U.N. DEVELOPMENT PROGRAMME — WORLD HEALTH ORGANIZATION

DESIGN PHASE

OF THE

WATER SUPPLY AND SEWERAGE WORKS

FOR THE

ACCRA-TEMA METROPOLITAN AREA, GHANA

WEIJA NEW DAM

FOR MUNICIPAL WATER SUPPLY AND IRRIGATION

— Addendum —

CONSULTING ENGINEERS:

TAHAL CONSULTING ENGINEERS LTD.
in Association with
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TEL AVIV
AUGUST 1968
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C O N T E N T S

A. INTRODUCTION

The report on the Weija New Dam* for Municipal Water Supply and Irrigation as part of the Design Phase of the Water Supply and Sewerage Works for the Accra-Tema Metropolitan Area was submitted to the W.H.O., the Ghana Water and Sewerage Corporation and other Government bodies, and members of the Advisory Panel, for their review.

The report was discussed at the Panel Meeting held in Accra in June, 1968. The meeting was attended by members of the Advisory Panel, representatives of the World Health Organization, representatives of the Ghana Water and Sewerage Corporation, and representatives of the Consultants - Tahal Consulting Engineers and Engineering Science Inc.

The findings of the report were discussed, and the recommendations to build a new dam at Weija for municipal supply and irrigation with NWL at 47 feet O.D., and a reservoir of 25,000 mg capacity, were accepted. However, further clarification of certain points was requested, and detailed explanations of these are given in the following Addendum.

The sanitary aspects of Weija impounding reservoir, and pollution of the river by upstream consumers were also raised at the meeting. A study of these subjects will be carried out in due course, and coordinated with the investigations of the WHO Mission. The findings of this study and the recommendations will be submitted to the WHO, together with the Design Reports, at the end of the present contract.

It is recommended that Ghana Water and Sewerage Corporation should immediately take steps to ensure the acquisition of the land designated for the impounding reservoir, demarcate its exact boundaries, halt any planned development and initiate the resettlement programme for the farmers living within one mile of the area to be flooded.

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* Tahal, March 1968, P.N. 828

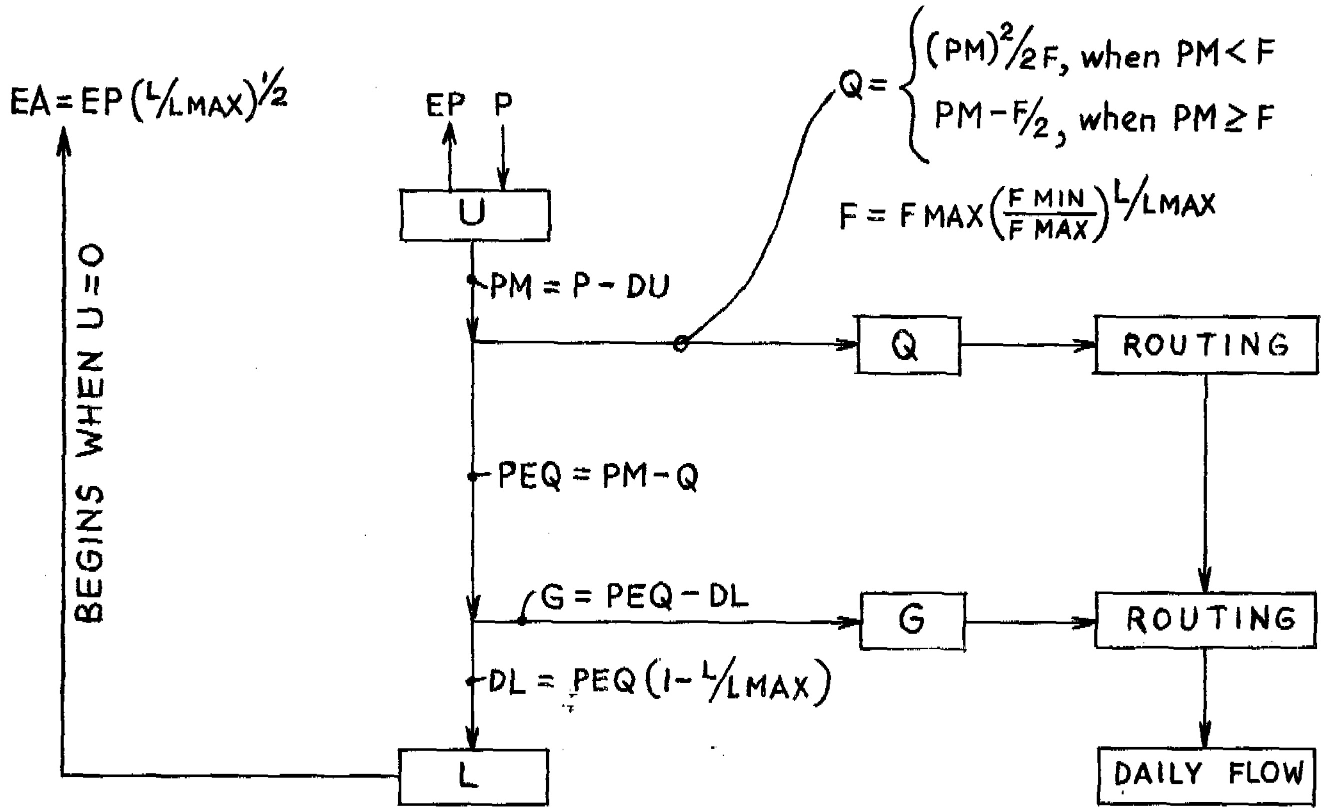
B. <u>RIVER DENSU HYDROLOGY</u>

<u>Method Used For Calculating Runoff</u>

1. <u>The Computer Flow Chart</u>

The essentials of the hydrological method used are outlined in the main report. The method is an adaptation of hydrological simulation techniques developed by Stanford University, Stanford, Calif., U.S.A.*

The computer flow chart and the functions used are given below.



* Ray K. Linsley and Joseph B. Franzini, Water Resources Engineering, McGraw-Hill Book Co., New York 1964, pp 46, 47.

2. <u>Explanation of Computer Symbols</u>

(symbols with a subscript t represent time variables)

P_t = Daily rainfall at a station, inches

EP_t = Potential daily evapotranspiration, inches (obtained by dividing the mean monthly evaporation at Accra by the number of days in the month, effective only when $U_t > 0$).

U_t = Upper zone moisture storage, inches
$= U_{t-1} + P_t - EP_t$ (represents interception by vegetation, depression storage and forest litter, limited by UMAX).

UMAX = Upper limit of U_t, inches

DU_t = Upper zone moisture deficit, inches
$= UMAX - U_{t-1}$

PM_t = Daily rainfall in excess of initial abstraction by the upper zone, inches

Q_t = Daily contribution to surface runoff, inches (subsequently converted into units of discharge by assigning the rainfall station an "effective" area, and routed downstream).

L_t = Lower zone moisture storage, inches
$= L_{t-1} + DL_t - EA_t$

DL_t = Daily retention by L_t, inches

EA_t = Actual daily evapotranspiration, inches (effective only when $U_t = 0$).

LMAX = Upper limit of L, inches

F_t = Maximum infiltration capacity, inches/day (effective only when $U_t = UMAX$).

FMAX = Upper limit of F_t, inches/day

FMIN = Lower limit of F_t, inches/day

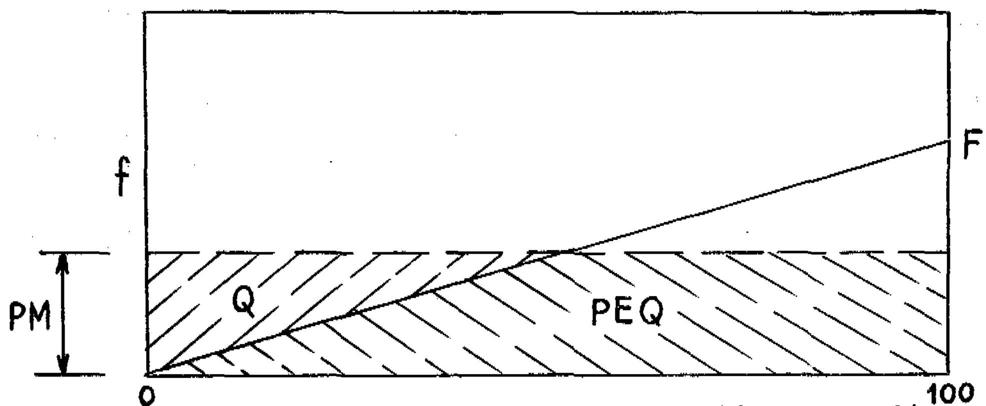
PEQ_t = Rate of percolation into L_t, inches/day

G_t = Daily contribution to subsurface flow, inches (treated the same as Q_t above)

3. The runoff functions

The functions that are shown on the flow chart are essentially empirical. However, they are based on current knowledge of the physical processes involved, and have already been tried out on other watersheds with good results. The following should be noted, with regard to the functions.

The functions for Q* takes into account non-uniform infiltration conditions in the watershed. It is assumed that the percentage of watershed area with an infiltration capacity less than or equal to some value f, varies linearly with f in the range $0 \leq f \leq F$. This is illustrated in the sketch below.



percentage of area with infiltration capacity less than or equal to f.

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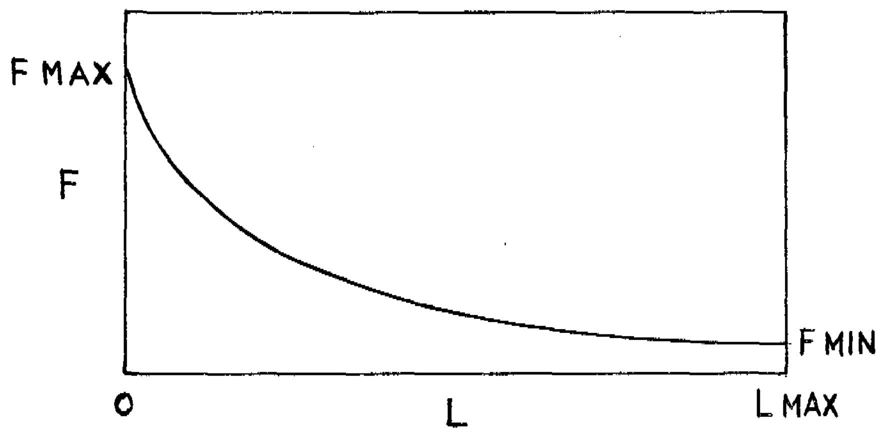
* The subscript t will be omitted from here onwards for the sake of simplicity.

It follows that $Q = \frac{PM^2}{2F}$ when $PM < F$,

and $Q = PM - F/2$ when $PM \geq F$.

Thus, the quantity that percolates into the ground is PEQ which is computed by subtracting Q from PM.

F, in turn, is assumed to vary with the soil moisture, L, by the relation $F = FMAX(FMIN/FMAX)^{L/LMAX}$ as depicted in the sketch below.



The function for F satisfies 2 preset conditions;

F = FMAX when L = O (i.e. soil moisture at wilting point), and

F = FMIN when L = LMAX (i.e. soil moisture at field capacity).

The function for G can also be written as G = PEQ (L/LMAX). It yields G = 0 at L = 0 and G = PEQ at L = LMAX. Thus, L will approach LMAX but will never surpass it.

The depletion by evaporation from L begins when U = 0. Hence EP and EA are not applied simultaneously. The function for EA yields EA = 0 at L = 0 and EA = EP at L = LMAX.

4. <u>The Runoff Constants</u>

The watershed above Nsawam was divided into 3 sub-areas. The rainfall over each sub-area was assumed to be represented by the three stations (advancing downstream), Tafo, Suhum and Nsawam, respectively. The watershed below Nsawam was considered as a single sub-area with rainfall equal to 0.8 of that at Nsawam.

The following constants were used for the sub-areas.

Constants	Sub - Areas			
	Tafo	Suhum	Nsawam	Nsawam x 0.8
Effective area sq. miles	194.0	194.0	195.0	367.0
UMAX	3.0	2.5	2.0	1.5
LMAX	28.0	28.0	24.0	12.0
FMIN	0.5	0.5	0.5	0.5
FMAX	80.0	80.0	60.0	30.0

In determining the "effective" areas it was ascertained that the weighted averages of the rainfall used corresponded with the mean rainfall over the watersheds.

The constants which appear in the above table were derived by a trial and error procedure guided by the apparent physical characteristics of the watershed. The selection of the constants proceeded as follows:

> First, a set of constants was assumed and fed into the computer. The flow results were then compared with the observed data and the constants changed accordingly. The set of constants finally chosen was that yielding the best fit between the computed and the observed flows. It should be pointed out that not all combinations have been tried and that it is probable that more than one set of constants will yield reasonable results. It should also be noted that the constants FMIN and FMAX are treated in the programmes as mean daily values. If the duration of most daily rains is, say, 3 hours, the per hour values of these constants would be the figures in the table divided by 3.

5. The Routing Procedure

The computed surface runoff of each sub-area was lagged and routed to the outlet of the watershed at Nsawam or Weija. The procedure utilized the construction of time-area diagrams which were subsequently routed by the Muskingum method with X = 0*. The isochrones with respect to Weija were as follows:

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* Ray K. Linsley, Max. A. Kohler and Joseph L.M. Paulus, Hydrology for Engineers, McGraw-Hill Book Co., New York 1958, pp 228, 229, 237.

Sub-Area	Isochrones
Nsawam x 0.8	0 - 2 days
Nsawam	2 - 3 days
Suhum	4 - 5 days
Tafo	5 - 6 days

The routing coefficients that were used in conjunction with the Muskingum method were derived by assuming that K* equals the mean flow time from sub-area to watershed outlet. The derived coefficients* at Nsawam and Weija are given below.

Sub-Area	Coefficients at Nsawam			Coefficients at Weija		
	C_o	C_1	C_2	C_o	C_1	C_2
Tafo	0.11	0.11	0.78	0.08	0.08	0.084
Suhum	0.14	0.14	0.72	0.09	0.09	0.82
Nsawam	0.33	0.33	0.34	0.14	0.14	0.72
Nsawam x 0.8	-	-	-	0.33	0.33	0.34

The following table shows the peak discharges and time lags (the time lapse between the day of rain and the day of peak runoff) resulting from a storm of one day's duration yielding 1 inch of run-off in each sub-area.

* Ray K. Linsley, Max A. Kohler and Joseph L.M. Paulus, Hydrology for Engineers, McGraw-Hill Book Co., New York 1958, pp 228, 229, 237.

Sub-Area	At Nsawam		At Weija	
	Peak discharge cfs	Time lag days	Peak discharge cfs	Time lag days
Tafo	910	5	710	7
Suhum	1,000	4	780	6
Nsawam	2,020	1	1,080	4
Nsawam x 0.8	-	-	3,390	2

The routing coefficients for the subsurface flow were derived from the actual rates of flow depletion during low flow periods at Nsawam. A mean recession rate of 0.95 per day at Nsawam was assumed applicable in all sub-areas, yielding identical routing coefficient of 0.025, 0.025 and 0.95 for C_0, C_1 and C_2 respectively.

C. CONSIDERATION OF PROPOSAL TO CONSTRUCT WEIJA NEW DAM IN STAGES

An alternative proposal to implement Weija New Dam in stages has been studied and has not been found economically justified. The Weija New Dam is part of the Second Stage Works which are to be commenced in 1974.

The dam is planned to reach its full capacity during 1978, 4½ years later, so that construction in stages would have to be done during this short period. At the beginning of the Second Stage - 1974, the domestic supply from Weija to Accra Low Pressure Zone will be 70% of the ultimate capacity of the 40 mgd Weija Second Stage Domestic Water Works, and at the end of 1974 it will be 75% of this ultimate capacity. Domestic and irrigation supply together will require, at the beginning of the Second Stage in 1974, a capacity of 85% of the Weija impounding reservoir ultimate capacity, and at the end of 1974, 87.5% of the Weija impounding reservoir ultimate capacity. It will not be practical to build a dam for the Second Stage for less than the first year's requirements of 87.5% of the reservoir gross capacity, or 0.87 x 25,000 = 21,700 mg. This capacity is provided at a reservoir level of 45.5 ft O.D., or 1.5 ft below the ultimate level. The spillway, piers, gates, intake, upper and tailwater channel will have to be constructed in the First Stage for the maximum flood. Thus, only 1.5 ft of the top of the embankments could be postponed, with a relatively minor saving.

If it is assumed that the dam will be built in two phases with the first phase having a normal water level of 40 ft, about 50% of the ultimate storage capacity, and the second phase for the ultimate capacity being constructed at the end of the Second Stage Scheme in 1978, this reduced capacity means either a risk of shortage during 1974-1978 if consumption follows irrigation and domestic design forecasts, or the postponement of the irrigation project until after 1978. Calculations indicate that even with a reduced height of 7 ft in the First Stage, it would not be economically justified to build the dam in two phases.

Under this alternative the entire gated spillway section would have to be built during the first phase of construction, and only 6 ft of the top section of the embankment could be postponed. It was found that the cost of this part of the Works is about $120,000, with interest at 6% over 4½ years amounting to $36,000. This amount of saving will be outweighed by the mobilization cost for the second phase of construction and the higher unit costs which will be required for this relatively small work. It should also be noted that the borrow area of the filling material for the proposed dam embankment has been found within the area of the proposed impounding reservoir. If the material in this borrow area is not used in the first phase of construction, it will obviously not be available for the second phase of construction. Similar material will thus have to be imported from a much greater distance with a concomitant increase in cost.

D. DREDGING OF RESERVOIR AREA FOR STORAGE

The possibility of dredging the reservoir area and thus creating part of the required storage while at the same time reducing the dam height, has been studied and has not been found economically justified. Dredging is an expensive procedure requiring the determination of the location of silt by accurate sounding, the transportation of the dredger to the site (or shipment from overseas), the installation of floating pipes for the delivery of silt, and operating the dredger. The comparison has been based on dredging cost of $0.9 per cu.yard, or $5,000,000 per 1,000 mg. The required capacity is 25,000 mg and this is provided by the full height of the dam with normal water level of 47 ft O.D. at a cost of only $3,200,000.

E. DETERMINATION OF FREEBOARD

In the following, the "normal freeboard" and the "minimum freeboard" will be determined.

The term "normal freeboard" is defined as the difference in elevation between the crest of the dam and the normal reservoir water level as fixed by design requirements. The term "minimum freeboard" is defined as the difference in elevation between the crest of the dam and the maximum reservoir water surface that will result if the inflow design flood occurs and the spillway functions as planned.

The normal freeboard must meet the requirements for long term storage. It must be sufficient to prevent overtopping of the embankment by abnormal and severe wave action of rare occurrence that may result from an unusually sustained wind of high velocity from a critical direction, or as a result of an earthquake. Minimum freeboard is provided to prevent overtopping of the embankment by wave action which may coincide with occurrence of the inflow design flood. Minimum freeboard provides a safety factor against many contingencies, such as settlement of the dam to a greater extent than anticipated when selecting the camber. The minimum freeboard is also established on the assumption that the dam should not be overtopped as a result of malfunctioning of the controlled spillway resulting from human or mechanical failure to open the gates.

The formulation of adequate freeboard requirements is important since overtopping of the earth dam under discussion by waves or floods could be disastrous for the stability of the dam.

DETERMINATION OF FREEBOARD

(feet)

	Normal water level	Maximum reservoir water level	Wave height	Wave rideup	Safety factor	Total free-board	Dam crest level
Minimum freeboard		+50 O.D.	3.3	5.0	1.0	6.0	+56 O.D.
Normal freeboard	+47 O.D.		4.1	6.1	2.9	9.0	+56 O.D.

The above figures do not include provision for settlement of the embankment.

F. WATERSHED MANAGEMENT

At present, various consumers draw water from the Densu River at various points along its course. Plans for the further exploitation of the river have recently been drawn up, and water requirements from it will probably increase in the future. However, pollution of a river is usually proportional to the increase in the use of water. In order to regulate the river throughout the entire basin for the benefit of all potential consumers, a watershed management authority will have to be created.

Watershed management is the scientific application of the principles of the watershed development process, and its objective is to meet the expected requirements for water by varying the quantity and quality of supplies delivered and by modifying the time and place of the use of these supplies as conditions permit. Normally, available supplies would have to be allocated among potential uses on an economic basis, but this approach may be affected to a considerable extent by legal, political, or other considerations. In formulating the development plans for a watershed, all possible uses of the water should be studied, as otherwise a truly comprehensive development plan will not be produced. For example, municipal, industrial, and irrigation supply, flood control and pollution control—may all be motivating factors in the development of river basin projects.

It is only in exceptional cases, in arid regions, that streams are exploited or regulated to their maximum potential. The degree to which the physical limit of regulation is approached is a function of technological, political, economic, sociological and ethical factors.

In order to select the optimum combination of alternative competitive and complementary water uses while capitalizing on the economy and resource allocations permitted by mulit-purpose project development, a coordinated system analysis of engineering, economic, ecological and social principles is required.

The achievement of an efficient integrated system of multi-purpose projects to promote the development of the watershed, requires the creation of a legal public agency which will represent the interests internal to the management plan. These internal interests are associated with various parts of the watershed development; however, they are not necessarily in agreement with each other. In fact, conflicts of interest generally exist, and the main task of the public agency will be to reach the optimum interest among the conflicting interests. The public agency will include representatives of Ghana Water and Sewerage Corporation, the Ministry of Agriculture, the Ministry of Economic Affairs, the Ministry of Works and Housing, the Ministry of Local Government, municipal authorities, local councils and others.

The public agency will have to derive its authority from a law based on the water-rights system. The importance of the water-rights system is that it defines the legal privileges of water use in terms of a property right, and prescribes legal remedies for infringement of these privileges.

As a first step towards a comprehensive development of the watershed, the public agency will have to put into effect the principles of watershed planning. These principles can be defined as follows:

(a) The determination of current and future needs.

(b) The appraisal of all possible means to meet these needs.

(c) The selection of the most economic approach to satisfy the anticipated requirements.

The detailed investigations and studies that should be initiated by the public agency, are as follows:

(1) Population, economic land use, and other associated planning studies which are essential for the prediction of the water quality and quantity requirements.

(2) Hydrological investigations to provide estimates of the available quantities of surface and ground water.

(3) Investigations to evaluate the physical, chemical, and biological characteristics of the surface water of the watershed.

(4) Studies of the storage capacities and uses of existing and proposed reservoirs in the watershed. It will be necessary to carry out topographical and geological investigations of the basin with the aim of determining additional reservoir sites and of estimating the potential storage.

(5) Studies to provide information on existing and anticipated sources of wastes which will be introduced into surface or ground water bodies. Information on the optimum treatment that these wastes could economically be expected to undergo will also be required.

(6) Estimate of low flow requirements, as these play an important role in fish and wild life preservation and in combating pollution.

(7) Evaluation of socio-economic benefits which will result from the proposed water resources programme.

A plan for the future development of the basin will be prepared, based on the above investigations and studies.

This plan will provide the public agency with a most important instrument which will enable it to initiate the financing and, eventually, the implementation of the various development projects.

It is recommended that the GWSC immediately take the necessary steps to create the said public agency so that decisions on further development of the Densu river can be taken before the mid-seventies.