava, is only encountered below 50 to 60 m depth. Since the deep hot springs formerly in the basin must have broken through the impervious top formation, water from the reservoir may be finding its way down these passages, mixing with the warmer water at depth and finally reemerging with it in the springs' area. Tests with dye might show whether the passages in the bottom of the reservoir really communicate with the springs, or whether the increased discharge of the latter is due to some other cause. Temperature measurements and chemical analysis would also produce interesting information.

Other investigations to be carried concurrently with the above would be : (i) further surface inspection and borehole surveys to complete present knowledge of the geological structure of the springs area; (ii) the study of water levels observed in the boreholes, and the use of piezometers for determining the local ground water flow conditions; and (iii) further flow gauging operations to establish more reliable spring discharge data.

IV - 3. Silt Control

The origin of the solid materials depositing in Lake Gelilea are mostly the Awash itself (a systematic sediment load measurement program has been in progress at Melka Gorge since September 1962) and the river Mojo, which discharges directly into the reservoir. Measurements at the railway bridge gauging station have been supplying sediment load data for the Mojo since August 1962.

Despite its size (2,300 km2), the complementary catchment area above Lake Gelilea only features steep slopes in a small part of its area and is only drained by small streams. Its sediment contribution can be neglected for initial analysis purposes.

Full data for the Melka Gorge and Mojo gauging stations are covered in the hydrological surveys discussed in Volume III. These suggest that the total quantity of sediment brought into the reservoir during a normal year must be less than 10,000,000 tons. Thus the problem of the useful working life of the Koka reservoir is not nearly so serious as had been anticipated. Even if the full quantity of sediment were to settle out permanently in the reservoir (under present operating conditions, the water seems to discharge clear for most of the time) the average annual storage loss would only amount to about 6 hm3; i.e., 0.33 % of the total storage capacity. At this rate, the reservoir would take about 300 years to silt up completely. In fact, however, the sedimentation process is probably such that the finer materials settle out in horizontal layers in the deepest part of the reservoir, near the dam. The coarser materials presumably settle out shortly after entering the reservoir and, due to the shallow bed slope in the latter (about 50 cm/km), their deposit must only build up very slowly downstream. Though no exceptional inflows have been experienced (at present, the difficulty is to ensure normal filling of the reservoir) it may be necessary to discharge some of the water downstream without passing it through the turbines. This should be done via the bottom outlet so as also to flush out some of the sediment at the foot of the dam.

Although erosion control measures are difficult and take a long time to become effective, it is hoped that a general erosion control program along the lines discussed in Volume II will begin to produce results after a few decades and help to reduce the sediment loads carried by the rivers.

The already mentioned idea of deliberately accelerated warping of the marshland upstream from the Sidamo road by total diversion of the river Mojo is also a means of direct sediment control. The Mojo now spreads out over part of this area through its natural defluents, but dykes and some earth-moving work will be needed to divert all its floods from the deeper parts of the reservoir. The most urgent requirement, however, is a study of present sedimentation conditions, with the assistance of EELPA. It should be fairly easy to organise a program of surveys by depth-sounder, at say regular intervals of five years, so as to establish up-to-date curves of reservoir areas and capacities against water levels, and also to assess the progress of sediment deposition, i.e. to compare volumes of sediment inflows and deposits, and to locate these deposits. The implementation of this program depends on reliable preparatory topographical surveys, and especially on the setting up of a chain of geodetic survey bench marks around the lake.

IV - 4. Improvement of Average Filling Conditions

Lake Gelilea has not been filling up as completely as had been anticipated. The following points emerge from the water level variation plots produced since the dam first went into service :

(i) The highest water level during the year (generally from late September to late October) and the lowest water level (late June to early July) increased appreciably between 1960 and 1962.

(ii) The reservoir level has fluctuated around an average level of 107 m since 1962.

(iii) The dam spillway has not had to operate once in four years of service (spilling level 110.30 m), and the top tenth of the storage capacity - i.e., about 130 hm3 - has never been used.

(iv) During the last annual cycle, the mean monthly volume of water through the turbines never reached 100 hm3. This is equivalent to an average flow of 38 m3/s instead of the 42 m3/s provided for in the original design.

(v) Effective annual production at Koka has never exceeded 100 GWh, even during the last recorded cycle (November 1963 - October 1964), whereas a production figure of 110 GWh was originally expected (after deducting transmission losses).

This discrepancy is due to an under-estimate of the evaporation (which amounts to over 300 hm3/year, instead of the 200 hm3/year allowed for) and to substantial water losses (400 hm3/year) beyond the control of the reservoir.

The idea of improving the average filling conditions for Lake Gelilea resulted from the following fact, noticed a long time ago, relating to the special topographical configuration of the region along the southern edge of the Awash Basin :

The ground rises gradually from the Sidamo road bridge across Lake Gelilea to a vast plateau near Alemtena, which extends far towards the south and is run through by a chain of partly interconnected lakes (Ziway, Hora Abiyata, Langano) collecting all the runoff from an independent catchment area. Lake Ziway is the biggest of the four (500 km2) and at its altitude of roughly 50 m above Lake Gelilea also the highest. Its total catchment area extends over 7,000 km2, receives comparatively large quantities of rainfall and produces high flows (mean total annual flow probably between 500 and 1,000 hm3). In view of the large supplies available from this area, its hydraulic command of the Awash, and the absence of natural obstacles between them, it seemed worthwhile examining the possibility of diverting some of the water, the area now supplies to Lake Ziway to the Awash - more precisely to Lake Gelilea. A full technical examination confirms the feasibility of diverting supplies from the river Meki, one of its two main tributaries. This study is discussed in Chapter II.

IV - 5. Compensation for Turbine Flows

Flows passing through the turbines at Koka will also drive the turbines at other power stations downstream on the Awash, probably according to similar programs as at Koka because these plants will be of the run-of-the-river type. As a result, the hydraulic regime of the river at its entry into the Middle Valley - i.e., after all the natural falls likely to be harnessed for power production between Koka and Awash station - will still be modulated by the electric power demand. This prevents the full use of supplies in the Awash for run-of-the-river irrigation, but an attempt can be made to damp it by a compensation dam downstream from the falls. It should, however, be as far up-river as possible from the irrigation areas in the Middle Valley; i.e., in the Awash station sector.

Three different types of modulation have to be considered.

First type is daily modulation due to hour-by-hour demand variations during the day. A mean turbine flow variation curve applicable for all working days has been plotted from the Koka reservoir operating records (see Fig. 2), which shows :

(i) Two peaks during the day, one 25 % in excess of average discharge at about 10.00 hrs, and the other, 50 % in excess of average discharge between 19.00 and 20.00 hrs.

(ii) Two minimum consumption points, one at 04.00 hrs, and the other around 14.00 hrs.

The graph also shows irrigation demand for an assumed peak month (irrigation during 18 hours out of the 24), when all the water discharging from the turbines is used for irrigation.

It is found by planimeter measurement of this graph that the compensation capacity is equivalent to 5 1/2 hours at the hourly turbine discharge averaged over the day. Except for the month of May 1963 when the demand was exceptional, the quantity of water through the turbines seldom reached 4.5 hm3 daily. The capacity required for daily compensation works out approximately at :

$$\frac{5.5 \times 4.5}{24} = 1 \text{ hm3}$$

The graph, however, shows that a discharge fluctuation damps out progressively as the wave proceeds down-river. Moreover, irrigation demand at a compensation site upstream from the Middle Valley will tend to spread out along the time scale (compared to the irrigation schedule on the graph), because the times taken by the supply flows to reach the various irrigation areas down-river will be different. For both these reasons, the daily compensation reservoir capacity seems to allow a wide safety margin.

Second type is a weekly modulation, which shows a drop in consumption on Sundays, when the turbine flow is only 70 % of the average daily flow over the previous six days.

Thus, to ensure flows evenly distributed over the week in the Awash downstream from Koka, it would be necessary to provide for an additional capacity equivalent to about 4 % of the weekly flow through the turbines. This has so far amounted to, at most, about 35 - 40 hm3 (except in May 1963). A capacity of 1.5 hm3 is thus sufficient to ensure compensation of turbine flows during the week.

Third type is monthly modulation, which does not occur according to any evident law, as can be seen from the monthly turbine flows plotted together with Koka reservoir water level variations in Fig. 2. Its casualness is similar to that of the inflows downstream from Koka. In assessing methods of compensation from one month to those following it, therefore, it is better to consider not the flows passing through the turbines at Koka, but those in the Awash in the area in which a compensation dam is desirable; i.e. near Awash Station. The flow data "A" to consider, therefore, are those recorded at the Awash gorge gauging station, from September 1962 to October 1964.

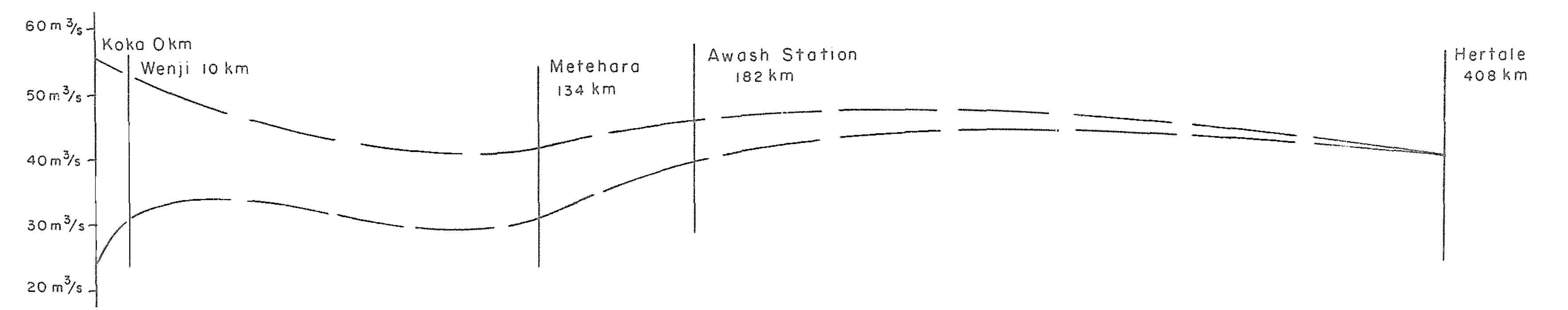
The recorded flow values "A" indicate a critical period from December 1962 to March 1963, during which the flow was lowest. Yet, for forecasting the probable future flow of the river, present values "A" should be further reduced to account for an increase limited to 50 hm3/year in the water consumption in the upper reaches of the basin, which is liable to follow the rapid economic development; and also to account for the net consumption of water for irrigation purposes in the Nura-Era and Metehara region, where the total development area may be tentatively estimated at about 15,000 ha.

The estimated amounts of cumulative water consumption "B" to be deducted from the currently measured flow, are shown on line 2 of the table :

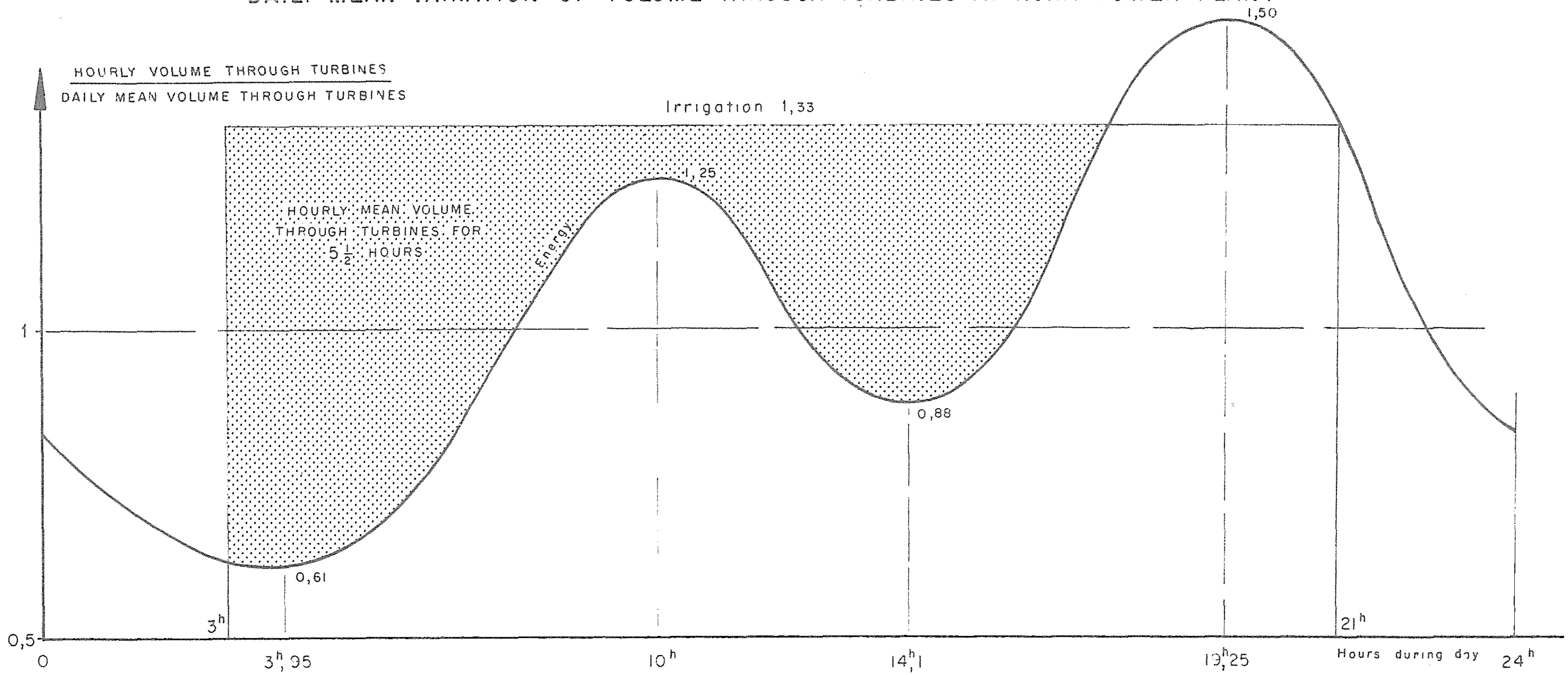
	N	D (1962)	J (1963)	F	M
A (hm3)	80	59.5	68	63	104
B (hm3)	15.5	25	23.5	23	18.5
A' (hm3)	64.5	34.5	44.5	40	85.5
0.8 bd (m3/ha)	765	1415	1315	1270	955
$\frac{A'}{0.8\text{ bd}}$ (ha)	84 000	24 500	34 000	31 500	89 500

FIG. 2

MEAN EXTREME DAILY DISCHARGE VARIATION OVER ENTIRE LENGTH OF AWASH FROM KOKA TO HERTALE



DAILY MEAN VARIATION OF VOLUME THROUGH TURBINES AT KOKA POWER PLANT



Line 3 thus gives the probable flow of the Awash A' = A-B available in the future.

By comparing the values A' with that of per hectare irrigation requirements in the Middle Valley, and assuming that about 1/5 of the applied water will be drained back to the Awash (0.8bd), it becomes possible to estimate the gross maximum area which might have been irrigated in the Middle Valley by simply diverting all the water available in the river during the critical period. These figures are given on line 5.

The effective storage capacities required to compensate flows from one month to another during the critical period are listed below in relation to the areas which may be actually irrigated by each given capacity.

Area (ha)	Storage (hm3)
24,500	0
25,000	0.5
30,000	8
35,000	21.5
40,000	41.5
45,000	59.5
50,000	82.5
55,000	103.5

These results do not appear to have any general bearing at first sight. They merely show the impossibility of supplying more than 24,500 ha of land in the Middle Valley downstream from Awash Station with water during the critical period. Yet the soil survey put the maximum overall amount of irrigable land in the Middle Valley below Awash Station at 75,000 ha. Even disregarding the area commanded by the Kesem reservoir, this still leaves 50,000 to 55,000 ha of land irrigable by the Awash. A compensation storage capacity of roughly 100 hm3 would thus have been necessary to ensure adequate supplies during the critical period for all the land irrigable economically by the Awash. Nevertheless the following points should be considered :

(i) An available storage volume of 100 hm3 would have required a dam probably costing as much as the Kesem dam (350 hm3) and far less effective for developing irrigation in the Middle Valley.

(ii) The inflow shortages are unlikely to recur while Awash I remains the only hydroelectric power plant in service on the Awash, as operation of the Koka plant will in future become increasingly regular in meeting growing industrial demand. Output from Awash I was already fairly continuous from July 1963 to October 1964, so much so that no compensation would have been required throughout that period.

(iii) Although supplies contributed by the Middle Valley tributaries (Arba, Kesem, Kebena) were neglected in the calculations, they are not entirely negligible during the December to March low-water period.

(iv) Full development of the Middle Valley also implies irrigation for the Kesem-Kebena plain, which depends on the construction of the Kesem dam. Regulation by this dam will considerably affect flows in the Awash.

Although these points are reasons for deciding against a compensation dam, it should be realized that a particularly severe succession of sub-normal inflows may cause operation of the Koka reservoir to be restricted and that overall turbine flows may be reduced as Awash II, and later Awash III, go into service, at least while their additional potential production is not absorbed.

It appears prudent, therefore, to keep to the principle of a compensation dam with a storage capacity of about 50,000,000 m3. Its full benefit would not be felt before developing an appreciable amount of land in the Middle Valley; i.e. 20,000 to 30,000 ha.

It has been pointed out that a dam compensating for turbines flows from power stations depending on the Koka dam should be sited downstream from the natural falls, though also well upstream from the large Middle Valley plains. The roughly 50 km along Awash gorge running from the last major fall (Gotu) to the ford at Melka Sedi at the entrance to the plains is a geographically suitable site. Typical structural features of the gorge are :

(i) Tall cliffs to either side of the river, becoming higher upstream. From a height of about 20 m near Melka Sedi, they rise to about 80 m at the railway bridge, and 100 m at the Awash Station. Below the confluence with the Arba Dima, the gorge is nearly 200 m deep. These features are eminently suitable for a reservoir of substantial capacity.

(ii) The width of the gorge : from only about 100 - 150 m wide with near-vertical sides over its initial 25 km downstream, the gorge then suddenly opens out above the railway bridge to a width never less than 500 m and frequently exceeding 1,000 m. This points to a site near the railway bridge, where the gorge is about 80 m deep and roughly 150 m wide across its mouth.

The only topographical surveys carried out so far are a few levelling runs. The results are shown in Fig. 3. Though fragmentary, these data enabled reservoir curves to be plotted for a dam across the Awash at the railway bridge. They show that a dam about 40 m high should store about 50 hm3 of water. This dam site is mentioned only tentatively. Its technical feasibility would depend on detailed topographical surveys and hydraulic, geological and civil engineering investigations well outside the restricted scope of the present project. If necessary, it could be included in a second investigation phase.

In July 1964, a set of four boreholes 1/ was sunk in to obtain additional information on the possibility of building a road bridge across the Awash immediately above the railway bridge. At the 810 m level 40 m approximately below the railway bridge, all four boreholes ran into a bed of basalt in sound condition over most of its depth. One borehole penetrated this bed to a depth of 30 m.

If a comparatively high dam should be found technically too difficult or too expensive to build, the only remedy would be to lay down a form of compensation for operating the chain of power stations dependent on the Koka dam. They could be made to discharge volumes which ensured that the flow in the Awash at its entry into the Middle Valley never fell below 70 to 80 hm3/month (i.e., 27 - 30 hm3/s continuous flow). For example, if it had been necessary to irrigate all the land in the Middle Valley from November 1961 to October 1964, the power loss at Koka would have amounted to about 15 GWh, whereas the total production for the three years was about 275 GWh.

1/ By de Leuw, Cather and Company responsible for designing the Awash-Tendaho road.

An important question is whether it might not subsequently prove difficult for the power plant to accept program changes requested by third parties.

V. DEVELOPING THE HYDROELECTRIC POTENTIAL OF THE AWASH DOWNSTREAM

It is advisable to harness all the natural falls downstream from the Koka dam so as to make the most of its regulating action.

Downstream from the Awash II, III and IV projects - i.e., below the hot springs at Sodere - the Awash still features falls and rapids all the way to Melka Sedi, where it finally emerges from the Awash Station gorge. The bed level falls from 1,345 m to 748 m along this 170 km stretch of the river; i.e., an overall level difference of 600 m and an average slope of 3.5 m/km. The average slope of the Awash from Melka Sedi to Lake Abe is little over 50 cm/km, despite the existence of several irregular river sections.

In view of the gross head it offers and the proximity of the consumption areas, the section between Sodere and Melka Sedi is the one most worth developing in the near future. Actually only a fraction of this gross 600 m head can be used economically, but this is not surprising. An initial reconnaissance survey of the hydroelectric potential of this part of the Awash was carried out by a Yugoslav mission early in 1958. Its data have since been amplified by reconnaissance surveys relating to the Project. Among the main findings were a number of very fine falls along this part of the river. Going downriver from Sodere, these are :

(i) Era fall, about 12 m high, 23 km from Sodere just before Nura bridge;

(ii) Melka Bokara fall, about 10 m high, 47 km from Sodere between Nura bridge and Metehara;

(iii) most important of all, a 20 m high fall at Gotu 115 km from Sodere, marking the outlet from the Metehara plain and the beginning of Awash Station gorge.

The available head can be quite easily increased by a low dam raising the water level by a few meters or so and resulting in an effective and inexpensive project of the type of Awash II to IV. Thus it should be possible to bring the total head available for use at the three falls to about 50 m. (To raise the level at Gotu too much, however, might interfere with the natural drainage of the Metehara plain.)

There are several series of rapids between these falls. They could also be developed for power production, but less profitably because longer supply tunnels or higher dams would be needed to give adequate operating heads. These rapids are :

(i) in a defile extending for 20 km to either side of Nura Bridge, which, without Era fall, adds up to a gross fall of about 55 m ;

(ii) in a 10 km long defile leading to the Metehara plain, in which the level difference probably adds up to as much as about 100 m.

(iii) in the Awash Station gorge, but their hydro-power production possibilities are limited by the compensation dam needed for irrigation. The most it might harness is a head of about 10 m. at the foot of the dam.

Roughly, therefore, it should be possible to harness an overall net head of about 50 m by short-circuiting some rapids in the Awash.

In the absence of large-scale topographical survey data, this is about as far as one can go in defining the hydroelectric potential of this part of the Awash.

A preliminary idea of the productivity of these schemes is summed up in the table below, based on the assumed net heads and on an average flow of 42 m3/s; i.e., the same as at Koka, once it is working to full capacity. Comparison between conditions in the Awash below Koka and at Metehara shows that the tributaries (Geleta and Orenso) only affect the Awash flows during the wet season. They contribute no useful supplies for power production.

Power site	Average head (m)	Productivity (GWh)
Koka plant (Awash I) - in service	36	110
Awash II and III - under construction	119	360
Awash IV - at design stage	58	180
Three downstream sites - first priority	50	150
Rapids - second priority	50	150
T O T A L	313	950

VI. OTHER DAM-BUILDING POSSIBILITIES IN THE AWASH BASIN

Prospection for additional dam sites in the Awash Basin is primarily needed to meet irrigation requirements in the Middle Valley and Lower Plains. It has also the secondary purpose of enabling a certain amount of electricity to be produced for newly developed regions. This prospection will not be concerned with the Upper Basin of the Awash, but will consider the river itself (downstream from Awash Station) and those tributaries supplying the greatest quantities of water.

Attention has been drawn to certain sites in earlier surveys, especially by the Yugoslav mission in 1958; others were found during the aerial surveys for the Project. Field surveys were carried out at several of these sites, and at the two considered the most promising-Kesem and Tendaho - systematic topographical and geological surveys were undertaken and preliminary civil engineering designs prepared.

VII. DAM SITES ON AWASH TRIBUTARIES

The main tributaries are now reviewed, going down the Awash after its entry into the Middle Valley. All sites already developed, in course of development, investigated or simply considered are shown in Fig. 4.

VII - 1. Arba Dima

This is the biggest right bank tributary of the Awash. Gaugings at a point about 10 km above its confluence with the Awash revealed a total volume of about 336 hm3 conveyed during the annual cycle from November 1963 to October 1964. Its regime is very uneven, with low-water flows likely to fall as low as 100 l/s. (February/ March 1964). This tributary does not lend itself to run-of-the-river use of its supplies. Storage possibilities seem very limited. The lower part runs through a narrow canyon, but the upper valleys immediately upstream slope steeply. No appreciable control is possible over the flows but the dam proposed for the Awash Station gorge will partly iron out flow irregularities caused in the Awash by this tributary's flood peaks.

VII - 2. Kesem

This is the most prolific tributary of all. Observations at Awora Melka gauging station between August 1962 and October 1964 showed an average annual volume conveyed of 550 hm3, but its flows are even more irregular than those of the Arba Dima, with low-water flows of less than 500 l/s, and flood flows exceeding 500 m3/s. Attention was given right from the start to regulating this river, and a group of sites was identified about 10 km up-river from Awora Melka, chiefly because they seemed suitable for erecting a big dam giving a storage capacity of 350 hm3 in this section, which is sufficient to regulate the river so as to meet the needs of regular irrigation for almost 20,000 ha of land in the Kesem and Kebena plains. This dam would also produce an appreciable amount of electricity. Because of the site's scope and promise, a set of special studies were carried out and a preliminary civil engineering design prepared. They are discussed in Chapter III.

VII - 3. Kebena

The Kebena with its catchment area immediately to the north of the Kesem's cannot supply as much water (yearly average of 350 hm3 from November 1962 to October 1964), but it features a dam site likely to give a storage capacity of over 50 hm3 at a point 20 km upstream from the river's confluence with the Awash. Use of the flows in the Kebena is not essential for the developing of the plain lower down, where the irrigation supplies from the Kesem alone would suffice. Any plan to use the Kebena to supply other irrigation areas (e.g., Bolhamo) would create major difficulties over channelling supplies to them. Thus it was considered pointless to investigate this site as thoroughly as the Kesem site. Studies and their findings on it are presented in Chapter IV.

A further site on this river was observed at a point about 10 km farther up-river (below the Kori river confluence), but the river section there is much wider and the slope of the bed such that only a small storage capacity could be achieved.

VII - 4. Awadi

The first permanently flowing river farther down the Awash Valley is the Awadi, which springs from the Termaber uplands. A small narrow section formed by a volcanic outflow was reconnoitred a short distance up-river from the Melka Askwafen ford. It is suitable for erecting a dyke of alluvial material roughly 350,000 m3 in volume and costing approximately E$ 6,000,000, but the capacity of the reservoir between fairly high alluvial terraces would not exceed 5 - 10 hm3. Thus, investigations remain at the preliminary reconnaissance stage. Geological data are given in Appendix 1.

VII - 5. Other Tributaries of the Middle Awash

North of Debre Sina, the road from Adis Abeba to Dese crosses tributaries in the following order : Robi, Jawaha, Ataye, Jara and Borkena. It follows the upper valleys of these tributaries above 1000 m altitude, which are 'suspended' above the Awash rift, with which they communicate by narrow abrupt breaches in the mountains. Most bridges cross the rivers at the lowest points of these high valleys. Here use was made of the narrowness of the river sections to set up gauging stations. Flows at these levels are permanent and add up to about 800 hm3 in the year, half of which comes from the Borkena. This is an appreciable contribution of water to the Awash.

What are the flows in the lower portions of these rivers; i.e., beyond the Rift cliff ? As the Project's limited means made it impossible to set up gauging stations farther down the river, no more than assumptions can be made. It seems likely, however, that these rivers must be supplying water to the very faulted old alluvial formations marking the Rift piedmont terrace, for the superficial flow in some tributaries disappears at low water condition. This results in a certain natural regulation of the flows contributed by them to the Awash. The fact that the flood flows must remain roughly as they are in the higher valleys justifies regulation.

The possibility of providing reservoirs in these upper valleys was considered, and the narrow sections closing the valleys appear to be the most obvious sites. Unfortunately, even low dams at these points would result in the immediate permanent submersion of a lot of valuable land already under cultivation, or of land easily reclaimable for farming; e.g., in the Borkena marsh. This idea must be rejected. There are other sites higher up in these valleys, but they are frequently wider. What is more, their basins are very limited in size.

Solely from the power production angle, several authors have drawn attention to a possibility of diverting these rivers in tunnels through the mountain and 'run-of-the-river' turbine operation, but this is unsuitable for local conditions, because the river flows are liable to dwindle to a few hundred litres/seconds, i.e., a very low guaranteed power output. The high cost of tunnelling through the rock would not be economic. It would be preferable to meet additional power demand along the Adis Abeba-Dese itinerary by progressively building small local thermal power stations until - as will eventually happen - a power transmission line from powerful plant (Kesem, Blue Nile) is available.

VII - 6. Mile

Beyond Dese towards Asmera the road runs across high valleys of Awash tributaries ultimately forming the Mile and Logiya. Though dams could be sited on these rivers, they would seriously affect present agriculture. More interesting are the sites reconnoitered during operations carried out under the Project on the lower Mile on both sides of the Aseb road. Probably the most interesting is at a point 10 km up-river from the road bridge. There is not yet a track to this point, but from direct aerial observation and photographs, a dam there would probably not exceed about 30 m in height and measure 200 - 300 m along its crest, giving a storage capacity somewhere between 100 and 200 hm3.

A further site, immediately upstream from the road bridge, could be closed off by a main dam about 15-20 m and measuring 150 m along its crest, with a 400 m long dyke on the left bank; but the storage capacity would probably be less than 100 hm3. A small dam could be considered at a point 13,500 m downstream from the bridge, where the river passes through a small transverse range of hills in a short cutting.

The value of these sites should be considered chiefly from the viewpoint of their ability partly to regulate inflows from the Mile, which can be considerable (539 hm3 from November 1963 to October 1964) and to trap sediment. The Mile supplies part of the sediment conveyed by the lower Awash. The purpose of developing one or more of the sites reconnoitered would be to prolong the life of the Tendaho reservoir, which, as will be shown later, is the keystone in developing the Lower Plains of the Awash.

VIII. DAM SITES ON THE AWASH ITSELF

Dams on the Awash downstream from Koka are necessary to iron out flow irregularities caused by inflows from each successive tributary. The need for this additional regulation, after that afforded by Koka, becomes increasingly marked down the river. The advantage of a dam downstream rather than upstream - i.e., concerning the Lower Plains more than the Middle Valley - becomes even greater with the assumption that the biggest tributaries upstream - the Kesem and Kebena - can be regulated independently. Even assuming these general principles, however, a systematic review of all the dam construction possibilities on the Awash itself remains of interest.

VIII - 1. Mount Dofan

Mount Dofan, a volcanic salient, restricts the width of the flood plain of the Awash near Melka Warar ford, so that it is possible to erect a low dam across the Awash from the foot of Mount Dofan on the left bank to the foot of the elongated hill on the right bank. Its length would be about 250 m. Unfortunately this idea must be discarded. The reservoir would have to be fairly extensive to be effective, and cover a lot of the best land in the basin. It would also interfere with the draining of the Kesem-Kebena and Melka Sedi irrigation areas. In the investigations discussed in Volume V, only a water intake structure to supply the Amibara area (and, by extension, the Angelele and Bolhamo areas) will be considered for this site.

VIII - 2. Dabita Ale

The Middle Valley plains are isolated from the Gewani area in the north by volcanic outflows, through which the river has opened a passage gradually becoming more like a gorge, with falls and rapids, before rushing down the canyon ending at the Maro Gala plains and Lake Hertale. This configuration is eminently suitable for installing regulation and power production structures, despite the small total difference in level (less than 100 m from Angelele to Maro Gala). This part of the river cannot supply the potential available at Koka. It would also be pointless to consider massive regulation facilities at the Dabita Ale, as the Gewani marsh a few dozen kilometers down-river (and especially the riverside flood plain extending downstream of this marsh) represent an enormous natural storage capacity. One should merely consider the possibility of creating a reservoir of limited capacity, followed by harnessing the steepest falls. A reservoir of this type could be created alongside the Dabita Ale, where the valley begins to narrow. To avoid impairing drainage of the Angelele plain, the dam should not be more than 10 m high and 1,000 m long. The valley widens out again below this site, but a similar dam might also be built at a narrow section farther northward, if its topographical and geological features proved more suitable.

A substantial fall less than 2 km downstream from the Awadi confluence might become the main element in the hydroelectric development of this part of the Awash. A second power plant could also be built at a smaller fall 5 km farther north, where the river enters its final canyon. This is, however, a fairly isolated region where a demand for electricity is unlikely to develop for many years.

Although past volcanic upheavals are still not fully understood, there are grounds for believing that they may have had something to do with the diversion of part of the flow in the Awash upstream from the Dabita Ale. This may explain how Lake Hertale manages to supply the permanent flows it does by its outlet. The rainfall basin is itself too small to account for them. Difficulties might arise in filling a reservoir at the Dabita Ale, because of the increased leakage. This would be a phenomenon similar to the one now experienced at Koka. For these reasons it was not considered necessary to pursue the hydroelectric potential study for this part of the Awash any further.

VIII - 3. Buri

At a point 14 km north of Lake Gedebasa, the gallery forest through which the Awash winds narrows slightly between two mounds about 1,200 m apart. Here a dyke would provide a fairly vast reservoir, but it would submerge an area far greater than the present Gewani marsh, so that evaporation losses would probably be even higher than the considerable rates already recorded there. This idea must be discarded.

VIII - 4. Between the Chaleka and the Ledi

In this 100 km long section, the Awash breaks through a further volcanic range marked by two pronounced gorge-type restrictions : one immediately above the Chaleka confluence, and the other downstream from the Ledi. The total level difference must be slightly over 100 m, with a canyon occurring just before the Ledi confluence. The downstream restriction, to which the Yugoslav mission drew attention in 1958 and named the 'Layagili massif', has suitable topographical features for erecting a substantial dam with its reservoir possibly extending up all the rivers and streams converging fanwise near this point. The area is very remote and a dam would merely duplicate the much better placed Tendaho dam, which is downstream from the Mile. This site, therefore, did not justify any special studies for the time being. It could be employed either for a water intake supplying the small irrigable plain extending upstream from the Mile confluence - i.e., on the right bank of the Awash - or for retaining some of the sediment accumulating in the Tendaho reservoir (like the dams considered for the Mile river).

VIII - 5. Tendaho

The site at Tendaho was discovered during the first reconnaissance surveys. It was clearly recognizable because it lies beside the main Aseb road. Its exceptionally favourable position at the entrance to the Lower Plains of the Awash and downstream from all the most prolific tributaries and the possibility of creating, by means of a 35 m high dam, a reservoir of a capacity approaching 1,000,000 m3 and ideal for irrigation requirements in the Lower Plains, are both decisive arguments in selecting Tendaho and eliminating all other sites in this region. Numerous problems arose concerning the feasibility and design of a dam. In an attempt to solve them, a set of special topographical, geological and civil engineering design investigations were carried out and a summary preliminary design prepared. They are described in Chapter V.

VIII - 6. Upstream from Lake Abe

The overall fall along the final part of the Awash between the lower marshland of the Asayita delta and Lake Abe is nearly 100 m. The section runs between the volcanic Damahale hills in the north and outflows from the Asmera and Masera volcanoes in the south. This fairly uneven portion of the Awash features two small falls. They might be developed at some future date to supplement the electricity supplied by Tendaho to the Lower Plains.

IX. MORE DETAILED INVESTIGATIONS

As a result of the foregoing review of the major hydraulic development possibilities in the area, emphasis can now be laid on the four main developments, which are discussed in Chapters II to V. They are :

(i) schematic study of the diversion of the river Meki into the Koka reservoir.

(ii) summary preliminary design for the Kesem dam.

(iii) schematic preliminary design for the Kebena dam.

(iv) summary preliminary design for the Tendaho dam.

Preparatory topographical, geological, hydraulics and civil engineering research prior was necessary before accomplishing these studies.

IX - 1. Topographical Surveys

Summary surveys by compass, range finder and clinometer were undertaken on the major dam sites during the preliminary reconnaissance stage. Once the sites were selected, regular surveys were carried out to a scale of 1/1000. The reservoir basins were mapped to a scale of 1/20,000 by plotting from air photographs. A few complementary levelling surveys were made to determine the exact altitudes of particular points; e.g., saddles alongside the reservoirs, and boreholes.

IX - 2. Geological Conditions at the Sites

The particular features of the geological structure of the Awash basin, to which attention was drawn in Volume II, largely determine the implementing of major hydraulic development projects in this region. The only potentially suitable areas are where the rivers have dug into gorges through hard volcanic formations, contrasting with the wider alluvium areas upstream and downstream. It is expected that, as a general rule, the alluvium surrounding these volcanic formations also occurs above and below them, though not at all the sites, where the volcanic layers are frequently very thick.

Difficulties may result from the variegated ground between individual hard volcanic rock bars. Frequently this is fine alluvium, loam or tuff, ash and slag, which might be subject to underground erosion through piping or decomposed volcanic rock (especially montmorillonite, which acquires plastic properties and swells when moist). This type of formation must not be included in any structural foundation zone. Permeable volcanic rock may cause leakage problems near and around the dams.

Leakage problems also arise if the reservoirs widen out into the alluvium surrounding the volcanic formations. The more pervious the alluvium and the shorter the seepage path (e.g., towards a lateral valley), the more serious the risks of leakage are likely to be. A much more unpredictable form of leakage is caused by faults through which water is already circulating and which offer leakage paths for water in the reservoir, as may be happening at Koka. Reservoir leakage losses are both likely to occur and difficult to predict almost anywhere in such substantial alluvial series.

Like all volcanic subsidence regions, Ethiopia is subject to earthquakes which are sometimes violent. According to the geophysicists, the Fentale area is one showing the most permanent activity. Though this activity is generally moderate, this risk should be recognized in designing any major structures. Thus the overall geological conditions in the Awash basin are not particularly suitable for hydraulic or hydroelectric development schemes. Various difficulties may be expected. Some are predictable, as the geological studies show. Others are less predictable; e.g., the leakage problem at Koka.

Although these foundation and leakage problems were recognized during the initial expert surface surveys, borehole surveys were necessary to obtain the information needed for assessing the practicability of certain structures.

Because funds were limited, it was not possible to extend the borehole operations to cover many sites. Only the Kesem and Tendaho sites were prospected by this means.

IX - 3. Hydraulic and Civil Engineering Investigations

The numerous factors which must be analyzed in dimensioning dam structures include :

(i) Natural river inflows, as well inflows provided to allow for changes in conditions due to plant upstream going into service.

(ii) Reservoir evaporation.

(iii) Irrigation water requirements.

(iv) Damping out normal floods and discharging exceptional floods.

(v) Providing a minimum guaranteed hydroelectric power output without adversely affecting irrigation requirements.

The analysis gave reasonable structural dimensions for Tendaho and the Kesem development.

With the civil engineering design studies founded on these basic dimensions and the available geological data, and after examining a number of alternatives, it became possible to submit a set of summary preliminary design documents for an economic scheme for each major dam. These preliminary designs include a summary estimate of project construction costs based on the unit costs summed up in this section of Volume IV, Appendix 2.

CHAPTER II. DIVERSION OF FLOWS FROM THE RIVER MEKI TO LAKE GELILEA

I. CHOICE OF DIVERSION CHANNEL ROUTE

As shown in Chapter I, one way of making the Koka scheme more effective would be to divert all or part of the water supplies in Lake Ziway to Lake Gelilea. A number of topographical surveys carried out by the Project enabled the water levels in Lakes Gelilea and Ziway to be tied in with the Franco-Ethiopian Railway levelling survey. Thus, the water level in Lake Gelilea fluctuates between altitudes of 1580 m and 1590 m (dam gauge levels 100.30 and 110.30) while that in Lake Ziway remains around 1635 m with seasonal variations of only a few decimeters. Thus, there is a permanent difference between the levels in these two lakes. It varies between 45 m and 55 m.

According to the map (Fig. 5) of the area between the two lakes, the natural choice appears at first sight to be a direct link along the shortest distance, i.e., from the north-eastern tip of Lake Ziway to the southern shore of Lake Gelilea, and keeping east of the Sidamo road. A ground reconnaissance survey showed, however, that this would run into Mount Baricha at an altitude well above 1700 m; even the road rises to an altitude of 1693 m at a point 6 km south-west of Alem Tena. It was necessary, therefore, to look for an alternative route west of the Sidamo road which, though longer, would run at an acceptable altitude. Aerial photographs revealed a small intermittently flowing river, the Dubeta, a few kilometers to the west of the road and discharging into the marshland at the head of Lake Gelilea. This river comes to within 5 km of the Meki, which is a main tributary of Lake Ziway. The idea of a direct link from Lake Ziway was finally abandoned for a scheme to divert the Meki into the Dubeta.

Inspection of the area between these two rivers revealed a sinuous depression connecting them, between Melka Wokele on the Meki and a point on the Dubeta about 7 km directly due north-east. This depression, which becomes a marsh during the rains, shows every sign of once having been a river bed, of which the Dubeta was probably a direct extension. This seems an eminently suitable route for the proposed diversion.

II. WATER SUPPLIES TO BE DIVERTED

Lake Ziway is supplied basically by two rivers, the Katar and the Meki, with catchment areas of 3,500 km2 and 2,400 km2, respectively, in the 7,200 km2 of overall lake basin. Owing to the local relief, the diversion channel will have to start from the Meki, so that it will only be possible to divert the supplies available in this river, which drains only a third of the overall lake catchment area. It would be impossible to draw too heavily on natural inflows into Lake Ziway as the resulting partial drying-out of the Lake and the elimination of its outflow into Lake Hora Abiyata would have unacceptable consequences.

Measurements at the Meki village gauging station since May 1963 show an annual inflow of 234 hm3 between November 1963 and October 1964, 95 % of which discharged between June and October. These measurements and the rainfall/runoff correlations established for the Awash Basin are substantiated by the hydrology survey in Volume Three, which gives the normal volume discharged by the Meki annually as 336 hm3. The 1963/64 cycle thus appears to have been sub-normal as regards flows, but the cycle extending from November 1962 to October 1963 seems to have been almost normal with 328 hm3.

Flows in this river are very irregular, and it is liable to run completely dry at times (as in March 1964), or else to carry substantial flood flows with peaks of up 100 m3/s (which are recorded at least once a month during the rains), or occasionally even exceeding 150 m3/s (e.g., 155 m3/s on May 1, 1963). From November 1963 to December 1964, mean daily discharge exceeded 40 m3/s only on ten days.

In making a provisional estimate of the volume to divert to Lake Gelilea, it is important to maintain the present low-water flows downstream from the diversion in the Meki and to divert, for instance, only the excess above 1 m3/s. It is also important to divert only part of the highest flood flows, for instance, under the following assumptions :

(i) that the entire flow in the Meki will be diverted if under 40 m3/s (except for the 1 m3/s reserve flow).

(ii) that when flow in the Meki exceeds 40 m3/s, it will divide between the lower course of the river and the diversion channel in proportions depending on the total flow to be apportioned, but probably about equal at the exceptional flood discharge of about 200 m3/s.

Calculation on this basis shows that about 240 hm3 could have been diverted during the near-normal inflow cycle from November 1962 to October 1963. This should be considered as a gross inflow, however, for the diversion of the Meki to improve the filling of Lake Gelilea will also result in higher evaporation and seepage losses from this lake, though it is hard to estimate by how much at this stage. It seems reasonable to allow only for a net normal inflow of 200 hm3/year, which is an appreciable prospect.

III. TOPOGRAPHICAL CONSIDERATIONS

A further series of levelling surveys provided the necessary data for longitudinal profiles of the lower Meki, the middle Dubeta and the sinuous depression between them. Altitudes of the corresponding landmarks are shown in Fig. 5, which is a tracing taken from the aerial mosaic.

The actual longitudinal profiles are shown in Fig. 6, from which the surprising fact emerges that the average slope of the depression is towards the Meki and not the Dubeta. This seems to imply that an orogenic movement - always possible in a volcanic region - caused this whole area to tilt at some comparatively recent period, and also the Awash flow to be diverted northwards.

Though this slope inversion is a disadvantage for the diversion scheme, the fact that the Dubeta has a steeper slope than the Meki means that a canal starting at Melka Wokele with a slope of about 50 cm/km would connect up with the bed of the Dubeta after a run of 18 km, and that in the highest ground along the route, its bed would not be more than 22 m below surface level. Not only is this a reasonable scheme to consider, but no better alternative routes appear to be available.

IV. HYDRAULIC CONDITIONS FOR THE DIVERSION SCHEME

Assuming that the diversion channel will run through ground similar to that into which the Meki has dug its bed, and that the total river flow will be diverted, the equilibrium slope of the new river bed should be similar to that of the old bed; i.e., about 1 m/km. The slope of the Dubeta, however, is much steeper. This was to

be expected as it carries much less water. It would be pointless to give the canal the same slope and cross-sectional dimensions as the Meki and much more reasonable to consider it as an initial channel which the water will subsequently deepen by regressive erosion. The natural development of the diversion channel could then be expected to proceed along these lines :

(i) Initial phase. Deepening and possibly extension due to meander formation of the lower course of the Dubeta and the downstream canal section; incipient sediment deposition in the upstream part of the channel and the Meki due to retardation of the flow by the inadequate size of the channel cross-section.

(ii) Second phase. Regressive channel erosion catching up with the sediment deposition process.

(iii) Third phase. General deepening of the bed to a continuous slope equal to its equilibrium slope.

Throughout this process (see Fig. 6), material eroded from the bed would form an alluvial cone on reaching the marsh at the head of Lake Gelilea. This would build up until the equilibrium slope was established. The eroded materials would deposit in this particular area at the head of Lake Gelilea. As it is marshland, it is the one most likely to require priority warping treatment. Similar use of the Mojo river floods is considered for the opposite end of this marsh.

Once the diversion channel has stabilized its bed, it will convey into Lake Gelilea the sediment normally brought in by the Meki. Very small quantities are involved, compared with those brought by the Awash and the Mojo (only 370,000 tons from November 1963 to October 1964).

V. DANGERS OF THE OPERATION AND PRECAUTIONS REQUIRED

The operation is hydraulically perfectly feasible, but may involve certain risks and disadvantages. For example, the higher water levels near Melka Wokele might have caused the Meki to overflow during the initial phase. Fortunately, a survey traverse along the right bank of the Meki showed that the present bed is sufficiently deep to rule this danger out.

The most serious problem concerns the type of ground run through. The area between the depression and the Sidamo road contains extremely light erodible soils, probably originating from volcanic ash. Real crevasses have formed, possibly as a result of intense infiltration. A string of crevasses is seen on the left bank of the river and half-way between Melka Wokele and the town of Meki. They are a sure sign of underground flow towards the Meki. Is this an isolated case associated with some local geological accident, or are the crevasses due to extensive highly permeable formations ? A painstaking geomorphological study is needed to clarify this problem, as well as a borehole program to establish the nature of the subsoil directly underneath the diversion, especially in the sinuous depression.

The third natural development phase of the new Meki also provokes misgivings. For one thing, it rules out any possibility of reverting to the initial state, or even maintaining some of the flow in the present bed, as a control structure at the diversion channel intake would be eventually destroyed by regressive erosion. The alluvial cone at the head of Lake Gelilea might itself be a danger if it tended to displace the Dubeta eastwards; i.e., towards the Sidamo road.

Before undertaking the operation, one must be quite sure that the diversion of the Meki will not too severely affect present conditions in Lake Ziway and its distributary. Suitable climate and hydrometric measurement facilities (rainfall, tributary lake and distributary flow gauging, evaporation tank) should be installed as soon as possible to provide the basic data for a full hydrological survey of the basin. It seems advisable to take the following precautions :

(i) progressive barring of the Meki by a sill initially low, but subsequently made higher if the diversion worked satisfactorily.

(ii) construction of gated control works at the head of the diversion channel.

(iii) Provision of buried sills along the final 9 kilometer stretch of the Dubeta to control regressive erosion and stabilize the bed around its present level. At a rough estimate, seven sills about 5 - 6 m high will be required, one being combined with the small bridge taking the track from Alem Tena to Melka Gorge across the river.

VI. SCHEMATIC DATA FOR THE DIVERSION SCHEME

Topographical and hydrological data are still insufficient for preparing preliminary designs for the corresponding structures. The following characteristics are only approximate :

Initial channel	Invert width 5 m Side wall batter 3 on 4 Longitudinal slope 50 cm/km
Head works .... :	Three gates 2.5 m wide by 3 m high
Temporary sill in the Meki :	Top 1.5 m above bed level
Final sill in the Meki .... :	Top 3 m above bed level

Based on the above data, about 10 m3/s could be diverted during the initial phase. During the second phase, the entire Meki flow up to 40 m3/s could be diverted, but, if necessary, some of this could be returned to the original river bed downstream by setting the gates at the appropriate spilling levels. Flows in excess of 40 m3/s would divide between the Meki and the diversion channel, in roughly equal proportions at the maximum anticipated flood discharge, which is tentatively estimated at 200 m3/s.

VII. SUMMARY COST AND QUANTITY ESTIMATE (in Ethiopian Dollars).

(i) Initial channel

Excavation requirements would amount to 6,000,000 m3 in loose ground bringing the cost to:

1.2 x 6,000,000 E$ 7,200,000

		Reports : E$	7,200,000

(ii) Headworks

Excavation in loose ground : $1.2 x 5,500 m3	6,600	
Metal sheet piling, 6 m av. width : $225 x 1.12 x 640 m3	161,300	
Concrete : $100 x 800 m3	80,000	
Additional for reinforced concrete : $50 x 560 m3	28,000	
Gabions : 1st phase : 580 m3; 2nd phase : 120 m3; 700 m3 x $58	40,600	
Rock fill : $12 x 160 m3	1,900	
Total for civil engineering work		318,400
Mechanical equipment		201,600

(iii) Sills in the Dubeta

Excavation in loose ground 1.2 x 2310 m3		2,800	
Reinforced concrete 150 x 250 m3		37,500	
Rock fill 12 x 1140 m3		13,700	
For 7 similar structures :	7 x	54,000	378,000
Total			8,098,000
Additional for unforeseen items, design and supervision 30 %			2,429,400
GRAND TOTAL .... E$			10,527,400

VIII. USE OF DIVERTED SUPPLIES

VIII - 1. Power Production

Results of the last annual cycle show that the present mean monthly volume of water through the Koka plant does not exceed 100 hm3; i.e., an average rate of flow of 38 m3/s. The additional annual inflow obtained by partial diversion of the Meki - estimated at an average 200 hm3 during a normal year - will increase by about 6 m3/s the mean power plant discharge. This can easily be absorbed by the present plant equipment at Koka, as its three power units can handle an overall flow of 135 m3/s, even under the minimum head. The net result will be 15 % higher productivity for Koka and for the power projects now being built or designed between Koka and Awash station. An increase in definitely saleable power output will only result, however, if the additional Meki inflows can be used to run the turbines strictly according to power demand. This demand is illustrated by the volumes passing through the turbines at Koka each month, which are tending to even out. If Meki inflows are to boost output, they should pass through the turbines at as regular a rhythm as possible; i.e., at the following rate :

$$\frac{200}{12} = 16.65 \text{ hm3/month}$$

Under these conditions, the following increases in output could be expected :

Plant now operational (Koka)...................	16 GWh
Plant under construction (Awash II and III)....	57 GWh
Potential future power plants..................	76 GWh

Diversion of the Meki would thus boost power production by some 150 GWh yearly. This more than offsets the corresponding construction costs.

VIII - 2. Development of Irrigation

With an additional 200 hm3 yearly, it is in theory possible to irrigate roughly 12,000 ha of land in the Middle Valley at a rate of 16,740 m3/ha each year. In fact, however, this could only be achieved if the Koka reservoir were designed to ensure regulation strictly according to irrigation requirements. This is by no means the case, for the dam tends to discharge roughly equal quantities of water (100 hm3) each month, whereas irrigation demand varies throughout the year.

Thus, it is important to consider two extreme cases of use of the Meki inflows:

(i) Use exclusively for power production. The additional 16.65 hm3/month will only be available for irrigation during certain months, when there is a shortage; in other words, this quantity will serve no irrigation purpose during months in which water supplies are already in excess of irrigation demand. The effect of the diversion of the Meki for irrigation is then comparable to a partial compensation effect, and calculation confirms that diversion of the Meki during the November 1962 to March 1963 short-supply period considered in Chapter I would have enabled roughly an additional 10,000 ha of land to be irrigated in the Middle Valley.

(ii) Use exclusively for more widespread irrigation. Here the compensation effect is much more pronounced, as the entire 200 hm3 release can be allotted solely to the short-supply months. Calculation shows that in the same critical period, this use of flows diverted from the Meki would have enabled over 40,000 ha of land to be irrigated, not only in the Middle Valley, but in the Lower Plains as well.

If the Meki diversion scheme is practicable, it will be most important to decide for which purpose to use these additional supplies. This choice will be taken into account in determining the broad outlines of water planning policies for the Awash Basin. This is the subject of Volume V.

CHAPTER III. REGULATION OF THE REGIME OF THE RIVER KESEM

I. PROSPECTION FOR DAM SITES AND TOPOGRAPHICAL SURVEY

In Chapter I, regulation of the regime of the Kesem by a big dam was suggested as a most valuable measure for developing the Middle Valley of the Awash. On the basis of normal flows, the Kesem is the most abundant tributary to the Awash. Provided these resources can be regulated by a dam, they should be sufficient to irrigate more than 20,000 hectares of good land and also to generate a substantial amount of electricity.

The configuration of the Kesem basin was surveyed from the air at the start of the Project. The flights showed that the river runs in narrow valleys through several volcanic outflows between the point where it leaves the high plateau and its confluence with the Awash. One outflow, formed of lava from the Fentale craters, is deeply cut to form a gorge 1500 meters long and 70 meters deep, 8 km to the east of Awora Melka. This gorge is relatively easy to reach overland by a route some 15 km long, on the left bank from Awora Melka. Thus, it was possible, right at the outset, to start a topographical and geological survey of the gorge and to identify all likely sites for a dam. Shortly afterwards, a series of topographical operations furnished a regular 1/1000 survey of the gorges and a 1/20,000 map of the reservoir plotted from existing aerial photographs. These documents, attached to this Volume, were a basis for subsequent geological, hydraulic and civil engineering studies.

Inspection of the 1/20,000 map shows that the future reservoir could widen out fairly far northwards, but that above the 1360 to 1365 m levels, it would communicate through several separate passes with thalwegs outside the catchment area. To reduce the size of ancillary works, the normal water level in the reservoir must, in practice, be limited to a relative altitude around the 1365 m mark. The 1/20,000 map also provided the filling curve for the reservoir, with the figures for the upper section :

Relative altitude (m)	Surface area (ha)	Capacity (hm3)
1360	1850	270
1365	2300	370
1370	2850	500

These figures prove that a work of reasonable size can dam a very large mass of water and play a dominant part in regulating the flow of the river.

II. SURFACE GEOLOGY SURVEY

II - 1. Choice of Dam Site

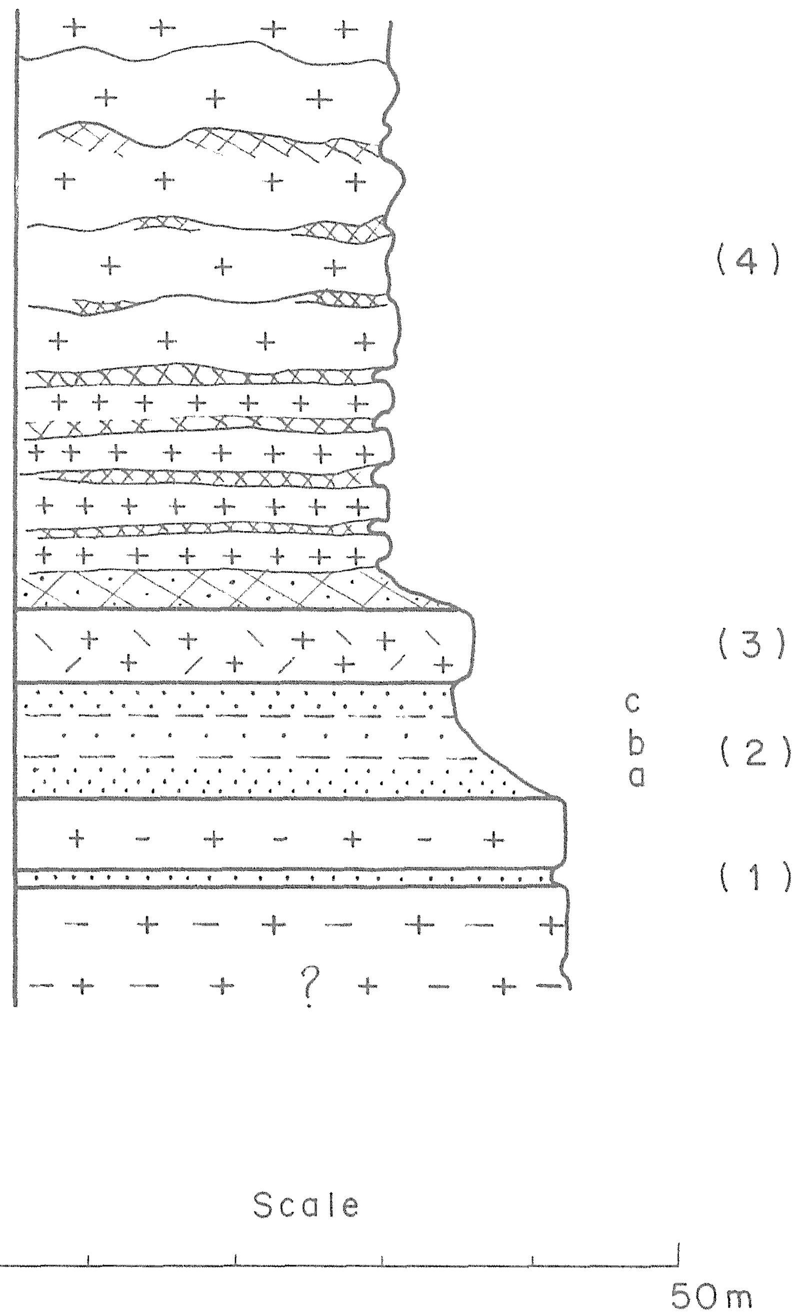
Two sites are worth considering in the Kesem gorge : one upstream (between x - coordinates 9,000 and 9,100 on the local squaring) and the other one downstrear (between x - coordinates 9,700 and 9,800). The two sites are geologically simila

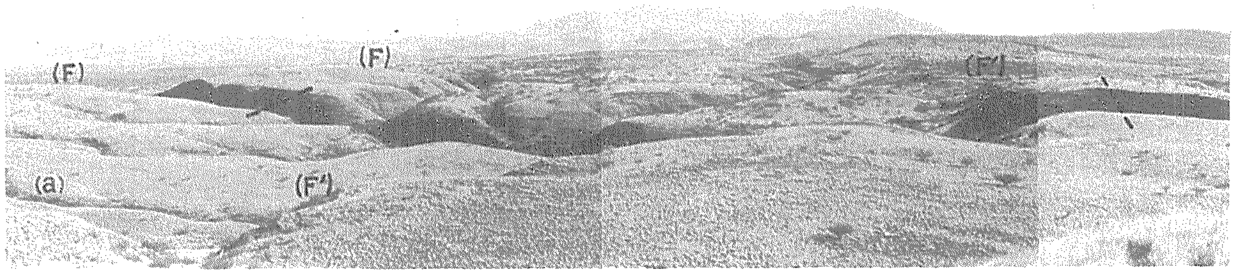
FIG.7

KESEM DAM SITE

Schematic Geological Cross-section established by surface survey

(left bank)





THE UPSTREAM AND DOWNSTREAM SITES SEEN FROM THE POINT AT WHICH THE TRACK REACHES THE LEFT BANK PLATEAU

This view clearly shows these alluvial plateaux (a) overlying the volcanic outflows through which the gorge runs. Note the two major faults (F) and (F') interrupting and upsetting the alignment of the volcanic series downstream from both sites. The volcanic Fentale range is in the background.

THE LEFT BANK AT THE UPSTREAM SITE

(1) Light-coloured volcanic rock (Ignimbrite) forming a cornice

(2) Soft tufa forming an embankment

(3) Volcanic breccia forming a reddish bar

(4) Large high-level outflow of grey rock (andesite)

but the upstream one has disadvantages; it is distinctly wider, and there are many hot springs near a fault some way downstream.

The downstream site has the double advantage that it is not affected by these phenomena and is much narrower. On the other hand, the cliffs are a little lower, so that the normal water level at the dam can scarcely exceed the relative height of 1365 m, which is the limit already contemplated. The site has also the major disadvantage of being only about 200 m from the gorge exit, marked by a straight line of cliffs at right-angles to the river and corresponding to a great fault. The leakage paths from the dam to this escarpment are fairly short, and a grouting curtain may be necessary beyond the dam if the volcanic formations are permeable, which is by no means impossible, especially in the upper cliff.

II - 2. Geological Conditions of the Hillsides

The strata dip a few degrees upstream, but downstream the foot of the slopes is covered in scree. Thus, it is difficult to tell from a surface survey, just what are the strata at the actual dam. On the left bank the series can be clearly seen between x - coordinates 9,700 and 9,800. The layers, working downwards, are sketched in Fig. 7 :

(4) The great upper cliff, about 40 m high. This is a thick, irregular volcanic outflow with levels of lava forming outcrops alternating with softer beds of slag or small tuff levels. At the base, several regular banks are seen. They consist of olivine andesite and contain a high proportion of glass as well as fairly large microlites and large phenocrysts of corroded plagioclase. The olivine is in the form of granules, which are often weathered. This whole layer, which is often chaotic, is probably fairly permeable.

(3) A thick reddish-brown bank, brecciated and cellular at the base and about 5 m deep. It consists of ignimbrite.

(2) A slope of soft ground, ochre-yellow or red in colour. This consists of tuff, often very friable and sometimes sandy. Under X-ray analysis a sample taken from the middle proved to consist almost entirely of plagioclastic feldspar.

(1) At the water's edge, there is a hard cliff, about 5 meters high, consisting of a bedded volcanic formation more or less brecciated in appearance. The rock is light-coloured and mottled, but small blackish vitreous parts reveal it to be an ignimbrite originating from very fluid lava flows which consolidated before they became completely crystallized. When studied in thin sheets, it proves to contain light-coloured slightly ferruginous glass and a few large broken or crazed plagioclase phenocrysts with glass in the breaks. There is no pyroxene.

On the right bank, a little further downstream, tuff reappears in places, alternating with volcanic outflows, but apparently predominant. It explains the scree slopes and the absence of cliffs. This is the type of terrain one may expect to find underneath the structure. It should be investigated by means of boreholes.

II - 3. Geological Conditions of the Alluvium

At no point in the gorge are volcanic rocks seen to cross the river. The rare points where they are encountered are shortcircuited by former thalwegs filled by alluvium forming short epigenous sections. Under these conditions the probable depth of this alluvium cannot be estimated merely by a surface survey. This is a problem which the borehole survey may clear up.

II - 4. Technical Consequences

There should be no major difficulty in building an earth or rockfill dam at this site, even if the areas of sandy or clayey tuff were liable to subsidence. Even if the alluvium were very deep, it could be kept under the structure by a grouting curtain. The site is, however, suitable for a concrete dam. For this type of structure, the volcanic outflows augur well, provided that the site is thoroughly cleared of loose rock, which may involve a lot of work. Difficulties may be encountered in the actual river bed, if the layer of alluvium is very thick (which is not certain) or if tuff exists at foundation level. These two possibilities must be explored by boreholes.

The most delicate problem with the abutments is set by the layer of more or less pulverulent tuff. Its depth will have to be established by the boreholes. If it is not very deep, the foundation excavations can be taken right down to the underlying bar of rock. Great care would be needed downstream of the spillway to prevent scour, which could be extremely fierce in both tuff and basalt.

Two other points require special care, and need to be examined by boreholes :

(i) The big level of tuff lying halfway up the valley-side : although porous, it does not appear to be very permeable, but it would be as well to preclude risk of piping or surface degradation by providing a stone facing or graded rubble mat forming a filter for some distance on the upstream side.

(ii) The upper outflow, which is liable to be highly permeable, especially through its brecciated or scoriaceous levels, and also liable to require in view of its proximity to the fault downstream, an extensive and costly cut-off wall some distance further away.

II - 5. Reservoir Problems

In theory, it is not impossible that the hydraulic pressure of the reservoir may drive the hot springs at the upstream site to escape elsewhere, thus offering a possible line for leaks. This, however, is not certain. It is difficult to see how to study this problem. A certain element of risk must be accepted.

Extensive outcrops of volcanic rocks are found in the gorge upstream of the site and in most of the thalwegs joining the gorge in this area, but there are no thalwegs very close together where the rocks reappear, and where leaks could occur. Here no major risk should be expected except for the great fault and the hot springs.

Old, probably Tertiary, alluvium directly overlies the volcanic formations of the gorge. It has the same dip, sometimes showing up in beds of silt or stones and, further away, in a volcanic bed forming a cuesta or capping a few hills. Most formations are masked by scree and vegetation, but cross-sections are visible at certain points and mainly consist of beds of friable coarse tuff or light brown loam. Actual stony beds are limited and are most often near the tops of the hills. It is hoped that this material will not be very permeable when considered in comparison to the scale of the storage basin and that no leakage will occur.

On the other hand, more recent alluvium fills the edges of the main valley and the former thalwegs cut into the earlier alluvium ; this alluvium has formed terraces remaining horizontal. That is why they are probably Quaternary. The main terrace lies at about 1360 - 1365 m, where it takes the form of several fairly wide, flat saddles on the left bank. The largest saddle forms the large shelf (with a little abandoned village) with the road to the dam site running alongside it. This means that the normal storage level of the reservoir will have to be limited to the 1365 m level. Even with a reservoir level limited in this way there will be a risk of leakage through the more stony alluvia, if the saddles separating the reservoir basin from the neighbouring thalwegs are narrow. Saddles in such soft formations could well be subject to rapid and deep degradation.

II - 6. Reconnaissance Work Program

It is worthwhile to carry out a minimum of forest and brush clearance work in the dam site area. The tuff levels should be bared on the surface, by means of trenches at right-angles to the river, dug out right down to the bed rock along the full length of the trench. In theory, the trenches should be placed as near as possible to the dam centerline, but a certain tolerance is allowable if it makes the work easier.

The following boreholes are the most urgent and should be made along the the probable centerline of the dam :

(i) Borehole No 1, about 60 m deep, located if possible on the river centerline, to explore the depth of alluvium and the foundation zone (presence of tuff, if any) and the permeability of the formations.

(ii) Borehole No 2, on the left bank about 30 m from the edge of the cliff, to avoid the zone of more or less uncompressed rock. It should be taken down 10 m below the point where No 1 meets bed rock, to give a cross-check on the series and to make sure it has been thoroughly explored.

(iii) Borehole No 3, the same as No 2, but on the right bank.

Second priority should be given to the following boreholes, especially if the first ones reveal high degrees of permeability, with the possible need for a grouting curtain further out, to avoid the risk of leakage round the dam :

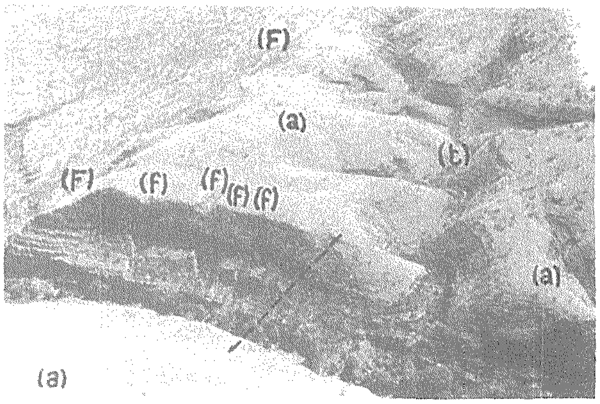
(i) Borehole No a. on the left bank, 50 to 90 m deep and some 150 m out from No 2, about half-way between the future reservoir and the great fault cliff.

(ii) Borehole No b. on the right bank, about 200 m from borehole No 3 and also in the middle of the narrow plateau separating the last thalweg upstream and the great fault downstream.

If high degrees of permeability are encountered throughout the above boreholes, they would have to be driven down below the river level ; but if, as is probable, permeability is high only in the upper outflow, the boreholes need be taken no deeper than a few meters into the underlying tuff, just to check that its permeability is low. According to the results given by borehole No 1, the exact profile of the rock under the alluvium would have to be filled in with the help of two or three boreholes all lined up in the same cross-section, after a minimum penetration of 5 m in the rock, to avoid errors due to large detached boulders.

THE DOWNSTREAM SITE
SEEN FROM UPSTREAM

Note how the gorge has dug down into the volcanic formations and its sudden interruption downstream by an escarpment at a major fault (F). Plateaux with indefinite surface contours formed by various alluvial deposits (a) overlie the volcanic formations.



THE DOWNSTREAM SITE SEEN
FROM THE LEFT BANK

Several small faults (f) which cannot be seen from the gorge form breaks in the volcanic outflows on the right bank. Note the narrowness of the plateau on the right bank between the escarpment at fault (F) and the tributary thalweg upstream (t).

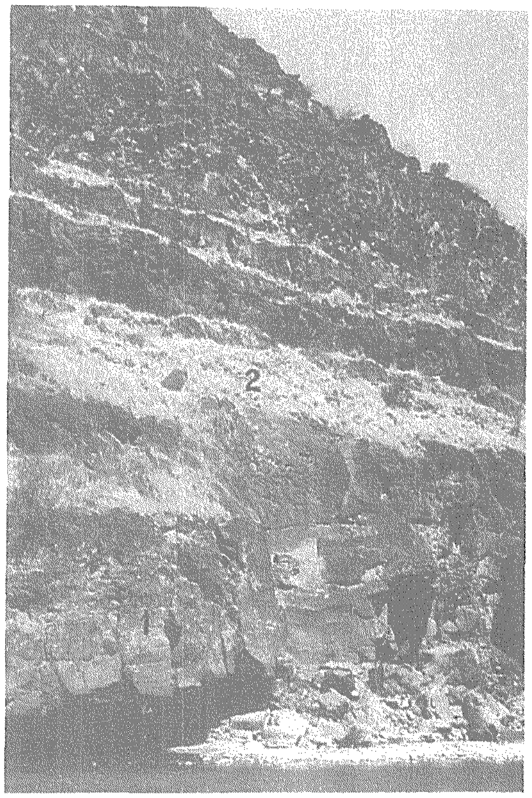
GENERAL VIEW OF THE DOWNSTREAM
SITE, AS SEEN FROM UPSTREAM

The escarpment, marking the passage of the large fault downstream from the site is seen on the right. The main elements in the series show up on the left.

THE DOWNSTREAM SITE SEEN FROM DOWNSTREAM

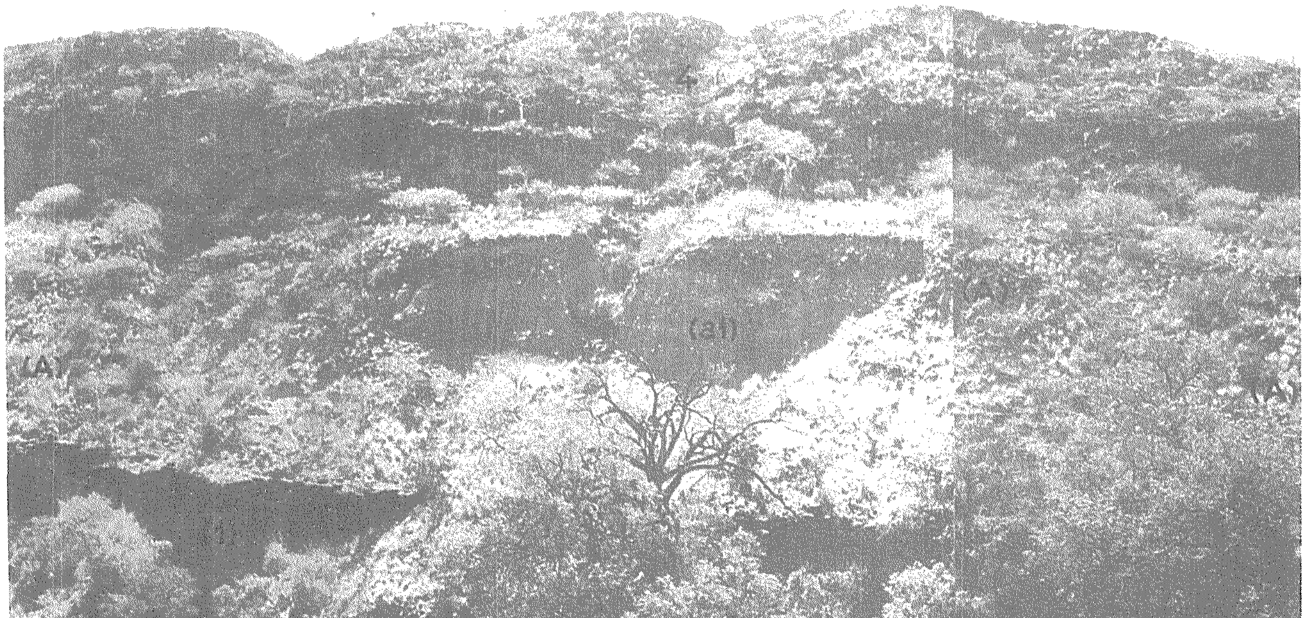
Note the escarpment at the fault (F) at the downstream limit of the gorge. The site is by the escarpment on the right; only its large high-level outflow (4) is visible on the photograph.

THE LEFT BANK SEEN FROM DOWNSTREAM



THE GEOLOGICAL SERIES
ON THE ACTUAL SITE

This series is similar to
that at the upstream site



THE RIGHT BANK AT THE DOWNSTREAM SITE

The prominent features of this bank are the bottom formation (1) showing up as a cornice and the big upper outflow forming a cliff; the rest of this formation is partly hidden under scree (A) and old alluvium (a1).

Difficulty with these boreholes will be drilling through both very hard rock like basalt and other volcanic rocks, as well as very soft and sometimes even pulverulent formations. It seems worthwhile working with a hole diameter large enough for the samples to be properly taken in this loose ground; if necessary, with the help of a double-tube sampler.

The aim should be to obtain something like a 90 % core yield. The cores should be properly marked and sorted into appropriate boxes, and those from loose ground should be waxed and placed in protective plastic casings or metal tubes.

The permeability measurements are extremely important and call for great care. In theory, they could be planned for every five meters, as is usual, where conditions permit, depending on changes in the formations (basalt or tuff). The tests should be carried out to Lugeon standards ; i.e., permeability expressed in liters absorbed per minute, each test lasting ten minutes. As far as possible, increasing then decreasing pressures will be used, care being taken to start with low pressures for the tuff formations (e.g., 2 to 3 kg per cm2) to avoid shattering the layer. As far as possible, pressures of up to 15 to 20 kg per cm2 will be used. Wherever possible, the static water level in the boreholes will be noted (to check possible presence of water tables).

Priority should be given to two reconnaissance boreholes, Nos 4 and 5, at the two little saddles north of the dam site. They will make it possible to get under the alluvium (perhaps not very thick) and investigate the permeability of the underlying basalt, if it is near enough to the surface. The expected depth of the boreholes will be 20 to 30 m, unless the permeability is still very high at the bottom. The alluvium will be further explored by open test pits used for sampling and full-scale permeability test. They could also be carried out in a large diameter pilot hole.

Second priority should be given to work of a similar nature at the various other saddles where there is a likelihood of leakage owing to a short leakage path.

III. BOREHOLE AND TEST SURVEY

III - 1. Description and General Results

The reconnaissance program was put into effect in February and March 1964. It involved six boreholes from 38 to 87 m deep and two pits dug by hand, 10 m deep. Despite recommendations, no test trench was dug in the tuff, as the information obtained from both core samples and examination of outcrops was considered adequate for a feasibility study. The location of the pits and boreholes is shown on the general 1:20 000 scale map. In addition, the boreholes nearest the river are shown with great accuracy on the 1:1 000 scale site plan. Diagrams are attached showing the lithogical cross-sections, with permeability values indicated.

III - 2. Geology of Foundation and Abutment Areas

The description of the geological formations at the dam site was confirmed and filled in by the five main boreholes (i.e., Nos 1,2,3, a. and b.) and the results shown diagrammatically in the attached cross-section, described below :

(4) Large volcanic series, 45 m deep, the surface of which forms the plateau overlooking the Kesem.

This formation was completely traversed by borehole No a. and partially by Nos 2 and 3, which were situated slightly below the structural surface of the plateau. Lithologically, it subdivides as follows :

(4b) andesite outflow, fairly fractured on the whole.

The core samples from several horizons were reduced to small fragments, which indicates either heterogeneous rock with its softer constituents ground to pieces by the drill, or, more likely, extremely fractured rock.

(4a) Cellular basalt, overtopped by a 1 m horizon of ash and lapilli and itself lying on another bed of lapilli and ejected volcanic matter, 2 m thick.

The basalt was very fractured and on several occasions the bit encountered cavities several dozen centimeters across, which may have been caused by gas pockets in the cooling lava. Petrographic analysis of a core sample taken at 33.60 m in borehole No 2 revealed dolerite with plagioclastic phenocrysts ; the olivine is weathered.

(3) & (2) Red, grey or fawn volcanic tuff, generally soft and friable.

Heterogeneous aspect, as the tuff contains inclusions of andesite, basalt and basaltic slag. The rocky outcrop (3), forming a very regular cliff which is visible on both sides of the gorge, consists of a harder layer interbedded in the tuff and probably of the same lithological nature. Borehole No 2 gave the total depth of this formation as 24 m.

(1b) Bank of ignimbrite.

This shows up in the form of the lowest cliff, only a few meters above the river. It is 4 m thick at borehole No 2 but becomes thinner from south to north, as can be seen from the cross-section.

(1a) Soft, purplish-blue ignimbritic tuff brecciated structure.

Outcropping at the Kesem low water level, it is 9 m thick.

(-1b) Bank of hard rock, light in colour and brecciated in aspect.

This forms a reference level which can be used to correlate boreholes Nos 1 and 2. The rock can be interpreted either as consolidated tuff or as lava of the andesite type. The layer is 5.5 to 6 m deep.

(-1a) Light yellow soft tuff, depth 2 to 3 m.

(-2) Large basaltic series.

The rock, fairly fractured, frequently contains small honeycombs. At certain levels they are extremely numerous and seem to link up with each other. Laboratory examination showed that the rock is dolerite with plagioclastic phenocrysts, but certain horizons contain cellular basalt with large microlites tending to doleritic in structure and the cells filled with calcite and sometimes zeolite. A horizon of red rock, 0.8 m deep, is interbedded with the basalt. The total depth of the formation is about 25 m.

(-3) Volcanic ash or very soft tuff, which broke up on contact with the drill.

This material contains many inclusions of volcanic matter belonging to the basalt family.

(-4) Complex basalt formation, made up of : (-4c) 7 m of hard but very fractured basalt. Fine structure, with some plagioclastic and titaniferous augite phenocrysts ; (-4b) a three meter layer of volcanic sand and soft ash tuff, containing fragments of basaltic slag at the base ; (-4a) hard very fractured basalt, traced over a depth of 2.3 m.

Borehole No 1 ended here, at the relative level of 1236.4, i.e. roughly 59.5 m below the low water level of the river.

III - 3. Alluvium Reconnaissance Survey

Light was thrown on the problem of the recent alluvium by borehole No 1, passing through 5 m of these deposits and giving the following results :

(i) for the first three meters : a large andesite boulder, the result of a local rock fall from the cliff, and rounded pebbles.

(ii) for the remaining two meters : muddy sand, with remains of rotting wood, a few pebbles and some gravel and some black or violet loam produced by the weathering of tuff boulders from (1a).

As no other boreholes were made in the bed of the Kesem, our knowledge of the rock profile underneath the alluvium is incomplete. In view of the lay of the land and the narrowness of the alluvial strip, it is reasonable to suppose that borehole No 1 gives a fairly close approximation to the maximum depth of the alluvium, i.e. 5 m. Old alluvia forming a superficial deposit on the right bank cliffs will have to be removed before the dam is built.

III - 4. Permeability Measurements

One objective of the borehole survey was to study the permeability of the formations near the proposed dam site and the saddles bounding the reservoir so as to assess the leakage danger and the work necessary to counteract it. Darcy's permeability coefficient, conventionally called K, was established for the various formations studied from tests made in the pits and boreholes. Two types of permeability were encountered : interstitial, depending on the nature of the rock (e.g., tuff alluvium) and permeability of a secondary type due to dislocation (andesite and basalt). The K values are shown on the cross-sections opposite each section of ground tested and, wherever possible, the permeability is expressed in Lugeon units (i.e., the average number of liters absorbed per minute by a one-meter length of borehole, at a pressure of 10 kg per cm2, each test lasting ten minutes).

The risk of leakage escaping round the ends of the dam, and the resulting resurgence downstream, can be assessed once the permeability of the valley sides upstream and downstream of the dam has been ascertained.

The table below gives the ranges of permeability values measured in boreholes Nos 1, 2 and 3 :

Relative height		K range (m/s)
No 1	1280 - 1290	2 to 5 x 10^{-6}
	1272 - 1280	8 x 10^{-8} to 2 x 10^{-7}
	1246 - 1272	4 x 10^{-7} to 10^{-6}
	1236 - 1246	2 to 4 x 10^{-6}
No 2	1315 - 1364	2 x 10^{-6} to 10^{-5}
	1290 - 1315	10^{-5} to 10^{-5} probably
	1282 - 1290	10^{-6} to 2 x 10^{-6}
No 3	1353 - 1364	10^{-7}
	1325 - 1353	10^{-6} to 5 x 10^{-6}
	1289 - 1325	5 x 10^{-7} to 10^{-6}

These results show an unsatisfactory degree of permeability on the left bank abutment : in borehole No 2 the absorption was so high between levels 1290 m and 1315 m that Lugeon tests could not be carried out, as the discharge available was not sufficient to provide the required pressure. The K range was estimated from observation of the absorption conditions affecting the drilling water. The lateral boreholes Nos a and b, further away from the Kesem gorge, gave more encouraging results :

Relative height		K range (m/s)
No a	1328 - 1361	5 x 10^{-7} to 10^{-6}
	1286 - 1328	4 x 10^{-7} to 6 x 10^{-7}
No b	1338 - 1379	7 x 10^{-7} to 2 x 10^{-6}

The area of high permeability seems to be restricted to a strip of land parallel to the Kesem, situated on the left and not more than 200 m wide.

III - 5. Reservoir Boundary Formations

Three areas warrant special attention. The first area is the thalweg joining the river on the right bank upstream of the site. It will be submerged along some 500 m by the reservoir. This gully is only 300 to 400 m from the great escarpment downstream and the leakage path at the 1365 m level is even less, being under 200 m as measured from a small secondary gully. Borehole no b. was located just there, near the saddle giving the shortest leakage path. The results obtained from this borehole showed that the general leakage danger is low, the average permeability of the rocky barrier being fairly low, but there is a definite risk of leakage along preferential flow paths, accentuated by the existence of a difference in height between the reservoir level and the plateau below, which will be about 20 to 25 meters.

The second area comprises the rounded hills forming the northern boundary of the storage basin, where there is a risk of underground leakage towards the thalwegs joining up with the Kesem downstream of the gorge. The surface layers include coarse and loamy alluvium overlaying tuff, which is extremely light and friable in certain horizons. Boreholes Nos 4 and 5 were located in the two areas most likely to be subject to leakage, as indicated by the lie of the land. The shortest leakage paths measured were respectively 200 m and 350 m, at the 1365 m level (expected normal maximum storage level).

Borehole No 4 revealed that, under a thin layer of low permeability alluvium, were more permeable tuffs, themselves lying on the great andesite outflow which forms the plateau to each side of the Kesem. This andesite is highly crevassed, and here the highest permeability values were registered. Approximate K values are :

Relative height	K range (m/s)
1359 - 1367 (alluvium)	2×10^{-7}
1352 - 1359 (tuff)	10^{-6} to 4×10^{-6}
1339 - 1352 (alternate andesite and basalt)	10^{-7} to 4×10^{-6}
1332 - 1339 (fractured andesite)	horizons at 10^{-4}

The third area is a saddle linking two very flat valleys, one inside the reservoir basin and the other outside. Here 10 m test pit, No 6, was dug. The saddle is situated at the 1360.7 level, and a fairly long, but not very high, embankment will have to be built across. The ground into which the pit was dug was uniformly composed of clayey loams, apparently originating in the weathering of tuff or tuff ash, which are both plentiful in the nearby hills. As a pit No 5, the tuff becomes less and less permeable the deeper one goes :

Relative height	Permeability - K (m7s)
1357.2	6×10^{-6}
1355.3	4.2×10^{-6}
1353.1	1.5×10^{-6}
1350.7	5×10^{-7}

III - 6. Conclusions

The geological structure of the site is simple : except for a few variations in thickness, there is good agreement between the layers on both sides of the river and the same alternation of outflow and intermediate layers continues underneath the Kesem where the depth of alluvium is about 5 meters.

As far as the dam foundations are concerned, this alternation of very hard and soft layers does not set any great problems for a flexible earth or rockfill dam likely to adapt itself to large-scale differential subsidence. For a concrete dam, however, these dangerous levels would have to be eliminated. This is quite feasible with the little level under the Kesem. This consists entirely of Montmorillonite, which can be cleared away and the foundations set in the large level of dolerite, but the right and left bank abutments are a different matter.

As far as the leakage problem is concerned, boreholes Nos 2 and 3, actually on the dam site did not reveal an excessive degree of permeability, except for a certain section in No 2 (left bank), where the permeability was so high as to prevent testing, and in certain stretches of the three boreholes, corresponding to either soft porous layers (possible source of pipings) or large cracks in the hard volcanic outflows (where there were cavities). An extensive grouting curtain will be necessary, but its exact nature can only be defined by more detailed studies later. In the areas beyond the dam the alluvium has a fairly low degree of permeability, whereas the levels of tuff or ashes are more permeable, as are also, on occasions, the underlying volcanic rocks. There is thus a definite risk of leaks, especially through preferential zones which are sometimes difficult to detect. On the whole, the studies so far made suggest that this problem is not insoluble.

Later studies should chiefly concern the leakage problem and clarify the grouting curtain requirements, both near the dam and out beyond it. Grouting curtains will be difficult to carry out owing to the varied nature of the rock strata along any one vertical, ranging from hard and cracked to soft and porous, and also the mechanical qualities of the various soft layers separating the beds of hard rock, where clay is abundantly, and sometimes exclusively, present.

IV. HYDRAULIC SURVEY AND BASIC DIMENSIONS OF THE SCHEME

IV - 1. Limitation of Storage Capacity

The topographical and geological surveys of the dam site and the reservoir basin showed that the highest normal water levels could not easily be set at a height of more than 1365 m. This maximum elevation will be a basis for the operating calculations which follow. The level of the reservoir must not drop below a certain altitude, so that :

(i) the water retained will include a bottom layer (dead storage) in which most of the sediment carried down by the river will collect;

(ii) the head of water available in the power station at the foot of the dam does not fall below a figure compatible with the technical and economic requirements of the generating sets.

The second requirement takes precedence over the first, and it is assumed that the lowest water level will not normally fall below about 1345 m. These upper and lower limits of reservoir level give the following final figures for capacity:

Dead storage (below 1345 m mark) 79 hm3

Live storage (between 1345 and 1365 m marks) 288 hm3

Total normal storage 367 hm3

The sole purpose of the following calculations, based on monthly figures, is to determine the optimum operating conditions for this capacity ; on the basis of average probable inflow, this capacity must satisfy as fully as possible all requirements for irrigating the land commanded and generating electricity.

IV - 2. Choice of Probable Flow Figures

The following monthly readings were taken at the Awora Melka hydrometric station which, being slightly downstream from the dam site, is fully representative of flow A at that site :

A (hm3)	O	N	D	J	F	M	A	M	J	J	A	S	Year
1962											101.5 −	78.5 −	
1962-1963	11.5	2.5 −	2 −	2.5 −	1.5 −	1 −	7 +	6.5 +	3	118 −	332 +	72	559.5
1963-1964	18.5	5 +	4 +	4 +	1 −	2.5 +	24 +	12.5 +	17	184 =	253.5 =	97.5 =	623.5
1964	24												

Statistical study of rainfall in the upper basin shows how the monthly inflow varies from normal conditions; the following symbols are used :

Minus sign : Sub-normal

Equals sign : Normal

Plus sign : Above normal

Although rough, this assessment suggests that variations from normal approximately cancel out within the two complete cycles given, so that 600 hm3 can be regarded as a substantially normal flow. Operating calculations will be based preferably on the first of these two cycles. It was the more irregular and corresponded to a lower total figure for the year. It represents, therefore, the minimum results expected in operating the dam.

IV - 3. Life of the Reservoir

October 1963 to September 1964 is the only yearly period for which complete and significant returns of sediment load in the Kesem are available (thousands of metric tons) :

O	N	D	J	F	M	A	M	J	J	A	S
1.8	0.5	0.4	0.4	0.1	0.3	72.2	123.3	50.7	1997	3034	306.5

In all, the Kesem carried down almost 5,600,000 metric tons of material, 99.95 % of this total being between April and September; the studies in Volume III show that this quantity is well above normal (3,570,000 tons/year), giving an average silting rate of about 2-3 hm3/year. Consequently, the theoretical life of the reservoir is certainly more than a hundred years. In this respect, the Kesem would be less dependable than Lake Gelilea. The following points should be taken into account :

(i) Sediment is carried down almost wholly by the floods which stir the water in the reservoir to some extent and help to keep the finer sediment in suspension (e.g., sediment made up of particles with an average diameter less than 50 microns). In a normal year, the reservoir should regularly overflow at the end of the rainy season (August, September), thus discharging part of the sediment brought down.

(ii) Above the dam, the slope of the bed is about 1 in 200, so that deposits tend to accumulate towards the bottom of the reservoir, where they can more easily be collected and evacuated by flushing through the dewatering conduit of the dam ; the gorge leading to the dam will help to carry the flushing current a fair distance upstream, into the thickest part of the sediment.

IV - 4. Passage of Floods

Kesem flood conditions are examined in Volume III and the probabilities are shown by the following peak discharges :

Biennial flood......................	310 m3/s
Maximum 1964 flood (13th August)....	340 m3/s
Maximum 1963 flood (6th August).....	570 m3/s
Ten-year flood......................	840 m3/s
Fifty-year flood....................	1100 m3/s
Hundred-year flood..................	1180 m3/s
Morphological flood.................	1400 m3/s

For safety, the dimensions of the dam outlet system are calculated on the basis of a nominal flood of 1500 m3/s. In reality, the dam must be so planned that it does not return excessive discharges, except in unusual circumstances. Otherwise the downstream works, comprising the irrigation intake and the embankments alongside the Kesem in the irrigable plain, would have to be made too big. On this basis it is assumed that the dam will not return more than 500 m3/s. To lower the level of the nominal flood, provision must be made for storing some flood waters in a section of reservoir above the 1365-meter mark. If the nominal-flood hydrograph is represented by an isosceles triangle with a base of 3 days and a height (1500-200*) m3/s, the calculation gives a storage of 100 hm3. This assumes that the highest water level in the reservoir could reach the 1369 m mark. The elevation of the crest will be 1370 or 1371 m, according to the type of dam.

IV - 5. Evaporation Losses from the Reservoir

Knowledge of evaporation from the reservoir is based on data recorded at the Awora Melka meteorological station. They are processed and interpreted in detail in Volume III and provide the specific figures set out in the first line of the table below. For accuracy, natural evapotranspiration in the reservoir area must be

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* 200 m3/s , which is the basic flow from which the exceptional flood starts.

deducted from each of the above figures ; virtually, this means rainfall (because surface runoff is negligible). Net atmospheric losses "e" from the future Kesem reservoir can be calculated from the figures in the third line of the table, expressed in m3/ha of reservoir area.

1962-1963	O	N	D	J	F	M	A	M	J	J	A	S
Evaporation (mm)	-	174	167	186	190	232	166	208	268	(240)	242	260
Evapotranspiration (mm)	-	23	18	0	1	18	71	84	7	(221)	121	11
Loss "e" (m3/ha)	(2460)	1510	1490	1860	1890	2140	950	1240	2610	190	1210	2490

Multiplying the annual total (20,040 m3/ha) by the average area of the reservoir (1750 ha) gives a total loss of about 35 hm3 per annum.

IV - 6. Irrigation Requirements "B"

The regulated flow of the Kesem will be used primarily to irrigate the plain known as "Kesem-Kebena", where an area of 17,550 ha can be served by a gravity system (see Volume V). Specific requirements of irrigation water for the Middle Valley, as estimated in Volume II, apply to the Kesem-Kebena plain and give the following annual water requirement :

$$17,550 \times 16,740 = 294 \text{ hm3}$$

This is well below total inflow (even after deducting losses E), and it is possible that the Kesem reservoir may cover irrigation requirements exceeding those of the Kesem-Kebena plain. Unfortunately, it is impossible economically to build the dam higher, and there is no question of using the whole annual inflow for extending the irrigated area.

Assuming the reservoir to be full on 1st October, it will empty continuously until 30th June, when it should theoretically be at its lowest level. Over these nine months, inflow (37.5 hm3 from October 1962 to June 1963) will be largely lost by evaporation from the reservoir, so that a volume slightly in excess of the reservoir's live storage - i.e., about 300 hm3 - will provide enough water to irrigate the following area :

$$\frac{300,000,000}{13,465} = 22,500 \text{ ha}$$

On a first estimate, some 5000 ha could be supplied, in addition to the Kesem-Kebena area. This land would have to lie within the area of the Middle Valley which is to be supplied direct from the Awash (Melka Sedi, Amibara, Bolhamo, Angelele).

IV - 7. Electricity Generating Requirements

If the volume passing through the turbines month by month were limited strictly to the water needed for irrigation, the amount of electricity generated during months of small irrigation demand would fall too low to justify the power station equipment, which can be economic only if a minimum firm amount is generated. During those months the turbines would need more water than the volume required for irrigation. These tappings would slightly reduce the irrigable area when the reservoir was emptying. They would have no effect while it was filling. The tailwater level below the dam can be put at about 1296 m. This means that the hydro-electric plant will normally operate with a geometric head of between 69 and 49 meters.

IV - 8. Final Operating Calculations

A compromise had to be reached between the needs of the area to be irrigated and the need for a firm power output. Optimized scheme operation gives :

Total irrigated area .... 22,000 ha

Firm power .... 3.5 GWh per month

The figures' validity is confirmed by the following table :

1962-63	O	N	D	J	F	M	A	M	J	J	A	S	Year
Z_1 (m)	1365.0	1364.2	1363.0	1360.8	1358.8	1356.5	1354.6	1353.0	1350.0	1344.9	1353.4	1365.0	-
V_1 (hm3)	367	350	325.5	285.5	248.5	212	183.5	162	124.5	78	166	367	-
S_1 (ha)	2330	2250	2130	1930	1735	1545	1400	1285	1070	770	1305	2330	-
A (hm3)	11.5	2.5	2	2.5	1.5	1	7	6.5	3	118	332	72	559.5
B (hm3)	22.5	21	39	36	35	26.5	27	42.5	47	25	17	30	368.5
R (hm3)	0.5	2.5	0	0	0	0	0	0	0	5	8	0	16
T (hm3)	23	23.5	39	36	35	26.5	27	42.5	47	30	25	30	384.5
A - T (hm3)	-11.5	-21	-37	-33.5	-33.5	-25.5	-20	-36	-44	88	307	42	175
V'2 (hm3)	355.5	329	288.5	252	215	186.5	163.5	126	80.5	166	367	367	-
S'2 (ha)	2280	2145	1945	1750	1565	1415	1290	1080	785	1305	2330	2330	-
S (ha)	2305	2200	2040	1840	1650	1480	1345	1185	930	1040	2000	2330	-
E (hm3)	5.5	3.5	3	3.5	3	3	1.5	1.5	2.5	0	2.5	6	35.5
H (m)	68.6	67.6	65.9	63.8	61.7	59.6	57.8	55.5	51.5	53.2	63.2	69.0	-
P (GWh)	3.51	3.53	5.71	5.10	4.80	3.51	3.47	5.24	5.38	3.55	3.51	4.60	51.91

The symbols have the following meanings :

Z_1	Elevation of water level at start of month
V_1	Reservoir capacity at start of month
S_1	Reservoir area at start of month
A	Natural inflow
B	Irrigation requirements (22,000 ha)
R	Additional volume of water to be returned in order to secure firm power output
$T = B + R$	Volume passed through turbines
$A - T$	Rise or fall in reservoir capacity
$V'_2 = V_1 + (A-T)$	Storage capacity at end of month, not allowing for evaporation
S'_2	Reservoir area at end of month, not allowing for evaporation
$S = \frac{S_1 + S'_2}{2}$	Average reservoir area
$E = eS$	Net loss by evaporation
H	Average geometric head
$P = \frac{T.H}{450}$	Potential power output

The table shows :

(i) At the start of July, the level of the reservoir is at its lowest (1344.9 m), but the rise does not exceed 20 m, except perhaps for short periods when the floods overflow, from mid-August to mid-October.

(ii) Average annual reservoir level is 1358.2 m, so that average geometric head is 62.2 m.

(iii) A total additional volume of R = 16 hm3 must be passed through the turbines to bring firm power up to 3.5 GWh per month. As these amounts are mainly drawn off while the reservoir is filling, they do not affect total irrigated area (22,000 ha).

(iv) Loss by evaporation, amounting to 35.5 hm3 per annum, represents only one-tenth of the total reservoir capacity.

(v) Total annual energy output (P = 51.91 GWh) corresponds to a theoretical productivity reached only if there were no limit to the installed power. Firm annual energy output is :

$$3.5 \times \frac{365}{30} = 42.5 \text{ GWh}$$

IV - 9. Size of the Power Station

The firm energy output of 3.5 GWh per month must be attainable even when the reservoir is down to the 1345-meter mark; i.e., when the geometric head is 49 m. Average power to be generated in these conditions is :

$$W_{av.} = \frac{3.500.000}{31 \times 24} = 4,700 \text{ kW}$$

Applying the same assumed load factor as for the Koka plant, (i.e., 55%), firm peak power is :

$$W_{peak} = \frac{4700}{0.55} = 8500 \text{ kW}$$

It would be a pity, however, to limit installed power to this figure and it is reasonable to assume that the generating sets must also work at full power with the average head of 62.2 m, so that installed power is :

$$W_{inst} = W_{peak} \left(\frac{62.2}{45}\right)^{3/2} = 12,000 \text{ kW}$$

Thus, full power is not guaranteed when the head is below 62.2 m. In practice, the station will have one 4000 kW set and one 8000 kW set. Total plant flow is 24 m3/s with the average head of 62.2 meters.

IV - 10. Supply of Power to Irrigation Pumps

The foregoing calculations apply only if the electricity is used by regular domestic and industrial consumers. The position would be different if a substantial fraction of the electricity generated by the Kesem station were reserved for irrigation pumping and had to vary according to changes in water requirements. The outline development study for the Middle Valley will show, however, that irrigation by pumping is not an operation of the greatest economic value. It would not require more than a relatively small amount of power (1000 kW for the Bolhamo area).

IV - 11. Compensation for Flows Through the Turbines

The flows returned by the Kesem station will be affected by daily and weekly variations in the demand for electricity. They are not the same as the variations of irrigation demand. The subject was considered in Chapter I, where compensation for the turbine flows at Koka was discussed. Making all due allowances, a capacity of some 500,000 m3 should be sufficient for the daily and weekly compensation of flows through the turbines at Kesem. With a structure 5 to 10 m high, a reserve can easily be established in the gorge stretching 9 km downstream from the site of the dam to the point where the river enters the plain at Awora Melka. Inspection of aerial photographs and the line of the gorges showed three possible sites :

(i) 3000 m downstream from the site of the main dam ;

(ii) 2500 m upstream from Awora Melka ;

(iii) At Awora Melka itself, with the compensating dam also serving as the irrigation intake dam.

The final choice of a site will be based on a general large-scale survey of the Kesem gorge and a geological study.

V. OUTLINE PRELIMINARY DESIGN

The remainder of this study is concerned only with the downstream site, which was preferred to the upstream site on geological and topographical grounds.

V - 1. Catchment Slopes

The conformable strata on the two banks slope along the line of the river, dipping some degrees in the upstream direction. The lower slopes are covered with scree and part of the escarpment on the right bank is masked by an old alluvial deposit culminating near the 1355-meter mark. Five borings taken along the dam axis showed the following succession of the strata :

(i) Between the 1345 and 1375 m marks, an andesite, fairly fractured in general ; this stratum rises above the crest of the dam which will be at the 1370 or 1371 m mark.

(ii) Between the 1330 and the 1345 m marks, a vesicular basalt outflow, separated from the previous stratum by a thin horizon of ash and lapilli. Like the andesite, this rock is very hard, but badly fractured.

(iii) Between the 1305 and 1330 m marks, a complex succession of mainly soft and friable strata (volcanic tuff).

(iv) On either side of the 1300 m mark, a bed of ignimbrite, thickening from the left to the right bank (4 meters at boring No. 2, 11 meters at boring No. 5) and with its base near to river level (1296 meters). It stands on soft purplish tuff.

V - 2. Bed of the River

The Kesem runs over recent alluvial deposits, roughly 5 meters thick, with no volcanic rocks crossing the river at any point. Borehole No. 1 revealed the following strata below the alluvium :

(i) a hard brechoid layer, 5 to 6 m thick ;

(ii) 2 to 3 m of soft tuff ;

(iii) a major basaltic series, some 25 m thick, with its highest point at 1282 m - hard, but rather fractured rock - identified as dolerite.

V - 3. Watertightness near the Dam

Permeability tests in the borings showed that the risk of the dam being by-passed by percolation in the volcanic formations seemed confined to a band with a maximum width of 200 meters running parallel to the Kesem, on the left bank.

V - 4. Watertightness of the Reservoir Basin Around the Dam

(i) Saddle on right bank : low permeability, but with no grout curtain, the percolation gradient would be around 1/20.

(ii) Saddle at boring No. 4 : high permeability in the andesite, where the percolation gradient would be around 1/30.

(iii) Saddle at pits No. 5 and 6 : clayey loams, very watertight to a sufficient depth to rule out any risk.

V - 5. Type of Dam

After cement grouting, the basalt and andesite outflows and the ignimbrite bed will have a very high mechanical strength and modulus of elasticity. This does not apply, however, to the intermediate tuff strata. An arch dam seems to be ruled out, but a gravity dam, hollow if possible, with the blocks bedded on the hard strata would be well suited to the geology of the site. A rockfill dam is also feasible. An earth dam seems unsuitable for a number of reasons; there is little alluvium in the river, the soils of the plateau are very clayey and the base thickness of the dam would be excessive.

Two possibilities - a hollow gravity dam and a rockfill dam - were selected for final study. Each study produced three sets of drawings which are attached to this Report and show the main structural features.

(1) Hollow gravity dam : The work comprises 11 blocks, each 18 m wide, topping at the 1370 m mark. Numbering these blocks from the left bank to the right bank :

(i) Blocks 1, 2 and 11 are based on the andesite, at the 1360, 1350 and 1360 m marks respectively ;

(ii) Blocks 3, 4, 8 and 9 are based on the ignimbrite at 1302.5 m.

(iii) Blocks 5, 6 and 7 are based on the dolerite, at 1280 m.

(iv) Block 10 is based on the cellular basalt at 1340 m.

(2) Rockfill dam : All strata are strong enough to carry the dam. The scree would have to be cleared and all old surface deposits of alluvium on the right bank removed. The dam would be made watertight by a core of clayey earth from the plateau on the left bank ; in the river bed, it goes down to the brechoid level revealed near to the 1290 m mark by boring No 1. The upstream and downstream facings are obtained from the basalt and andesite on both banks, particularly from the excavations for the spillway and the water intake. The crest of the dam is levelled off at the 1371 m mark to provide 1 m freeboard greater than fixed for the concrete dam.

V - 6. Layout of Appurtenant Works

(1) Hollow gravity dam : All appurtenant works are incorporated in the dam or set against it ;

(i) the water intake and penstock for the power station are incorporated in block 5 (installed flow 24 m3/s);

(ii) the power station is immediately downstream ; it contains two vertical Francis sets generating 4000 kW and 8000 kW respectively. Access is from above, at the 1310 m mark, because of the substantial rise downstream when the river is in flood ;

(iii) the flood spillway with its sill at the 1356 m mark comprises 2 gates 12 m wide and 9 m high, capable of passing the nominal flow of 1500 m3/s. The first gate straddles the joint between blocks 5 and 6 and the second the joint between blocks 6 and 7 ;

(iv) the dewatering conduit is in the middle of block 6. Its shut-off gate can be raised through the center pier of the flood spillway. Its control gate is a 12 m2 sector gate ;

(v) there is no temporary diversion tunnel, because the water in the river remains low for a long time ; the bottom part of the center blocks can be built in two stages, and it is assumed that, at the end of the first stage, the flood waters will flow over the uncompleted blocks without destroying any final works.

(2) Rockfill dam : The appurtenant works are sited on the river banks :

(i) Right bank : the flood spillway with the same gates and standing on the upper andesite ;

(ii) Left bank : a temporary diversion tunnel *, of 5-meter inside diameter, which will finally become the dewatering conduit. Its roof will be in the ignimbrite, but its floor in the purplish tuff.

(iii) Also on the left bank : the intake, headwater tunnel, surge tank, penstock and power station (all for the same flow and power as in the first scheme). The headwater tunnel is driven through the cellular basalt carrying the intake and the chamber for the penstock stop-valve. The power station stands on the brechoid bed.

V - 7. Grout Curtain

Whichever dam is chosen, the grout curtain must extend from a point 300 or 400 meters away from borehole b, on the right bank to a point some 150 m away from borehole 4 on the left bank, with a possible break around borehole a. The gross total area of the curtain can be put at 100,000 m2, less the tuff strata, which are very watertight. The rest consists of fractured basalt and andesite levels which can be easily penetrated by cement grout. There will be a good deal of absorption, but no technical difficulties requiring the use of special products.

* A tunnel is indispensable. It will even be essential that work on the dam should begin in October so that it can be raised to the 1345 m mark by the following July. Calculations show that, with an inside diameter of 5 m and with allowance for lowering of the flood level by the bottom storage layer in the reservoir, the tunnel could safely evacuate the flow from a ten-yearly flood lasting 3 days and with a peak discharge of 900 m3/s.

V - 8. Summary Cost Estimate of Works (in Ethiopian dollars)

(1) Hollow gravity dam

Civil engineering		
Cofferdams	2,000,000	
Rock excavation 5 x 240,000 m3	1,200,000	
Concrete structures 85 x 320,000 m3	27,100,000	
Additional for reinforced concrete 65 x 20,000 m3	1,300,000	
Grouting	3,000,000	
Secondary dam	1,000,000	E$ 35,700,000
Electrical and mechanical equipment		
Dam		
Flood spillway	387,000	
Dewatering conduit	97,000	
Intake	165,000	E$ 649,000
Power station		
2 turbines	550,000	
2 alternators	930,000	
2 main butterfly valves	102,000	
Overhead crane	145,000	
2 transformers	200,000	
Switchyard	370,000	
Auxiliary equipment and services	500,000	
Loading gantry	145,000	E$ 2,942,000
TOTAL		E$ 39,291,000
Contingencies, design and supervision of works : 30 %		E$ 11,787,300
G R A N D T O T A L		E$ 51,078,300

(2) Rockfill dam

The estimate for this type of dam, which seems cheaper, gives separate costs for the following :

(i) the dam alone and all elements necessary for its operation, so that it could be built in a first stage, if required for irrigation purposes only ;

(ii) the power station and such parts of the dam as belong only to power generation, which could be installed as a second stage.

First stage : irrigation

Civil engineering

Cofferdams	2,000,000	
Rock excavation 5 x 24,000 m3	120,000	
Rockfill 12 x 690,000 m3	8,280,000	
Watertight core 5 x 110,000 m3	550,000	
Filters 20 x 95,000 m3	1,900,000	
Concrete 150 x 22,000 m3	3,300,000	
Dewatering) Shaft 4950 x 40 m	198,000	
Dewatering) Tunnel 3530 x 400 m	1,412,000	
Initial intake tunnel 1665 x 50 m.................	85,000	
Gangway giving access to control room	223,000	
Grouting	3,000,000	
Secondary dam	1,000,000	E$ 22,066,000
Electrical and mechanical equipment		
Dam		
Flood spillway	595,000	
Dewatering conduit	298,000	
Intake	66,000	E$ 959,000
TOTAL		E$ 23,025,000
Contingencies, design and supervision of work: 30 %		E$ 6,907,500
G R A N D T O T A L		E$ 29,932,500

Second stage : electric power

Civil engineering

Rock excavation 5 x 11,000 m3	55,000		
Concrete 150 x 8000 m3	1,200,000		
Intake 1665 x 140 m	233,000		
Surge tank 3850 x 24 m	96,000	E$	1,584,000

Electrical and mechanical equipment

Dam - Intake	337,000		
Power station	2,942,000	E$	3,279,000
TOTAL		E$	4,863,000
Contingencies, design and supervision of work : 30 %		E$	1,458,900
G R A N D T O T A L		E$	6,321,900
COMBINED TOTAL FOR FIRST AND SECOND STAGES		E$	36,254,400

V - 9. Conclusions

The rough estimates strongly favour a rockfill dam. There is a risk, however, that the driving of the diversion tunnel may encounter serious difficulties. It would be wise to drive a trial length of tunnel before calling for tenders, although it is unlikely that the difficulties would outweigh the economic advantage of a rock-fill dam. Even excluding contingencies, the gravity dam still costs more ; and the cost of civil engineering works for the diversion tunnel is only 1,600,000 E$ as against a total of 7,000,000 E$ for contingencies.

If there are no disastrous results with the trial tunnel, the choice should go to a rockfill dam, which might cost slightly over 35,000,000 E$, including 30,000,000 E$ for the first stage, leaving the remainder to be invested progressively whenever the demand for electricity justifies the installing the first generating set and a power transmission line.

CHAPTER IV. REGULATION OF THE REGIME OF THE RIVER KEBENA

As stated in Chapter I, the regulation of the Kebena is not a key element in the hydraulic and agricultural development of the Middle Valley. Nevertheless, it is useful to summarize the information collected on this subject.

I. CHOICE AND TOPOGRAPHICAL STUDY OF A SITE

Some 20 km upstream from its confluence with the Awash, the Kebena runs through a narrow gorge, some 200 m long, which seems at first sight to be very suitable for constructing a fairly high dam. A 1/1000 plan was drawn and a 1/20,000 map of the reservoir basin was produced by plotting from aerial photographs ; this map was used to draw characteristic reservoir filling curves.

The gorge is 200 meters long ; the plateau through which it is cut slopes steeply downstream and the greatest depth, of about 70 m, is at the upstream entrance. The stretch about one-third of the way along the gorge from the upstream end is very narrow at the base. The dam should be sited at this point.

At a relative altitude of 1265 m the gorge is barely 125 to 130 m wide. Above this level, it opens out, and at the 1275 m mark it is more than 200 m across. Solely for topographical reasons, the crest of the dam should be set at about 1265 m.

With the crest at this elevation (i.e., with the highest normal level in the reservoir at 1260 m) the total capacity will be 50 hm3. For this reason alone, the scope and value of the scheme are much less favourable than that of the Kesem dam. As a variant, the possibility of raising the dam by 10 m (highest normal level at the 1270 m mark and capacity 75 hm3) will be examined.

II. GEOLOGICAL RECONNAISSANCE SURVEY

II - 1. General Structure

The valley of the Kebena runs through a thick mass of varied alluvial deposits, at some places including regular, dark bands of volcanic rock, and frequently broken or displaced by faults. At the site considered the gorge cuts through a particularly strong volcanic series, with an apparent thickness of more than 80 m. The bed-rock cannot be seen because the series, which dips downstream (and rises upstream), is sharply broken by a big fault running south-east and north-west, bringing it into contact with further alluvial deposits which are probably more recent.

II - 2. Volcanic Series in the Gorge

The volcanic series, illustrated in Fig. 8, comprises the following strata, reading downwards :

(8) Old alluvial deposits covering the volcanic formations.

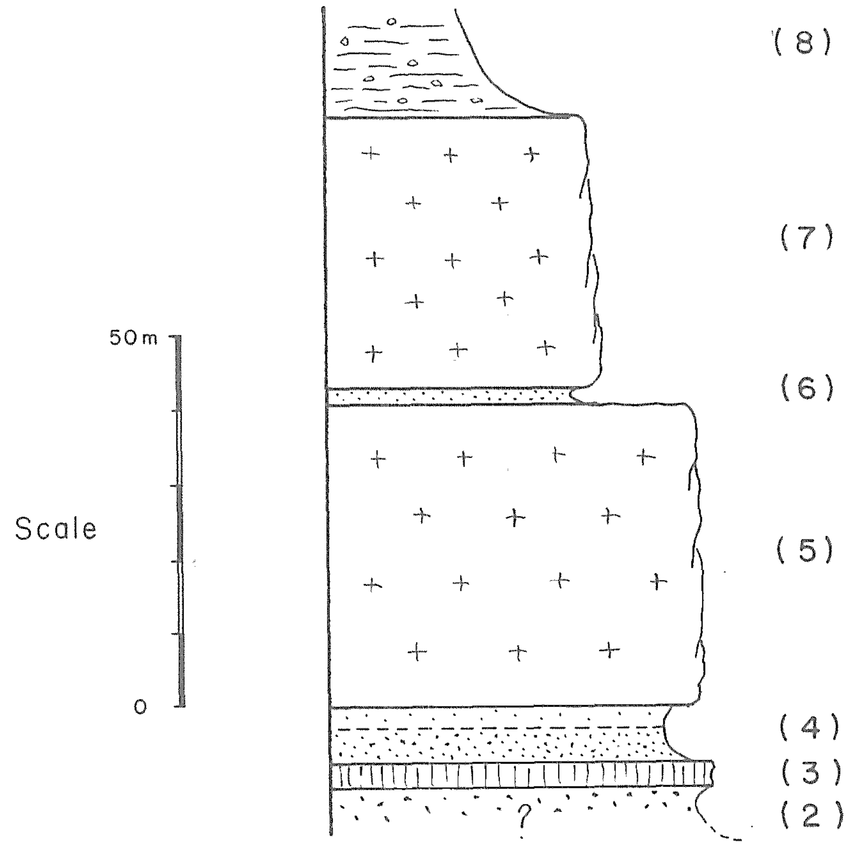
(7) An upper basaltic outflow generally well-bedded and prismatic, of regular appearance, but heavily fissured. Some levels are scoriaceous or cellular. The outflow is about 30 m thick. It consists of a fluidal dolerite, containing large microlites and serpentinized olivine, together with some big plagioclastic phenocryst.

(6) Intermediate bed of tuff with varying amounts of loam ; this tuff is friable, yellow at the bottom, red and sometimes very hard at the top ; the bed is

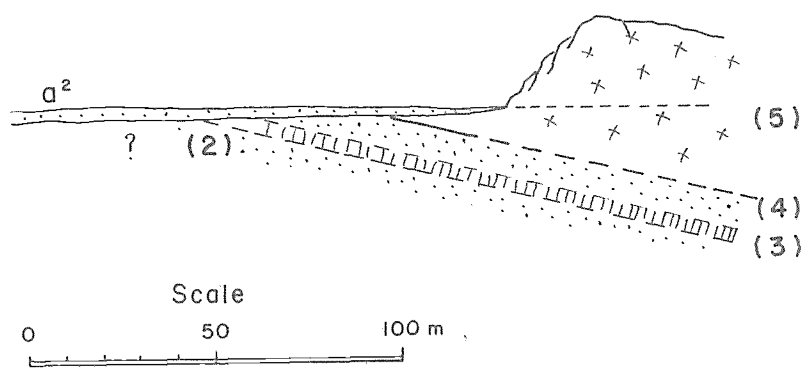
FIG.8

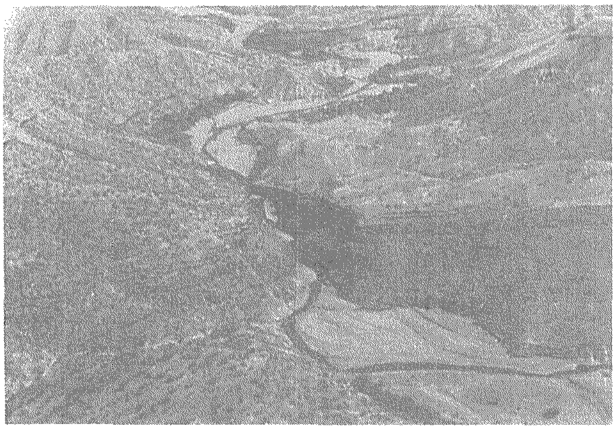
KEBENA DAM SITE

SCHEMATIC GEOLOGICAL CROSS-SECTION ESTABLISHED BY SURFACE SURVEY
(UPSTREAM LEFT BANK)



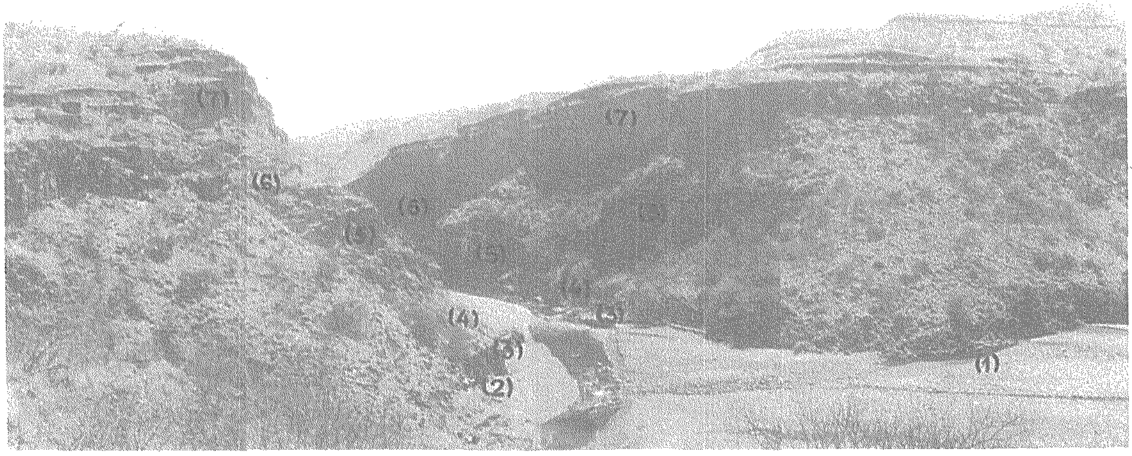
SECTION ALONG THE LINE OF DIP GOING DOWN.
THE THALWEG TO THE RIGHT BANK DAM ABUTMENT.





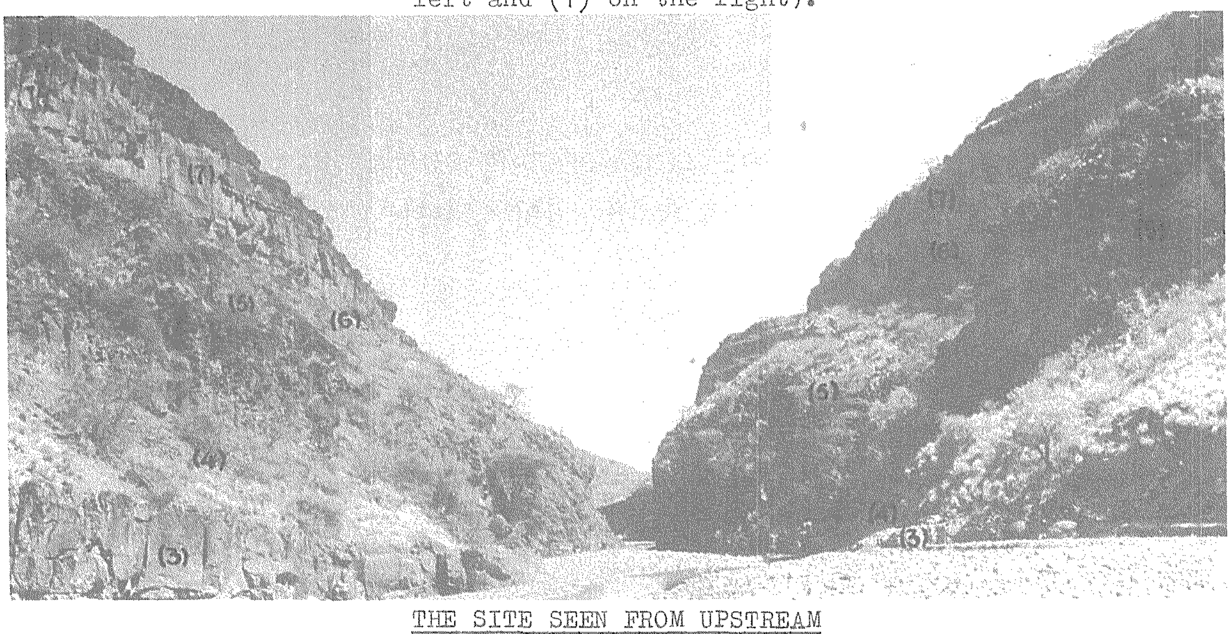
GENERAL AERIAL VIEW OF THE SITE FROM UPSTREAM

The narrow gorge runs between volcanic outflows set in a vast expanse of alluvial terrane forming the remainder of the region (above and below the gorge outflows).



GENERAL VIEW OF THE SITE, TAKEN FROM THE LEFT BANK UPSTREAM

Both the cliff formed by the high-level outflow and the intermediate-level outflow shoulder directly above the site are clearly visible. Note also the alluvium above the main volcanic outflows in the gorge.
The two supplementary formations can be seen in the foreground ((2) on the left and (1) on the right).



THE SITE SEEN FROM UPSTREAM

The site is in the narrowest section, where the substantial lower outflow has formed the distinctive shoulder seen on the right bank. Formations dipping deep underneath the site downstream are seen emerging on the left.

2 to 3 m thick. Analysis of a sample taken at the top from the part reburned by the upper outflow (7) gave these constituents in decreasing amounts : plagioclasic feldspars, calcite, illite, quartz and haematite.

(5) Large intermediate outflow, some 40 m thick, forming the shoulders at the bottom of the site. This is made up of a basaltic rock, which is generally strong but irregular (continuous half-bedding and single half-prismation) and fissured (due to shrinkage and tectonic action). This is a dolerite with a distinct intersertal structure, containing big microlites, titanium-bearing augite and a few large phenocrysts of plagioclasic feldspars.

(4) Ashy tuff level, from 7 to 8 m thick, consisting, at the bottom, of a fairly hard, pink or ochre, very light, brechoid tuff ; montmorillonite and feldspars predominate ; and, at the top, of a fairly coarse, white layer, which is not very coherent and breaks down completely in water ; despite the presence of granules, only montmorillonite is revealed by X-ray. Dip is such that this layer must pass fairly close under the bed of the river directly below the site.

(3) An approximately 3 m band of long-vesicle brechoid tuff, containing a few crystallized glomerules of plagioclastic feldspars, augite and haematite, together with a few scattered granules of quarty and feldspar. The rock is light, but strong, it is heavily fractured, perhaps because of the proximity of the big fault Fl.

(2) Somewhat friable grey and pink tuff, with small beds of ash (cinerite). This layer comes to the surface upstream, near to the river's edge on the left bank. Its thickness is not known. Here soft formations could be 5 to 10 m thick.

(1) Following a break in the series, due to the river and its alluvial deposits, there is a narrow escarpment of very hard, cellular, basaltic rock, at the foot of the slope, upstream on the right bank. Detailed analysis shows it to be a dolerite (basalt containing olivine and very big microlites) with intersertal structure. The olivine is present as numerous small granules. Small vesicles are filled with calcite. There is no pyroxene. This must constitute the sub-stratum of the previous layer, but its thickness cannot be ascertained as it is abruptly cut short by the big upstream fault Fl. Consequently, there is even less information regarding the bed-rock (alluvium?) of this volcanic series.

II - 3. Old Alluvial Deposits

The normal alluvial deposits covering the volcanic series appear to predominate on the left bank, where they form the whole hill up to the narrow volcanic band at the top. Deep gullies up the gorge show mainly fairly fine loams, which are often compact and various shades of pink. On the other hand, there is a stony alluvium, containing large blocks, above the escarpment on the right bank ; this is probably more recent, forming gullies through the previous layers. This formation must be much more permeable than that on the opposite bank. As a tributary thalweg joining downstream of the site cuts through it, the risks of leakage must be considered.

II - 4. Recent Formations

As the rock nowhere across the river appears above the recent alluvial deposits, there is no way of arriving at their thickness. To judge by the narrowness of the gorge and the tuff at the base of the series they may be fairly thick. The scree sometimes consists of large blocks and is consolidated by limestone tuff in many places. At the site itself scree covers the shoulders of the middle outflow, but does not seem very thick, particularly on the right bank.

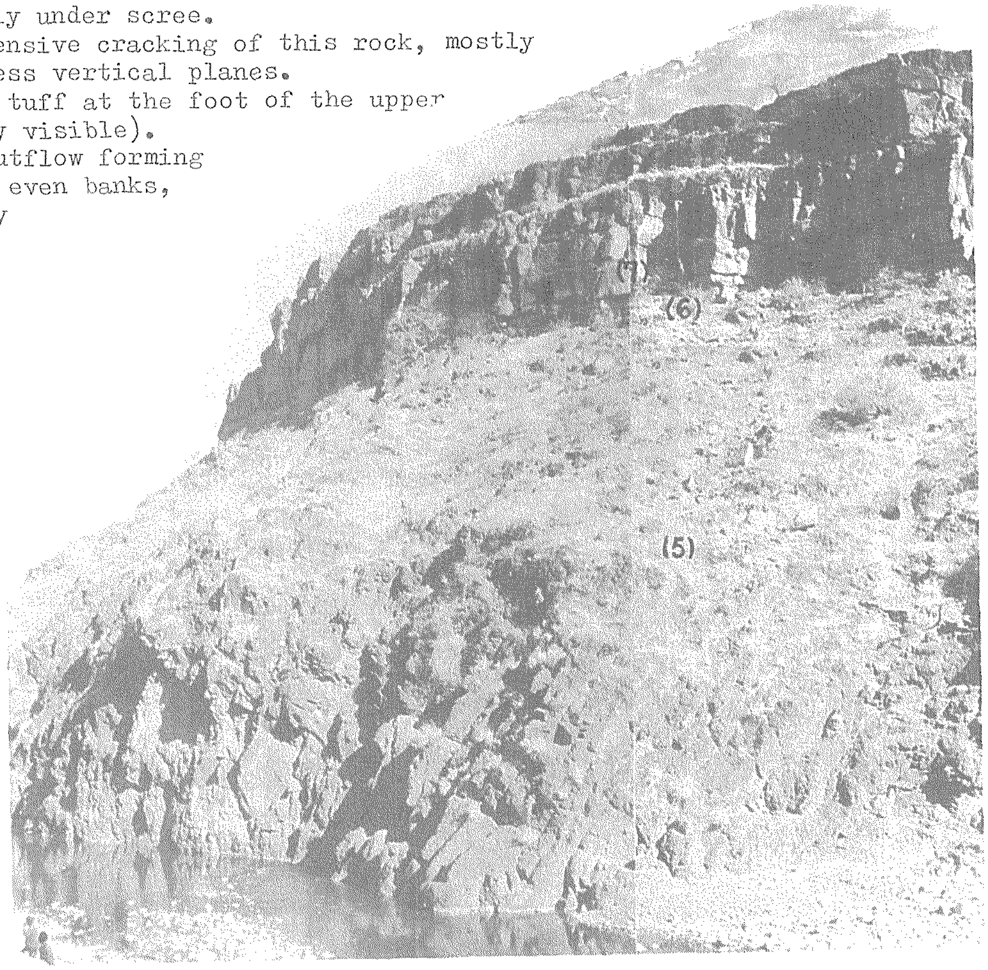
THE LEFT-BANK ABUTMENT SEEN FROM DOWNSTREAM

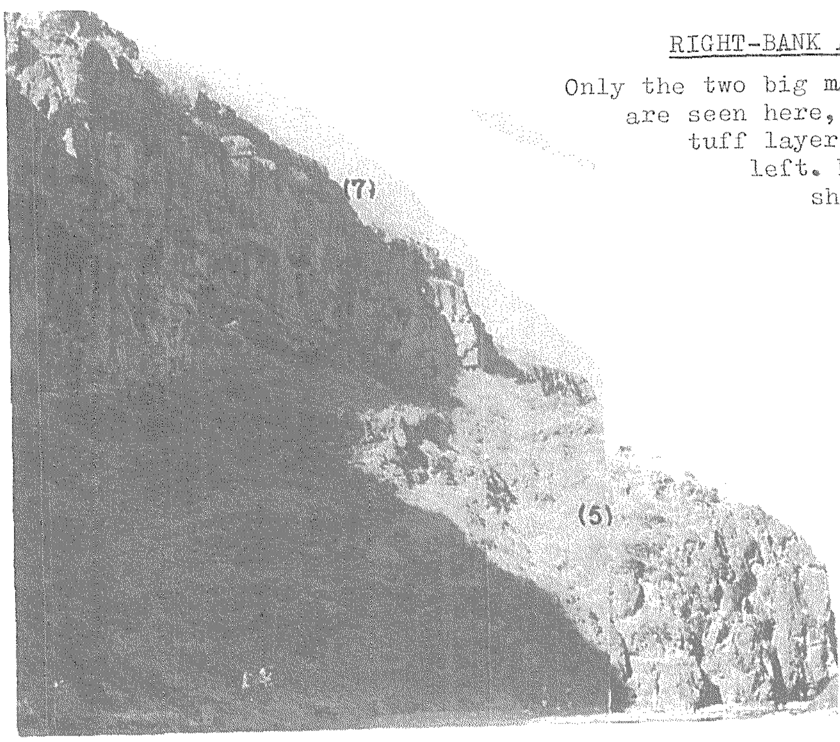
Only the three top formations are visible on the photograph :

(5) : An intermediate-level outflow forming a shoulder with its top surface partly under scree.
Note the extensive cracking of this rock, mostly in more or less vertical planes.

(6) : Clay-bearing tuff at the foot of the upper cliff (hardly visible).

(7) : High-level outflow forming thick fairly even banks, but with very pronounced vertical cracking.





RIGHT-BANK ABUTMENT

Only the two big main outflows (5) and (7) are seen here, with the intermediate tuff layer (6) visible on the left. Note the characteristic shoulder-shaped lower outflow.

II - 5. Tectonic Structure

Faults of varying extent are associated with the site, (see Fig. 9). The big upstream fault F1 cuts off the series up the gorge, while the strata dip regularly by 10 to 12° downstream under the dominating alluvium. Its throw is difficult to assess, but may be as much as several hundred meters. Its prolongation is clearly visible in the upstream thalweg on the left bank, where the volcanic rocks of the gorge reappear locally below the alluvium of the upstream section of the fault. The whole zone is fairly heavily fissured and losses might occur there.

Fault F2 upstream on the left bank worsens geological conditions at the entrance to the gorge ; on the surface, it produces a gully, revealing very fractured rocks, covered to a varying extent by scree.

Fault F3 is downstream from the site and has a very limited throw. Its main effect is a gaping fracture, mainly on the left bank, where it becomes a large vertical crevasse in the upper cliff. Above the ledge, it disappears below large-block alluvium.

As it is almost parallel to the axis of the dam, it should not give much trouble, unless it continues other fissures which might cause leakage.

II - 6. Foundation and Stability of a Concrete Dam

Even though the thickness of the recent alluvial deposits is unknown, they may extend to 10 or 20 m. The upstream tuff series must run not far below the actual site. This may complicate the problem of foundations. The middle outflow forming the shoulders of the site is irregular and criss-crossed by fissures. Despite the tenerally good appearance, there could be fairly substantial decompression of the rock; if an arch-dam were decided upon, it would be necessary to consolidate by grouting, which often proves effective in this type of fissured, hard rock. The same applies to the upper outflow, but to a lesser extent, because only the upper part of the structure would be concerned. The intermediate layer of tuff can easily be cut through during the excavations, so that the concrete can bed on the underlying basalt.

II - 7. Watertightness Around the Dam

Permeability is seldom great at depth in fissured basalts, but this should be confirmed by borings. Particularly on the left bank, the upper alluvial deposits may have protected the basalts against scouring underground flows, but this must be verified. In depth, in the underlying tuff formations, it should be possible to find an almost impermeable layer to which the grout curtain can be joined.

A lateral leakage is possible particularly on the left bank, due to faults F1 and F2. An oblique boring of the fault F2 would be helpful. If there were a substantial risk, the topographical survey would have to be continued into the small valley upstream on the left bank and borings taken to see if the grout curtain could be extended across fault F2.

II - 8. Watertightness of the Reservoir

The storage basin has high, steep banks, and the highest water will always be some distance from the saddles leading to adjacent valleys. The risk of leakage from the reservoir into other valleys seems to be very limited, even though some old alluvial strata may be fairly permeable.

II - 9. Program of Borings

(1) First priority. A 50 m boring along the thalweg, which will provide information on the depth of the alluvial deposits and indicate both the nature of the formations (adequate core-sampling of the soft tuff layers) and their permeability in the series underlying the basalts at the site. This boring could also be made on a river bank, the alluvial deposits being investigated by at least two borings on a line at right angles to the center-line of the river.

As decompression of the shoulders of the middle outflow appears to be a major difficulty in building an arch-dam, two borings should be made from the top ledges shoulders, i.e., midway between the two escarpments; the boreholes should penetrate several meters into the underlying tuff (about 40 to 50 m).

The continuity of the series around the site and variations in the permeability of the basalt in these directions must also be studied. Other borings should, therefore, be carried out far enough from the rock-wall to ensure that they are clear of the decompressed surface layer (about 30 m). They must go down at least to river level and, if possible, to the tuff below the basalts (approximately 60 to 70 m).

A boring running east-west, at an angle of about 60 to 70°, will be made at the upstream end of the upper basalt ledge on the left bank, to explore the area of the downstream fault. It should go down at least to river level (70 to 80 m).

(2) Second priority. If the lateral borings reveal very high permeabilities, two others must be made 100 to 150 m further outward (possibly not going so deep, because the slope of the ground would rapidly increase the "dead section" run through by the borings). This would apply to the area of fault F2 which might be retraversed deeper inside the hill, to ascertain whether permeabilities decrease in that direction.

III. SHORT HYDRAULIC STUDY OF THE RESERVOIR

III - 1. Reservoir Capacity

On the basis of the topographical survey and operating requirements, reservoir capacity is limited to the following values :

	Normal water level		Capacity (hm3)		
	Maximum	Minimum	Bottom storage layer (dead storage)	Live storage	Total storage
First solution	1260	1240	16.5	33.5	50
Variant	1270	1250	30.5	44.5	75

III - 2. Flow Cycle

The following figures, recorded at the Kebena hydrometric station, represent flows at the dam site :

Inflow (hm3)													
Date	O	N	D	J	F	M	A	M	J	J	A	S	Year
1962-63	8.5	6	4.5	1	2	2	10	15.5	3	42	114.5	28	237
1963-64	5	4	4.5	4	2.5	5	78	12.5	15	96	130	30.5	387

The second cycle is consistently higher than the first. As the proposed capacities are much less than the annual inflow, the dam must overflow during the flood season, and act as regular from October to June only. If we take only this nine-month period, it is seen that the first cycle is much more normal than the second, which is irregular in April. Operating calculations will be based on natural inflow from October 1962 to September 1963.

III - 3. Life of Dam

The Kebena hydrometric station recorded the following sediment inflows :

(i) October 1962 to September 1963 : 1,325,000 metric tons

(ii) October 1963 to September 1964 : 3,160,000 metric tons

The second figure is substantially affected by the exceptional figure for April 1964 (1,460,000 t). The expected annual sediment inflow can be put at about 1,500,000 t. Thus, if all the sediment were deposited permanently 1 hm3 of capacity would be lost annualy. On this basis, the theoretical life of the reservoir would be well under 100 years, so that the Kebena dam would be even less favourably placed than the Kesem dam. As with the Kesem dam, however, a large fraction of the sediment should not really contribute to permanent silting of the reservoir, either because it would remain in suspension or because it would be flushed away; long life for the dam would be favoured by the slope of the bed of the Kebena (1 in 100), the relative narrowness of the reservoir and the stirring of the water by annual floods in the rainy season.

III - 4. Flood Flows

The Kebena flood flows are discussed in Volume III, which estimates the 100-yearly flood at about 1000 m3/s. On this basis, a nominal flow of 1200 m3/s can be used for calculating the size of the dam's spillway. Too much reliance should not be placed on the dam freeboard for any substantial reduction of so high a flood, which may produce more than 100 hm3; indeed, the reserve capacity provided by a 5-meter section above the highest normal water levels is only 10 to 15 hm3. Embankments will be needed to prevent the Kesem-Kebena plain from flooding by the Kebena.

III - 5. Evaporation Losses

As the two sites on the Kesem and the Kebena are close together (15 km), the figures adopted for losses from the Kesem reservoir to atmosphere can also be used for the Kebena project. With an average water surface of 200 ha, total annual losses could be about 4 hm3.

III - 6. Operating Calculations

From October 1962 to June 1963, the reservoir would empty almost continuously, while inflow (less evaporation) over these nine months would be 49 hm3. Thus the volumes of water available for irrigation would be :

(i) First solution (crest at 1265 m),
49 + 33.5 = 82.5 hm3, allowing the irrigation of 6,000 ha.

(ii) Variant (crest at 1275 m),
49 + 44.5 = 93.5 hm3, allowing the irrigation of 7,000 ha.

We must keep a little below these maximum figures so as to generate enough firm power. Thus, for the first solution, the area to be irrigated has been limited to 5,000 ha. so as to allow a firm power output of 1 GWh/per month. This is verified by the following operating table (which uses the same symbols as the table for the Kesem reservoir).

1962-63	O	N	D	J	F	M	A	M	J	J	A	S
Z_1 (m)	1260.0	1260.3	1259.5	1257.2	1253.0	1248.6	1242.5	1241.7	1245.5	1239.2	1259.2	1260.0
V_1 (hm3)	50	50.5	48.5	43.5	35.5	28	19.5	18.5	23.5	15.5	48	50
A (hm3)	8.5	6	4.5	1	2	2	10	15.5	3	42	114.5	28
B (hm3)	5	5	9	8	8	6	6	9.5	10.5	5.5	4	7
R (hm3)	2.5	2.5	0	0.5	1	4	5	1	0	4	4	0.5
E (hm3)	0.5	0.5	0.5	0.5	0.5	0.5	0	0	0.5	0	0.5	0.5
A-(B+R+E)	0.5	-2	-5	-8	-7.5	-8.5	-1	5	-8	32.5	106	20
H (m)	60.15	59.9	58.35	55.1	50.8	45.55	42.1	43.6	42.35	49.2	59.6	60
P (GWh)	1.00	1.00	1.17	1.04	1.01	1.01	1.03	1.02	0.99	1.04	1.06	1.00

For the variant solution, a similar calculation proves that 6,500 ha. can be irrigated while maintaining the same firm power output of 12 GWh/per annum.

III - 7. Size of Power Station (first solution)

Calculations exactly the same as those for the Kesem dam give the following powers :

(i) Average power with minimum head (39.2 meters) :

$$W_{av} = 1,350 \text{ kW.}$$

(ii) Firm peak power : W_{peak} = 2,450 kW.

(iii) Installed power with average head (52.2 meters)

$$W_{inst} = 3,800 \text{ kW.}$$

The station will have two generating sets of 2,500 kW and 1,300 kW, respectively. Thus, the plant can be developed and operated more flexibly. Total flow through the plant is 9 m3/s with a head of 52.2 meters. A small additional dam would be needed to compensate for the water passed through the turbines. It might be located on the site of the Kebena measuring station.

IV. OUTLINE PRELIMINARY DESIGN

IV - 1. Choice of Type of Dam

The width/height ratio and the remarkable symmetry of the gorge suggest the construction of an arch dam. There are, however, many difficulties. Formations of doubtful strength might be found down to a depth of about 20 m below river level before the lower outflow is reached. The intermediate and upper outflows are separated by a horizon of no great strength and may perhaps have fairly dissimilar mechanical properties. The arch supports at several levels may behave differently. Some relative movement between the upper and lower parts could be disastrous for arch resistance. These problems could be solved only by detailed studies on the spot, in the laboratory and in the design office, going beyond the scope of the project.

For the moment we offer only an outline preliminary scheme, for a structure less well adapted to the topography of the site, but more reliable : a six-block buttress dam, with two blocks based on the lower outflow around the 1180 m mark and the remainder on the intermediate outflow at levels between 1210 and 1240 m.

IV - 2. Description of the dam (first solution : crest at 1265 meters)

From the left to the right bank, the layout is as follows::

(i) block 1, resting on the upper outflow rock mass ;

(ii) block 2, which is the first section with running buttresses; vertical upstream face, downstream batter of 0.75 and width of 22 m.

(iii) straddling blocks 3 and 4, the "ski-jump" spillway, with a capacity of about 1200 m3/s.

(iv) between blocks 4 and 5, a dewatering return conduit and the small power station. The dewatering conduit runs through block 4, the water intake and the power station penstock through block 5. Access is from the left bank, under the spillway.

(v) block 6, linking up with the upper outflow on the right bank.

IV - 3. Higher Dam (variant : crest at 1275 metres)

The crest of the dam can probably be raised by 10 meters. On the left bank, the 1275 m mark should be about 40 m from the escarpment edge, along the axis of the dam; on the right bank, which is a little lower than the left, the 1/1000 plan does not go far enough to determine the length of the extension. Additional expenditure would relate mainly to the highest blocks. It can be estimated at roughly 25 % on the cost of civil engineering works.

IV - 4. Summary Cost Estimate of Works (in Ethiopian dollars)

(i) First solution (crest at 1265 meters)

Civil engineering			
Cofferdams	1,500,000		
Rock excavation 5 x 80,000 m3	400,000		
Concreting 85 x 135,000 m3	11,500,000		
Extra for reinforced concrete 65 x 20,000 m3	1,300,000		
Grouting	1,500,000		
			16,200,000
Electrical and mechanical equipment			
Dam : Flood spillway	434,000		
Dewatering conduit	129,000		
Intake	66,000		
Gantry	85,000		
		714,000	
Power station :			
2 turbines	242,000		
2 alternators	522,000		
2 main butterfly valves	36,000		
Overhead crane	53,000		
2 transformers	179,000		
Switchyard	240,000		
Auxiliary services	251,000		
		1,523,000	
			2,237,000
T O T A L			E$ 18,437,000
Contingencies, design and supervision of work: 30 %			5,531,100
G R A N D T O T A L			E$ 23,968,100

(ii) Variant Solution (crest 1275 meters)

Civil engineering	20,000,000
Electrical and mechanical equipment	2,237,000
TOTAL	22,437,000
30 %	6,731,100
G R A N D T O T A L ....E$	29,168,100

IV - 5. Conclusions

These estimates reinforce the arguments in Chapter I against a dam on the Kebena. The variant solution can be ruled out straightaway, because it involves investing a further E$ 5,200,000 to extend the irrigated area by only 1,500 ha. Comparison with the figures for the Kesem dam shows that even the first solution is too costly.

	Kebena (1265)	Kesem
Cost (E$)	24,000,000	36,300,000
Irrigated area (ha)	5,000	22,000
Electricity generated(GWh/per annum)	12	42.5

Failing some economic means of carrying water to the Bolhamo plain (which cannot be supplied easily from the Awash), the Kebena dam could only serve the area of some 5,000 ha north of the lower Kebena. The scheme would be of value only if the Kesem dam supplied only 13,000 ha. in the Kesem-Kebena plain and if the area which it could serve in the other irrigable parts of the Middle Valley were increased to 10,000 ha.

It would be difficult and costly to draw off water flowing over the Kebena dam because a very big and vulnerable structure would have to be built to divert water from the wider river downstream, and also because the establishment of an intake higher upstream - e.g., on the site of the gauging station already considered for a compensating dam - would require a long and delicate feed along the foot of the escarpment on the left bank.

CHAPTER V. REGULATION OF AWASH FLOWS AT TENDAHO

I. IMPORTANCE OF THE TENDAHO SITE: TOPOGRAPHICAL FEATURES

The large amount of good potential farmland in the Lower Plains of the Awash was indicated by the irrigability study, which estimated that nearly 70,000 ha of this land are in classes II and III. Irrigation is essential for its development, but is at present practiced only on a fraction of the overall potential farmland reconnoitered, and only by intermittent extensive and often rudimentary flood water spreading methods. Present farmland is exposed to unpredictable flooding and particularly active forms of river instability. The only permanent solution is the fullest possible regulation of flow conditions in the Awash, to ensure :

(i) New flow conditions capable of meeting irrigation requirements ;

(ii) Damping out of flood peaks to limit river overflowing and bed shifts.

These measures imply erecting a dam to form a large reservoir, integrating most of the flows affecting the Lower Plains and adequately commanding the entire region. It is remarkable that a single site running through the village of Tendaho and directly accessible from the Aseb road can meet all these conditions at once. It is suitable for erecting a main dam running north-west to south-east, with its left-bank extremity abutting against the hillside, on which the village is built, and its other end at a long narrow spur jutting out into the river from the right bank.

A 1:1000 scale topographical survey of this site area was carried out for geological survey and civil engineering design requirements. It extended a fair distance beyond the river banks in areas likely to give rise to leakage problems. Heights on the map produced are given as relative altitudes (i.e., the altitude of Tendaho village above sea level is 409 m).

As this map shows, the hillsides to either side of the river fall away fairly abruptly, so that the 'gorge' only increases from 200 m in width at the foot of the hills at level 575 m to 380 m at the plateau on top of the hills at level 615. Thus, even a dam rising to a height of 40 m above the river bed would still be comparatively short along its crest and assume excessive length proportions only if built to above the 615 m level. If the top of the dam were above the 600 m level, a secondary dam would be needed across the saddle at Tendaho village.

Thus, it was essential to establish the relationship between storage capacity and dam height. This information was ascertained by photogrammetric plotting of the aerial photographs to produce a 1:20,000 scale map of the reservoir area. It showed that a dam impounding at a 600 m level would produce a reservoir extending 30 km upstream and covering the Mile river confluence, as can also be seen from the reservoir filling curves plotted. A few significant values are listed in the table below.

Relative altitude (m)	Area (km2)	Capacity (hm3)
596	63.5	376
598	81	516
600	104	716
602	131.5	970
604	165 (approx.)*	1300 (approx.)*
606	210 (approx.)*	1800 (approx.)*

* Extrapolated.

These figures indicate that if the normal storage level exceeds about 605 m, the storage volume might be far greater than necessary for flow regulation purposes. Though the examination indicates that the highest storage levels are likely to be below 605 m, only a complete reservoir operation study allowing for available water supplies, water requirements, evaporation losses and flood damping can determine the highest level it would be uneconomical to exceed; i.e., the optimum storage capacity of the reservoir.

II. SURFACE GEOLOGY SURVEY

II - 1. General Structure of the Region

Although the ground in the Tendaho region resembles that around Kesem and Awash station in that volcanic outflows predominate compared to the ground elsewhere (alluvium or tuff), it has a peculiar broken-up multiple-parallelogram relief, which is clearly visible on the aerial photographs. One fault system running south-east to north-west has produced elongated hills, which are run through roughly from north to south by a second fault system (especially west of Tendaho). Though these faults show up very distinctly in the general structure, they are fairly difficult to locate accurately on the ground, as deep scree layers cover the hillsides and the thalwegs have filled up with more recent tertiary and quaternary deposits. Here too, these major faults have produced hot and boiling water springs (e.g., the Alala-bada geyser).

II - 2. Volcanic Formations

These form the 'skeleton' of the entire region and, though traversed by faults, appear to feature homologous strata sequences (at least near the dam site, and especially to the east of the road).

The best, most complete section is observed in the low hill between the Awash and the Tendaho saddle, where the following formations are visible in their order from top to bottom. (See Fig. 10)

(7) An upper outflow, forming an overhang towards the Awash above the bend in the track up the hill. This is hard scoriaceous-base black rock with fairly pronounced prismatic and cellular features and very numerous cracks on the surface. Examination of thin plate samples revealed fine-grained basalt, except for a coarse corroded plagioclase from a first consolidation ; calcite occurs around the cavity edges and also in diffused zones.

(6) An intermediate level which, at the same bend in the track, consists of scoria and indistinct loamy brown tuff with white calcareous concretions, red with evidence of reheating near the top.

(5) A second black outflow, with a yellowish patina, which is frequently both stratified and prismatic. This is fine-grained basalt with a microlitic tendency (seldom with olivine, non-cellular, fairly frequent glass, magnetite).

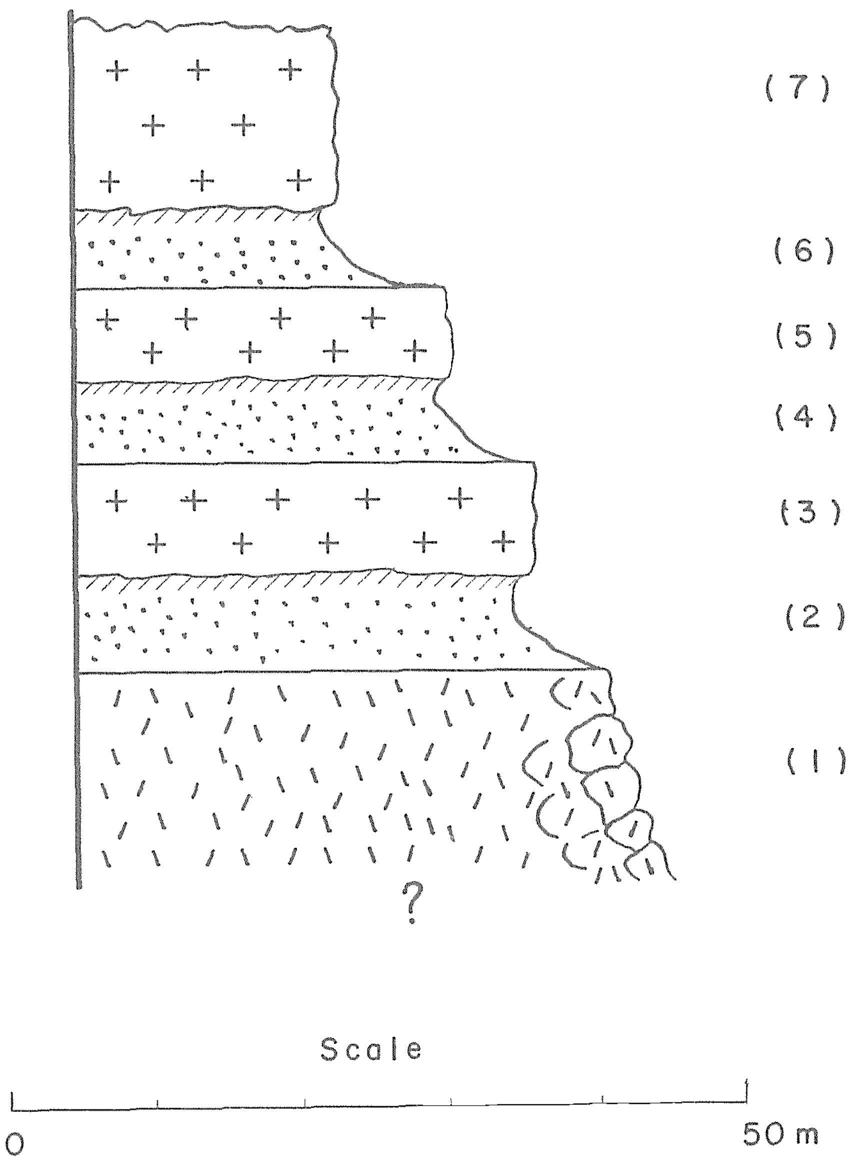
(4) A second soft intermediate level, which cannot be identified completely as it lies under a scree embankment. It seems to consist mainly of scoria, but may also contain various types of tuff.

FIG. 10

TENDAHO DAM SITE

Schematic Geological Cross-section established by surface survey

(small hill between the Awash and the saddle at Tendaho)



(3) A third outflow, of very compact black rock, thicker and more massive than (5), with more pronounced cracking on the surface. A scoriaceous level is observed at its base. This is basalt with a doleritic tendency. Thin plate samples were found to contain irregular zones with a very finely crystallized, partly vitreous, ground mass of darker colouring containing small microlites ; the rest is lighter in colour and contains big microlites, less magnetite and practically no augite or olivine.

(2) A third soft intermediate level which, though not actually visible in the section, is probably the same as the one observed in a small cave above the road pass. This consists of ochre or light brown clayey tuff with a distinctly red zone in its upper portion, which has been metamorphosed by the third outflow (3). X-ray analysis reveals mainly illite and montmorillonite.

(1) The bottom outflow, which differs considerably from the others. The rock is invariably dark in colour, coarsely granular and breaks up into large balls. Except for their dark colouring, these outcrops are rather like coarse granite in appearance ; but thin plate sample analysis shows, that they invariably consist of fine-grained basalt with small augite granules, occasional olivine, a few calcite-filled cavities, and fairly frequent glass.

This level (1) also occurs along the right bank downstream from the dam site, where it is about 20 - 30 m thick, more or less prismatic at its base, and overlies tuff.

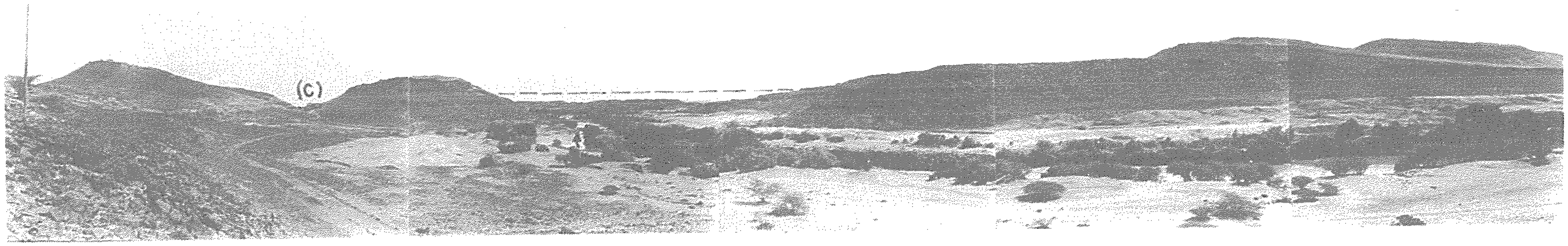
Though the intermediate layers above the dam site on the right bank are hidden from view, the small cornices in the upper portion of the abutment zone seem to be of the same material as those on the left bank. The deeper intermediate layers (1), (2), and possibly (3) disappear underneath the scree and alluvium.

Examination of the strata appearing on the surface west of the Tendaho saddle reveals an isolated outflow (9) over the upper outflow (7), consisting of medium-grained dolerite with a distinct intersertal tendency, which is partly concealed by numerous cavities. It contains numerous augite and olivine granules, also glomerular plagioclase feldspar, fairly large quantities of clear glass, and magnetite or intersertal ilmenite.

A different series occurs along the first kilometer south of Tendaho, featuring two main outflows ; one below, and the other above the road. Both resemble the upper outflows. A soft level runs between them at the road, which varies in thickness up to about 4 - 6 m and mainly consists of light brown loam with lime concretions and a generally very red upper portion (contact metamorphism). This level faintly resembles level (2) in the typical section, but its bottom run differs considerably from level (1). As this is not the same sequence, a fault must run through the Tendaho saddle.

II - 3. Recent Sedimentary Formations

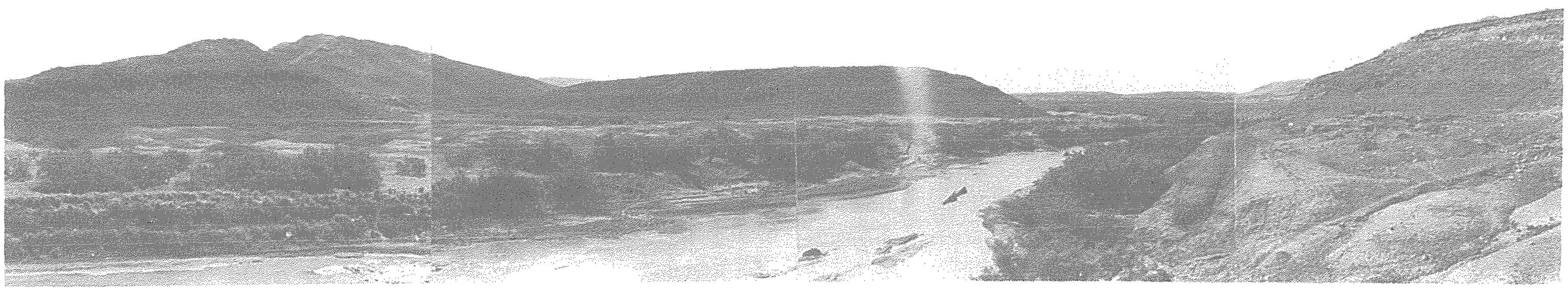
A basically sandstone series, the colour of which varies from red (frequently) to yellow (occasionally), occupies the entire downstream part of the dam site area (a small gorge in the Awash about 1 km north of the actual site). These formations bear directly against the north-eastern hill slopes in the site area (by direct transgression, not by faulting) and generally end up as a completely silicified very hard horizon forming a distinctive little flat-topped cornice (mostly at 600 - 605 m relative altitude). The formations filling the large left-bank tributary thalweg west of the road are probably also part of this series ; the deposits associated with them are generally of a finer loamy material, but these formations too end up as the same peculiar silicified crust. Between the two formations, sandstone forms isolated outcrops in the bed of the Awash or in its banks, in the dam site area. This level may be of Pliocene origin (late Tertiary).



GENERAL VIEW OF THE SITE FROM THE ROAD

The saddle over which the road passes and where an ancillary structure will be required is at (C) on the left, followed from left to right by :

- the hill forming the left-bank abutment, with the various basalt outflows showing through the scree ;
- the actual Awash valley and its loam terrace ;
- the right-bank abutment, with its outflows only just visible under a substantial scree covering.



THE SITE SEEN FROM DOWNSTREAM

The essentially loam terrace is seen on the left, and in the bed of the Awash sandstone banks accompanying the clay or loam strata clearly visible on the right. The subsidiary right-bank tributary valley isolating the hill providing the abutment on this side is visible on the left.

A generally grey-coloured terrace of loam, coarse sand occasionally consolidated to sandstone, and gravel. Downstream from the site, this terrace 'dovetails' into the above sandstone formation ; it also occurs at the foot of the hills up-river from the dam site. This is presumably of early quaternary origin ; it frequently extends up to 600 m relative altitude. Where this loose material has been carried away, traces of the original level are still observed as an 'onion-skin' type of tuff encrustment ; e.g., all the way along the right-bank hillside and at the foot of the left-bank abutment.

Light brown loam forming several terraces at lower altitudes along the banks of the Awash. The topmost terraces frequently show evidence of severe ravine erosion.

II - 4. Tectonics

The strata do not dip very much in the general site area. At the actual dam site, they dip slightly from Tendaho towards the right bank, so that the lower strata disappear below the surface ; they then run roughly level all the way up-river along the right bank. West of the road, they initially run level in the long straight stretch south of Tendaho, then rise towards the north-east in the hill on the western side of the Tendaho saddle. This non-conformity indicates a fault hidden under quaternary alluvium in the upstream part of the area and running along the small thalweg from Tendaho to a point upstream from the dam site. Another fault probably follows the straight upstream scarp of the elongated hill on the right bank and then crosses the first fault near the Tendaho saddle. This explains the breaches visible from the road on the western and north-western sides of the saddle.

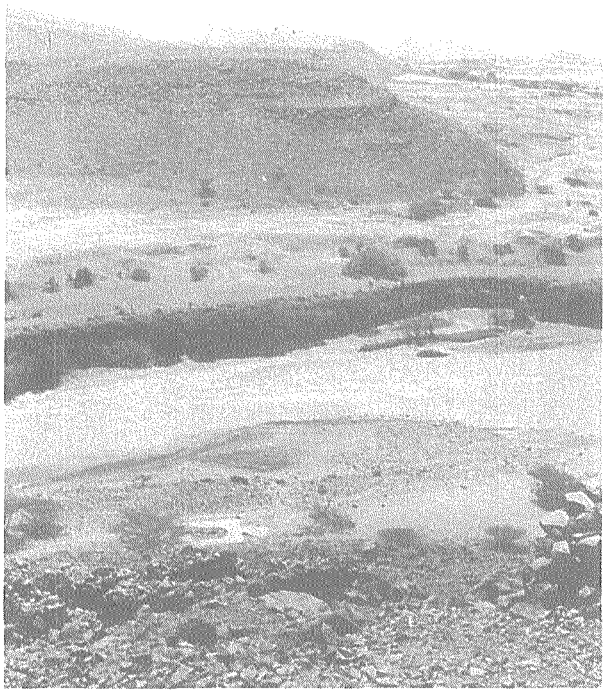
At least two faults in a bundle running from north-east to south-west intersect the western series at the small road bridge about 200 - 300 m south of Tendaho. They are probably the same as the faults observed about 500 m up-river from the dam site at the small saddle on the elongated hill on the right bank. Though faults are numerous in this area, none appear in the Awash at the actual dam site. Differences observed in the two abutments appear to be due to the dip of the strata. The boreholes should provide further information.

III. TECHNICAL CONSEQUENCES FOR THE PROPOSED STRUCTURE

III - 1. The Main Dam

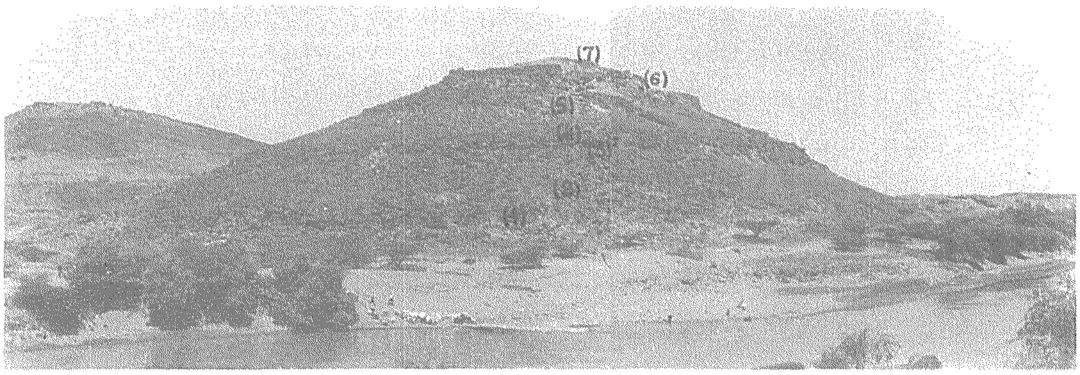
With a 'flexible' structure (e.g., an earth or rock fill dam), there should be no appreciable difficulty with the hillside abutment foundations, despite the tuff levels in the intermediate formations between the outflows. It will merely be necessary to provide suitable protection for these soft formations, so as to prevent them from breaking up either during the excavation work or by underground erosion. The problem can be solved with grout curtains. Between the hillside abutments, on the other hand, only a few recent sandstone formations have been identified in the river bed. The depth at which the volcanic rock lies and the thickness of possible sandstone and clay formations above it are unknown, and can be established only by the borehole survey. The right bank is taken up by a wide band of quaternary loam which becomes extremely plastic when wet. This material will have to be removed from underneath the foundation, but the quantities involved can be determined only by the borehole survey.

The leakage problem around the site is similar to that associated with the Kesem dam ; it depends on the permeability of the deeper volcanic outflow formations, knowledge of which depends on the borehole surveys.



THE RIGHT-BANK ABUTMENT SEEN FROM ACROSS THE RIVER

This view clearly shows the terrace essentially of light brown loam on both sides of the river. The actual abutment consists of alternating hard black basalt and softer tuff layers, with only the former apparent under the scree covering.



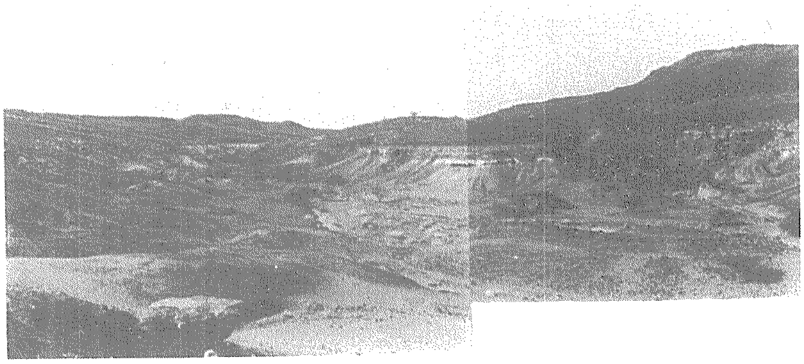
LEFT-BANK ABUTMENT HILL

A loam terrace and alluvial sand are seen in the foreground, and behind these the following, from bottom to top :-

(1) The bottom outflow of weathered globular basalt.
(2) A tuff or slag bank. (3) A thick basalt outflow.
(4) A tuff bank under scree. (5) A basalt outflow.
(6) A tuff and slag layer visible at the bend in the road.
(7) The top basalt outflow.

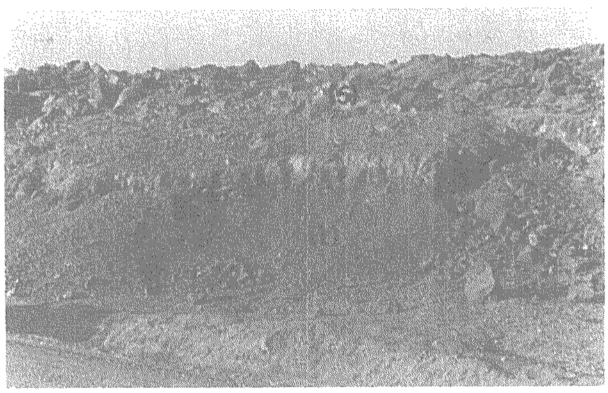
THE ROAD SADDLE SEEN FROM UP-RIVER

Though the ground is hidden under scree, a basalt outflow can be seen above the road and an intermediate tuff layer above the houses.



THE SUBSIDIARY LEFT-BANK TRIBUTARY VALLEY

This small valley may cause some leakage towards the saddle in the background, but is of ancient origin and has filled up with practically impervious clay loam (probably Tertiary) under silicified limestone (note the small cornice).



A RED CLAY TUFF FORMATION BESIDE THE ROAD

This is an example of the soft layers (t) separating the various basalt outflows (b). Such formations are seldom visible as they are almost invariably concealed by basalt scree.

III - 2. Secondary Structure at the Tendaho Saddle

This site is difficult to prospect because the ground is almost completely hidden from view by the road, houses and a scree covering. If one or two faults run through the area, the ground may be very pervious, possibly with piping where the faults have filled in. Though this difficulty may be overcome by grout curtains, it needs serious attention.

It is also necessary to determine the depth of loose surface ground (e.g., scree or alluvium) to be removed from underneath the secondary dam foundations.

III - 3. The Reservoir

The boreholes each side of the reservoir between the two dams will give a first indication of its watertightness. Even if it proved necessary to provide an unbroken grout screen between them, the project should still be practicable. A more comprehensive study of the question should be considered later when the results of the initial borehole work are known. Definite leakage risks are expected in the elongated hill on the right bank, in which the strata run roughly horizontal. They are under water on their western side, and leakage is likely to occur into the thalweg forming the downstream foot of the hill, and even more so near the small saddle with two faults about 600 m from the Awash. If the leakage path were longer, the rock might become more compact with depth than in its very broken-up surface region, so that it would be less permeable on the whole, but the leakage path is short, even at the 600 m level. The upper outflow is in very broken-up condition all over the small plateau it has formed.

The most important thing is that this problem appears to involve the full length of the hill ; i.e., a distance of 1500 to 2000 m ; priority will be given to investigating the immediate surroundings of the actual dam site and the small saddle, which is the point where, at the moment, there is the most uncertainty.

Tuff crusts are found right up to 600 m relative altitude ; though they are mainly associated with the scree, they may have some effect in reducing leakage from this side of the reservoir.

The left-bank tributary thalweg saddle is at a relative altitude of about 610 - 615 m. The material filling this area up to about this altitude, which is probably of Tertiary origin and mainly consists of loam with varying proportions of clay, may be sufficiently impervious to prevent local reservoir leakage, especially around the 600 m level.

III - 4. Material for the Main Dam.

This question should be given priority consideration. If it is impossible to obtain certain types of material near the site, this may affect the choice of structure adopted for the main dam. The more or less recent alluvium along the Awash upstream from the site might be worth considering for the core of the dam, although it becomes very plastic under water and generally shrinks considerably; it may not lend itself well to compaction. This point should be examined when assessing the available quantities. The thickness of the alluvium layer could be established by a geophysical survey or by borehole measurements.

There are few prospects of finding alluvial material suitable as fill. The recent alluvium contains loam, and the older alluvium is more sandy than stony and invariably at least partly consolidated as sandstone or pudding stone.

It will be necessary also to consider the possibility of a rock fill dam. No immediate precise prospection work would be required as there should be no difficult in finding a suitable basalt rock borrow near the site. One could even consider material from the extensive scree coverings on most of the valley hillsides. It consists mainly of fragments of the hardest rock bars. A rock fill dam with an impervious blanket upstream should be technically feasible even if the loam turned out to be unsuitable for use as core material for an earth structure.

IV. RECONNAISSANCE WORK PROGRAM

The positions of all the proposed borings are shown on the 1 : 20,000 scale general map. They are :

IV - 1. Right-bank Hill

(i) No. 1 borehole at the center of the first small plateau, between the site and the small faulted saddle.

(ii) No. 2 borehole in the faulted zone of the small saddle, to establish any abnormally higher permeability rates than in neighbouring borings, and hence signs of shorter leakage paths to the thalweg below.

(iii) No. 3 borehole, also on the plateau, about 300 - 400 m beyond the saddle.

These three boreholes should extend down to Awash level; i.e., to a depth of about 40 m.

IV - 2. Main Dam Site

(i) No. 4 borehole on the upper shelf in the right bank hillside, about 30 m inside its edge to keep out of the uncompressed surface zone. This borehole start at the 610 m relative level and be taken down to 10 m below Awash level; i.e., to a depth of 40 - 50 m.

(ii) No. 5 borehole, also on the right bank shelf, will serve the same purpose as a deep borehole in the river, as its position will be roughly halfway between the two abutments. To be taken to a depth of about 40 - 50 m to reconnoiter all the strata; i.e., loam, sandstone and, possibly, volcanic rock.

(iii) No. 6 borehole starting at the 610 - 615 level on the left bank (neither too near the river bank, nor on top of the hill, where much 'dead' material would have to be drilled through). To be taken to at least 10 m below Awash bed level; i.e., a depth of 40 - 50 m.

(iv) Two shallower borings, in line with the dam like the above, to provide additional information on the surface ground between the two hills. One (No. 'a') to be sunk in the river, and the other (No. 'b') between borings Nos. 4 and 5 on the right bank.

It was also proposed to reconnoiter the inter-outflow strata, or if this proved impracticable, to excavate test pits starting as close as possible to the upper outflow and extending down to the bottom one.

IV - 3. Tendaho Saddle

The three following boreholes were necessary to reconnoiter this complex area:

(i) No. 'c' borehole 10 - 15 m above the road to the west of the saddle.

(ii) No. 'd' borehole between the road and the houses on its eastern side.

(iii) No. 'e' borehole in the hill on the eastern side of the saddle, starting 15 m above the road.

These boreholes would be sunk to at least 20 m below the level of the saddle, except for the middle one, which should extend to a depth at which sufficiently low permeability values are found. In addition to studying the foundation problems for a structure which will probably not be high, these boreholes would serve to investigate permeability and the possibility of deep and lateral leakage.

IV - 4. Upstream Thalweg Saddle on the Left Bank

A borehole about 30 m deep (No. 'f') would be useful in more closely establishing permeability in the flat part of the saddle, also the existence of comparatively impervious tertiary or quaternary formations (if any) underneath the superficial scree.

As on the Kesen and Kebena sites, the purpose of this reconnaissance work was to establish the technical feasibility of the project, before a final scheme is worked out. Provision have of course to be made for additional investigations.

V. BOREHOLES AND TESTS

In the event, because funds did not suffice, only part of the program was executed. Six reconnaissance boreholes (Nos. 2, 3, 4, 5, 6 and 'a') and three test pits were put down. Their positions are shown on the 1 : 1,000 scale geological map. However each borehole was planned to enable an initial approach to be made to the most important geological problems while making full allowance for available data, especially those obtained from previous borings. The lithological sections of these boreholes and permeability test results are shown on the drawings with this report.

The boreholes supplied information on the nature of the volcanic and alluvial formations, layer thicknesses and the relationships between the different types of formation. All this information is featured on the general cross-section through the site, which also shows diagrammatic sections for all six boreholes.

Borings No. 2, 3 and 4 were sunk in the narrow elongated hill running along the right-bank side of the reservoir. Boring No. 6 concerned the small hill between the Awash and the Tendaho saddle on the left bank. These boreholes ran through superimposed basalt and soft material layers. Borehole No. 4 also ran into the alluvium at the base of the volcanic formation.

V - 1. Basalt Outflows

Their thickness usually varies between 3 m and 6 m, but may exceed 15 m in extreme cases. The rock is generally sound and compact, but contains some horizons honeycombed with small pea-size cavities. They do not appear to be in communication with each other or to increase the permeability of the formation.

Fairly numerous fractures run through nearly all the basalt banks ; there are sometimes so many that the rock is reduced to a heap of disjointed fragments. This was most noticeable in borehole No. 2, where the particularly pronounced state of dislocation of the basalt bedrock must be due to a nearby fault. Cavities in a few horizons prospected (borehole No. 2 from 0 to 6 m) were partly filled up with calcite deposited by infiltrating water.

The core samples do not show any prevailing direction of fracturation. From the lithological point of view, few differences are evident by direct visual observation between the basalt in the successive outflows, except for a bank with a rather 'gritty' appearance and a few big feldspar crystals scattered about in the basalt ; this latter type of basalt is the one in outflow (1) ; i.e., the bottom formation reconnoitered during the preliminary survey. A new basalt unit with a fine intersertal structure was identified underneath it in borehole No. 6, which is shown as (- 1) on the cross-section.

V - 2. Intermediate Levels Between Outflows

Their thickness varies between 2 m and about 10 m ; i.e., roughly as for the basalt outflows. Comparison of the overall extents of both types of formation along a given vertical shows that the basalt predominates on the right bank, where it accounts for 60 to 70 % of the overall depth of ground reconnoitered in boreholes Nos. 2, 3 and 4 ; but borehole No. 6 in the left-bank hill ran through roughly the same thicknesses of basalt and intermediate formations.

Slag and volcanic ash in various degrees of consolidation are the most frequent forms of rock in the intermediate formations. The top levels in the left-bank hill, plainly visible in the test pits dug there, consist of heaped slag or scoriaceous basalt fragments roughly the size of a fist. The spaces between them have filled up with pulverulent materials sometimes difficult to identify ; e.g., whether volcanic ash from eruptions, or alluvial loam deposited by wind. Abundant quantities of calcium carbonate in certain horizons support the second assumption. Elsewhere, the intermediate levels also contain soft tuff, frequently in a reddened condition, cinerite and eruption materials such as lapilli, which form beds at most 2 m thick between the slag horizons.

V - 3. Reconnaissance of Recent Sedimentary Formations

Boreholes Nos. 5 (55 m) and 'a' (30 m) in the high-water bed of the Awash, ran through formations of alluvial origin with no sign of volcanic rock. They are grey loam with varying proportions of clay and containing intercalations of fine sand partly transformed into soft sandstone, and stone or rounded pebble beds with a few conglomerate banks. Most contain lime which, where present in greater quantities in certain horizons, forms the cement bonding the sandstone and conglomerates.

V - 4. Permeability Measurements

Infiltration tests were carried out under pressure or at ambient conditions in all the borings to ascertain the permeability of the ground. The permeability coefficient K can be applied for alluvium and porous volcanic ground. Basalt rock itself is impervious, but the basalt banks have as a whole a certain water-conducting capacity because of their fractured condition. As the infiltration tests were carried out on vertical sections several meters in depth, this rock can be considered permeable to such a scale and given a permeability coefficient as for a porous medium. Approximate values of coefficient K measured in the boreholes are listed below.

(i) Volcanic ground

No. 6 - Surface to 15 m : 5×10^{-7} to 10×10^{-7} m/s
15 m to bottom (49 m) : 10^{-7} to 3×10^{-7} m/s

No. 4 - Most horizons : 2×10^{-7} to 4×10^{-7} m/s

Higher values were found in a few horizons, but never more than 2.6×10^{-6} m/s.

No. 2 - Values ranging between 5×10^{-7} and 2×10^{-5} m/s, the highest rates being measured in the basalt banks.

No. 3 - Surface to 35 m : 10^{-7} to 5×10^{-7} m/s
35 m to bottom (47.5 m) : about 5×10^{-8} m/s.

(ii) Sedimentary formations

No. 5 - 10^{-6} to 6×10^{-6} m/s

No.'a'- 6×10^{-7} to 4×10^{-6} m/s

No. 4 - Alluvium horizon (50.5 - 55 m) : 1.4×10^{-7} m/s.

V - 5. Geology of Dam Foundations

The results of the borehole survey have led to a clearer view of problems associated with the dam foundations and the impermeability of the ground in the vicinity.

The two hills against which the dam would abut consist of very heterogeneous rock, with soft occasionally friable rock, such as tuff, ash, slag, and even alluvia, alternating with hard basalt outflows. This structure is clearly visible on the surface on the left-bank hillside and has also been identified in the boreholes in the right-bank hill, where the successive strata are largely hidden under scree.

No volcanic rock was encountered underneath the high-water bed of the Awash, but material of undoubtedly alluvial origin was found, such as sand partly bonded with a calcareous cement, loam with varying proportions of clay and horizons containing large quantities of gravel or rolled pebbles. This ground lacks consolidation and does not feature any rock of sufficient strength to be a foundation for a concrete structure.

V - 6. Leakage Problems

(i) Near the dam : few leakage problems are associated with the ground underneath the Awash, as it is practically impervious. Though not especially high, permeability rates in the volcanic hills on both river banks and in the abutment zones are sufficient to require grout curtains at least in the more pervious upper ground layers (K varying between 2×10^{-6} and 3×10^{-6} m/s).

(ii) Reservoir right-bank : the fault running through the small saddle at borehole No. 2 has produced a dislocation area, so that permeability rates in the local basalt outflows are bound to be fairly high (maximum K about 10^{-4} m/s) ; a grout curtain would be necessary in this area. Farther south, borehole No. 3 showed that the elongated hill on the right bank is comparatively impervious beyond the small saddle. Permeability rates between this saddle and the Awash are not known as funds did not permit the carrying-out of borehole No. 1 ; a grout curtain may also be necessary in this hill, but probably need not extend for more than a short distance south of the saddle.

(iii) Reservoir left-bank : the special leakage problem associated with the Tendaho saddle was clearly shown up by the preliminary survey. Lack of funds ruled out the three boreholes which had been proposed for this area, as well as borehole No. 'f' in the left bank tributary thalweg saddle. The work required to seal the left-bank area is still largely a matter of conjecture.

(iv) Deep infiltration : under present conditions, seepage from the Awash merely shows up as capillary rise, which sustains narrow strips of vegetation along both river banks. The nearly impervious alluvium below about 8 m depth underneath the river prevents any major seepage through to a greater depth. This explains why no aquifers were encountered, except in borehole No. 'a'. which ran through sandstone banks in communication with the river at shallow depth. If a dam is built, the reservoir will extend up the local volcanic hillsides, and water will infiltrate into their permeable rock. Actual leakage rates at greater depth cannot be estimated. They depend on the nature of the ground underneath the visible volcanic series.

V - 7. Two Assumptions

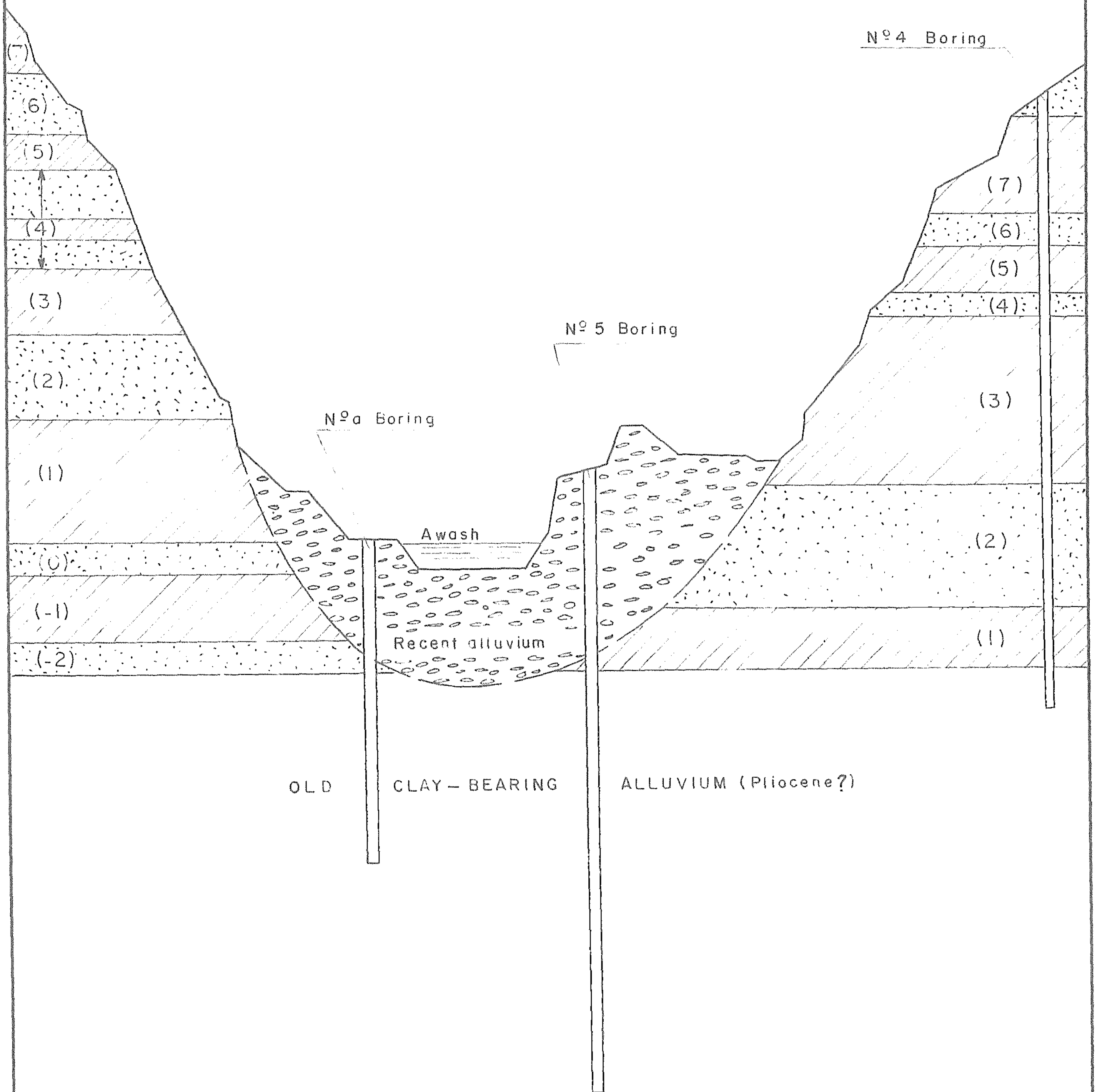
Two assumptions can be made on the strength of the information obtained so far, as shown diagramatically on the sketches with this report :

The first assumption (Fig. 11) is that, slightly below the level of the Awash, the volcanic formations overlie a thick old alluvial deposit probably dating back to the Pliocene and containing substantial clayey horizons. This means that borehole No. 4 penetrated into the alluvial formation to a depth of about 5 m, also that after running through a 14 m layer of recent alluvium, borehole No. 5 penetrated the old alluvium formation to a depth of 41 m without reaching its base. In fact, this old formation may extend to a depth of several hundred meters. If this is correct, the alluvium dating before the volcanic phase presumably forms a largely almost impervious floor reducing the risk of deep infiltration losses to negligible proportions, except in a few coarser horizons.

The second assumption (Fig. 12) is that, the alluvia in the bottom of borehole No. 4 are the remnants of an interim episode in a volcanic series of unknown overall depth. This implies that, a long time ago (possibly as far back as the Pliocene), the Awash must have dug out a deep valley in this formation and that this valley then filled up with clayey alluvia and the recent alluvial loam deposits over them. If this is correct, the basalt and tuff series (which may also contain intermediate alluvium beds) may have considerable depth. It so, water infiltrating from the reservoir would gradually fill up the voids in the formation and finally form a water table extending throughout the surrounding region. This might result in greater water losses than in the previous case, as volcanic rock is generally more pervious than fine alluvium.

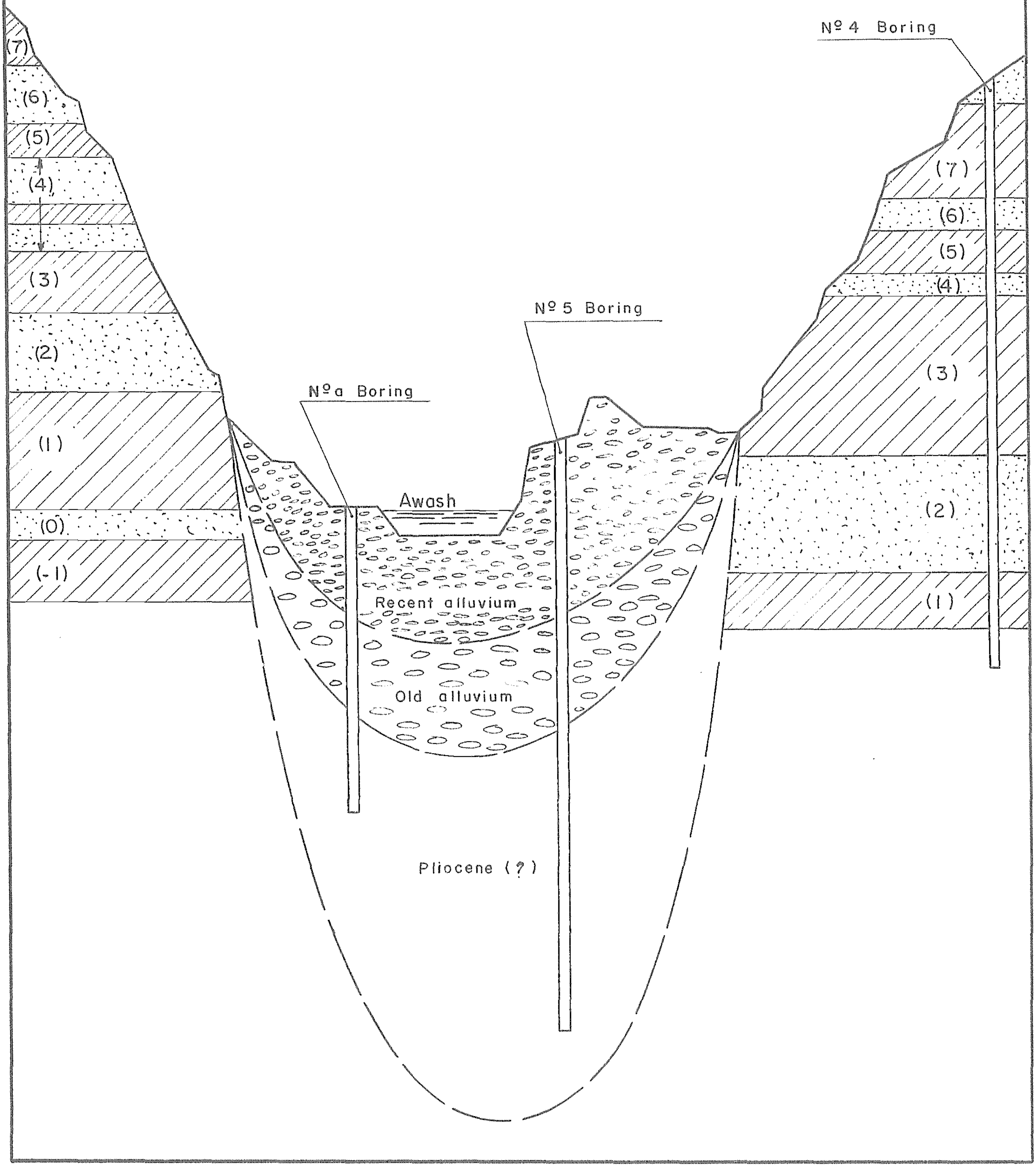
TENDAHO DAM SITE

First geological assumption



TENDAHO DAM SITE

Second geological assumption



V - 8. Conclusion

The borehole survey provided closer information about the geological structure of the site, especially as regards :

(i) Variations in the lateral extent of the basalt outflows and the off-setting of the strata probably by faulting between the river banks.

(ii) The substantial relative size of the soft intermediate strata in the basalt outflows.

(iii) The considerable thickness of the - probably Pliocene - loose material formations underneath the Awash.

From the structural foundation point of view, it is important to dismiss any idea of building a concrete dam ; only a structure with sufficient 'give' in it (i.e., a rock-fill dam) could adapt itself to the very heterogeneous nature of the ground and the differential settling effects to be expected.

From the leakage point of view, very high permeability rates were observed only at the small saddle on the right bank, or on the surface. They generally decreased fairly quickly with depth. Precise data are still lacking for the left bank, and a serious problem is that no evidence was found of a water table, even in the deepest borings.

Further reconnaissance surveys are essential. The leakage problem should be given priority consideration throughout the left bank area - especially at the Tendaho saddle - and also in the deeper formations along the right bank so as finally to settle which of the two assumptions is right. Only then will it be possible to decide on the extent of the grout curtains to be provided. Their positioning involves a number of difficult problems because of the alternating hard fissured and loose finely porous rock in the ground.

VI. HYDRAULIC OPERATION OF THE RESERVOIR AND DIMENSIONS OF BASIC PROJECT COMPONENTS

The previous discussions allow fairly wide latitude in choosing a storage capacity for Tendaho, but determining the optimum storage capacity is a complex problem. Its solution calls for studies on the following points :

(i) Modification of present flows supplied by the Awash at Tendaho due to various degrees of use of river water upstream.

(ii) Effect of reservoir water losses to atmosphere, which may rise beyond an acceptable value if the average reservoir level exceeds a certain altitude.

(iii) Formulation of the water requirement to be met below Tendaho for irrigation, river stabilization and power production purposes.

Each of the above headings will now be reviewed in turn.

VI - 1. <u>Present Water Supplies Available at Tendaho</u>

These can be assessed on the strength of data from Dubti gauging station ; they are summed up as monthly inflows for all three 'hydrological cycles' of the Project in the following table :

Date	N	D	J	F	M	A	M	J	J	A	S	O	Year
1961-62	-	89.5	105.5	102.5	124	103	81	45.5	78	520	340.5	254	(1961)
1962-63	127.5	92.5	118	49.5	57	239	319	131	161.5	397.5	571	213.5	2477
1963-64	125.5	110	129.5	107	93.5	183.5	122	86.5	593	1158	685	313	3706.5

Expressed in hm3

The annual inflows vary greatly between cycles. In comparing them with normal conditions, it is best first to consider rainfall in the whole region affecting the lower Awash ; i.e., basically the western hillsides along the Upper Basin (the Koka catchment area cannot be included as its flow contribution benefits from interannual regulation). The results of this comparison for the two predominant 'inflow quarters' of the year is summed up in the following condensed form :

	March	April	May		July	August	September
1962	=	-	-		-	++	+
1963	--	++	++		=	=	-
1964	-	+	+		++	++	=

-- : well below normal conditions

- : below normal conditions

= : normal conditions

\+ : above normal conditions

++ : well above normal conditions

There is a satisfactory correlation between monthly rainfall supplies and the corresponding runoff flows, except in September. The table also shows that flows supplied during the 1963/1964 cycle were above normal, whereas the two previous cycles were below normal ; especially the 1962/1963 cycle, when the over-abundant supplies of the first rainy season were amply compensated for by the subsequent shortage.

The direct flow comparison made in the basin hydrological inflow/outflow balance study in Volume III (i.e., between the flow as related to their normal values) confirms these results. Preferably, this comparison should not be concerned with the total flow recorded at Dubti, but should consider the difference between this flow and the volume of water through the turbines at Koka (which is not connected with any natural phenomenon). This gives the following differential flows :

1961-1962 : (1961) - 880 = 1,081 hm3

1962-1963 : 2,477 - 1,044.5 = 1,432.5 hm3

1963-1964 : 3,706.5 - 1,179.5 = 2,527 hm3

Normal condition : 3490 - 1200 = 2,290 hm3

In the absence of data covering a large number of years, from which 'typical inflow cycles' for a known probability range could have been constructed, the hydrological year from November 1962 to October 1963 will be considered in studying the use of Tendaho reservoir supplies. A good reason for continuing to consider this period is that it has already served for hydraulic studies of the other reservoirs. On the other hand, it is preferable to consider the 1963/1964 cycle in determining the effectiveness of Tendaho reservoir in protecting the Lower Plains against flooding.

For a more rigorous determination of Awash flows at Tendaho, it is necessary to deduct the quantity of water supplied by the Logiya from the flows arriving at Dubti. This is very small, and is estimated to have only amounted to 20 hm3 during the 1962/1963 cycle, equally distributed over the five months (April, May, July, August and September) during which flows in this river reached any appreciable proportions.

Awash flows supplied at Tendaho during the considered annual cycle were as follows :

Date	N	D	J	F	M	A	M	J	J	A	S	O	Year
1962-63	127.5	92.5	118	49.5	57	235	315	131	157.5	393.5	567	213.5	2457

Expressed in hm3

VI - 2. Probable Water Supplies after Development

Flow conditions in the Awash will change as development of the Upper Basin and Middle Valley proceeds, mainly through :

(i) A gradual reduction in present supplies due to the amounts of water used, particularly for irrigation.

(ii) Regulation of these supplies as other dams similar to the Koka dam become operational.

In predicting these changes, certain assumptions must be made for the actual water consumption trends and for the effects of changes in flow conditions due to those on conditions several hundred kilometers downstream. The hydrological analysis work described in Volume III has greatly eased the second difficulty by producing quantitative river loss data between Koka and Tendaho and by throwing some light on the 'deformation' of flows in the Awash due to the floodable areas in the Middle Valley.

Three 'development cases' have been singled out to illustrate probable increasing water consumption trends in the Upper Basin and Middle Valley up-river from Tendaho.

Case (i) Moderate development. This allows for any developments likely to be put into effect within a relatively short-term period (say, about ten years) without involving the State in any heavy financial commitments. Two possibilities were considered.

The first possibility was increased water consumption in the Upper Basin, which must be expected as the population increases and higher standards of living are attained, also with the expansion and increasing industrialization of Adis Abeba and its surrounding region (especially towards Nazret). Net consumption in the Upper Basin may be expected sooner or later to reach a limit deliberately imposed to restrict the possible effects on hydraulic potential depending on the Koka reservoir; e.g., it is assumed that reduction of present water resources in the Upper Basin will not exceed 50 hm3 yearly, i.e., 4.5 hm3 monthly. This is equivalent to a gross water consumption of 100 l/s by a population of 1,350,000, which seems largely adequate to cover domestic, municipal and industrial needs.

The second possibility is due to new irrigation areas going into service downstream from Wenji plantation, as follows :

- between Bofa and Metehara	5,000 ha
- in the Metehara plain	10,000 ha
- 'run-of-the-river' supplies from the Kesem and Kebena :	2,000 ha
- in the Melka Sedi and Melka Warar plains	7,000 ha
T o t a l	24,000 ha

A main reason for selecting these areas is that concessions for them have either already been granted or are under negotiation.

Case (ii) Advanced development. In addition to needs considered in case (i), this one allows for a number of developments in the Middle Valley, which are selected from among the least hazardous to implement and the most profitable to run. These are :

Irrigation of additional areas (for present purposes, only considering those irrigable by 'run-of-the-river' supplies from the Awash ; i.e., disregarding any needing new pumping plant). These areas are :

Amibara.............................	11,000 ha
Bolhamo (gravity)	2,500 ha
Angelele............................	4,500 ha
Initial area in the Maro Gala Plain ..	10,000 ha
T o t a l	28,000 ha

Note. Compensation of Awash flows where they enter the Middle Valley, by a dam in the gorge at Awash station, will be necessary to ensure run-of-the-river irrigation for these areas. Alternatively, if this proved too difficult or too costly, a new schedule of turbine operation at Koka power station and the other stations below it could provide the necessary adjustments.

Case (iii) Full development. This covers the full range of developments potentially applicable to the Middle Valley, including some operations which, though technically feasible, are less suitable for immediate application. This case should be considered only as part of a long-term policy. These potential developments are :

(a) Increasing Awash supplies by 200 hm3 annually by diverting the river Meki into Lake Gelilea. Though this would be a profitable operation, a number of serious hazards still remain, as discussed in Chapter II.

(b) Regulating flows in the Kesem by the proposed big dam.

(c) Maximum extension of irrigation in areas mentioned under cases (i) and (ii)above, as follows :

- from 2,000 ha to 17,500 ha in the Kesem-Kebena plain	15,500 ha
- from 2,500 ha to 8,500 ha in the Bolhamo plain by pumping in stages	6,000 ha
- from 10,000 ha to 22,500 ha in the Maro Gala plain	12,500 ha
T o t a l	34,000 ha

Probable flow conditions in the Awash at Tendaho can be deduced from the results of the hydrological survey of the basin and compared with present conditions for each of the above development cases, as follows :

MONTHLY FLOWS ARRIVING AT TENDAHO DURING THE 1962/63 CYCLE
(hm3)

	N	D	J	F	M	A	M	J	J	A	S	O	Year
A0	127.5	92.5	118	49.5	57	235	315	131	157.5	393.5	567.0	213.5	2457
A1 (i)	80	59.5	80.5	31	60.5	231.5	241	68.5	210	412.5	543	216.5	2234.5
A2 (ii)	.58	43.5	59.5	20	25.5	188.5	204	23.5	190	405.5	525	201.5	1944.5
A3 (iii)	70	44.5	61.5	22	30.5	190.5	200	27.5	128	280.5	505	199.5	1759.5

Annual flows arriving at Tendaho in the three cases differ by less than the net total water consumption of the irrigated areas, as changes in flow conditions due to water consumption and artificial flow regulation help to reduce losses along the entire course of the Awash. Quantities gained work out as : 149 hm3 yearly from present conditions to case (i) ; 84.5 hm3 yearly from case (i) to case (ii) ; and 70 hm3 yearly from case (ii) to case (iii).

Comparison of monthly inflows leads to frequently obvious though also occasionally conflicting, conclusions. They can be explained only by a detailed hydrological analysis of the phenomena involved. No allowance could be made in the hydrological calculations for the dykes which will be required for the protection of irrigation areas in the Middle Valley and which, in helping to limit the extent of flooding, may reduce water losses even further.

VI - 3. Risk of Silting in the Tendaho Reservoir

Sediment load data observed at Dubti gauging station are summed up for the entire Project observation period in the table below. Figures are in millions of metric tons.

Date	N	D	J	F	M	A	M	J	J	A	S	O	Total
1962	-	-	0.1	0.1	0.4	0.2	0.1	0	0.9	8.6	3.8	0.5	-
1962-63	0.2	0.1	0.2	0	0.1	5.1	5.2	0.2	1.1	4.8	6.8	0.7	24.5
1963-64	0.3	0.2	0.4	0.3	0.2	2.6	0.6	0.2	24.6	32.0	10.6	1.7	73.7

These data are seen to differ considerably between years ; e.g., the total sediment load during the last annual cycle shows up the very heavy rainfall in the northern part of the Awash Basin during the last rainy season.

The normal sediment load at Dubti calculated from the overall synthesis discussed in Volume III works out at 32,800,000 metric tons annually. The sediment load in the Logiya, which will not affect the Tendaho reservoir, can be established by the same method. Assuming a specific normal degradation of 875 t/km2 for its catchment area (4,330 km2), it works out at about 3,800,000 metric tons in a year. An average 29,000,000 metric tons of sediment may find its way into the Tendaho reservoir each year. If all this material settled out, the storage capacity would be reduced by about 15 to 20 hm3 annually. With this pessimistic assumption the useful working life of the reservoir would not exceed about fifty years.

Although silting is still a serious problem for this reservoir (and more so than for any of the other reservoirs), there are prospects of reducing this risk to more reasonable proportions. Although no grain size analysis data are available for ascertaining the relative quantity of sediment likely to remain in suspension for any length of time, visual observation shows that this sediment largely consists of very fine material, with big proportions probably originating from the 'badlands' along the Middle Awash. Even if certain marginal areas in the reservoir silt up rapidly with this material, the deepest areas along the reservoir center line might not be seriously affected by silting for a very long time.

Sediment-flushing operations through the dewatering conduit could be practiced as at the other proposed dams. This would entrain some sediment deposited, or settling-out, near the dam. These operations imply fairly delicate schedules to ensure that deliberate losses of storage do not adversely affect subsequent water requirements.

If, after a few years of observation, the sedimentation risk turned out to be more serious than had been anticipated, one could consider building secondary dams to retain some of the sediment load on the river Mile and on the Awash itself above Tendaho and below Ledi (at Layagili); the sediment could be removed with earth-moving equipment. Similarly, if the main reservoir silted up too rapidly, the desilting with floating suction dredgers could be considered.

In any event, it is essential to leave a substantial dead storage volume in the bottom of the reservoir to allow the coarsest sediment to settle out. A reasonable allowance for this should be about 100 hm3 ; i.e., the storage volume below the 590 m level.

VI - 4. Reservoir Losses to Atmosphere

The meteorological station at Dubti was unable to yield representative data for the climate at Tendaho. The only alternative was to rely on the general relationship developed in Volume III between altitude and normal annual evaporation from an extensive open body of water, and to modulate this annual total by applying a series of monthly correction factors. Normal evaporation figures for the absolute altitude of the future reservoir at Tendaho (400 m - 410 m a.s.l.) are listed in the first line of the table below. The third line gives the specific loss net after subtracting present evapotranspiration, which is virtually the rainfall at Dubti.

Designation	N	D	J	F	M	A	M	J	J	A	S	O	Year
Normal evaporation (mm)	208	208	206	228	291	322	318	331	276	324	333	255	3300
Present evapotranspiration (mm)	3	2	7	7	17	58	19	6	50	86	58	7	320
Loss e (m3/ha)	2050	2060	1990	2210	2740	2640	2990	3250	2260	2380	2750	2480	29800

Specific water losses to atmosphere from the Tendaho reservoir are thus 20 % higher than for the Kesem reservoir. They increase very rapidly with rising mean water level, as shown in the next table :

Relative altitude (m)	Mean depth H above bed (m)	Mean area S (km2)	Annual loss E (hm3)
596	26	63.5	189
598	28	81	241
600	30	104	310
602	32	131.5	392
604	34	164	491
606	36	210	625

The loss E varies exponentially with mean depth of water H, by the following relationship :

$$E = e^{\frac{H + 17.8}{8.35}}$$

where e is the natural logarithm base.

To ensure minimum evaporation loss, the reservoir should ideally be run with the least possible mean depth of water H, but the final choice of optimum live storage level depends on requirements mainly connected with electricity production by the power plant at the foot of the dam. This study cannot be concluded until all the water requirements to be met downstream from the dam are known.

VI - 5. Quantities of Water for Irrigation Requirements in the Lower Plains

The specific water requirements for the Lower Plains (b_d) are estimated at the end of Volume II. This gives the figures in the first line of the table below :

	N	D	J	F	M	A	M	J	J	A	S	O	Year
bd (m3/ha)	1590	1455	845	875	880	705	285	1075	1935	1615	2075	2010	15 345
a (hm3)	125.5	89.5	105.5	49.5	57	103	81	45.5	78	397.5	340.5	213.5	-
a/bd (10^3ha)	79	61.5	125	56.5	65	146	284	42.5	40.5	246	164	106	-

It is interesting to establish the maximum amount of land irrigable each month purely by 'run-of-the-river' supplies ; i.e., assuming no dam at Tendaho and only considering present inflows for which the quantity considered for each month is the smallest inflow "a" recorded during three years observation at Dubti. For the Lower Plains, it is unwise to rely too much on a certain percentage of the irrigation supplies ultimately finding their way back into the Awash. Most drainage water will be run off into marshland areas and lost for good. The required maximum irrigable area is given by the ratio $\frac{a}{bd}$.

These data show that no more than 40,000 ha of land in the Lower Plains could have been irrigated by 'run-of-the-river' supplies between 1962 and 1964, and the figure would have been even lower if it were possible to consider a longer hydrometric observation period in the calculation. The possibility of the Awash drying-up completely during certain months would have to be allowed for. These consequences become even more serious in a case of reduced supplies through increased consumption in the Upper Basin and Middle Valley. River flow regulation is an absolute necessity to ensure supplies for all the irrigable land in the Lower Plains throughout the year ; i.e., at least the land in categories II and III (as shown in the semi-detailed soil surveys in Volume II). The maximum area covered in the Volume V studies amounts to 66,500 ha, of which roughly two-thirds (43,500 ha) will require pumped supplies.

Irrigation water requirements for the Lower Plains are listed in the table below (B refers to the full 66,500 ha, and B' to the 43,500 ha to be supplied by pumping plants).

	N	D	J	F	M	A	M	J	J	A	S	O	Year
B	105.5	96.5	56	58	58.5	47	19	71.5	128.5	107.5	138	133.5	1019.5
B'	69	63.5	37	38	38.5	30.5	12.5	47	84	70.5	90	87.5	668

Expressed in hm3

VI - 6. Additional Volumes of Water Returned to the River

If no more than the amounts of water required for irrigation were passed downstream from Tendaho dam, the mean discharge of the Awash would gradually decrease from Tendaho to Lake Gamari. During low-demand months (i.e., January to May, and especially May), the river discharge might be reduced to only a few cubic meters/second past the points at which the substantial quantities required for the Dubti and Dit Bahri areas are diverted. This would no doubt complicate riparian economic conditions, and perhaps harm animal life in the downstream areas.

Since the normal annual inflow exceeds irrigation requirements (even allowing for Tendaho reservoir losses to atmosphere), it is advisable to discharge as much as possible back into the river downstream during most of the year. This purpose is also achieved in the operational calculations discussed in the following pages. Their object is to determine the "critical profitability" for the firm output to be produced by the future Tendaho plant. These calculations will be slightly more complicated than for the Kesem as some of the power produced at Tendaho will be needed for the pumping stations and, therefore, strictly geared to irrigation demand.

Although the required total pumping head may vary appreciably according to where the supplies are diverted and to flow conditions in the river, 7m for the small pumps mainly in service in the Asayita delta and 9m for major pumping stations should represent average values. Because the smaller pumps are less efficient, with values of about 60 % compared to 75 % for the larger pumps, energy in GWh required for the pumps is given by the following expression :

$$P' = \frac{B'}{450 \times 0.8} \times \frac{7 \text{ to } 9}{0.6 \text{ to } 0.75} \approx \frac{B'}{30}$$

(0.8 = 80% turbine efficiency)

VI - 7. Determination of Live Reservoir Storage Level

From the viewpoint of maximum reservoir productivity, losses to atmosphere E increase sharply with rising mean reservoir water level, and the quantity available downstream from the dam A-E is accordingly reduced. This quantity A-E will serve not only to supply irrigation but also to produce electricity and to maintain adequate flows in the river downstream. They can be expressed by the relationship :

$$A - E = B + R$$

where : B is the annual volume of water reserved for irrigation, and R is the additional volume of water to be returned to the river.

In addition to the energy $P' = \frac{B'}{30}$ reserved for the pumping stations, the energy output given by P also includes the additional quantity :

$$P - P' = \frac{(B + R)H}{450} - \frac{B'}{30} = \frac{(A - E)H}{450} - \frac{B'}{30}$$

where H is the mean geometric head available for the turbines (the tail water level is at about 570 m). The greater H, the greater also will be this energy, which will be available for sale to consumers other than irrigation operators. The product (A - E)H, the two terms of which vary in the opposite sense, must be made a maximum ; i.e., its differential coefficient must be reduced to zero by solving the following equation :

$$\frac{A}{\frac{H}{8.35} + 1} = e^{\frac{H + 17.8}{8.35}} = E$$

The roots of this equation are :

For case (i) with A_1 = 2234.5 hm3/year, H = 33.15 m (603.15)

For case (ii) with A_2 = 1944.5 hm3/year, H = 32.20 m (602.20)

For case (iii) with A_3 = 1759.5 hm3/year, H = 31.55 m (601.55)

To get the most out of the reservoir, its average level would have to be set at about 602 m.

From the viewpoint of project economy, if electricity production is sacrificed, the reservoir level could be set solely for the 130 hm3 dead storage volume, plus the reserve storage required for regulation, which works out at between 500 and 600 hm3, depending on the development case considered. This figure will be checked in the final reservoir operation calculations. If the only object is to limit the height of the dam and reduce its initial construction cost, the total storage capacity of the reservoir need not exceed 700 hm3, for which the average level during the year would be about 595 m and the normal maximum level 600 m. Certain leakage problems might become serious if the reservoir level were allowed to rise systematically above 600 m.

Though full consideration of this subject requires a general economic study outside the present scope of the Project, a reasonable course would be the following rough compromise between both points of view :

(i) To choose a storage level slightly above the economic optimum so as to have an exceptional regulation volume handy, which is invaluable in a sub-normal rainy season.

(ii) To choose a storage level a few meters below the optimum productivity level, which should not seriously affect power productivity.

An average storage level of 599 m will be assumed in the calculations discussed further on, or, which amounts to the same thing, a normal maximum water level of 602 m.

VI - 8. Reservoir Operation Tables

To establish more closely probable reservoir operation results, a month-by-month analysis of its behaviour is essential ; this information is given in the three following tables. Each one refers to one of the three considered Awash River Valley development cases. The notation, similar to that in the Kesem reservoir operation tables, is as follows :

Z_1 : Relative water level at the beginning of the month (reservoir assumed to be full on the 1st November 1962).

V_1 : Storage at the beginning of the month.

S_1 : Area of water at the beginning of the month.

$A_1 - A_2 - A_3$: Respective reservoir inflows for the three development cases.

B : Irrigation water requirements for 66,500 ha.

R : Additional flow to be returned to the river.

$T = B + R$: Volume passing through the turbines.

$A - T$: Increase or decrease in storage.

$V'_2 = V_1 + (A-T)$: Storage at the end of the month, neglecting evaporation loss.

S : Average water area during the considered month.

E : Net loss to atmosphere.

Z_0 : Relative power plant tail water level (varying with T)

H : Average head $(Z - Z_0)$ during the month.

$P = \frac{TH}{450}$: Total producible energy.

$P' = \frac{B'}{30}$: Energy reserved to supply irrigation pumping plant.

Case (i) : MODERATE DEVELOPMENT

1962-63	N	D	J	F	M	A	M	J	J	A	S	O
Z1 (m)	602.0	601.3	600.3	599.6	598.1	596.1	597.1	598.1	596.4	596.4	599.1	601.9
V_1 (hm3)	970	870.5	750	664	522	398.5	449.5	523.5	399.5	401	617	957
S_1 (km2)	131.5	120	107	98.5	82	66.5	73	82	66.5	66.5	92.5	128
A_1 (hm3)	80	59.5	80.5	31	60.5	231.5	241	68.5	210	412.5	543	216.5
B (hm3)	105.5	96.5	56	58	58.5	47	19	71.5	128.5	107.5	138	133.5
R (hm3)	48	60	90	95	105	115	125	97	65	70	35	30
T (hm3)	153.5	156.5	146	153	163.5	162	144	168.5	193.5	177.5	173	163.5
A_1-T(hm3)	-73.5	-97	-65.5	-122	-103	69.5	97	-100	16.5	235	370	53
V'_2(hm3)	896.5	773.5	684.5	542	419	468	546.5	423.5	416	636	(987)	(1010)
S (km2)	126	113.5	103	90	74	70	77.5	74	66.5	79.5	110	130
E (hm3)	26	23.5	20.5	20	20.5	18.5	23	24	15	19	30	32
H (m)	30.25	29.40	28.55	27.45	25.85	25.25	26.20	25.75	24.80	26.15	28.90	30.50
Z_0 (m)	571.4	571.4	571.4	571.4	571.4	571.5	571.4	571.5	571.6	571.6	571.6	571.5
P (GWh)	10.32	10.22	9.27	9.34	9.40	9.11	8.40	9.64	10.66	10.30	11.09	11.08
P' (GWh)	2.30	2.12	1.23	1.27	1.28	1.02	0.42	1.57	2.80	2.35	3.00	2.92
P-P'(GWh)	7.98	8.10	8.04	8.07	8.12	8.09	7.98	8.07	7.86	7.95	8.09	8.16

Case (ii) : ADVANCED DEVELOPMENT

1962-63	N	D	J	F	M	A	M	J	J	A	S	O
Z_1 (m)	602.0	601.3	600.4	599.7	598.4	596.7	597.2	598.0	596.0	596.0	599.0	601.9
V_1 (hm3)	970	871	759	675.5	549	418.5	455	517	373.5	376.5	609	955.5
S_1 (km2)	131.5	120.5	108	99.5	85.5	69	73.5	81	63.5	63.5	92	130
A_2 (hm3)	58	43.5	59.5	20	25.5	188.5	204	23.5	190	405.5	525	201.5
B (hm3)	105.5	96.5	56	58	58.5	47	19	71.5	128.5	107.5	138	133.5
R (hm3)	25.5	35.5	66	68	76.5	86	100	71.5	44	47	10	5.5
T (hm3)	131	132	122	126	135	133	119	143	172.5	154.5	148	139
A_2-T(hm3)	-73	-88.5	-62.5	-106	-109.5	55.5	85	-119.5	17.5	251	377	62.5
V'_2 (hm3)	897	782.5	696.5	569.5	439.5	474	540	397.5	391	627.5	(986)	(1018)
S (km2)	126	114.5	104	92.5	77	71.5	77	73	63.5	78	111	131
E (hm3)	26	23.5	21	20.5	21	19	23	24	14.5	18.5	30.5	32.5
H (m)	30.45	29.65	28.95	27.95	26.35	25.75	26.50	25.70	24.50	26.10	29.05	30.65
Z_0 (m)	571.2	571.2	571.1	571.1	571.2	571.2	571.1	571.3	571.5	571.4	571.4	571.3
P (GWh)	8.87	8.70	7.85	7.82	7.90	7.61	7.01	8.17	9.40	8.97	9.55	9.47
P' (GWh)	2.30	2.12	1.23	1.27	1.28	1.02	0.42	1.57	2.80	2.35	3.00	2.92
P-P'(GWh)	6.57	6.58	6.62	6.55	6.62	6.59	6.59	6.60	6.60	6.62	6.55	6.55

Case (iii) : FULL DEVELOPMENT

1962-63	N	D	J	F	M	A	M	J	J	A	S	O
Z_1 (m)	602.0	601.5	600.6	600.1	599.1	597.9	598.6	599.3	598.0	597.6	599.0	601.8
V_1 (hm3)	970	896	798.5	731.5	620.5	511.5	567	641	520	483	609.5	946.5
S_1 (km2)	131.5	122.5	111.5	105.5	93	80.5	87	95	82	77.5	93	127
A_3 (hm3)	70	44.5	61.5	22	30.5	190.5	200	27.5	128	280.5	505	199.5
B (hm3)	105.5	96.5	56	58	58.5	47	19	71.5	128.5	107.5	138	133.5
R (hm3)	12.5	21.5	51	53	57	66	80	48	18.5	26.5	0	0
T (hm3)	118	118	107	111	115.5	113	99	119.5	147	134	138	133.5
A_3-T (hm3)	-48	-73.5	-45.5	-89	-85	77.5	101	-92	-19	146.5	367	66
V''_2 (hm3)	922	822.5	753	642.5	535.5	589	668	549	501	629.5	(976.5)	(1012.5)
S (km2)	127	117	108.5	99.5	87	84	91	88.5	80	85	110	129.5
E (hm3)	26	24	21.5	22	24	22	27	29	18	20	30	32
H (m)	30.65	29.95	29.35	28.60	27.40	27.15	27.95	27.55	26.30	27.10	29.20	30.70
Z_0 (m)	571.1	571.1	571.0	571.0	571.1	571.1	571.0	571.1	571.3	571.2	571.2	571.2
P (GWh)	8.04	7.85	6.98	7.06	7.04	6.81	6.15	7.32	8.59	8.07	8.96	9.29
P' (GWh)	2.30	2.12	1.23	1.27	1.28	1.02	0.42	1.57	2.80	2.35	3.00	2.92
P-P' (GWh)	5.74	5.73	5.75	5.79	5.76	5.79	5.73	5.75	5.79	5.72	(5.96)	(6.37)

The iteration poles by which the data in these three tables were determined were :

(i) No spillway operation ; i.e., full use of all inflows.

(ii) Highest possible additional power production.

It was considered pointless to take the iterative procedures as far as their absolute poles. Failure to do so would result only in a negligible residual error and in no way affect the validity of the comparison between the three cases. The annual (total or average) values for characteristic operating data are listed in the table below :

	Case (i)	Case (ii)	Case (iii)
Inflows (hm3)	2,234.5	1,944.5	1,759.5
Volume of water through turbines (hm3)	1,954.5	1,655.0	1,453.5
Loss to atmosphere (hm3)	272	274	295.5
Volume over spillways (hm3)	8	15.5	10.5
Minimum recorded water level (m)	596.4	596.0	597.6
Mean water level (m)	598.9	598.9	599.6
Mean total storage (hm3)	627.0	627.5	691.5
Storage available for flow regulation (hm3)	571.5	596.5	487.0
Mean water area (km2)	93	93	100.5
Mean geometric head (m)	27.40	27.65	28.50
Firm power output (GWh) (after meeting pumping requirements)	96.55	79.04	69.06

The various monthly values listed for T show the regulating effect of the reservoir on flows downstream in the Awash, to a point at which variations under normal conditions do not exceed 20 %. Flow conditions are likely to become irregular again from Tendaho to Lake Gamari due to the diversion of flows for irrigation.

Reservoir water loss to atmosphere never exceeds 300 hm3/year.

Reservoir water level variation ranges are limited to 5.60 m, 6.00 m and 4.40 m, which are all perfectly compatible with satisfactory turbine operation. The mean reservoir water level is around 599 m in all three cases, as originally expected.

Quantities available for regulation range between 500 and 600 hm3, which, for the three cases, gives emergency storage volumes ranging from 250 to 350 hm3 above dead storage level.

The results for cases (i) and (ii) differ only as regards energy production. Their overall similarity is explained by the monthly inflows undergoing the same modulation in both cases. In case (iii), the range of monthly inflows benefits from the regulating effect of the Kesem dam. Thus, less storage need be available at Tendaho than in cases (i) and (ii). As expected, the firm output decreases from case (i) to case (iii).

Although this should not be taken as an imperative recommendation, the choice of a normal impounded level of 602 m for the reservoir appears to be adequately justified by the available information.

VI - 9. Reservoir Salinization Risks

Water samples already obtained from the Awash at Dubti and Asayita fully represent conditions at Tendaho. The measured conductivity of these samples is invariably 0.37 mmhos/cm which, for this category of water, is equivalent to a salt content s_o in the neighbourhood of 0.35 g/l.

A simple calculation shows that, starting from a value s_o, the salinity of a reservoir tends asymptotically towards the following value :

$$s_1 = \frac{A}{A - E} s_o$$

Where A and E are normal annual inflow and losses to atmosphere. The formula gives the following results :

For case (i) : $s_1 = 1.14 \; s_o = 0.40$ g/l

For case (ii) : $s_1 = 1.16 \; s_o = 0.41$ g/l

For case (iii) : $s_1 = 1.20 \; s_o = 0.42$ g/l

The reservoir salinity variation month by month can be determined from these asymptotic values by a slightly longer calculation, which shows that it almost never exceeds 0.50 g/l. As the natural inflows are fresh and their quantities sufficient to ensure that the water is continually changed, its salinity is never likely to reach proportions alarming for agriculture.

VI - 10. Flood Flows

Several correlations (though difficult to match, they are significant) enabled a careful analysis to be made in Volume III of flood conditions actually recorded at Dubti and (especially) their origin and probable occurrence to be determined. Thus, it was found that the substantial flood flows in August 1964, which thrice reached 540 m3/s, were caused by a normal flood peak superimposed on a fairly large basic flood (300 m3/s), giving an estimated recurrence period of the resulting flood flow of six years.

These results cannot yet be extrapolated as the various phenomena are too closely interconnected and too few observed data are available. Unlike with the Kesem, no range of flood flows of lower recurrence probability than the August 1964 flood can be predicted. The only useful indication for the exceptional peak flow liable to occur at Tendaho is the morphological flood at Dubti, which is estimated at 1,100 m3/s with a recurrence period of over 1000 years. This estimate does not allow for the effect of the Koka reservoir, which is expected to assist in damping out incoming flood peaks from the Upper Basin. This is why a flood discharge of about 1,000 m3/s (as estimated in Volume III) should be considered for the nominal spillway flow at Tendaho.

The best possible use must be made of the large reservoir dimensions at Tendaho so as to attenuate major flood peaks. The principle of this control is that the flows returned to the river below the dam should always be less than the critical flow-conveying capacity of the mean water channel of the river ; i.e., the greatest amount of water it can carry without overflowing. This fulfils one purpose of the dam ; i.e., the protection of the Lower Plains against flooding. As the overflow level at Dubti is reached at about 350 m3/s, a figure of 300 m3/s for the maximum flow to be returned to the river by the spillway and bottom outlet at Tendaho is reasonable. Based on this flow, the proposed form of flood control means storing any inflows in excess of 300 m3/s in the reservoir. It is interesting to see, how much storage capacity would have been required to control the exceptional floods during the 1964 rainy season by this method.

Flows remained above 300 m3/s throughout from 21st July to 9th September 1964, the period when the total gross inflow amounted to 1,843.5 hm3. Deducting quantities returned to the river (1,322 hm3) and lost to atmosphere, this leaves roughly 450 hm3 to be stored in the reservoir. Had the water in the reservoir been at its normal maximum level (602 m) on 21st July 1964, it would have risen to 604.50 m by 9th September. As the hydrological correlations discussed in Volume III showed, it is possible to calculate the monthly base flood discharge at Dubti about a fortnight ahead from rainfall data for the previous six weeks for the part of the Rift between the Kesem and the Borkena. In other words, it is reasonable to assume that a flood prediction service would have given adequate warning for all necessary precautions to be taken ; i.e., to drain the reservoir to below the 602 m level to ensure more reliable flood peak absorption.

A dam crest level of 605 m is considered to be adequate, as it should allow safe normal reservoir operation up to the maximum 602 m for protection of the Lower Plains.

VI - 11. Hydroelectric Power Station Dimensions

The proposed position of the power plant at the foot of the dam is amply justified, as electricity demand throughout this (still totally unequipped) region will increase with hydraulic and agricultural development of the Lower Plains. In addition to future day-to-day electricity requirements, the power plant will have to meet those of the various conversion and processing (e.g., cotton ginning plants) and the irrigation pumping stations. Potential electricity supply in development case (i) will probably exceed demand for a long time, and it is assumed that the two will not balance until the conditions of case (iii) are achieved. Case (iii) is the one to consider in dimensioning the power station. It provides for a total annual electricity production of 91 GWh, of which 22 GWh are to be set aside for the irrigation pumping stations.

a.) Pumping plant requirements. Demand will strictly follow that of irrigation. Thus, when available heads are lowest (H = 26.3 m, in July), 2.8 GWh will have to be produced for 18 hours out of every 24, which, for the considered head, requires the following output :

$$W_1 = \frac{2,800,000}{31 \times 24} \times \frac{24}{18} = 5,000 \text{ kW}$$

b.) Additional output. As the operation table for case (iii) shows, monthly production peaks are at about 5.75 GWh, except in September and October. The peak output required at minimum head (in July, assuming a load factor of 0.55) works out as :

$$W_2 = \frac{5,750,000}{0.55 \times 31 \times 24} = 14,000 \text{ kW}$$

c.) Total installed output. Assuming the turbines will operate fully open at all heads between mean (28.50 m) and minimum (26.50 m), the total installed output is :

$$W_{inst} = (W_1 + W_2) \left(\frac{28.5}{26.3} \right)^{3/2} = 21,000 \text{ kW}$$

The corresponding total discharge is 94 m3/s.

d.) Total output distribution. To ensure flexibility in meeting demand, the installed output should be shared out between one 4,000 kW unit and two of 8,500 kW. Initial requirements will even be supplied by the auxiliary Diesel generating equipment.

VI - 12. Turbine Flow Compensation

The time taken for flows to propagate down the Awash (about two days from Tendaho to Lake Gamari) is so long and the irrigation areas are so widely spaced down-river, that flows released at Tendaho for irrigation will spread considerably, and daily fluctuations of demand at Tendaho due to various local irrigation requirements will undergo appreciable damping. In spite of this, fluctuations due to irrigation demand will not coincide with any due to electricity demand. Operation is likely to be straightforward as long as the turbine flows do not exceed the tabulated values R for case (i). If it proved necessary to use some of the supplies B reserved for irrigation to produce electricity, one or other of the following possibilities would have to be considered in the absence of a daily compensation reservoir (for which no really suitable site seems to be available below Tendaho) :

(i) Compliance with a power production time table ; i.e., a consumption time table matched to the irrigation schedule.

(ii) Matching irrigation times to energy production times ; if necessary, by creating daily water storage facilities at the head of the irrigation areas.

In practice, a combination of these two policies will be sought, implying a strict reservoir operation schedule and close cooperation between dam and irrigation system operators.

VII. OUTLINE PRELINIMARY DAM DESIGN

VII - 1. Design and Layout of Structures

The topographical features of the site only allow one possible dam position. Boreholes have not produced evidence of a rock substratum below the river bed suitable for structural foundations, but both lateral abutments are known to consist of superimposed soft layers and variously fractured very hard basalt outflows. With the appropriate cement grout injections, the latter should provide an excellent foundation for concrete structures. An obvious choice of layout is :

(i) A 'flexible' dam in the bed of the Awash.

(ii) Ancillary concrete structures on one of the river banks.

In view of the oblique angle of the dam to the river center line, the left bank is far the more suitable side on which to return the flow to the river, but the ancillary structures will require substantial excavation work, and over half the spoil will consist of basalt rock.

The only loose material available in any appreciable quantity near the site is very impervious clay-bearing alluvium. A sample was tested for its mechanical properties (see Appendix 3). As a result, it is considered that this alluvium contains too much clay to be suitable for use in bulk to form an earth dam. No constituent exceeds 0.2 mm in size, and 70 % of the particles are finer than 50 microns; its internal friction coefficient is low, and it gives a typical plastic clay Proctor diagram. It is hoped that the prospection for suitable borrows will reveal less clayey ground or sand giving a suitable earth dam body material when mixed with the clay on the site.

Due to its impermeability, and if it is not too difficult to moisten and place, this alluvium should be suitable for the impervious core of a rock fill dam. In the light of present knowledge, this is the form of 'flexible' structure recommended, especially as rock fill excavated on the left bank can be used for its construction. The soil mechanics tests have also confirmed that the alluvium has sufficient strength to carry a rock fill dam.

VII - 2. Grout Curtain

Permeability tests in the borings showed the alluvium and layers between the basalt outflows to be adequately impervious, but there is some risk of leakage in the outflows. Grout curtains will be necessary throughout the zone in which seepage paths are fairly short. Further borings with water tests will be needed to determine the extent of the grout curtain on the right bank, but one can safely assume that the total length of grout curtain will be at least 1,800 m, giving a curtain area of 80,000 m2, which is about the same size as at the Kesem dam. These curtains are expensive, though not to the point of affecting the economy of the Project.

VII - 3. Description of the Summary Preliminary Layout

All structures lie along an axis parallel to, and about 15 m downstream from, the line of boreholes. They are shown on three drawings in this Volume and include the following, from left to right bank :

(i) At the 605 m level beside the Aseb road, a wide platform closing off the Tendaho saddle and providing a parking area.

(ii) An access way from this platform to the main dam structures, via a cutting in the upstream left-bank hillside.

(iii) The flood spillway with two 9 m x 9 m sector gates to pass the nominal 1,000 m3/s flood discharge. This will stand on outflow (1) reconnoitered in the geological survey.

(iv) An approximately 100 m long gravity-type concrete dam section standing on outflow (1) and incorporating the power plant water intakes, penstocks and dewatering conduit. In view of the outflow's fundamental bearing on the dams' strength and behaviour, it will be essential at a subsequent project stage to establish its depth and horizontal extent by fairly numerous boreholes over a semi-circular area including all concrete structures and extending down to about the 550 m level.

(v) The hydroelectric power station at the foot of the dam. Final power station equipment will comprise three Francis units (one of 4,000 kW and two of 8,500 kW). The bottom outlet will run underneath the power station erection apron. Access to the power plant will be from the right bank via an approach ramp on the downstream face of the rock fill dam. The downstream portion of the temporary diversion channel shown on the overall plan will later become the tailwater canal.

(vi) A retaining wall standing on outflow (1).

(vii) The main rock fill dam measuring 300 m along its crest.

The crest level of all structures will be 605 m. (See VI-10 above).

VII - 4. Summary Structural Cost Estimate

The distinction made for the Kesem rock fill dam estimate is also useful in the present scheme. It divides the overall layout into :

(i) The part solely required for irrigation and protection of the Lower Plains (by stabilizing flow conditions, and thus also the bed of the Awash).

(ii) The remaining part concerned with electricity production, which is considered as a by-product.

Unlike the Kesem scheme, these two parts of the Tendaho scheme cannot be associated with two completely distinct construction stages.

a.) Initial part (irrigation and protection)

The only assumptions needed for these requirements are a total storage capacity of 700 hm3, reducing the normal maximum water level to 600 m and the dam crest level to 603 m, and the non-existence of the power station and its equipment exist. This results in a correspondingly shorter concrete dam portion.

Civil engineering costs	E$
Cofferdams	2,000,000
Excavation in loose ground : 1.2 x 58,000 m3.................	70,000
Excavation in rock, without re-use of spoil : 5 x 150,000 m3	750,000
Rock fill, including storage and re-use : 14 x 260,000 m3	3,640,000
C/F	6,460,000

B/F :	6,450,000	
Filters and drains 20 x 77,000 m3	1,540,000	
Impervious core 5 x 77,000 m3	385,000	
Concrete structures 85 x 39,700 m3	3,375,000	
Additional for reinforced concrete 65 x 5,000 m3	325,000	
Grout injection	3,000,000	
Ancillary dyke	350,000	
		15,435,000
<u>Electromechanical equipment</u>		
Flood spillway	240,000	
Dewatering conduit (incl. gantry)	227,000	
		467,000
Rerouting of Aseb road (about 10 km) :...........		1,200,000
TOTAL		17,102,000
Unforeseen items, design and work supervision 30%:		5,130,600
TOTAL (i)....................		22,232,600

b.) <u>Second part (electricity production)</u>

This part requires the addition of 2 m to the height of the dam, an extension to the concrete part of the dam, civil engineering work for the power station, and its equipment.

<u>Additional civil engineering costs</u>	E$
Excavation in loose ground 1.2 x 10,000 m3	12,000
Excavation in rock, without re-use of spoil : 5 x 75,000 m3	375,000
Rock fill, including storage and re-use : 14 x 40,000 m3	560,000
C/F	947,000

		E$	
	B/F :	947,000	
Filters and drains 20 x 13,000 m3		260,000	
Impervious core 5 x 13,000 m3		65,000	
Concrete structures 85 x 35,300 m3		3,000,000	
Additional for reinforced concrete 65 x 5,000 m3		325,000	
Ancillary dyke		150,000	4,747,000

Electromechanical equipment

Water intake	114,000	
3 turbines	988,000	
3 generators	1,335,000	
3 butterfly valves	547,000	
2 transformers	345,000	
Switchyard	435,000	
Auxiliary equipment and services	1,340,000	
		5,104,000
TOTAL		9,851,000
Unforeseen items, design and work supervision, 30 % :		2,955,300
TOTAL (ii)		12,806,300

GRAND TOTAL (i) + (ii) = E$ 35,038,900

The additional cost of equipping the Tendaho scheme for an anticipated production of 91 GWh/year thus amounts to about E$ 12,800,000. This is amply justified if the installation of the power plant is kept matched to electricity consumption development.

VIII. ANTICIPATED EFFECTS OF THE TENDAHO SCHEME

VIII - 1. Beneficial Effects

The construction of a dam and a power station at Tendaho is the primary condition for general development of the Lower Plains of the Awash. The scheme ensures all the following benefits :

(i) Availability of the amounts of water required to irrigate the best land at the necessary times throughout the year, (i.e., more than 65,000 ha overall) for comparatively intensive cropping patterns, and often for two harvests during the year.

(ii) Higher average water levels than now in the Awash by discharging flows additional to the already substantial irrigation flows into the river. The new conditions will not only favour riparian activities, but also facilitate the diversion of flows for irrigation under gravity, and should even make some of the additional flow available for irrigation by water-spreading in irrigation boundary zones likely to contain most of the grazing land.

(iii) River stabilization, which should produce valuable results once the flood peaks are attenuated, making it possible to establish permanent farming areas and associated 'infrastructure' when sudden uncontrollable river bed shifts are no longer feared.

(iv) Flood protection of land. By prolonged submersions, the present uncontrolled floods are liable to affect farming severely.

(v) A certain degree of overall drainage, assisted by eliminating flooding, and introducing efficient irrigation instead. This may even cause some marshland areas to recede.

(vi) Availability of considerable quantities of electricity. With a transmission and distribution system extending at least as far as the Asayita delta, this will be an invaluable asset for developing the Lower Plains, if only initially for the irrigation pumps.

VIII - 2. Possible Adverse Effects

Possible adverse effects of a major dam interrupting the natural course of the river are :

(i) Natural leaching of the land due to present flooding would cease. The soil salinization risk is discussed in Volume II, with recommended suitable remedies, including especially artificial 'maintenance' leaching through substantial pre-irrigation water applications.

(ii) Changes in the behaviour of the water table in the irrigation areas and along their boundaries, due to more regular water table inflows from irrigation throughout the year and because of the high water levels maintained in the Awash, so that levels would generally tend to rise. In spite of this, it is preferable to consider the idea that eliminating uncontrolled flooding will be the decisive factor in the general improvement of land drainage, and that each irrigation area will have its own drainage system designed to keep the water table down to an acceptable level.

(iii) Alluvial deposition beneficial to the land would be reduced as the sediment load would be trapped in the reservoir and only returned to the Awash during the few weeks of flood discharge. As in any other development of this type, deterioration will have to be guarded against by suitable artificial fertilization methods, especially by introducing appropriate crop-rotation patterns.

(iv) Recession of the lower lakes. This should really only apply to Lake Abe. All the others are "transit" lakes and should remain unaffected. Net water consumption for irrigation will partly be offset by lower evaporation losses in all marsh areas in the Lower Plains if they recede by any appreciable extent. The reduction in inflows into Lake Abe may not be sufficient to affect seriously this very unusual geographical accident.

All these disadvantages seem to be of a minor nature or to represent risks for which ample adequate counter-measures are available.

APPENDIX 1

PRELIMINARY GEOLOGICAL RECONNAISSANCE SURVEY OF THE AWADI SITE

The Awadi site results from the interbedding of volcanic outflows in a thick alluvial series. These outflows appear distinctly as cliffs amid softer alluvium. The site illustrates the important part faults have played in creating interruptions in rock barriers and offsetting them in several directions (See Fig. 13). The two following volcanic formations occur above each other, with an intermediate formation between them :

(i) A volcanic formation appearing as a cliff and dipping down to river level downstream. This upper cliff corresponds to a type of trachyte mainly consisting of 'welded tuff', with big plagioclase crystals, numerous microlite clusters in a very fine matrix, and auite with ferruginous resorption and serpentinized olivine.

(ii) An excavated softer intermediate formation underneath the upper cliff (on the left bank downstream) mainly consisting of debris with occasionally fairly large rocks embedded to a varying extent in a loamy material which has often been washed away. This formation, about 3 m thick, may be more or less permeable.

(iii) The second volcanic formation, in the form of an irregular outflow with slag passages and often vacuolar rock. It is a doleritic basalt with fairly coarse microlites, numerous olivine granules (advanced ferruginous alteration), much smaller proportions of pyroxene than olivine, numerous magnetite or ilmenite granules, and large vacuoles.

Only the upper formation is clearly visible on the right-bank. Lower formations are completely hidden under scree and vegetation. The rock bar is regular without showing signs of any special accidents. On the left-bank, the volcanic formations rise fairly steeply up-river, and there are several small faults in the cliff. The most troublesome one isolates the block which could have become the left-bank abutment for the dam. It forms a very distinct cleft towards the tributary thalweg not far away.

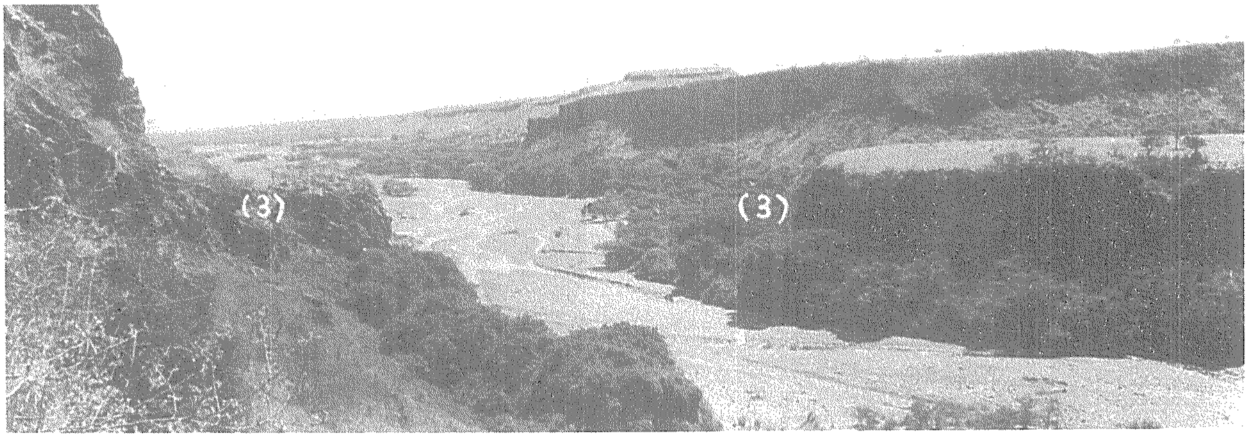
The dam will probably have to be sited farther up-river. This means building a slightly longer structure.

In addition to this left-bank fault, other problems to be investigated by boreholes would be :

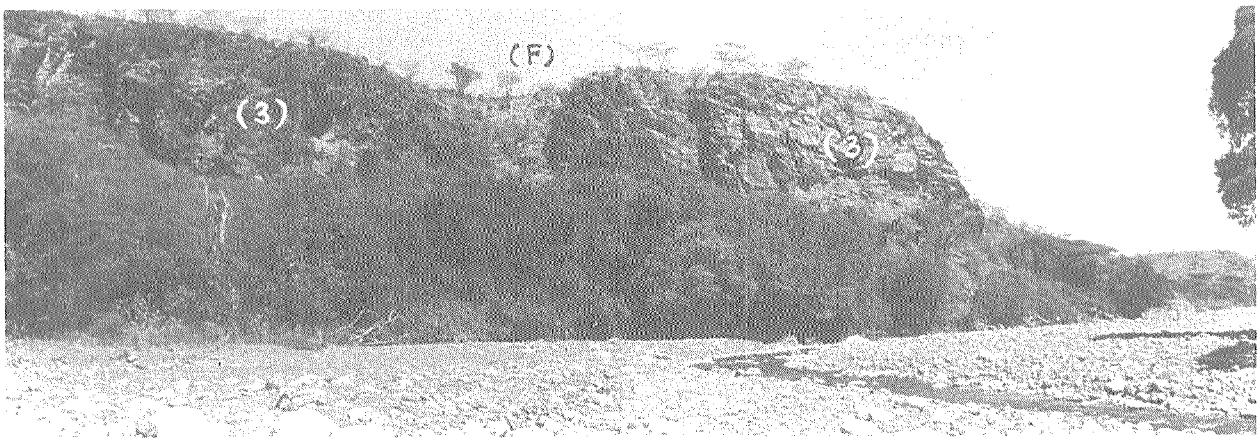
(i) The order of succession of the layers in the hillsides, and their permeability (risks of local leakage or farther afield to the tributary thalweg).

(ii) Thickness and permeability of the alluvium, which would no doubt be left in position underneath a 'flexible' earth or rubble structure. At least three boreholes would be needed for the width of this site, one of which should be sufficiently deep to check whether there are adverse features in the terrain underneath the alluvium.

General view of the site, taken from downstream - The comparative narrowness of the site is due to a substantial outflow of basaltic appearance.



The site seen from the left bank upstream - A substantial basalt outflow on the right forms a cliff with scree and dense vegetation at its foot.
On the left, a high-level outflow (3) rising up-river due to successive faults, and underneath it a bank formed by an intermediate loam and scree layer and a softer slaggy lower-level outflow.



Left-bank abutment area - This view clearly shows the high-level outflow (3) forming a cliff with a little saddleback at a fault.
The foot of the slope is thickly overgrown, but on closer inspection shows a soft intermediate loam and scree layer (2), and a slaggy irregular lower-level outflow (1).

APPENDIX 2

UNIT COSTS CONSIDERED FOR THE SUMMARY COST ESTIMATE FOR THE MAJOR STRUCTURES

This list of costs given applies to the major structures proposed for the hydraulic and agricultural development of the Awash basin ; i.e., big dams and their power stations, also water intakes and regulation works to be built on the main rivers.

Civil engineering costs have been established by a comparative analysis of quotations recently submitted with tenders for major projects and of data supplied by the authorities concerned with the main projects in the Awash basin, i.e., EELPA, HVA and TPSC.

- Bulk mechanical excavation in loose ground for major structure foundations	E$	1.2/m3
- Bulk mechanical excavation in rock for major structure foundations, no re-use of rock	E$	5/m3
- Extraction and placing of clay materials for major dam construction	E$	5/m3
- Extraction and placing of rock fill for major dam construction	E$	12/m3
Additional for rock fill storage and handling	E$	2/m3
- Provision of filters, drains and rip-rap for large dams..	E$	20/m3
- Bulk concrete	E$	85/m3
- Plain concrete for medium-sized structures	E$	100/m3
- Reinforced concrete, including reinforcement	E$	50/m3

Costs quoted for the cofferdamming of work sites, gabion works and grout injection have all been estimated from data relating to similar work carried out in various parts of the world.

Estimates for electro-mechanical equipment for dams, power plant and other major river structures are based on costs applicable in current international project tenders. An addition has been made to these costs to cover such items as transport and insurance, erection of the equipment at the site, tests and spare parts. No allowance has been made for possible Customs Duty or other taxes.

To complete these estimates, a flat-rate provision of 30 % of the total cost of the construction work will be added to cover unforeseen items, design and work supervision costs. This additional amount also covers the cost of providing any roads, telephones, buildings and other similar items required during the construction work and to run the project.

All cost quotations refer to economic conditions as on the 1st January 1965.

(Exchange Rate at that date: 2.5 E$ = 1 US$)

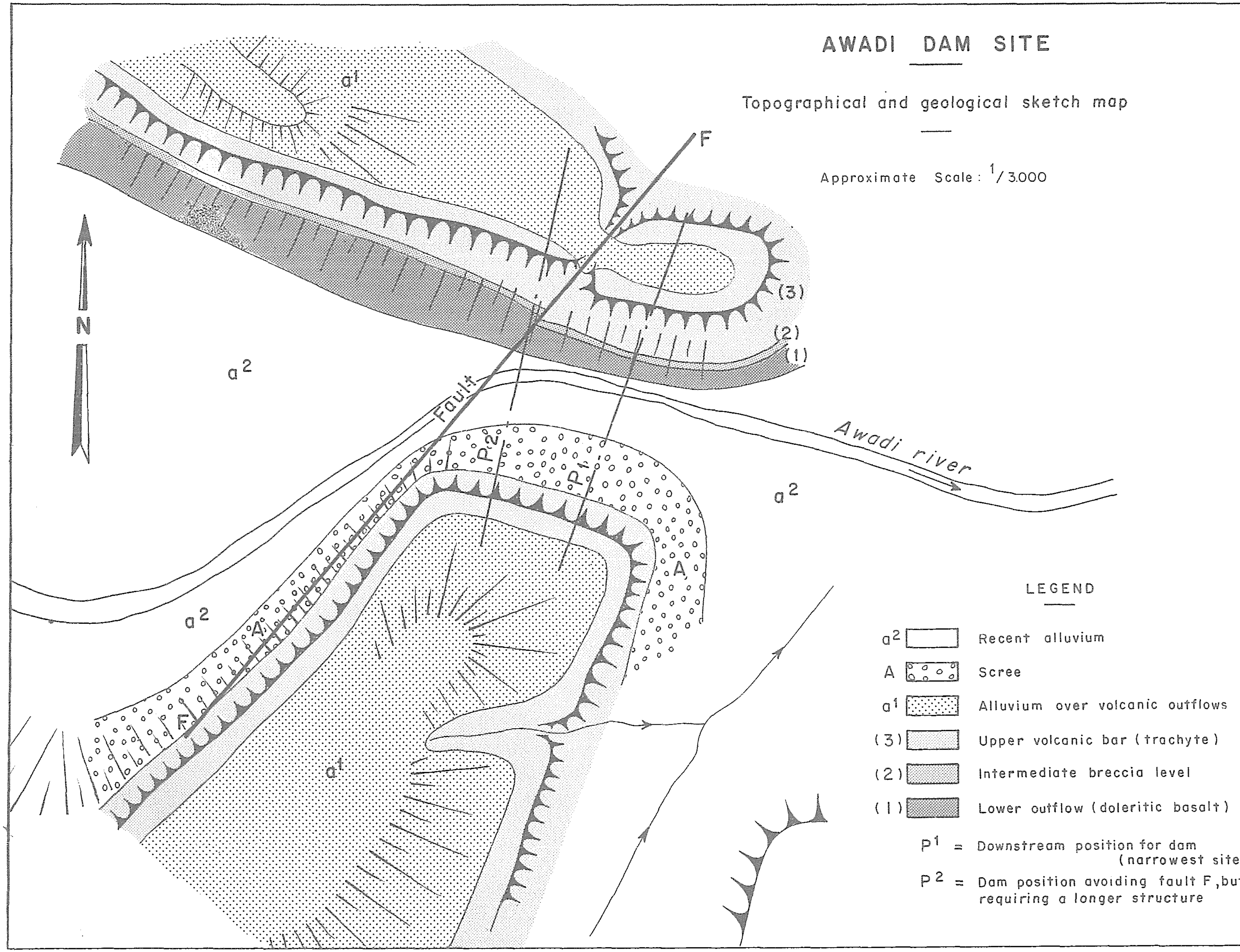

AWADI DAM SITE
Topographical and geological sketch map
Approximate Scale: 1/3.000
F
Fault
F
N
Awadi river
a1
a2
a2
a2
(3)
(2)
(1)
A
A
P2
P1
a1
LEGEND
a2 Recent alluvium
A Scree
a1 Alluvium over volcanic outflows
(3) Upper volcanic bar (trachyte)
(2) Intermediate breccia level
(1) Lower outflow (doleritic basalt)
P1 = Downstream position for dam (narrowest site)
P2 = Dam position avoiding fault F, but requiring a longer structure

FIG. 13

APPENDIX 3

SOILS MECHANICS TESTS ON AN ALLUVIUM SAMPLE FROM THE TENDAHO SITE*

a. Grain size analysis

The purpose of this analysis was to determine soil particle weights for various particle sizes. It involved the two following successive operations :

(i) Square-mesh sieving of particles coarser than 0.1 mm under water.

(ii) Sedimentation of particles finer than 0.1 mm

Particulars of the resulting grain size curve were :

- Abscissa (Grain diameter in decreasing order, to a logarithmic scale;
- Ordinate (Percentage of particles (by weight) finer than the corresponding abscissa, to a linear scale.

b. Moisture content

This was determined by weighing the samples before and after oven drying at 105°C for 24 hours. The moisture content is the ratio between the weight of water contained in the sample and the weight of dry soil.

c. Specific gravity of the solids

This is given by the ratio between the weight of dry soil per unit volume and the specific gravity of water at 0°C.

d. Consolidation test by consolidometer

A sample 70 mm in diameter and 24 mm deep is cut out of the original core sample and encased in a metal ring sandwiched between two porous filter plates. It is then subjected to a vertical load by a weight on the end of a sliding piston. Sample deformation is measured with a micrometer dial reading to within one-hundredth of a millimeter. Each load is applied for 24 hours. The results are plotted in the form of a consolidation curve showing sample deformation against vertical load.

e. Proctor (compaction) test -

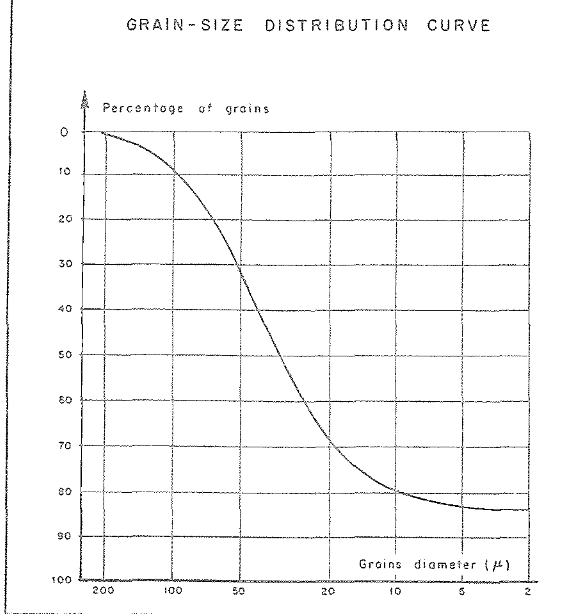
Its purpose is to determine, for a given compactive effort, the moisture content at which a soil should be compacted to achieve maximum dry density, which is usually referred to in France as the 'optimum Proctor moisture content'. The test is carried out by compacting a sample of the soil in a standard mould 35 mm in diameter by 70 mm deep with a standard hammer producing a compactive effort of 60 tm/m3 and according to a definite procedure, and by then determining the moisture content and dry density of the sample after compaction. The test is repeated several times, each time with a sample at higher moisture content, so that a number of points on the representative dry density vs. moisture content curve are obtained. This curve reaches

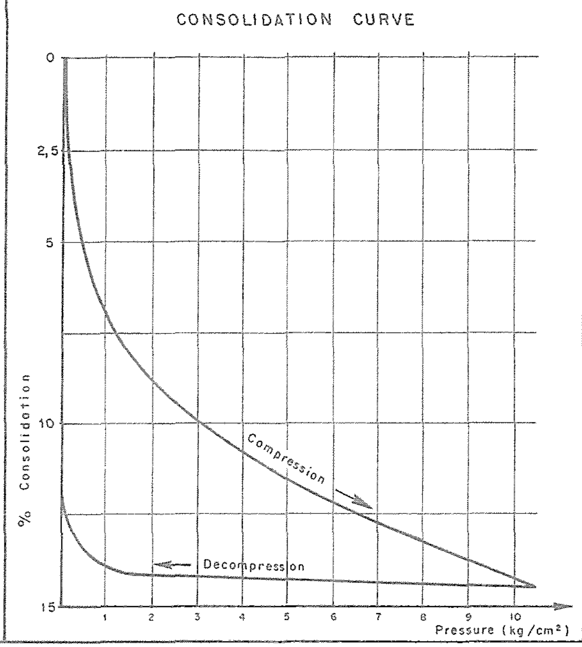
* Tests carried out at the Fluid Mechanics Laboratory of Grenoble University.

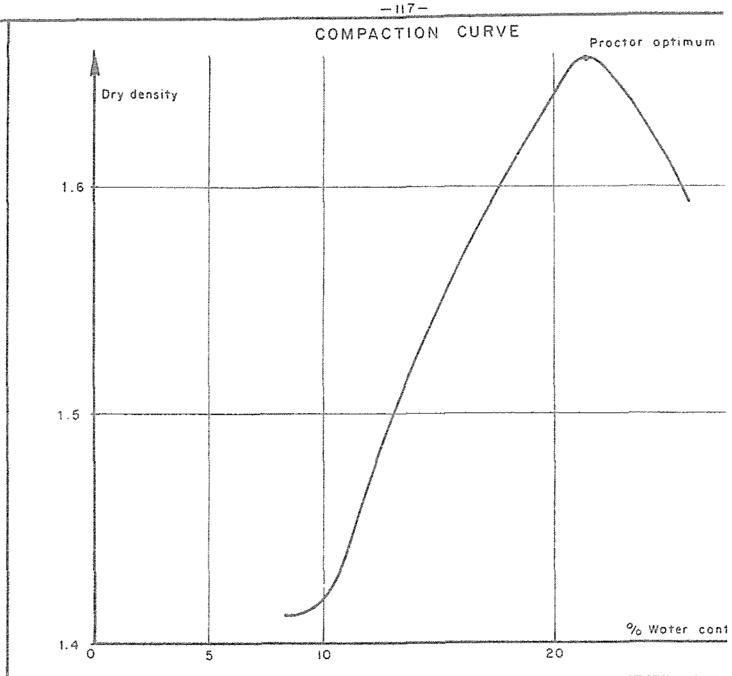
a maximum at the optimum moisture content (21 %) and maximum dry density (1.658) condition.

f. Triaxial shear test (consolidated undrained sample)

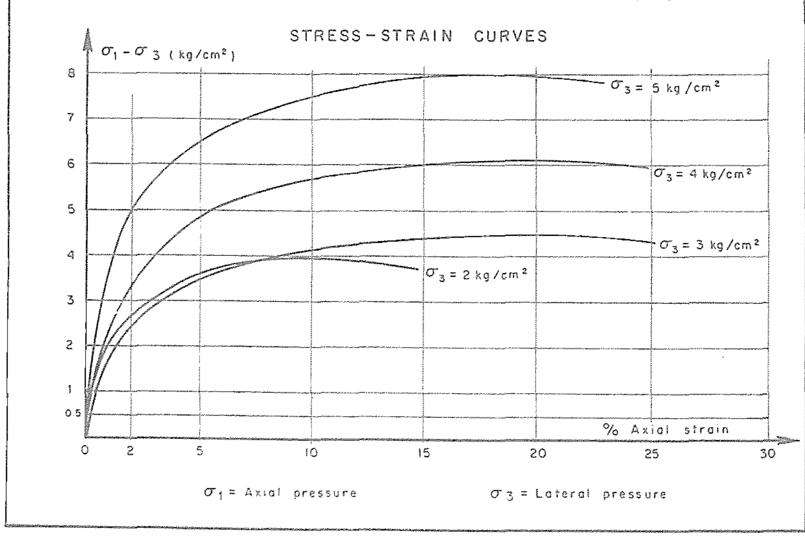
A sample compacted to 'optimum Proctor moisture content' is subjected to an isotropic stress applied by a rubber jacket. It is left to stand in that condition for a few days, noting changes in interstitial pressure (measured without draining any water from the sample). When the interstitial pressure becomes stable, the sample is consolidated until the interstitial pressure reduces to a low value, which is achieved by draining the sample. Finally, the sample is subjected to a low rate of shear (0.1 mm/min.) without draining, noting the interstitial pressure.

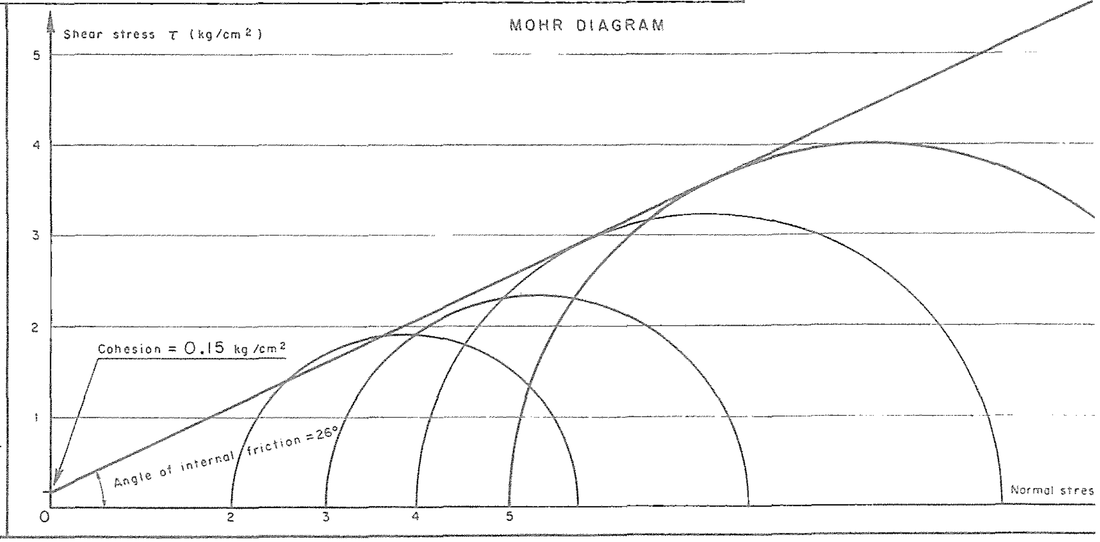






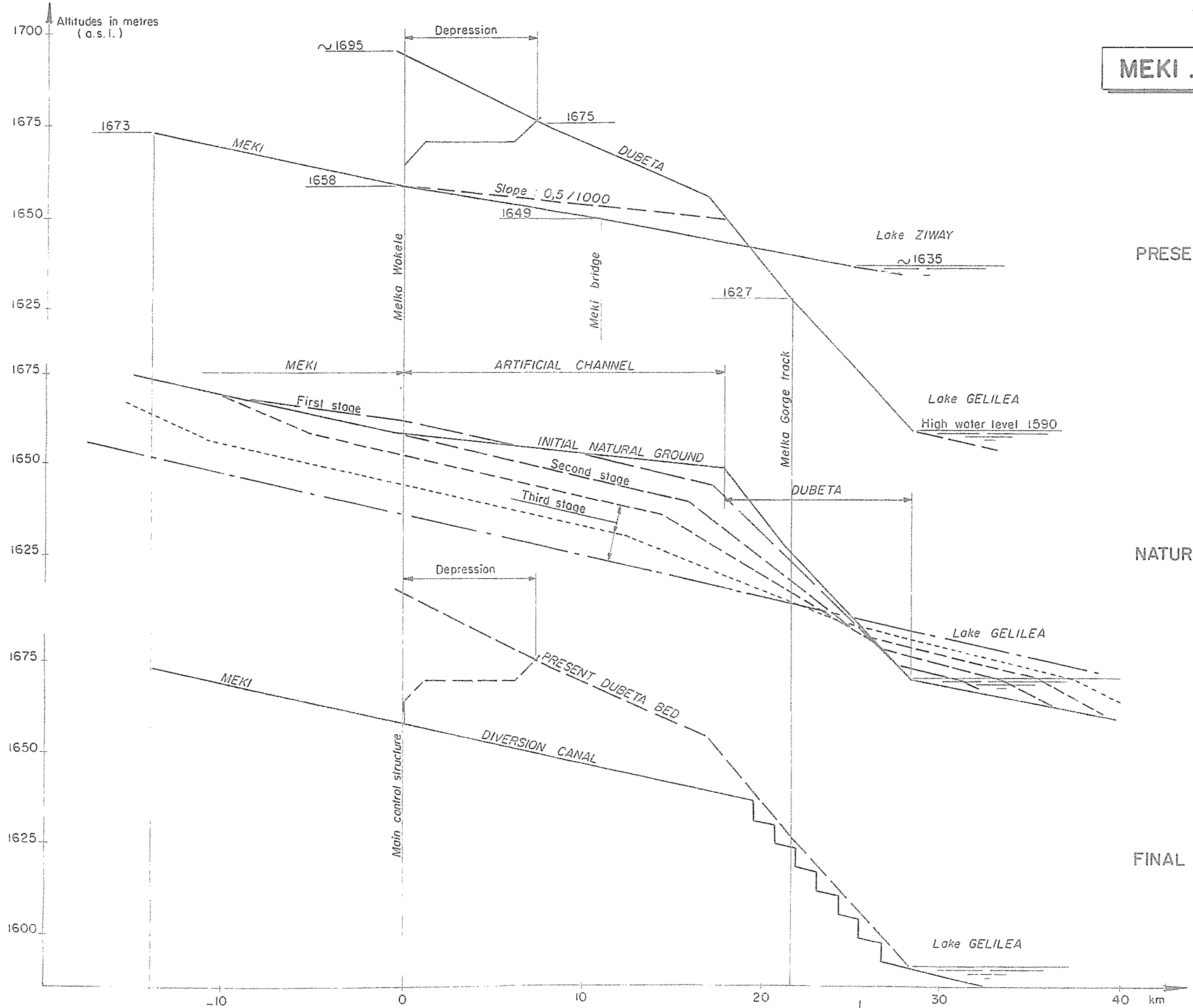
RESULTS OF SOIL MECHANICS TEST ON A SAMPLE OF TENDAHO ALLUVIUM





MEKI _ GELILEA DIVERSION

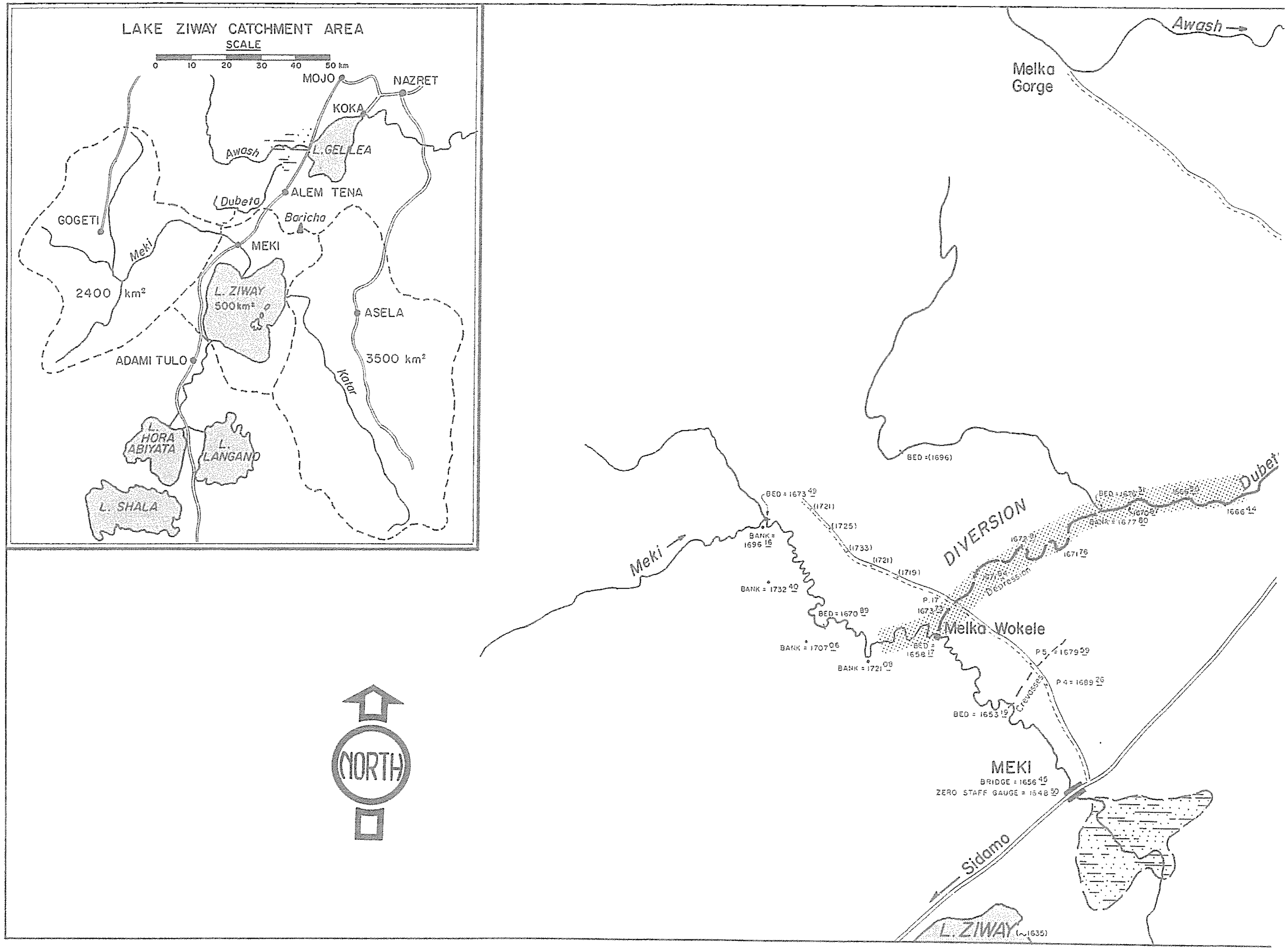
Scales { Vertical : 1 / 1000
Horizontal : 1/200,000

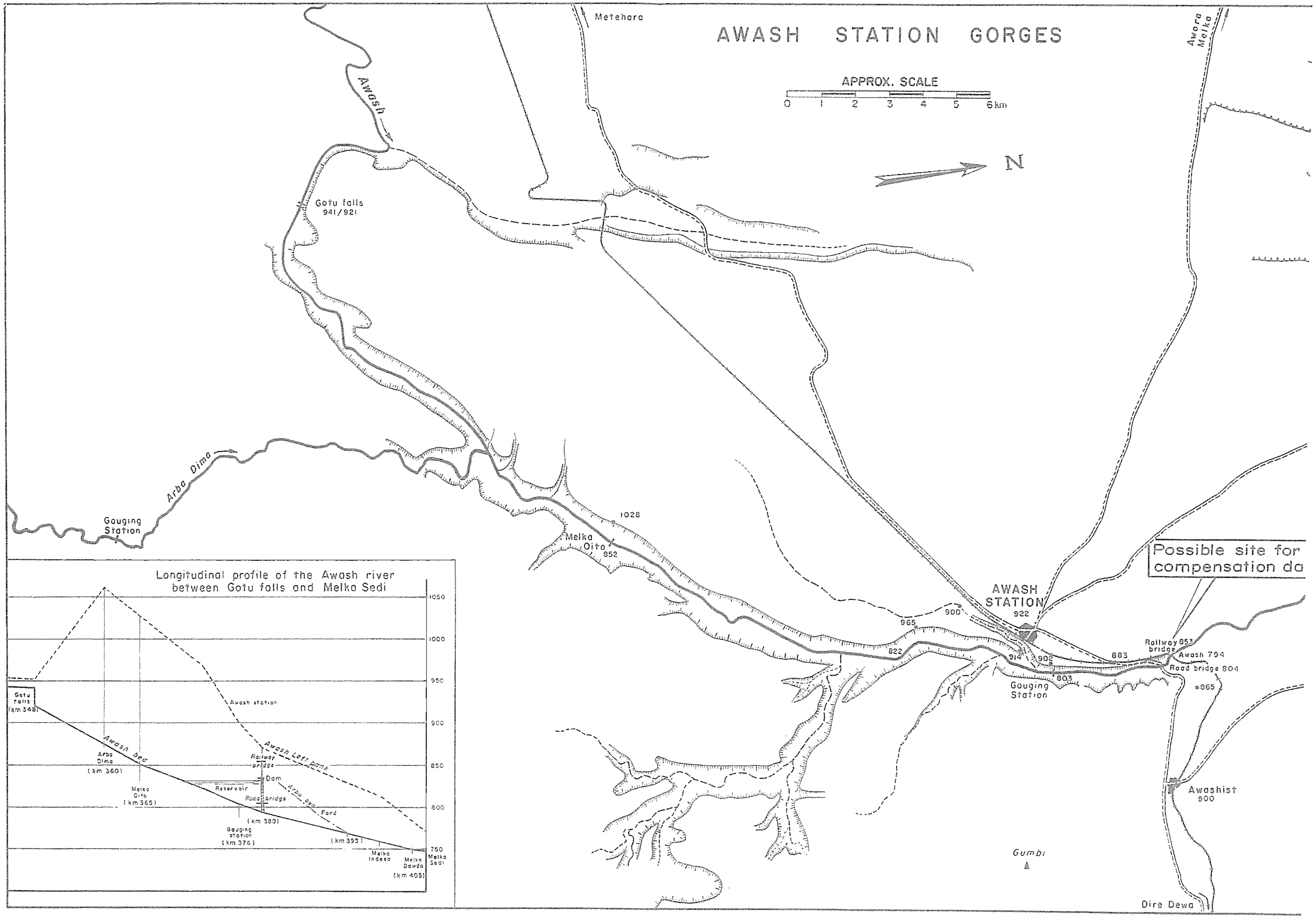


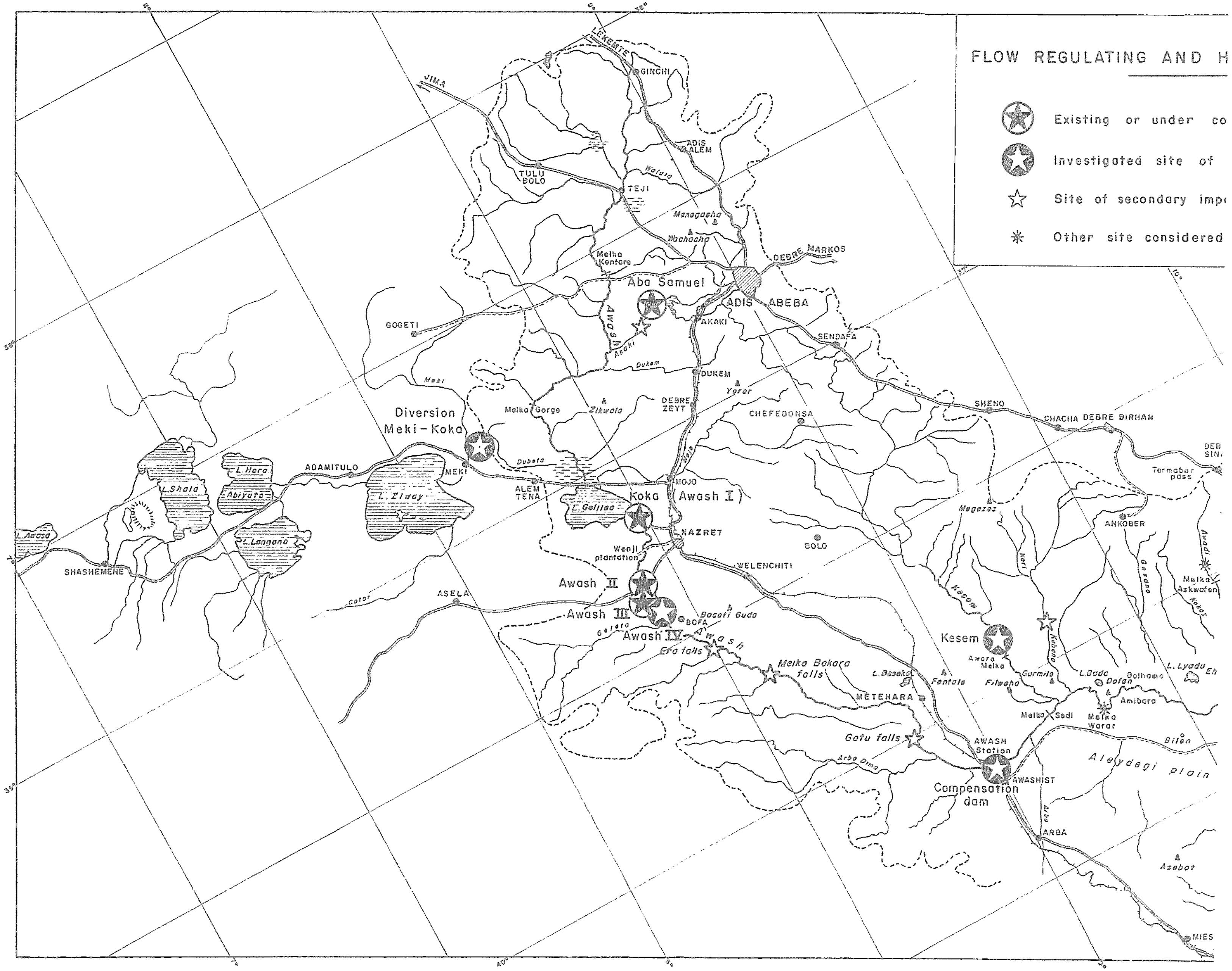
PRESENT LONGITUDINAL PROFILES

NATURAL EVOLUTION OF PROFILES

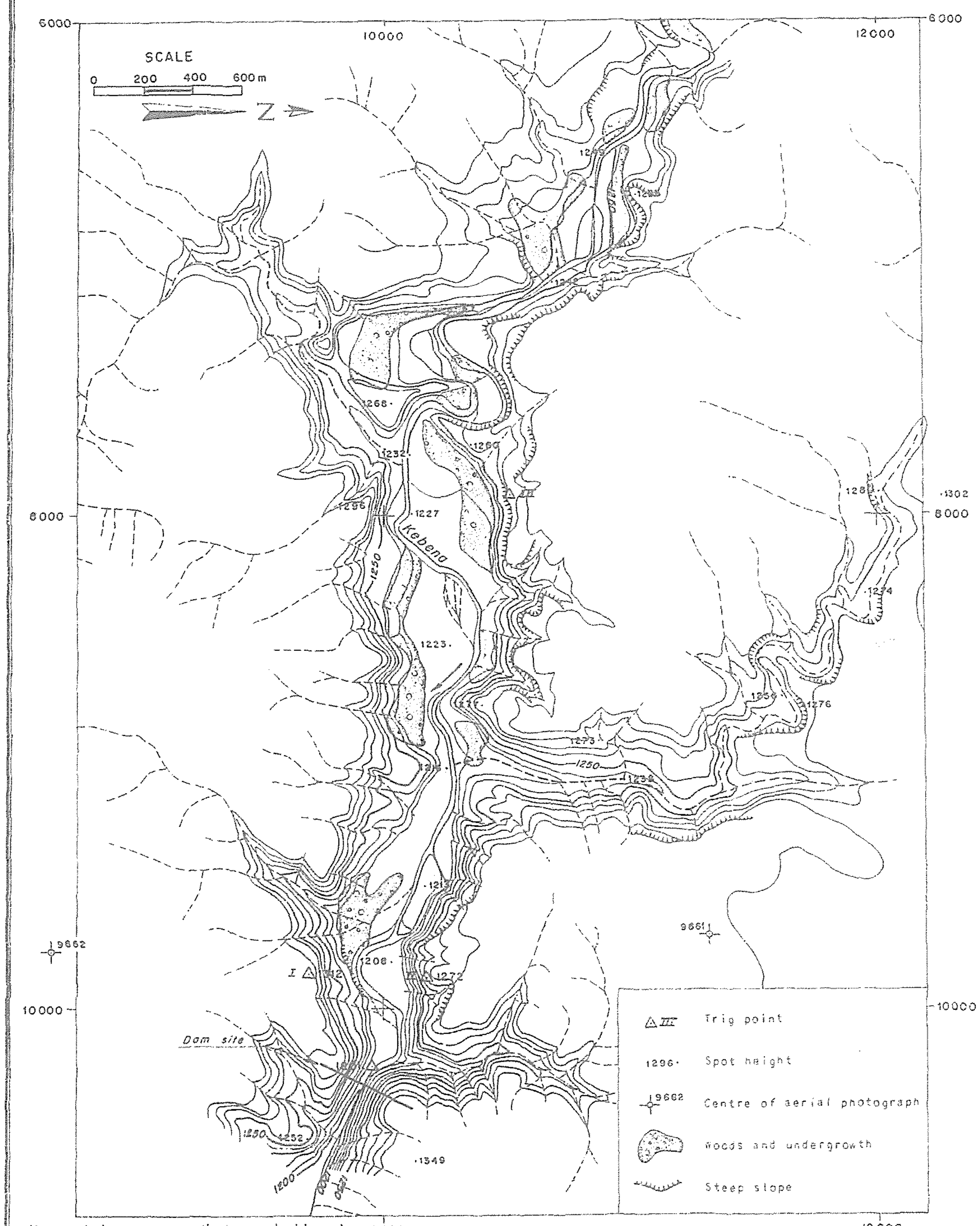
FINAL LONGITUDINAL PROFILE


LAKE ZIWAY CATCHMENT AREA
SCALE
0 10 20 30 40 50 km
MOJO
NAZRET
KOKA
Awash
L. GELILEA
Dubeta
ALEM TENA
Baricha
GOGETI
Meki
MEKI
2400 km²
L. ZIWAY 500 km²
ASELA
ADAMI TULO
3500 km²
Katar
L. HORA ABIYATA
L. LANGANO
L. SHALA
NORTH
Awash
Melka Gorge
Dubeta
DIVERSION
Meki
Melka Wokele
MEKI
BRIDGE = 1656.45
ZERO STAFF GAUGE = 1648.50
Sidamo
L. ZIWAY (~1635)
Crevasses
P 4 = 1689.26
P 5 = 1679.59
P 17
BED = (1696)
BED = 1673.49
BANK = 1696.16
BANK = 1732.40
BED = 1670.89
BANK = 1707.06
BANK = 1721.09
BED = 1658.17
BED = 1653.19
BANK = 1677.80
1671.76
1666.44
(1721)
(1725)
(1733)
(1721)
(1719)


AWASH STATION GORGES
APPROX. SCALE
0 1 2 3 4 5 6 km
N
Metehara
Awora Melka
Awash
Gotu falls 941/921
Arba Dima
Gauging Station
1028
Melka Oito 952
965
900
822
AWASH STATION 922
914
902
883
803
Gauging Station
Possible site for compensation da
Railway 853 bridge
Awash 794
Road bridge 804
865
Awashist 900
Gumbi
Dire Dewa
Longitudinal profile of the Awash river between Gotu falls and Melka Sedi
1050
1000
950
900
850
800
750
Gotu falls (km 348)
Awash station
Awash bed
Arba Dima (km 360)
Melka Oito (km 365)
Awash Left bank
Railway bridge
Dam
Reservoir
Road bridge
Arba bed
Ford
(km 380)
Gauging station (km 376)
(km 393)
Melka Indesa
Melka Dawda (km 403)
Melka Sedi


FLOW REGULATING AND H
Existing or under co
Investigated site of
Site of secondary imp
Other site considered
ADIS ABEBA
Aba Samuel
Koka (Awash I)
Awash II
Awash III
Awash IV
Diversion Meki-Koka
Kesem
Compensation dam
DEBRE MARKOS
LEKEMTE
GINCHI
JIMA
ADIS ALEM
TULU BOLO
TEJI
Melka Kentara
GOGETI
AKAKI
SENDAFA
DUKEM
DEBRE ZEYT
MOJO
NAZRET
WELENCHITI
SHENO
CHACHA DEBRE BIRHAN
ANKOBER
CHEFEDONSA
BOLO
MEKI
ALEM TENA
ADAMITULO
SHASHEMENE
ASELA
BOFA
METEHARA
AWASH Station
AWASHIST
ARBA
MIES
L. Ziway
L. Galilea
L. Shala
L. Hora
Abiyata
L. Langano
L. Awasa
L. Beseka
Wenji plantation
Era falls
Melka Bokara falls
Gotu falls
Aleydegi plain
Melka Sadi
Melka Warar
Amibara
Fentale
Termabar pass
Asebot

TOPOGRAPHICAL MAP OF KEBENA RESERVOIR



Map made by stereocompilation method based on the Blue Nile set of aerial photographs and using the Stereoflex - SOM equipment of SOGREAH GRENOBLE

Relative horizontal and vertical coordinates
Height between contour lines = 10 m

KEBENA DAM SITE

Geological survey