



CONFIDENTIAL

EFFECT OF PROPOSED HANGKLIP DAM ON PALMIET RIVER MOUTH

submitted to

DEPARTMENT OF WATER AFFAIRS, FORESTRY AND ENVIRONMENTAL CONSERVATION

**COASTAL ENGINEERING AND HYDRAULICS DIVISION
NATIONAL RESEARCH INSTITUTE FOR OCEANOLOGY
COUNCIL FOR SCIENTIFIC AND INDUSTRIAL RESEARCH**

CSIR REPORT C/SEA 8048

**Stellenbosch, South Africa
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SCOPE

In order to determine the effect of the proposed Hangklip dam on the stability of the mouth of the Palmiet estuary, computations were performed to quantify each of the parameters contributing towards the stability, namely,

- (1) The hydrology of the Palmiet, with particular reference to runoff and evaporation;
- (2) wave conditions in the coastal area immediately seawards of the estuary mouth; and
- (3) the rate of longshore wave-driven sediment transport into the estuary mouth.

The degree of stability of the mouth was quantified with the aid of the above information and the results were used to recommend a discharge policy for the proposed dam.

The work towards this report was performed by Mr. J. de V. Serdyn, Mrs. M.A. Hardman and Dr. D.H. Swart. Dr. Swart is also the author of the report.

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ACTING DIRECTOR

Stellenbosch
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1. INTRODUCTION

The Department of Water Affairs, Forestry and Environmental Conservation (DWAFFEC) are planning the Hangklip dam, to be built in the Palmiet River estuary in the mountain range situated on the Western Cape coastline (see Figure 1). The primary function of this dam will be to generate electricity for ESCOM but the more important, long-term, function will be to supply water to Cape Town.

The Special Tasks Division of DWAFFEC are at present studying the possible environmental impact of the dam. As part of this impact study the effect of building the dam on the estuary downstream of the probable building site has to be assessed. Through its letter B1/720 V/0740/04/E002 dated 27 August 1980, the Special Tasks Division asked the Coastal Engineering and Hydraulics Division (CEHD) of this Institute to look into the discharge policy required for the proposed dam to ensure that the estuary is maintained in its present (pre-dam) condition.

This study was performed by the Sediment Group of the CEHD as part of its *basic research* into estuary mouth stability, mainly for three reasons, namely,

(1) it would prove a good starting point for research into the dynamics of estuary mouths, which will be started shortly as a joint venture of the Sediment and Long Wave Groups of CEHD;

(2) the Special Tasks Division did not have the funds available during the present financial year to pay for the research; and

(3) it is considered essential that this information should become available as soon as possible, as it could affect the further planning and design of the scheme.

The stability of the estuary mouth is determined mainly by two factors, namely,

(1) the freshwater runoff and the tidal flow through the mouth which tends to flush the mouth, especially during peak runoff events; and

(2) the influx of wave-driven marine sediment into the mouth, which tends to block the mouth.

The hydrology of the area, the wave climate in the coastal area and its related longshore sediment movement are therefore discussed in detail. An assessment is then made of the stability of the estuary mouth under pre-dam conditions. A discharge policy for the dam, based on the findings of each of the above aspects, is then recommended in Section 7.

2. HYDROLOGICAL DATA

2.1. General

To assess the flow in the Palmiet under pre-dam conditions, flow and evaporation data for the 13 hydrological years 1963/1964 to 1975/1976 were analysed (in this report all references to time are made in *hydrological years*; a *hydrological year* running from October to September). The average total annual rainfall for this period was 1 163 mm, with a standard deviation of 148 mm, whereas the average total annual rainfall for the period 1923/1924 to 1975/1976 was 1 222 mm with a standard deviation of 245 mm. It can therefore be safely assumed that data for the 13-year period are representative and can be used as such. The data were obtained directly from DWAFEC on computer printouts, since the published data extend only to 1969/1970 (Division of Hydrology, 1978). Only those analyses of the original data which are of interest to the study reported on here are discussed below. The original data are summarized in histogram form in Appendix A, Figures A1 to A12.

2.2. Runoff

Since runoff is one of the two main flushing agents at the estuary mouth, the other being tidal flow, various statistical analyses were performed on the daily average flow records for the gauging station G4M07A (see Figure 1 for its location). The following four aspects were dealt with:

- (i) daily flow rates;
- (ii) total annual runoff;
- (iii) persistence of low flow; and
- (iv) persistence of high flow.

Daily flow rates

The average values for the maximum, mean and minimum daily runoff, as found for the above-mentioned 13-year period, are shown in Figures 2, 3 and 4, respectively, together with their respective standard deviations. These figures indicate that the mean maximum daily discharge rate could be as high as

51 m³/s (in July), whereas the mean minimum daily discharge rate could be as low as 0,3 m³/s (in January). The mean daily discharge rate varied between 0,5 m³/s (in January) and 16 m³/s (in August).

Total annual runoff

The mean annual runoff, based on the data contained in these three figures, amounted to $177 \times 10^6 \text{ m}^3$, with a standard deviation of $37 \times 10^6 \text{ m}^3$. The lowest recorded annual runoff in the 13-year period was $104 \times 10^6 \text{ m}^3$ and the highest recorded runoff was $239 \times 10^6 \text{ m}^3$. These runoff figures are used in Section 6 to specify the operation of the proposed Hangklip dam.

Persistence of low flow

An indication of the persistence of periods of low flow is given by plotting the percentage of the time in any one month that the discharge rate was below some critical low level, as was done in Figure 5 for flows of 1 m³/s and 0,5 m³/s. Figure 5 indicates that the largest possibility for low flow (as an indicator of 'no flow') occurred in January to March. Discussions with a Mr. Niemand, long-time resident at Kleinmond, indicated that the mouth of the Palmiet River is closed on average once every two to three years, mostly in February and/or March. Interestingly, flow rates of less than 0,5 m³/s, that is, flow velocities through the mouth of the order of only centimetres per second, occur for 50 per cent of the time in January to March (see Figure 5). This implies that such low flow rates occur, on average, every second year.

Furthermore, interesting information can be derived from a computer-produced table for the period 1923/1924 to 1975/1976, supplied by DWA FEC, which shows the runoff after compensation for upstream usage. Since the gauging station G4M07 came into operation only in 1963 (G4M07A after 1976), a monthly rainfall/run-off regression model was used to extend the period of observation to 1923. In the following it is assumed that no flow can be interpreted as a closed estuary mouth. The data for this 53-year period can be summarized as follows:

- (i) No flow is listed on 28 occasions, covering a period of 38 months. This implies that the chance of no flow is 53 per cent in any one year.

(ii) Of these 28 occurrences, 20 persisted for one month, seven for two months and one for four months.

(iii) The estuary mouth was continually open, that is, no occurrence of no flow was recorded during 29 of the 53 years, or for 55 per cent of the total number of years. Furthermore, the mouth was closed once/year during 20 years and twice/year during four years.

(iv) The biggest predominance of mouth closures was in February, with a probability of occurrence of 26 per cent, followed by May, March and November with 13 per cent, 11 per cent and 11 per cent each, respectively.

(v) The longest period without any mouth closure was seven years (1936/1937 to 1942/1943), whereas the most consecutive years during which a mouth closure took place was nine years (1927/1928 to 1935/1936).

The above information, especially points (i) and (iii), confirms the earlier conclusion, namely, that the mouth is apt to close on average once every two years, most probably in February.

Persistence of high flow

In order to get some impression of the duration of peak runoffs, the highest daily discharge for each month in each of the 13 years 1963/1964 to 1975/1976 was used to normalize the flow rates for the days before and after the maximum event. A mean normalized hydrograph over the 13 years was obtained in this manner for the highest runoff in each month. These normalized hydrographs with the standard deviation around the mean curve are shown in Figures 6 to 11. Also shown on these figures is the ratio mean daily runoff rate to maximum daily runoff rate for each month.

The normalized hydrographs vary only marginally over the year, as shown in Figure 12. Since the peak runoff rates occurred in June, July and August, a separate normalized hydrograph is also shown in Figure 12 for these three months.

In effect the area above the line representing the mean flow is therefore representative of the total additional runoff during peak events. The total runoff represented by this area is given for each month in Figure 13. This figure indicates that the total runoff greater than the mean during a single peak event varies between $0,15 \times 10^6 \text{ m}^3$ (in January) and $7,7 \times 10^6 \text{ m}^3$ (in July).

2.3. Evaporation

The discharge from the proposed dam should at least be sufficient to compensate for the evaporation from the surface of the estuary. An analysis was therefore also made of evaporation data supplied by DWAFFEC for the period 1963/1964 to 1975/1976 for the Hangklip dam location. The mean evaporation per month in millimetres with its standard deviation is plotted in Figure 14. As expected, the highest evaporation occurred in the summer months (201 mm/month in January) and the lowest in winter (32 mm/month in July).

By assuming that evaporation takes place over the whole estuarine area downstream of the road bridge (which has an area of $0,193 \text{ km}^2$) at the rates given in Figure 14, it is possible to transform these evaporation rates to effective volumetric evaporation rates.

2.4. Summary

The analyses of runoff and evaporation have for convenience been summarized in Figure 15. This figure should be read in conjunction with Appendix Figures A1 to A12, so that the variability of the runoff and evaporation during each month can be visualized. It is especially illuminating to study the histograms for maximum discharge and evaporation, since these indicate that the maximum runoff rate in, for example, July, can be as low as $20 \text{ m}^3/\text{s}$ and the volumetric evaporation rate in January, for example, can be as high as $0,018 \text{ m}^3/\text{s}$. Figures 2 to 15 and A1 to A12 and the data derived from them form the basis for the assessment of the effect on the estuary mouth of the Palmiet River hydrology.

3. WAVE INPUT

3.1. General

For the computation of wave-driven longshore sediment transport rates information about wave height, period and approach angle at the breaker line is required. The most reliable data on wave height and period are obtained from a waverider accelerometer buoy, whereas the best information on approach angle is normally obtained from either radar images or visual observations. No reliable simultaneous records of height, period and direction were available for the Kleinmond coastal area, shown in Figure 1.

The wave climate in the area is determined by the weather pattern in the Roaring Forties. The CSIR (1980) has shown that the wave-height distribution is very similar for Slangkop (near Cape Point) and the SOEKOR drill rig location offshore at Mossel Bay. Very comprehensive deep-sea wave data, determined by visual observation by ships passing through the area, are available for the Mossel Bay offshore area. The total number of available observations is 5 535. This data set is therefore assumed to be valid for the Kleinmond offshore area.

3.2. Deep-water Wave Data

The deep-water data obtained from ships' observations for the Mossel Bay area contain waves from directions covering the full 360°. When these waves are applied to the Palmiet River area, only waves in the sector 90° to 270°, that is, southerly waves, can reach the shore. The original data are grouped in directional sectors of 30°, with their centre points at 0°, 30°, etc. to 330°. Of the total number of 5 535 observations 289 (or 4,8 per cent) are classified as 'confused', whereas 265 (or 5,2 per cent) are in the sectors 300° to 60° (northerly waves) and 4 981 (or 90,0 per cent) are in the sectors 90° to 270° (southerly waves). Forty per cent (or 1 976) of the southerly waves are classified as either undetermined or having wave periods of less than 5 seconds. Therefore the occurrences of wave data from the various direction sectors are modified as follows:

- (1) all waves classified as 'confused' have been neglected;
- (2) all northerly waves are classified as 'calms' for the Kleinmond area, that is, 289 cases in total;
- (3) only those southerly waves which are not classified as either undetermined or having periods of less than 5 seconds are regarded as arriving from a southerly direction and their frequencies of occurrence are corrected in such a way that the total number of waves arriving from a southerly direction remains unchanged (that is, 4 981).

The corresponding overall table of adapted wave occurrence is shown as Table I after the text.

Similar tables were made for each month for use in the sediment transport calculations but these are not reproduced here.

3.3. Refraction Study

As waves propagate from deep to shallow water they steepen (shoal) and their direction of approach tends to become more perpendicular to shore (refract). These processes become important when the water depth is less than half the wave length. The longest wave period in Table I is 13 seconds, with a corresponding deep-water wave length of 260 m. Shoaling and refraction therefore commence in water of approximately 130 m depth. Since the available data are valid for deep water and the information needed for the computation of sediment transport rates is required at the breaker line, it is necessary to perform a so-called 'refraction study', to compute the changes in the wave characteristics as a result of shoaling and refraction. The computation is terminated when the wave reaches the breaker point, which is defined as that point where the ratio wave height to water depth is 0,6.

A FORTRAN computer program developed by Du Toit (1977), which is operational on the CSIR's CDC computer, was used to perform the calculations. The program requires as input the wave height, period and direction in deep

water and a rectangular grid of depths covering the area through which the waves propagate. The area covered by the grid which was employed in the program is shown in Figure 16.

Part of the output is a refraction diagram, which shows the path of selected points on the wave, as it travels from deep to shallow water. The refraction diagrams for those wave conditions in Table I which after refraction could arrive at the Palmet river mouth, that is, for waves with periods varying between 7 and 13 s and having deep-water approach directions between 150° and 270° , are shown in Figures 17 to 36.

3.4. Breaker-line Wave Data

The wave characteristics at the breaker line, as predicted by the refraction program, are summarized in Table II. The table indicates that the water depth at wave breaking varies between 0,56 m and 8,14 m, depending on the wave height, with a mean breaker depth of 2,7 m.

Keeping in mind that the coastline orientation at Palmet is 309° , a study of Tables I and II indicates that 39 per cent of the incoming waves (excluding calms), arriving from 240° and 270° , would generate sediment movement from west to east past the estuary mouth whereas the remaining 61 per cent of the waves (90° to 210°) would generate east to west sediment movement. On the other hand, the breaker-line wave angles are larger for the 240° and 270° deep-water angles of approach, which would most probably lead to higher sediment transport rates.

4. SEDIMENT TRANSPORT RATES

4.1. General

Because of wave action there is a continuous flux of momentum in the direction of wave approach. Inside the breaker zone, as waves dissipate their energy by breaking, the gradient in momentum flux causes a rise in water level and, if the waves approach the shore obliquely, a current runs alongshore, predominantly inside the breaker zone. Bottom sediment, which has been stirred up from the bed by wave action and especially by the turbulence due to breaking waves, is transported in the longshore direction by the longshore current. The direction of this longshore transport of sediment obviously depends on the direction of approach of the waves.

There are two main ways of computing the longshore sediment transport, namely, (1) by using an overall predictor, which gives for each wave condition the overall quantity of material moved but which does not specify to where it is moving, or (2) by using a detail predictor which supplies the transport rate at each water depth, thereby allowing one to obtain a cross-section of the sediment movement from the shore to beyond the breaker zone.

In the present exercise it was sufficient if the total volume of sediment moved in each direction was known. Therefore, an overall predictor, the CERC formula (SPM, 1973), was used since it was simpler to apply than the detail predictors.

4.2. Computations

The CERC formula simply relates the longshore component of the energy flux to the rate S_x of longshore sediment movement. The resulting factor of proportionality depends on the grain size of the bottom material (Swart, 1976).

$$S_x = K(D) f_i T_p H_{os}^2 K_{rb}^2 \sin 2\theta_b \quad \dots (4.1)$$

(with S_x in m^3/yr)

where $K(D)$ = parameter depending on the grain size

$$= 91 \times 10^4 \log_{10} \left(\frac{0,00146}{D_{50}} \right) \quad \dots (4.2)$$

D_{50} = median particle diameter;

f_i = fractional occurrence of the wave condition under consideration;

T_p = wave period which contains the highest energy in the wave spectrum for the specific wave condition;

H_{os} = significant wave height (that is, mean of the highest one-third of the waves in the specific wave spectrum) in deep water;

K_{rb} = refraction coefficient at the breaker line, that is, ratio between the breaker-line wave height and the deep-sea wave height; and

θ_b = angle between the wave crest and the local bed contour at the breaker line.

Values of the deep-water wave conditions (f_i , T_p and H_{os}) were obtained from Table I, whereas breaker-line wave data (K_{rb} and θ_b) were obtained from the refraction study (see Table II). Sand samples taken in the estuary mouth and along the beach to the east of the estuary mouth showed that the median grain diameter $D_{50} \approx 0,4$ mm, which implied that

$$K(D) = 51 \times 10^4$$

(see Table III).

Using the above-mentioned sources of information it was possible to compute the longshore sediment movement with the aid of equation (4.1). Since the stability of the estuary mouth was to be assessed on a monthly basis, these computations were performed separately for each month. The total accumulated eastbound, westbound, net and gross transport rates calculated in this manner are given in Table IV.

4.3. Discussion

Table IV indicates that the sediment transport was predominantly eastbound, with a mean annual eastbound rate of $1,35 \times 10^6 \text{ m}^3/\text{yr}$, as opposed to a mean annual westbound rate of $0,34 \times 10^6 \text{ m}^3/\text{yr}$. This implied a net eastbound movement of $1,01 \times 10^6 \text{ m}^3/\text{yr}$.

However, the actual sediment transport rate attained in the field does not only depend on the longshore component of the wave energy flux and the median grain size, but also on the availability of movable material. In a case such as Palmiet, the net annual transport should by necessity be much lower than the figure given above, because of the limited stretch of sandy beach (650 m), nested in a rocky coastline. Normally a longshore sloshing mode is established in cases like these with a resulting slight realignment of the coastline (and breaker angles correspondingly smaller than predicted). As a result the actual volume of sand moved in both directions is usually lower than the lowest predicted volume. This implies a gross transport rate (total of the transport in both directions) of less than $0,68 \times 10^6 \text{ m}^3/\text{yr}$ (that is, double the lowest of the west and eastbound annual longshore transport rates, or $2 \times 0,34 \times 10^6 \text{ m}^3/\text{yr}$) for the Palmiet situation, with a net transport of zero.

One should, however, also remember that the above argument applies to the centre of the beach to the east of the estuary mouth where the transport in both directions is well-established. The situation is different at the estuary mouth. Westbound sediment can be transported unhindered towards the mouth. On the other hand, there is most probably little eastbound supply of sand from the west of the estuary mouth since the area is completely rocky. Sediment would have to be picked up from the sea bed inside the breaker zone directly to the east of the mouth which would, under wave conditions which generate eastbound movement, be in the lee of the rocky promontory directly to the west of the mouth. In such a diffraction (lee) area the resulting amount of transport would be less than under normal conditions.

Furthermore, part of the sediment arriving at the estuary mouth will be transported into the estuary by the flood flow. Since the wave action inside the estuary mouth is less during ebb flow only part of this material can be transported out to sea again during falling tide. As a result, the volume of material arriving at the estuary mouth (from the east) would be more than the volume transported away (towards the east). Without a much more detailed study it would not be possible to accurately determine the extent of the difference. Using the wave height variation alongshore in a diffraction area as basis, one can estimate that about 50 per cent of the material transported into the mouth area from the east would again be

transported out towards the east. The rest would remain inside the estuary and would be flushed out again only during periods of peak freshwater runoff. Furthermore, some beach erosion can be expected to occur some distance to the east of the estuary mouth, to compensate for the deficit in the volumetric balance of longshore sediment movement.

Although the coastline to the west of the estuary mouth is rocky, as was stated earlier, it is possible that some sediment could be transported alongshore in deeper water, which could then supply sand to the beach in the eroded area. The magnitude of this transport, which will be only a fraction of the predicted eastbound capacity, if it occurs at all, is not important for the discussion of the stability of the estuary mouth, since this sand will bypass onto the beach without affecting the mouth.

5. STABILITY OF THE ESTUARY MOUTH

5.1. General

As stated in Section 1, the stability of an estuary mouth is determined by the tidal (or peak freshwater) runoff which represents the flushing mode and the longshore wave-driven sediment movement, which represents the blocking mode. For the purpose of the present study it was assumed that the flushing mode, during which the estuary mouth is flushed open to the maximum dimension it will attain during the year, is governed fully by the peak freshwater runoff during the year (and similarly for each month). Afterwards the ensuing tidal flow obviously helps to keep the mouth open for some period of time. Normally the longshore wave-driven sediment movement used in the determination of the stability of estuary mouth would be the gross transport, since sediment influx from both sides would tend to block the estuary mouth. However, the discussion in Section 4.3 implies that it would be more appropriate in the case of the Palmett to use an influx of sediment into the estuary mouth of the order of half the westbound sediment transport.

Bruun and Gerritsen (1960) suggested a quantitative criterion for the determination of the stability of an inlet, which was based on actual field observations at a number of inlets around the world. They used as determining criterion the ratio Ω/S_{xg} , where Ω is the total outflow (in m^3) during spring tide (in our case during a peak freshwater runoff) and S_{xg} is the gross value of S_x (in m^3/yr).

The following *grading for inlet stability* was obtained:

- | | |
|------------------------------------|--|
| "(1) $\Omega/S_{xg} > 150$ | conditions are relatively good, little bar and good flushing |
| (2) $100 < \Omega/S_{xg} \leq 150$ | conditions become less satisfactory and offshore bar formation becomes more pronounced |
| (3) $50 < \Omega/S_{xg} \leq 100$ | entrance bar may be rather large, but there is usually a channel through the bar |

(4) $20 < \Omega/S_{xg} \leq 50$

all inlets are typical 'bar-bypassers'; waves break over the bar during storms and the reason why the inlets 'stay alive' at all is often that they during the stormy season like the monsoon get 'a shot in the arm' from freshwater flows; for navigation they present 'wild cases', unreliable and dangerous

(5) $\Omega/S_{xg} \leq 20$

is descriptive of cases where the entrances become unstable 'overflow channels' rather than permanent inlets."

The description by local inhabitants (such as Mr. Niemand) of the state of the estuary mouth indicates that the Palmet typically falls within the fifth type during summer with a shift towards the fourth category during winter.

5.2. Computations

To test this observation, computations were done on a monthly basis with the aid of the maximum freshwater runoff per month as shown in Figure 2 and the blocking capacity of wave-driven longshore transport (being assumed half the westbound longshore transport). The results, given in Table V, indicate that the ratio Ω/S_{xg} varies between one (in January) and 52 (in July).

5.3. Discussion

The data contained in this table do indeed verify the observations on the physical state of the estuary mouth. The table shows that the estuary mouth 'gets a shot in the arm' as a result of peak freshwater runoffs, as Bruun and Gerritsen put it, for an average of five months per year, namely, May to September. This period can be regarded as the flushing period of the estuary. Furthermore, it was shown that the estuary mouth reached its lowest grading (poorest stability) during December through February, which can be considered the blocking period. These periods agreed well with the deductions made in Section 2 and with observations by local inhabitants of the behaviour of the estuary.

Furthermore, Table V indicates that the tidal flow will not be sufficient to keep the mouth in the fourth category, that is, in the stable situation, since the ratio Ω/S_{xg} for tidal flow reaches a value of only about 0,9. This implies that in periods when no floods occur and the tidal flow alone is available to flush the mouth, it will revert to an estuary mouth of the fifth category, that is, 'an unstable overflow channel'.

In order to quantify the cross-sectional areas of the mouth under these two conditions, namely, peak freshwater flow and tidal flow, some further computations were performed, the results of which are summarized below.

Peak freshwater flow

Using the critical mobility number, as defined by Ackers and White (1973), which defines the critical depth at which sediment movement will start (or cease), as basis, it can be shown that the average depth of the estuary mouth attained after a freshwater runoff with a maximum discharge rate of $50 \text{ m}^3/\text{s}$ (see Figure 2) is about 3,5 m below MSL. The corresponding maximum cross-sectional area is 90 m^2 below MSL.

Tidal flow

The cross-sectional area of a tidal inlet is directly related to the tidal prism, that is, to the total volume of flow into the inlet during spring tide (O'Brien, 1969). Most American researchers have included only large tidal inlets in their analyses of the relationship between cross-sectional area and tidal prism. O'Brien (1969) and Nayak (1971), however, also studied tidal inlets formed under laboratory conditions, whereas Moes (1979) plotted data for a few South African inlets onto the curves of O'Brien and Nayak. An estimate of the tidal prism for the Palmiet can be made by assuming a maximum tidal variation of 2 m at the mouth during spring tide and no tidal variation at the road bridge across the Palmiet River, about 1,6 km from the estuary mouth. The total wetted area of the lagoon, bound by the estuary mouth and the road bridge, is about $0,193 \times 10^6 \text{ m}^2$ which corresponds to a tidal prism of about $0,2 \times 10^6 \text{ m}^3$. According to the graph given by Moes this means that the cross-sectional area of the mouth can vary between 15 m^2 and 30 m^2 below MSL.

The above discussion suggests that tidal flow will be able to maintain a mouth of cross-sectional area of between only 15 and 30 m², whereas a peak freshwater runoff could open the mouth to about 90 m² (depending upon the magnitude of the runoff). These figures agree with observations at the Palmiet estuary mouth. It would be advisable to monitor the cross-sectional area in the throat of the estuary mouth both before and after the dam comes into operation to establish in this way the effect of the dam on the mouth.

When the mouth has been restricted to such a degree that it is in equilibrium with the tidal flow it is obviously easier for a sudden storm to cause the mouth to close than when the mouth is wide open. In reality the estuary mouth is therefore in a state of 'dynamic equilibrium': it opens wide during a peak freshwater runoff whereas it closes gradually to a cross-sectional area of between 15 and 30 m² below MSL after the peak event, only to be flushed open again during the next peak event. If the peak events do not follow each other with sufficient regularity, a severe storm at sea with accompanying heavy westbound longshore sediment movement could close the mouth.

The order of magnitude of the ratio Ω/S_{xg} (for peak freshwater runoff) and the fact that the flushing/blocking periods occur for roughly equal times indicate that the estimate of the magnitude of the blocking capacity of the longshore wave-driven transport, namely, one-half the westbound transport, was reasonably accurate.

6. OPERATION OF THE PROPOSED HANGKLIP DAM

6.1. General

It is obvious that the only way in which the estuary mouth can be maintained exactly in its pre-dam condition would be to allow all water to flow past the dam. Since this is not the object of building the dam, it is necessary to set some guidelines for the situation that have to be attained as minimum requirements. In doing so, one would like to keep the natural processes intact as much as possible; the amount of water used should not detract from the feasibility of the dam. From the discussions in Sections 2 to 5 it is apparent that the following processes are natural ingredients for the stability balance of the estuary mouth:

(1) During winter (June to September) peak freshwater events give the estuary mouth 'a shot in the arm', as Bruun and Gerritsen (1960) put it, and the mouth opens to a cross-sectional area of about 90 m² below MSL;

(2) with the mouth thus flushed open tidal flow through the mouth takes place unhindered and can maintain a cross-sectional area of the mouth below MSL of 15 m² to 30 m²;

(3) lateral sediment influx into the estuary mouth as a result of the longshore wave-driven sediment movement in the coastal area tends to block the mouth;

(4) during the summer months the restriction becomes so marked, evaporation so high and the freshwater runoff so low that closure of the mouth occurs on average once every two years in February and/or March;

(5) with the start of the rainy season in winter the cycle restarts with the flushing mode.

With this behavioural pattern as a basis, the key elements to be retained after construction of the dam are:

(1) the estuary should be flushed once per year to allow the proper 'cleansing' of the mouth of sediment;

(2) during the rest of the year a sufficiently high 'low runoff' should be maintained to compensate for the evaporation and so prevent hypersalinity if the mouth closes and no freshwater flow takes place.

These two aspects are elaborated on further below.

6.2. Minimum Flow Required

The mean *effective volumetric evaporation rate* recorded per month in the 13-year period 1963/1964 to 1975/1976 varied between approximately 0,015 m³/s in January and 0,002 m³/s in July (see Figures 14 and A1 to A12; and also Table VI), with a mean of 0,008 m³/s. If sufficient discharge is supplied to compensate for the total mean evaporation in each month the total volume let through the dam for this purpose would be 0,25 x 10⁶ m³, or 0,14 per cent of the mean annual runoff. In this way the water level would on average just be maintained in the estuary with no net loss due to evaporation or gain due to oversupply.

In years when the evaporation rate is less than the mean previously recorded rate some rise in water level will take place, should the mouth be closed. However, the increase in water level will not be sufficient to effect a breakthrough of the sand spit at the mouth, which normally has a crest level of approximately 3 to 4 m above LWOST. If the maximum evaporation rate for January were, for example, to be used as the supply rate throughout the year, the water level in the system would rise by only about 1,1 m if the mouth would be closed the whole year. Conversely, an evaporation rate greater than the mean previously determined evaporation rate would lead to a drop in water level with a corresponding increase in salinity.

On average the water level in the lagoon would therefore stay the same, with the possibility of some variation around this level if the mouth is closed. The variations in evaporation rate depicted on Figures A1 to A12 would lead to variations of ± 30 cm around the mean water level. It is therefore recommended that the mean evaporation rate per month, as set out in Table VI, be adopted as the discharge rate from the dam.

6.3. Stability of Mouth

The flushing which brings about the stability of the estuary mouth, as discussed in Section 5 and illustrated quantitatively in Table IV, varies from a maximum intensity in winter to a minimum in summer. Although the larger freshwater runoffs would determine the maximum cross-section to which the mouth could grow, the peak runoff in each month would in reality make a contribution towards flushing the mouth to yield a situation whereby the estuary mouth is on average closed for only about six weeks every two years (38 months in 53 years, see Section 2). It will obviously be impracticable to imitate this situation after the dam has been constructed. It is therefore recommended that only one flash flood be allowed through each year. Three aspects of the flash flood need to be considered, namely,

- (1) its discharge rate;
- (2) the best time to effect the flash flood; and
- (3) its duration.

These aspects are dealt with below.

Discharge rate for flash flood

Since (1) the maximum size to which the mouth can be scoured was shown to be directly related to the discharge rate through the mouth and (2) the time during which the estuary mouth will remain open after a single flash flood will be determined by the cross-section of the mouth, it is important that the magnitude of the flash discharge rate from the dam should be as close as possible to the maximum attained under pre-dam conditions.

Figure 2 indicates that the mean maximum monthly runoff rate has values of 44, 51 and 48 m^3/s in June to August, with the next highest monthly runoff rate being 24 m^3/s in September. It is therefore recommended that a flash discharge of 50 m^3/s be let out of the dam to ensure that the mouth cross-section reaches at least its maximum pre-flood value once a year. As shown in Section 5 the mouth will then reach a cross-sectional area of about 90 m^2 below MSL, provided that the flash flood lasts sufficiently long to enable the maximum cross-section to be attained.

Although it would appear from Figures A9 to A11 that the maximum daily discharge rate in June to August could be below $10 \text{ m}^3/\text{s}$ (reference to the original data shows the lowest maximum daily discharge rate in these three months to have been $6.1 \text{ m}^3/\text{s}$ on 21 June 1971) the maximum daily discharge rate in any given hydrological year varies between $29 \text{ m}^3/\text{s}$ in 1971/72 and $115 \text{ m}^3/\text{s}$ in 1973/74 with an average for the 13 years of $72 \text{ m}^3/\text{s}$. This implies that the recommended flash discharge rate of $50 \text{ m}^3/\text{s}$ is 31 per cent lower than the maximum daily discharge rate for the average year.

Therefore it is recommended that the flash flood at $50 \text{ m}^3/\text{s}$ should be administered every year, whether dry or wet.

Best month for flash flood

According to the stability criterion of Bruun and Gerritsen (1960) the minimum required value of the ratio flood discharge to influx of wave-driven sediment Ω/S_{xg} would be 20, in order to ensure that the estuary, as they put it, 'gets a shot in the arm' from the flash discharge, that is, that would allow it sufficient stability to remain open for some period of time. Table V shows that the maximum pre-dam value of this ratio is 52 (in July) and that a value of Ω/S_{xg} of 20 is exceeded during five months of the year (May to September).

If it is accepted that in order to limit the volume of water let through during a flash discharge the value of Ω/S_{xg} is limited to 20, it stands to reason that the discharge required to yield $\Omega/S_{\text{xg}} = 20$ will be lowest in the month in which the westbound transport is at its minimum. Bearing in mind that in order to maintain the pre-dam conditions as closely as possible the flash event should take place in the period June to August, Table IV indicates that the best month for the flash discharge would therefore be June, since during this month westbound longshore transport capacity is lowest. However, there are also other considerations.

According to Branch (personal communication) it would be advisable to flush the mouth as late in winter as possible, to ensure that the mouth is open in November, when the largest migration of fish through the mouth to spawn in the sea takes place. July or September might therefore be better months for the flushing. From DWAFEC's point of view it would be best to flush the mouth as early as possible during the wet season, thereby allowing more capacity for storage should high inflow into the dam occur.

It is tentatively suggested that the flash flood be administered in September. To further improve the stability, it would be advisable to choose the time for the flash flood to coincide with a period of *low wave action during September*. This could, for example, be judged by using the daily observations of wave height done at the Danger Point lighthouse, and administering the flash flood on a day when the wave height is less than, say, 1 m.

Duration of flash flood

Using the same technique as that used to determine Table V, it follows directly that a flash flood at a discharge rate of $50 \text{ m}^3/\text{s}$ should be maintained in September for about 15 hours to ensure that $\Omega/S_{\text{xg}} = 20$. The volume of water discharged during this period will be about $2,7 \times 10^6 \text{ m}^3$, or 1,5 per cent of the mean annual runoff. This figure represents the bare minimum total runoff during the flash flood that would ensure a reasonably well-scoured estuary mouth.

Information about the time needed to scour the Swartvlei on the South Western Cape coast open after the artificial opening of the mouth during September 1975 indicates that the estuary became tidal 36 hours after opening the mouth. If it is assumed that the maximum water levels in the Swartvlei and Palmiet estuaries are the same just prior to opening, the ratio of their tidal prisms could be used to estimate what time would be needed for the Palmiet to become tidal. Since the tidal prism of the Swartvlei is about $0,5 \times 10^6 \text{ m}^3$ it follows that 14 hours would be required after the artificial opening of the Palmiet before it would become tidal.

The fact that the estuary becomes tidal can be seen as an indication that the flash flood has completed its flushing process. The redistribution of material flushed out to sea by the flash flood will then have to be continued and completed by wave action and tidal currents.

Computations with the sediment transport formula of Engelund and Hansen (1967) indicate that, depending on the hydraulic roughness assumed for the bed, the time needed to scour the mouth of the Palmiet to its maximum cross-sectional area with a discharge of $50 \text{ m}^3/\text{s}$ would be between 8 and 16 hours.

It therefore seems that a duration of 15 hours, would, although it is the minimum time required to open the mouth to acceptable dimensions, be sufficiently long to enable the mouth to be flushed of its overburden of marine sediments. In this manner the gradual silting up of the estuary would be prevented.

Discussion

It is recommended that, to aid the initial scouring of a channel through the sand spit towards the sea, a guide channel be prepared in the location where the mouth breaks through at present, that is, directly to the east of the rocky promontory, in anticipation of the flood. This channel need not be very large; in fact, it could be dug by one worker with a spade.

It is important that the start of the flash flood be chosen in such a way that the arrival of the first water at the prepared guide channel at the mouth takes place on the falling tide in the sea when the water level is about MSL. In this way sufficient scour can take place to ensure that the wave action during the following high tide will not close the mouth again. Two separate floods, each of $50 \text{ m}^3/\text{s}$ and lasting $7\frac{1}{2}$ hours each, would not be as effective as one flood, even when administered on two consecutive falling tides, since the mouth will not have had a chance to scour to its equilibrium cross-section in $7\frac{1}{2}$ hours, thereby making it prone to closure during the following high tide.

Experience in operation would have to prove whether the procedure outlined above would lead to an acceptable configuration at the mouth. It is possible that practical experience would eventually show that it is possible to reduce either the flow rate or the duration of the flash flood although, as stated earlier, the duration is already most probably at or close to its minimum. On the other hand, it is also conceivable that the experience gained during actual operation of the dam would show that the total runoff during the flash flood will be too low to ensure a well-scoured estuary mouth which will have a chance to remain open for some period of time. Figure 2 and Appendix Figures A1 to A12 indicate that a flash flood at a discharge rate of $50 \text{ m}^3/\text{s}$ is very representative of the natural flood pattern. If it would therefore be found that the total runoff during the flash event is too low, this could be rectified by increasing the duration of the flash flood.

Discussions with DWAFEC have indicated that spilling (overflow through spillway channel) could be an annual event during a wet cycle, most probably during July. This spillway discharge could be of the order of $50 \text{ m}^3/\text{s}$. Nevertheless, it is recommended that the flash flood as discussed above should be administered as usual during such a wet cycle, to compensate for the fact that the recommended discharge rate for the flash flood will be less than the maximum daily discharge rate during an average year.

After completion of the dam in 1990 it will take approximately five years before the full yield of the dam will be used. During this period there will be surplus water available and it should be possible to arrange for various tests to be done to establish an efficient operating system (Dr. C.P.R. Roberts, DWAFEC, personal communication). These tests could be part of a regular monitoring programme, which should preferably be instigated before commissioning of the dam and which should run for some years thereafter, in order to ascertain how successful the procedure is. If changes to the recommended operating system should prove necessary, the minimum ecological requirements for the estuarine system after damming of the Palmiet would finally have to be reconsidered and secondly, if that should still prove necessary, the financial implications of changing the recommended policy for flash floods should be assessed.

7. RECOMMENDATIONS

In summary, the recommended discharge policy for the Hangklip dam is therefore:

(1) A continuous discharge at a rate equal to the maximum recorded monthly evaporation rate (see Table VI), to compensate for evaporation in the estuarine area, which would amount to about $0,25 \times 10^6 \text{ m}^3/\text{yr}$, or 0,14 per cent of the mean annual runoff; and

(2) A flash flood with a discharge rate of $50 \text{ m}^3/\text{s}$ during September which will last for about 15 hours, giving a total runoff of $2,7 \times 10^6 \text{ m}^3$, or about 1,5 per cent of the mean annual runoff. If this runoff were later to prove insufficient to maintain the estuary mouth properly, the total runoff can be increased by increasing the *duration* of the flash flood.

Although the above policy will not maintain the Palmiet River mouth in the same state as under pre-dam conditions, it will ensure that the mouth will be opened to pre-dam dimensions (about 90 m^2 below MSL) once per year and most probably revert to a cross-sectional area of about 15 to 30 m^2 below MSL within a few months after the flash flood. After closure, whenever that should occur, the continuous discharge will prevent hypersalinity due to evaporation. Therefore the conditions will go a long way towards meeting the pre-dam conditions.

The recommended policy represents the best estimate of the required flow that can be made at present. It is, however, conceivable that the policy regarding both the steady discharge to compensate for evaporation in the estuary and the flash flood to scour the mouth would have to be amended slightly to ensure optimum efficiency. This can be done only once the dam is in operation and should be based on the results of a regular monitoring programme.

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TABLE I: WAVE STATISTICS

(a) Occurrence

Sector Period(s)	Frequency of occurrence								
	90°	120°	150°	180°	210°	240°	270°	Calm	Total
7	0,041	0,026	0,026	0,039	0,083	0,101	0,048		0,364
9	0,024	0,020	0,035	0,040	0,096	0,094	0,033		0,341
11	0,008	0,006	0,008	0,019	0,054	0,049	0,014		0,157
13	0,002	0,004	0,003	0,006	0,035	0,025	0,005		0,080
Calm								0,058	0,058
TOTAL	0,076	0,055	0,072	0,104	0,268	0,269	0,099	0,058	1,000

(b) Wave height

Sector Period(s)	Significant wave height (m)							
	90°	120°	150°	180°	210°	240°	270°	Mean rms height
7	2,30	1,84	2,12	2,38	2,50	2,71	2,90	2,52
9	2,67	2,53	2,36	2,65	2,76	3,10	3,29	2,85
11	2,95	2,09	2,48	2,67	3,10	3,13	3,68	3,06
13	3,34	3,08	2,54	2,64	3,55	3,59	3,38	3,43
Mean rms height	2,51	2,26	2,30	2,56	2,87	3,02	3,19	

TABLE II: BREAKER-LINE WAVE DATA

Deep water approach angle	Wave period T (s)	Breaker-line wave crest orientation	Breaker angle θ_b^*	Refraction coefficient K_{rb}	Breaking wave height (m)	Water depth at breaking (m)
150°	7	310°	-1°	0,78	1,95	3,19
150°	9	311°	-2°	0,66	2,08	3,19
150°	11	310°	-1°	0,74	2,62	4,15
150°	13	305°	+4°	0,45	1,93	3,03
180°	7	303°	+6°	0,78	1,98	2,93
180°	9	309°	0	0,81	2,43	3,88
180°	11	317°	-8°	0,77	2,69	4,46
180°	13	317°	-8°	1,39	5,00	8,14
210°	7	305°	+4°	0,67	1,69	2,82
210°	9	308°	+1°	0,60	1,87	3,01
210°	11	302°	+7°	0,35	1,44	1,73
210°	13	318°	-9°	0,48	1,96	3,02
240°	7	352°	-43°	0,24	0,85	0,76
240°	9	324°	-15°	0,36	1,25	2,02
240°	11	332°	-23°	0,35	1,70	2,18
240°	13	314°	-5°	0,34	1,53	2,23
270°	7	13°	-64°	0,12	0,44	0,56
270°	9	350°	-41°	0,20	0,76	1,16
270°	11	351°	-42°	0,09	0,49	0,69
270°	13	345°	-36°	0,18	1,00	1,19

* Angle between wave crest and local bed contour at wave breaking

TABLE III: GRAIN SIZE DISTRIBUTIONS

Grain size	Beach		Estuary mouth	
	Sample 1	Sample 2	Sample 1	Sample 2
D ₁₀	247	241	290	305
D ₁₆	285	279	320	333
D ₂₅	309	313	352	359
D ₃₅	341	344	379	394
D ₅₀	378	396	402	416
D ₆₅	438	461	433	451
D ₇₅	485	499	462	484
D ₈₄	553	602	498	507
D ₉₀	621	642	527	533

Comments:

- (1) Diameters in microns = $\times 10^{-6}$ m.
- (2) D_i is that grain size for which i per cent by mass of the sample is finer.

TABLE IV: PREDICTED LONGSHORE TRANSPORT POTENTIAL

Month	Longshore sediment transport potential x 10 ⁶ m ³			
	Westbound	Eastbound	Net	Gross
January	0,411	1,734	1,322	2,145
February	0,307	0,928	0,621	1,235
March	0,252	1,109	0,857	1,361
April	0,325	0,919	0,593	1,244
May	0,290	1,172	0,882	1,462
June	0,254	1,477	1,223	1,731
July	0,299	1,675	1,376	1,974
August	0,530	1,675	1,145	2,205
September	0,275	1,562	1,287	1,837
October	0,400	0,833	0,433	1,233
November	0,418	1,246	0,828	1,664
December	0,347	1,893	1,546	2,240
Monthly mean for the year	0,342	1,352	1,010	1,694

TABLE V: ESTUARY MOUTH STABILITY

Month	Ω/S_{xg}	
	Peak freshwater runoff	Spring tidal flow
January	0,8	0,3
February	2,0	0,6
March	5,4	0,5
April	8,6	0,7
May	21,7	0,4
June	51,8	0,3
July	51,5	0,3
August	25,2	0,3
September	25,4	0,3
October	16,6	0,9
November	4,7	0,5
December	2,8	0,2

Comment:

Ω = tidal prism or total runoff during peak freshwater runoff event.

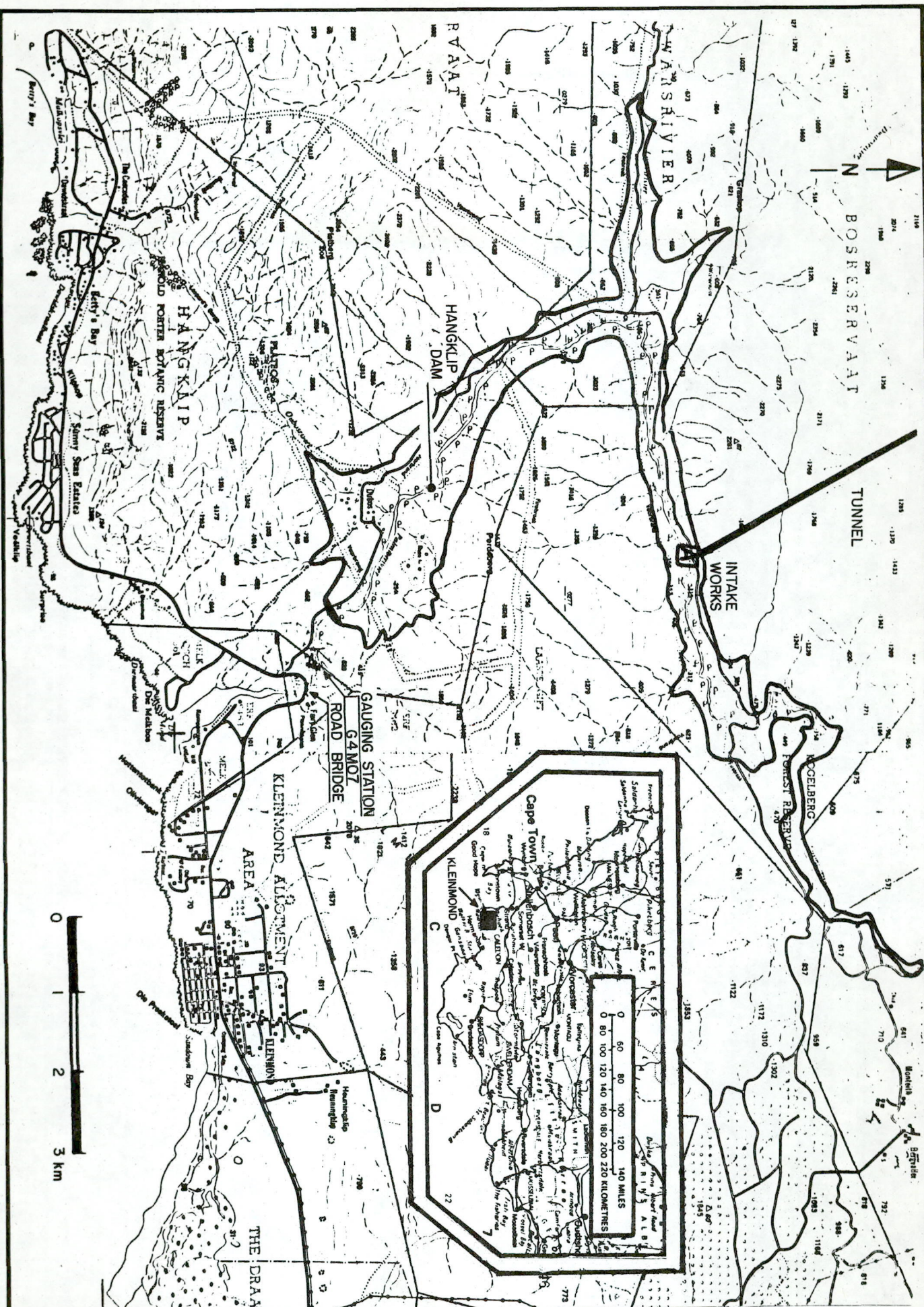
S_{xg} = 50 per cent of westbound sediment transport potential.

If $\Omega/S_{xg} \leq 20$: Estuary mouth is an unstable overflow channel.

If $20 < \Omega/S_{xg} \leq 50$: Estuary mouth gets a 'shot in the arm' from freshwater flow.

TABLE VI: MEAN MONTHLY EVAPORATION

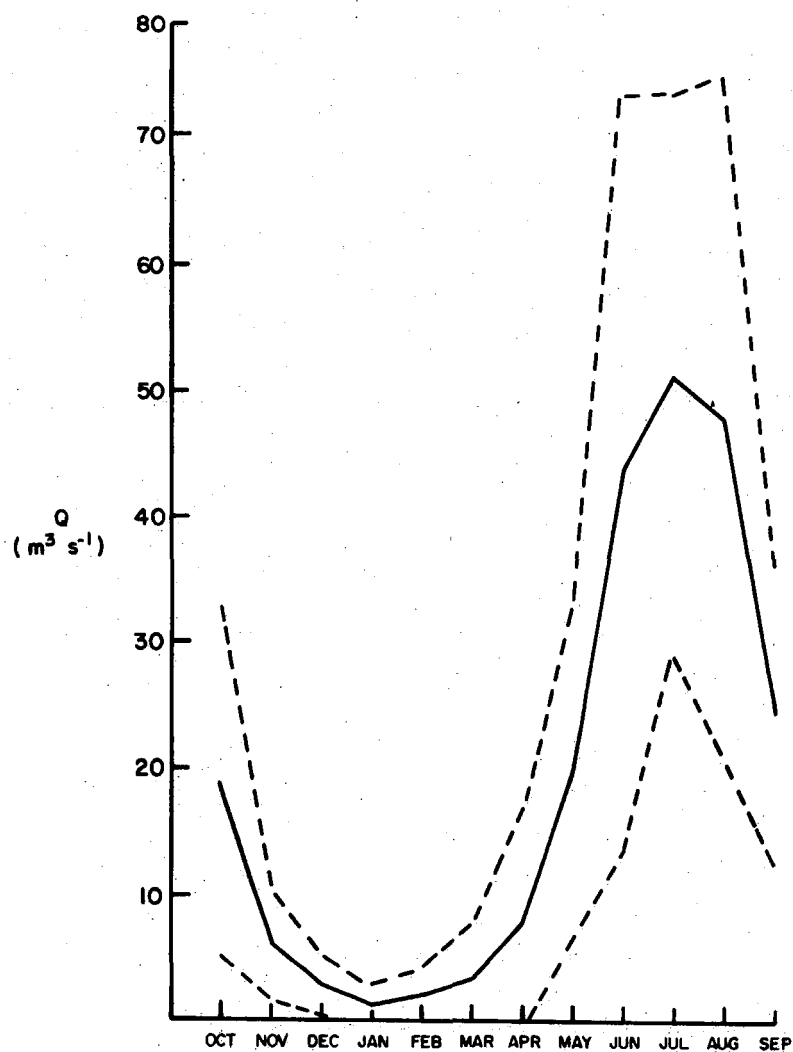
Month	Monthly evaporation (mm)	Equivalent discharge $\times 10^{-3} \text{m}^3 \text{s}^{-1}$	Total runoff $\times 10^6 \text{m}^3 / \text{month}$
October	112	8,07	0,0216
November	162	12,1	0,0313
December	191	13,8	0,0369
January	201	14,5	0,0388
February	166	13,1	0,0320
March	139	10,0	0,0268
April	84	6,25	0,0162
May	46	3,31	0,0089
June	36	2,68	0,0069
July	32	2,31	0,0062
August	40	2,88	0,0077
September	67	4,99	0,0129
Total	1 276	93,99	0,2462
Mean	106,33	7,83	0,0205



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 DATE : DEC '80
 REF : C/SEA 8048

HANGKLIP DAM LOCATION MAP

FIGURE
 I



———— AVERAGE OF MAXIMUM DAILY RUNOFF/MONTH
FOR 1963/64 TO 1975/76

----- ± ONE STANDARD DEVIATION

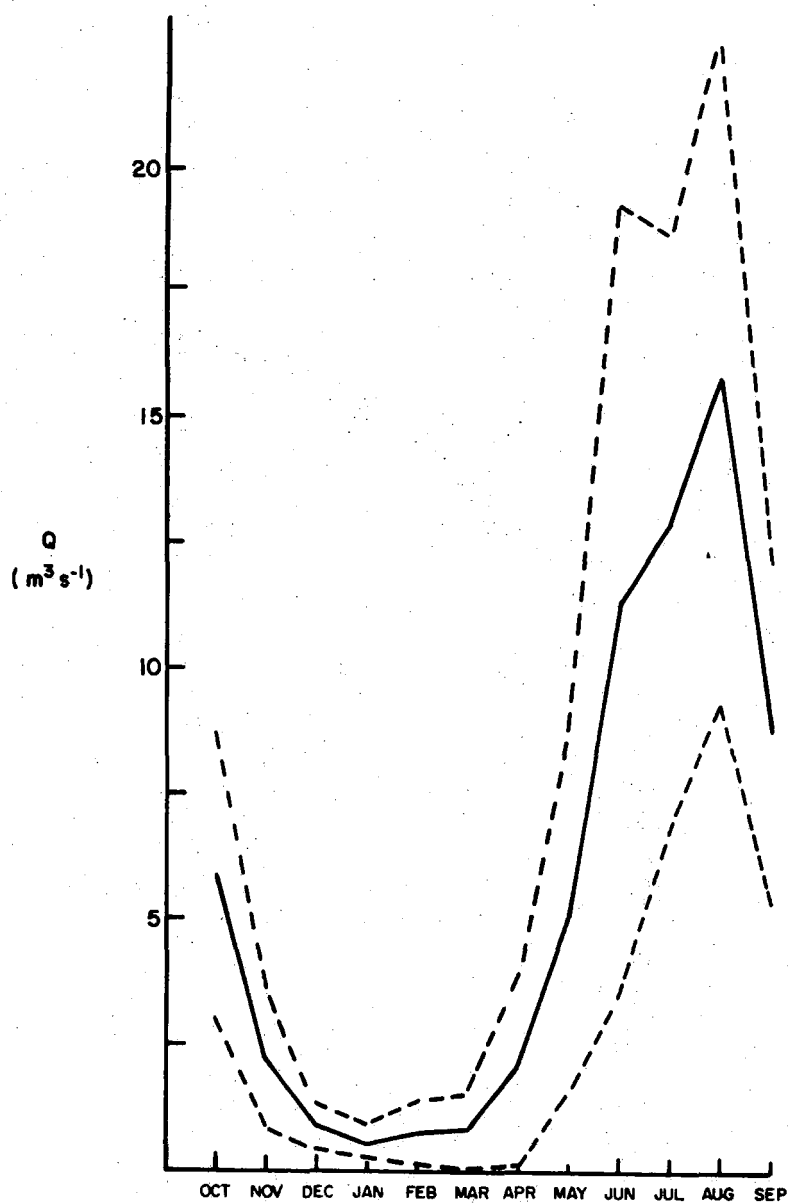
BASED ON DATA FOR GAUGING STATION G4M07

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DATE DEC '80
REF C/SEA 8048

HANGKLIP DAM

MONTHLY MAXIMUM RUNOFF

FIGURE
2



———— AVERAGE OF MEAN MONTHLY RUNOFF
FOR 1963/64 TO 1975/76

- - - - ± ONE STANDARD DEVIATION

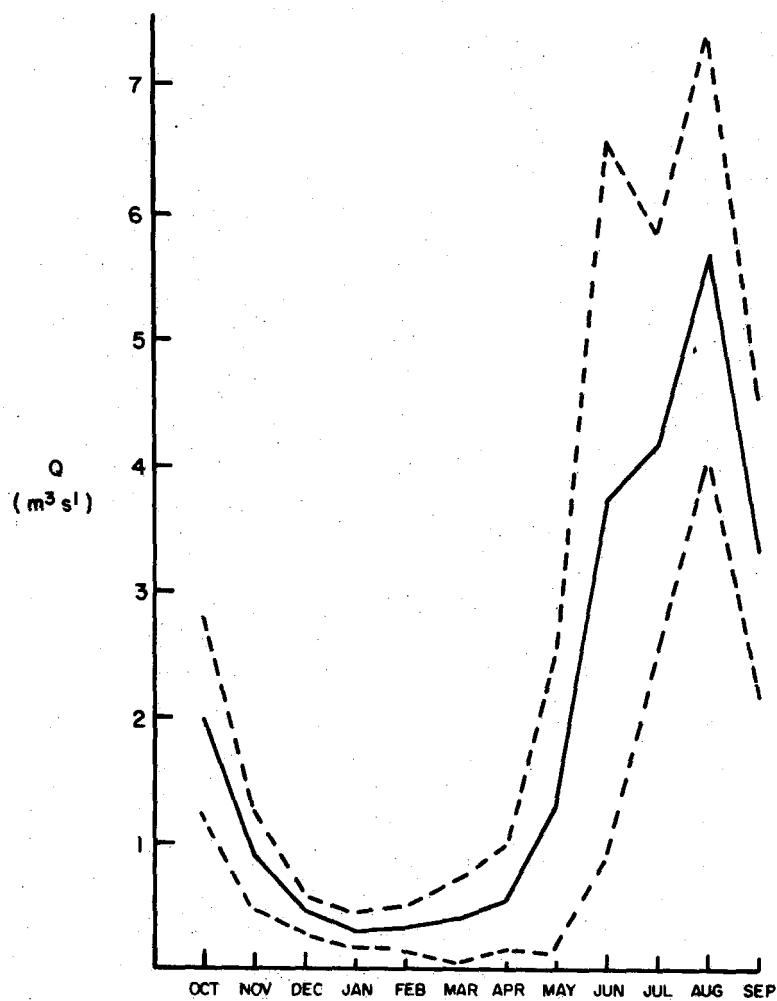
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HANGKLIP DAM

MONTHLY MEAN RUNOFF

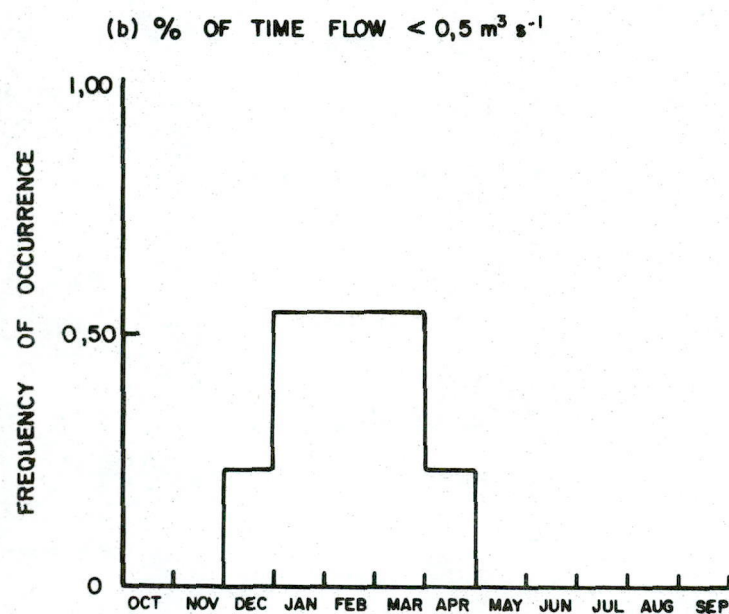
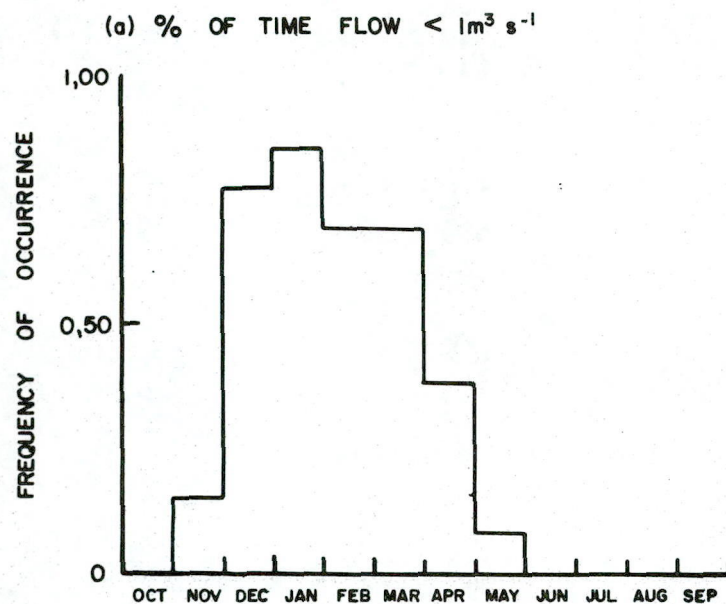
FIGURE
3



— AVERAGE OF MINIMUM DAILY RUNOFF/MONTH
FOR 1963/64 TO 1975/76

- - - ± ONE STANDARD DEVIATION

BASED ON DATA FOR GAUGING STATION G4M07

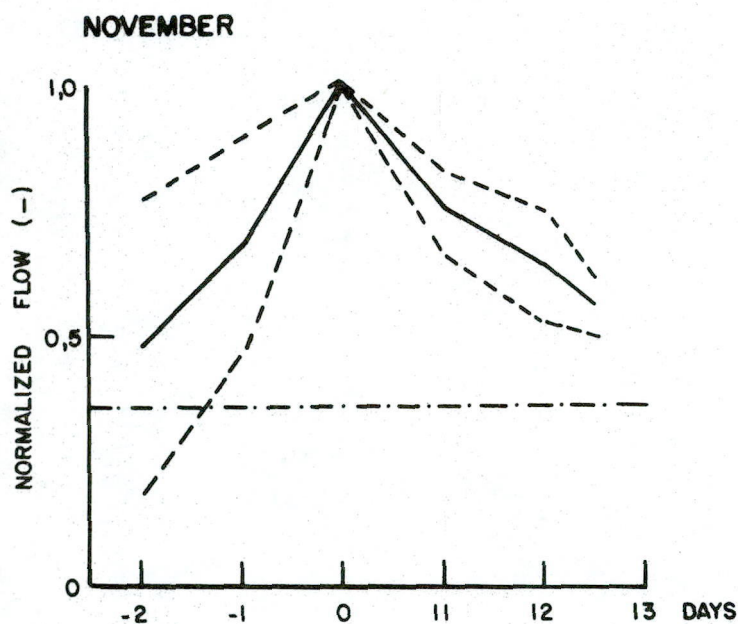
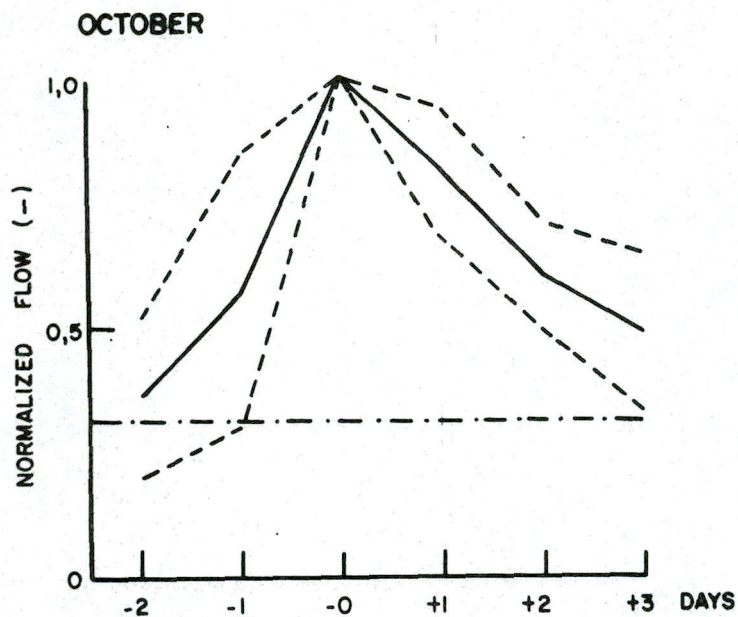


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HANGKLIP DAM

PERSISTENCE OF LOW FLOW

FIGURE
 5



——— MEAN HYDROGRAPH
 - - - ± ONE STANDARD DEVIATION
 - · - · - MEAN NORMALIZED MONTHLY FLOW*

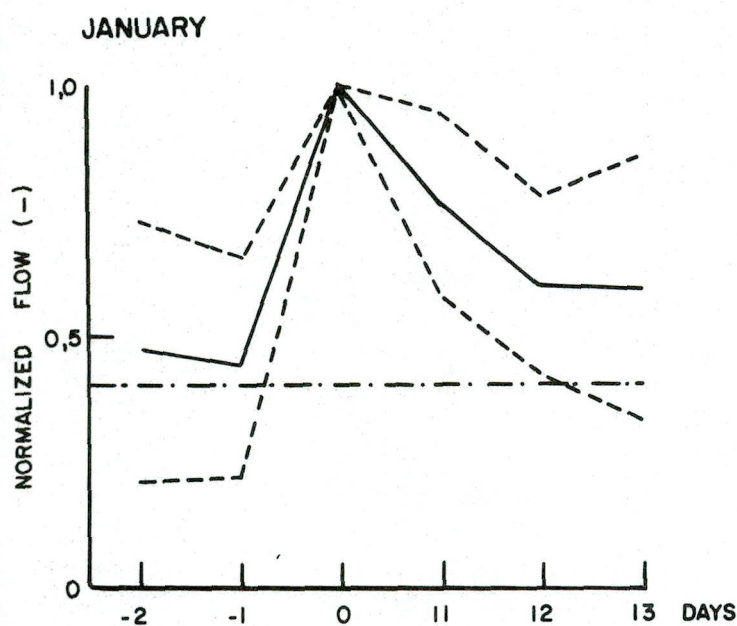
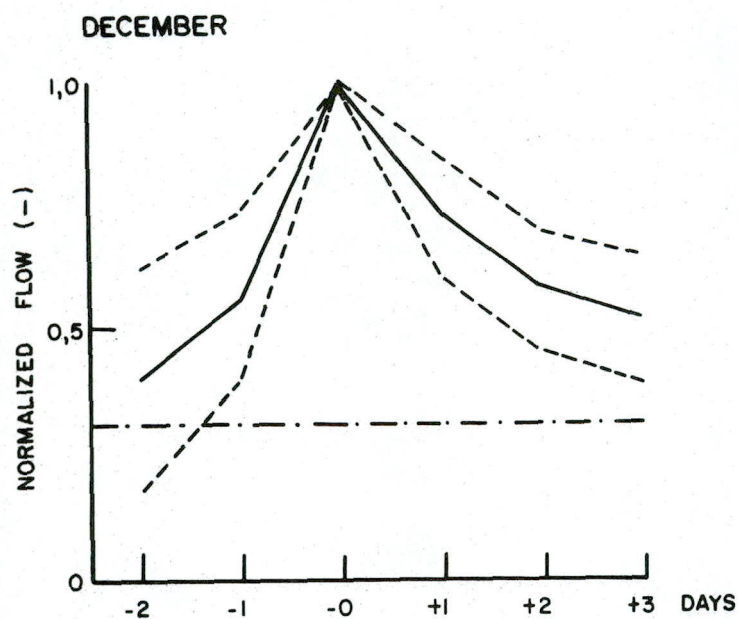
BASED ON DATA FROM G4M07

* THE MEAN DAILY FLOW FOR THE MONTH AS A FRACTION OF THE MAXIMUM DAILY FLOW FOR THE MONTH

TRACED JMO
 CHECKED
 DATE DEC '80
 REF C/SEA 8048

HANGKLIP DAM
NORMALIZED FLOOD HYDROGRAPHS
OCTOBER; NOVEMBER

FIGURE
6



——— MEAN HYDROGRAPH
 - - - - - $\pm$ ONE STANDARD DEVIATION
 - MEAN NORMALIZED MONTHLY FLOW*

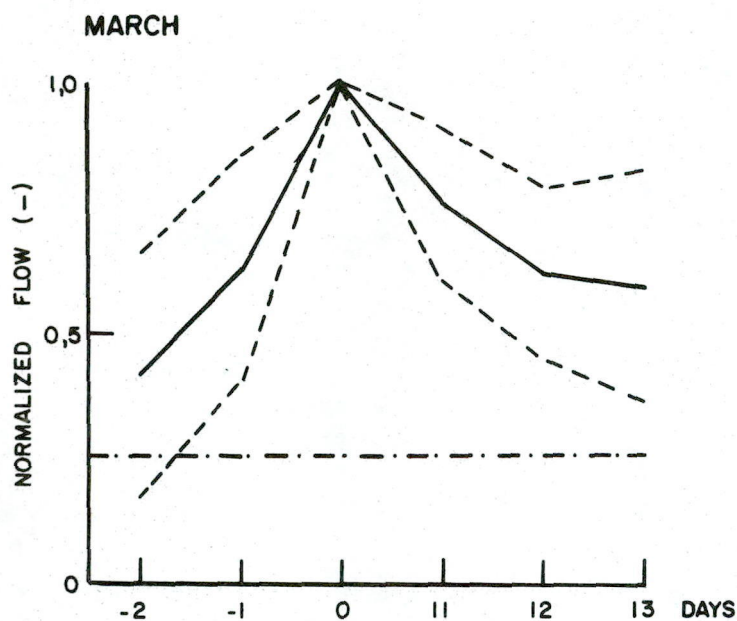
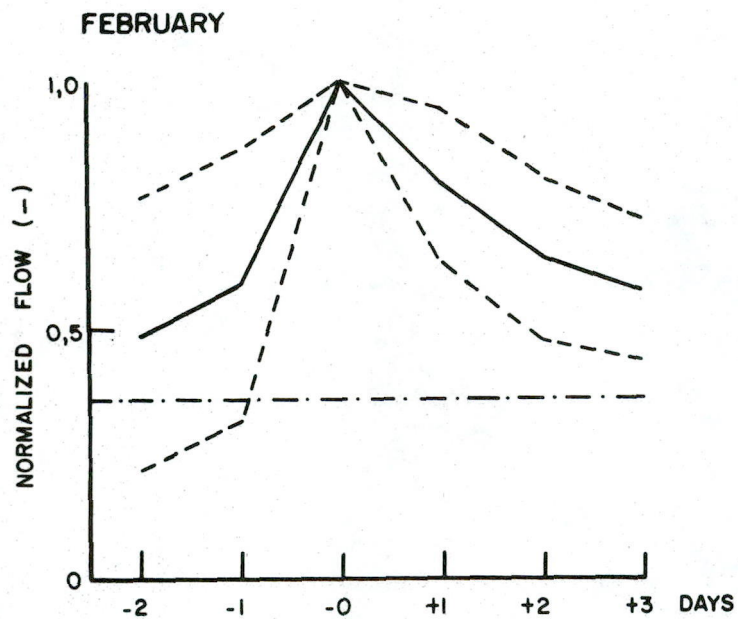
BASED ON DATA FROM G4M07

* THE MEAN DAILY FLOW FOR THE MONTH AS A FRACTION OF THE MAXIMUM DAILY FLOW FOR THE MONTH

TRACED JMO
 CHECKED
 DATE DEC. '80
 REF C/80A 8048

HANGKLIP DAM
 NORMALIZED FLOOD HYDROGRAPHS
 DECEMBER ; JANUARY

FIGURE
 7



——— MEAN HYDROGRAPH
 - - - ± ONE STANDARD DEVIATION
 - · - · - MEAN NORMALIZED MONTHLY FLOW*

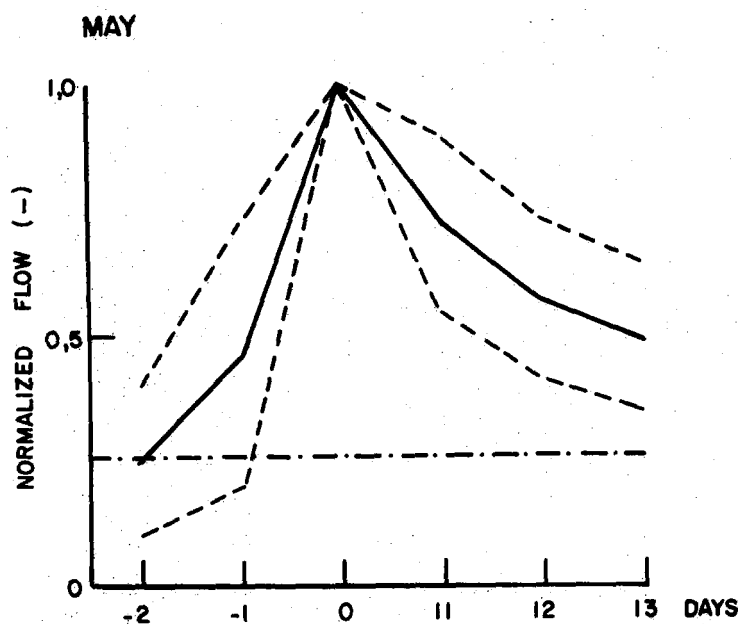
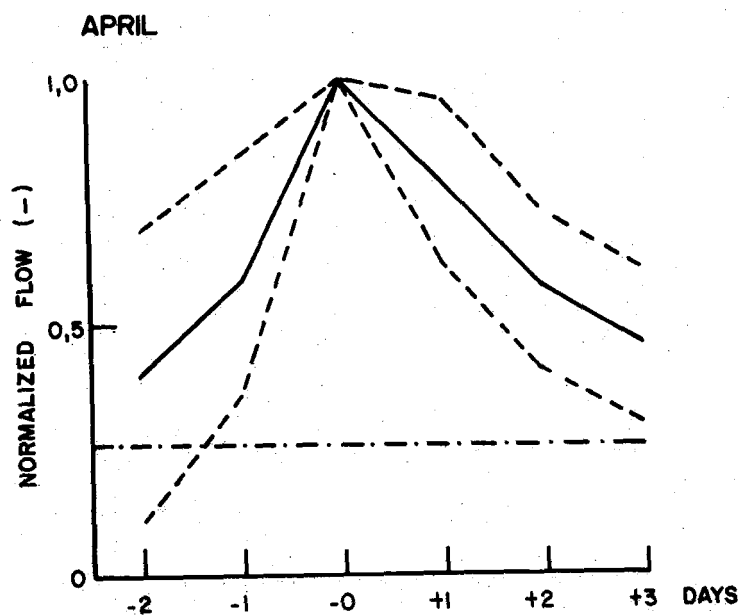
BASED ON DATA FROM G4M07

*THE MEAN DAILY FLOW FOR THE MONTH AS A FRACTION OF THE MAXIMUM DAILY FLOW FOR THE MONTH

TRACED JMO
 CHECKED
 DATE: DEC. 80
 REF: C/SEA 8048

HANGKLIP DAM
NORMALIZED FLOOD HYDROGRAPHS
FEBRUARY ; MARCH

FIGURE
8



— MEAN HYDROGRAPH
 - - - ± ONE STANDARD DEVIATION
 - . - MEAN NORMALIZED MONTHLY FLOW*

BASED ON DATA FROM G4M07

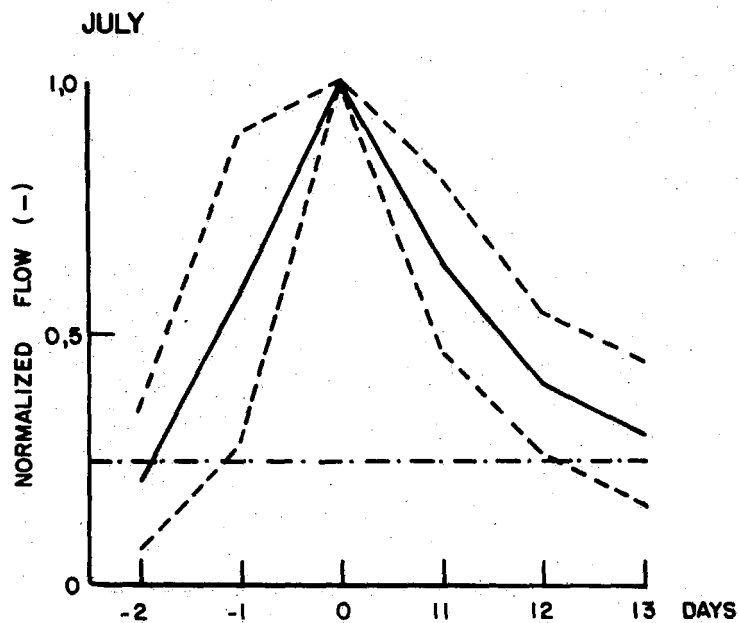
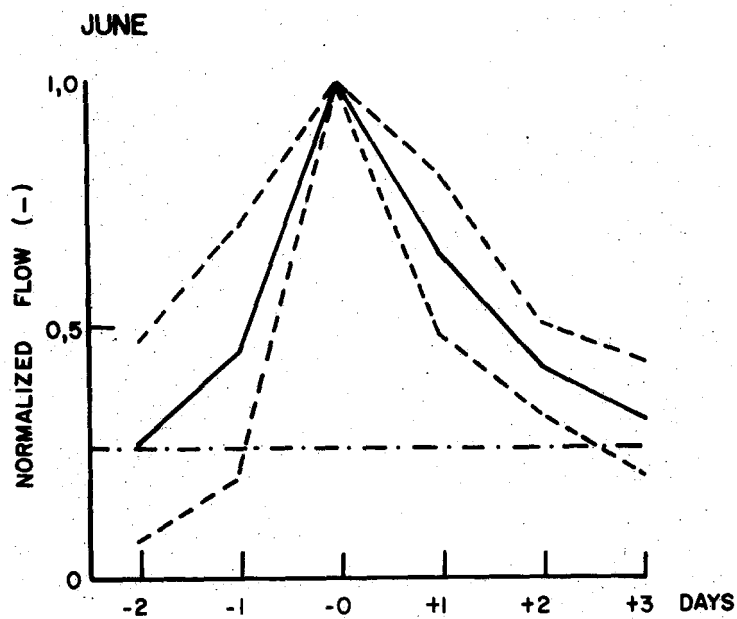
* THE MEAN DAILY FLOW FOR THE MONTH AS A FRACTION OF THE MAXIMUM DAILY FLOW FOR THE MONTH

TRACED JMO
 CHECKED
 DATE DEC. 80
 REF C/80A 8048

HANGKLIP DAM
 NORMALIZED FLOOD HYDROGRAPHS
 APRIL ; MAY

FIGURE
 9

NATIONAL RESEARCH INSTITUTE FOR OCEANOLOGY



——— MEAN HYDROGRAPH
 - - - + ONE STANDARD DEVIATION
 - . - . - MEAN NORMALIZED MONTHLY FLOW*

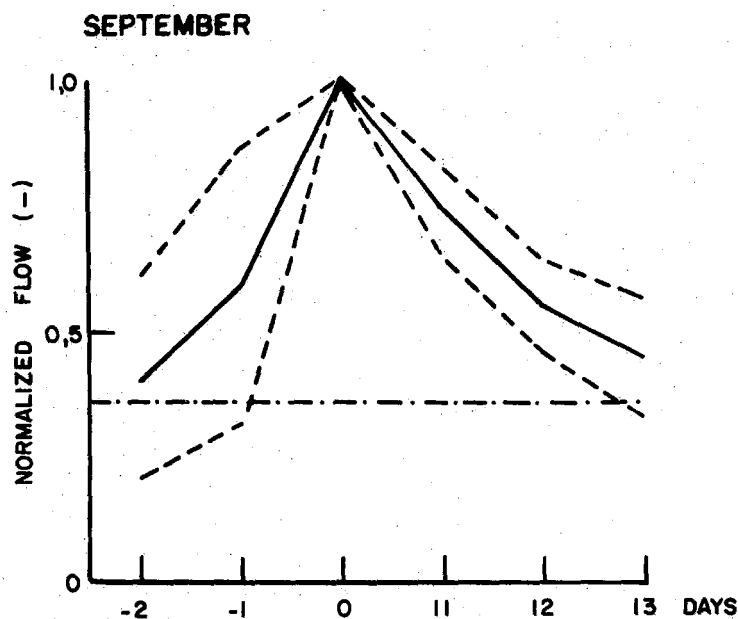
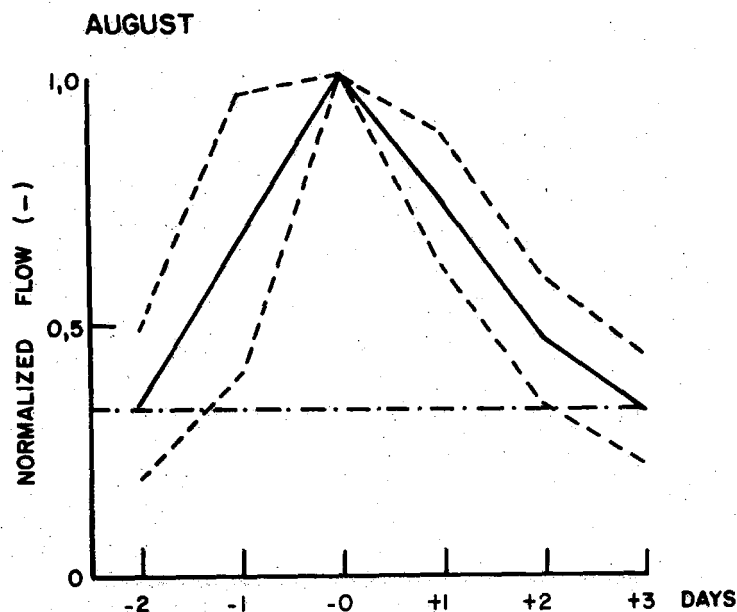
BASED ON DATA FROM G4M07

* THE MEAN DAILY FLOW FOR THE MONTH AS A FRACTION OF THE MAXIMUM DAILY FLOW FOR THE MONTH

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 DATE DEC. '80
 REF C/SEA 2048

HANGKLIP DAM
NORMALIZED FLOOD HYDROGRAPHS
JUNE ; JULY

FIGURE
10



——— MEAN HYDROGRAPH
 - - - - - $\pm$ ONE STANDARD DEVIATION
 - MEAN NORMALIZED MONTHLY FLOW*

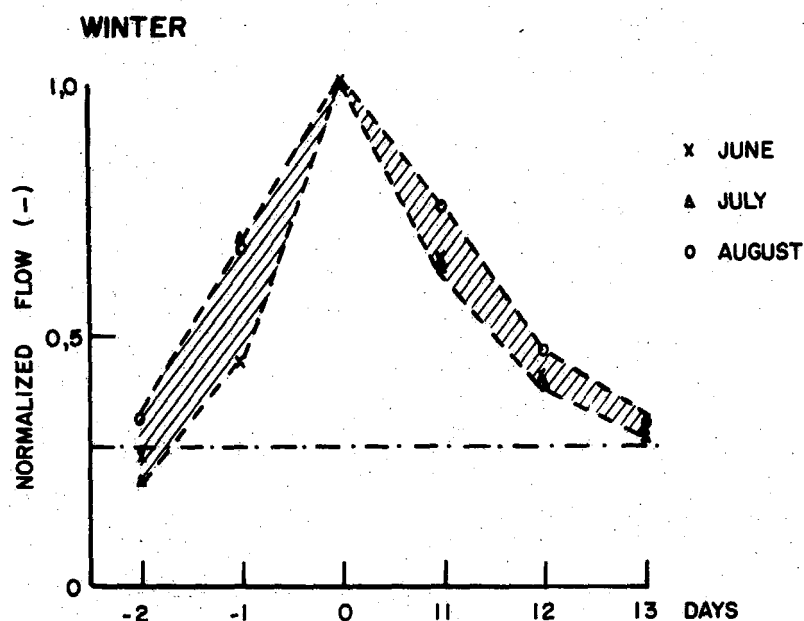
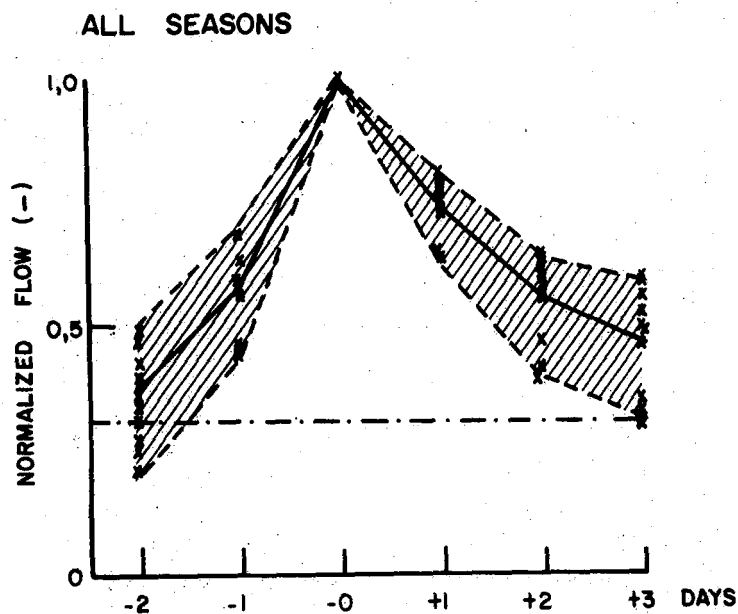
BASED ON DATA FROM G4M07

* THE MEAN DAILY FLOW FOR THE MONTH AS A FRACTION OF THE MAXIMUM DAILY FLOW FOR THE MONTH

TRACED JMO
 CHECKED
 DATE DEC. '80
 REF C/159 8048

HANGKLIP DAM
 NORMALIZED FLOOD HYDROGRAPHS
 AUGUST; SEPTEMBER

FIGURE
 II



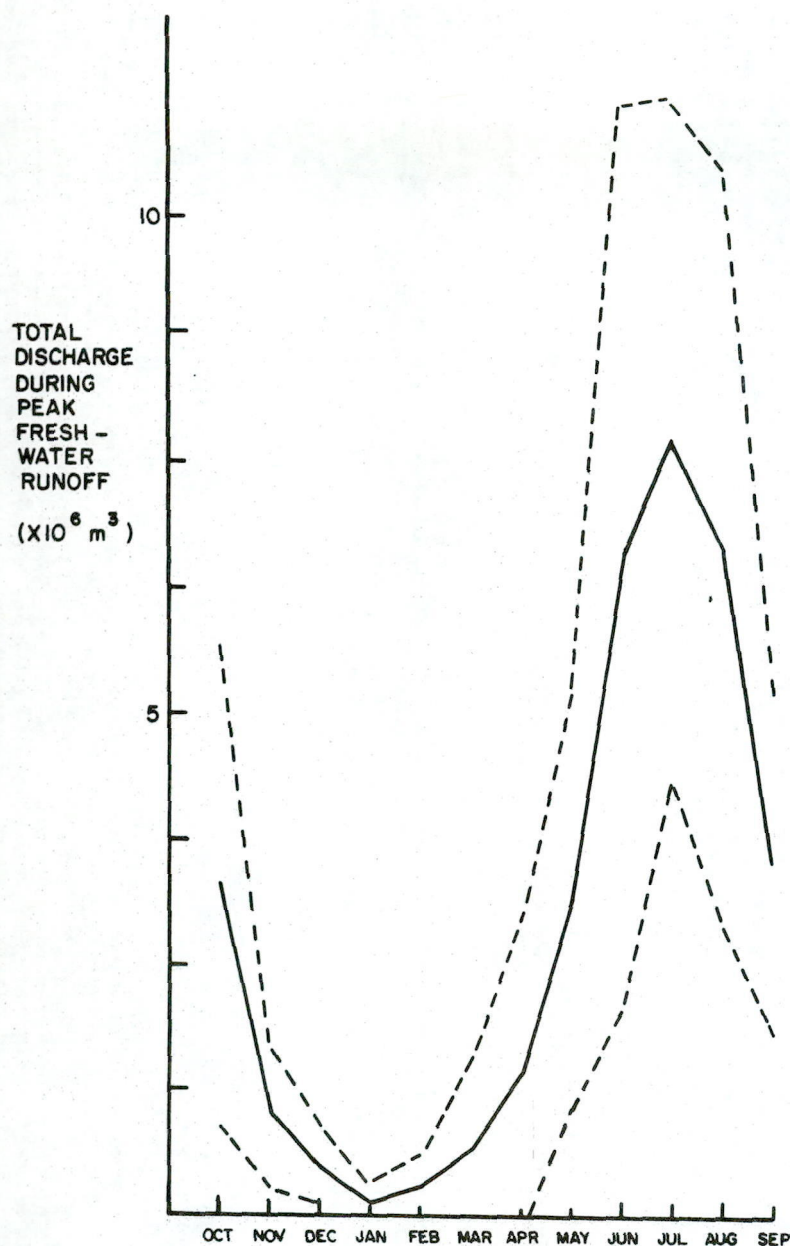
————— MEAN HYDROGRAPH
 ENVELOPE OF OBSERVED MONTHLY MEAN FLOOD HYDROGRAPHS
 - - - - - AVERAGE OF THE MEAN NORMALIZED MONTHLY FLOW FOR
 THE APPROPRIATE MONTHS IN FIGURES 6 TO 11

BASED ON INFORMATION CONTAINED IN FIGURES 6 TO 11

TRACED JMO
 CHECKED
 DATE DEC '80
 REF : C/SEA. 8048

HANGKLIP DAM
NORMALIZED FLOOD HYDROGRAPHS
ALL SEASONS ; WINTER

FIGURE
12



———— INTEGRATED VOLUME OF WATER REPRESENTED BY THE
AREA UNDER THE MEAN HYDROGRAPH IN FIGURES 6 TO 11

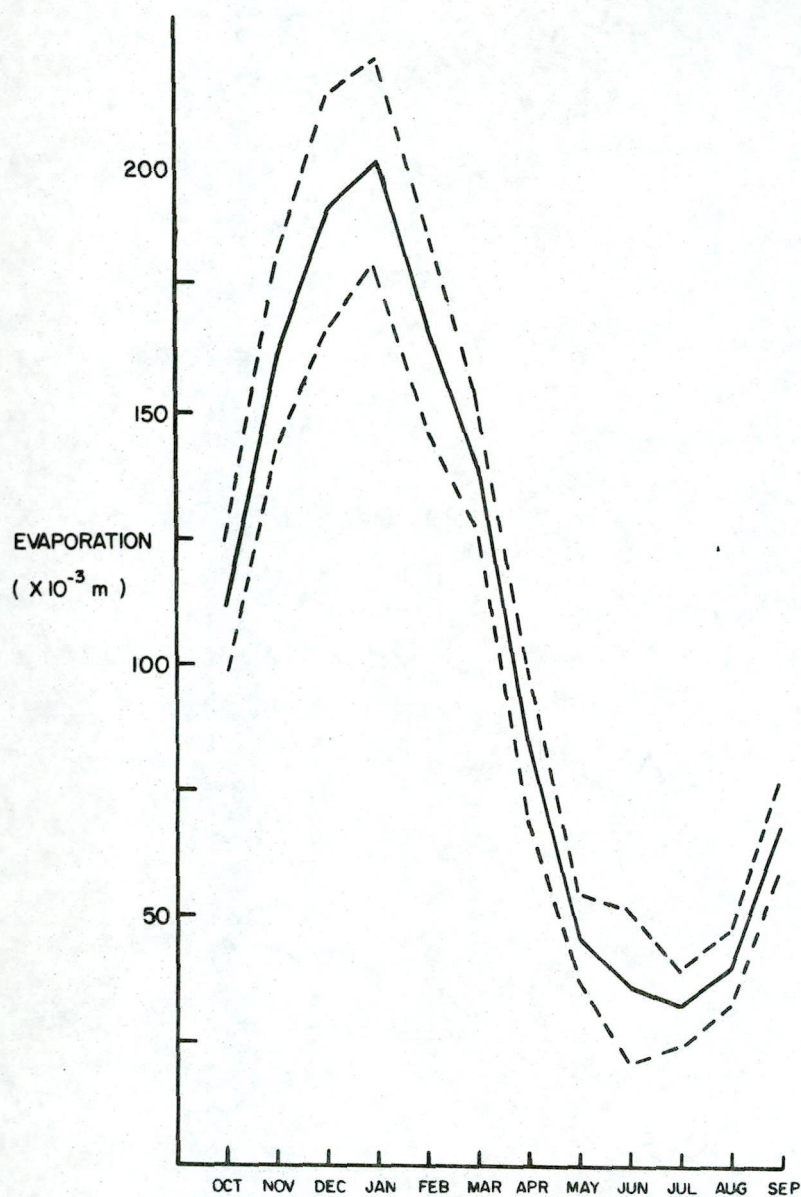
----- $\pm$ ONE STANDARD DEVIATION

BASED ON FIGURES 2 AND 6 TO 11

TRACED JMO
CHECKED
DATE DEC. '80
REF C/SEA 8048

HANGKLIP DAM
TOTAL DISCHARGE DURING PEAK
FRESHWATER RUNOFF

FIGURE
13



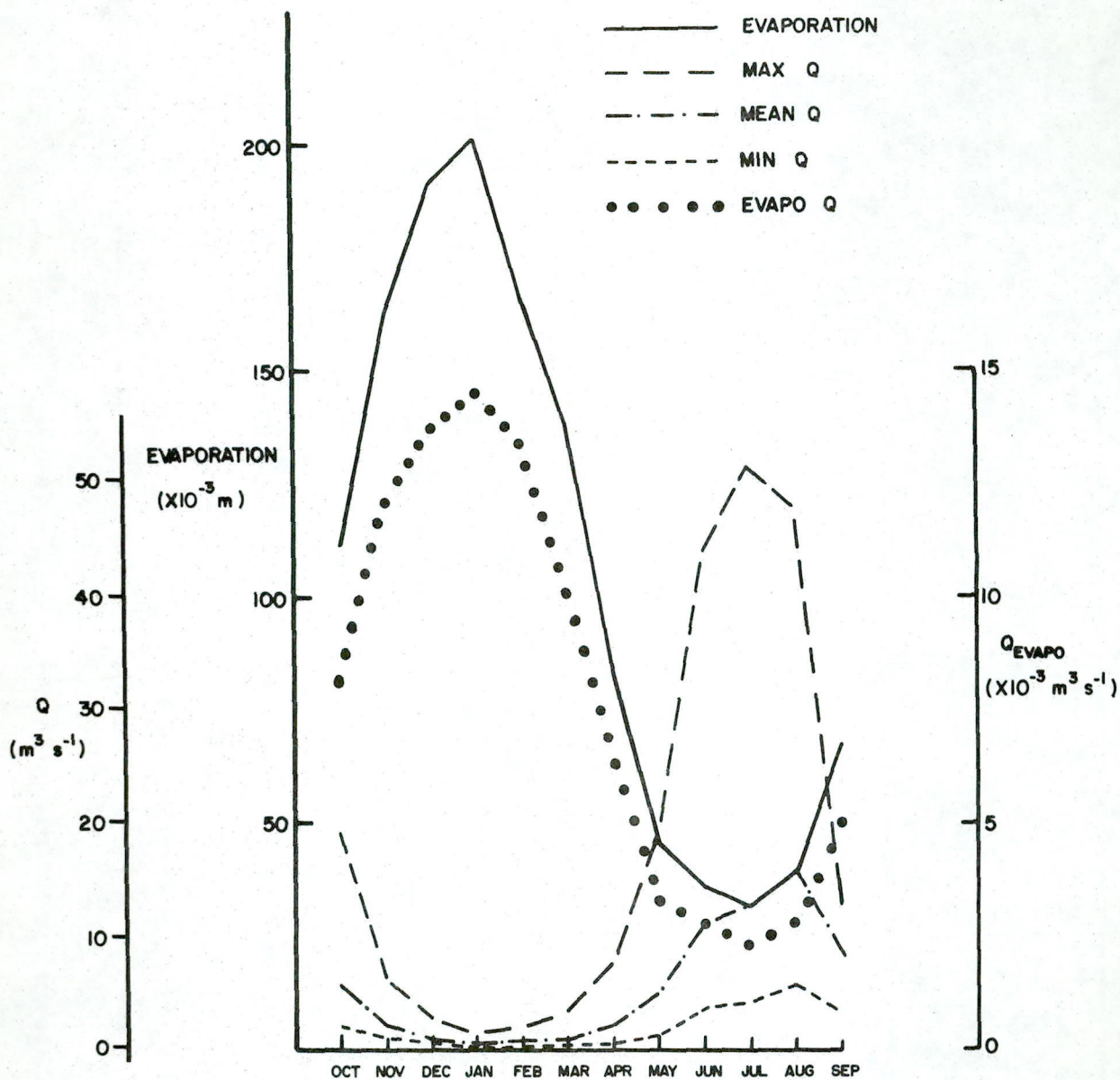
BASED ON MEASUREMENTS AT THE PROPOSED
DAM SITE IN 1963/64 TO 1975/76

TRACED JMO
CHECKED
DATE DEC. '80
REF C/SEA 8048

HANGKLIP DAM

MONTHLY MEAN EVAPORATION

FIGURE
14



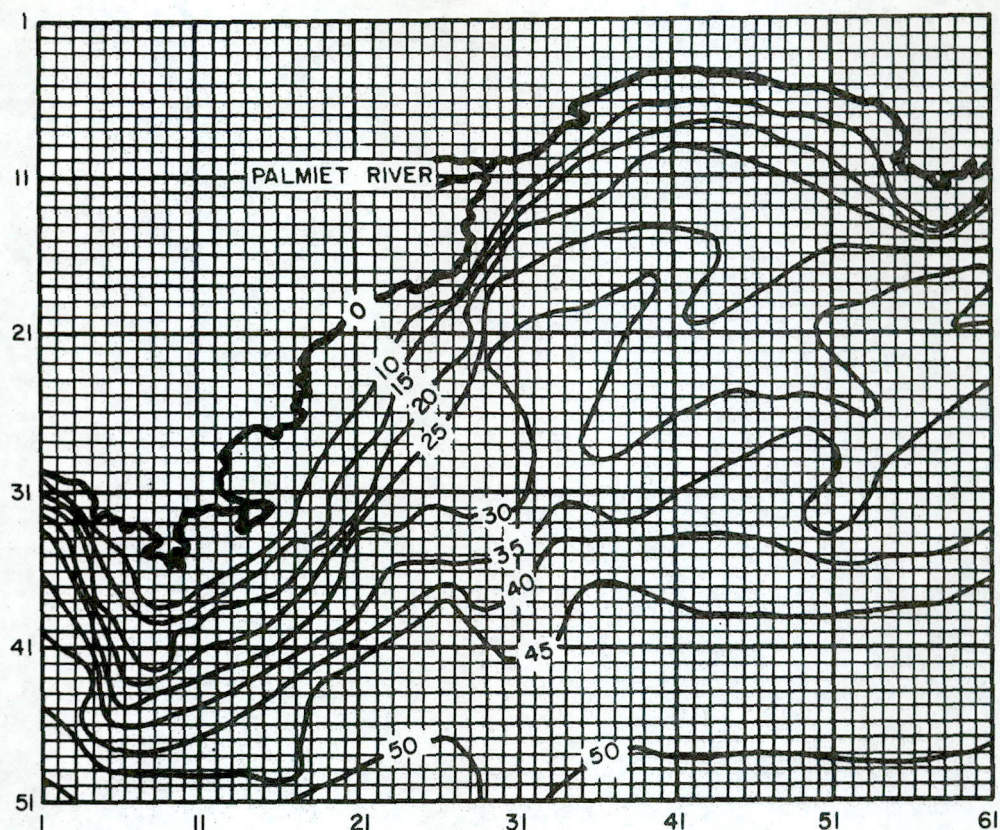
EVAPO Q : EQUIVALENT DISCHARGE COMPENSATING
FOR MEAN EVAPORATION PER MONTH
Q : DAILY DISCHARGE
BASED ON FIGURES 2 TO 4 AND 14

TRACED : JMO
CHECKED :
DATE : 80 DEC.
REF : C/SEA 8048

HANGKLIP DAM

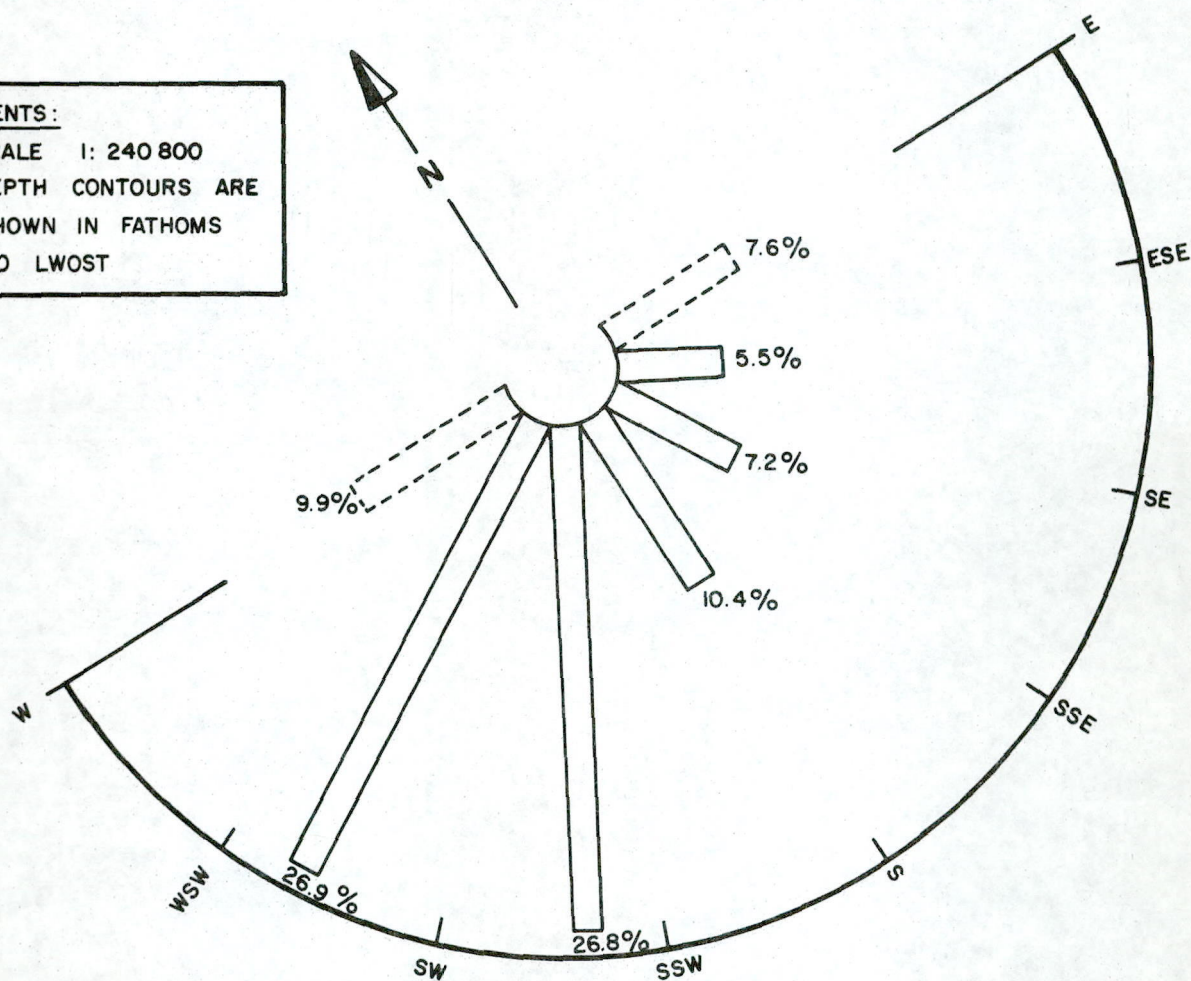
MEAN MONTHLY HYDROLOGICAL PARAMETERS

FIGURE
15



COMMENTS:

- (1) SCALE 1: 240 800
- (2) DEPTH CONTOURS ARE SHOWN IN FATHOMS TO LWOST

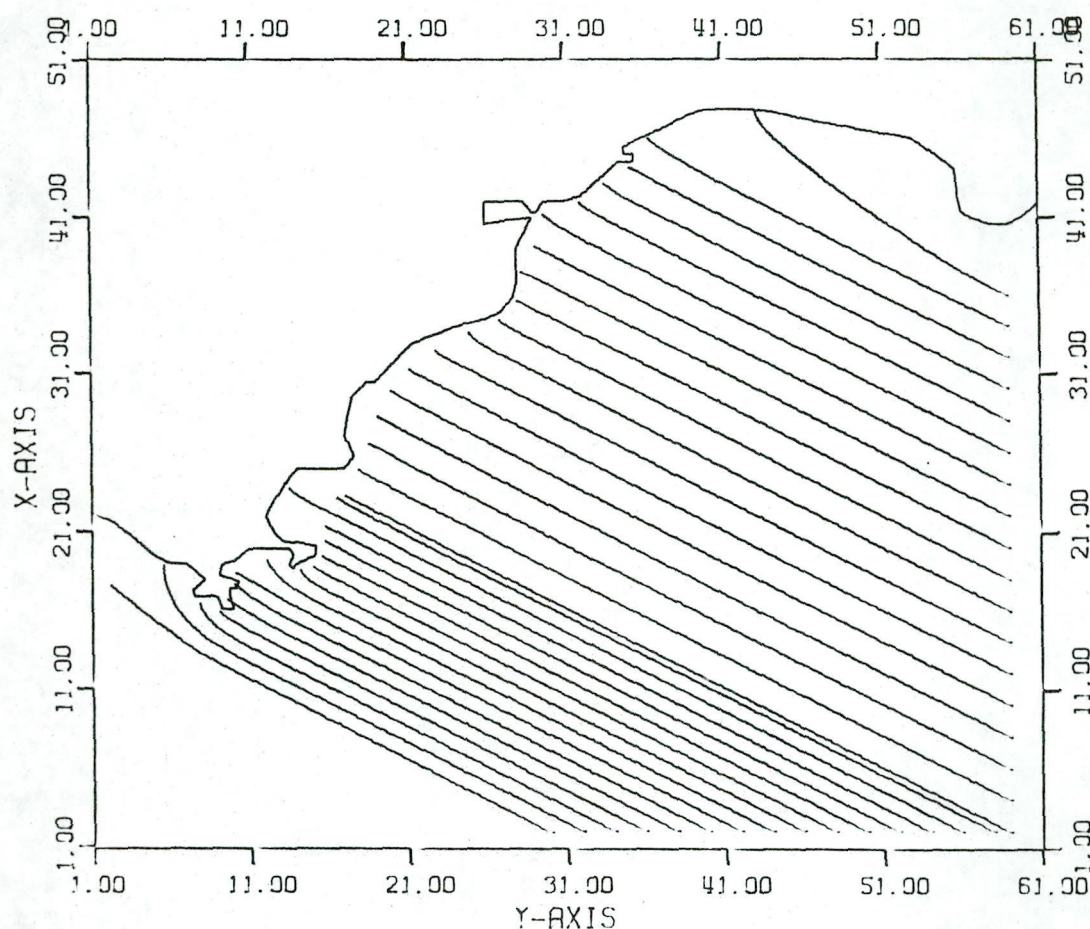


TRACED JMO
CHECKED
DATE: DEC. '80
REF C/SEA 8048

HANGKLIP DAM
REFRACTION GRID

FIGURE
16

REFRACTION (DUTOIT) - PALMIET
 WAVE DIRECTION AND PERIOD 150 - 7S
 SCALE OF PLOT = 1: 196850.00

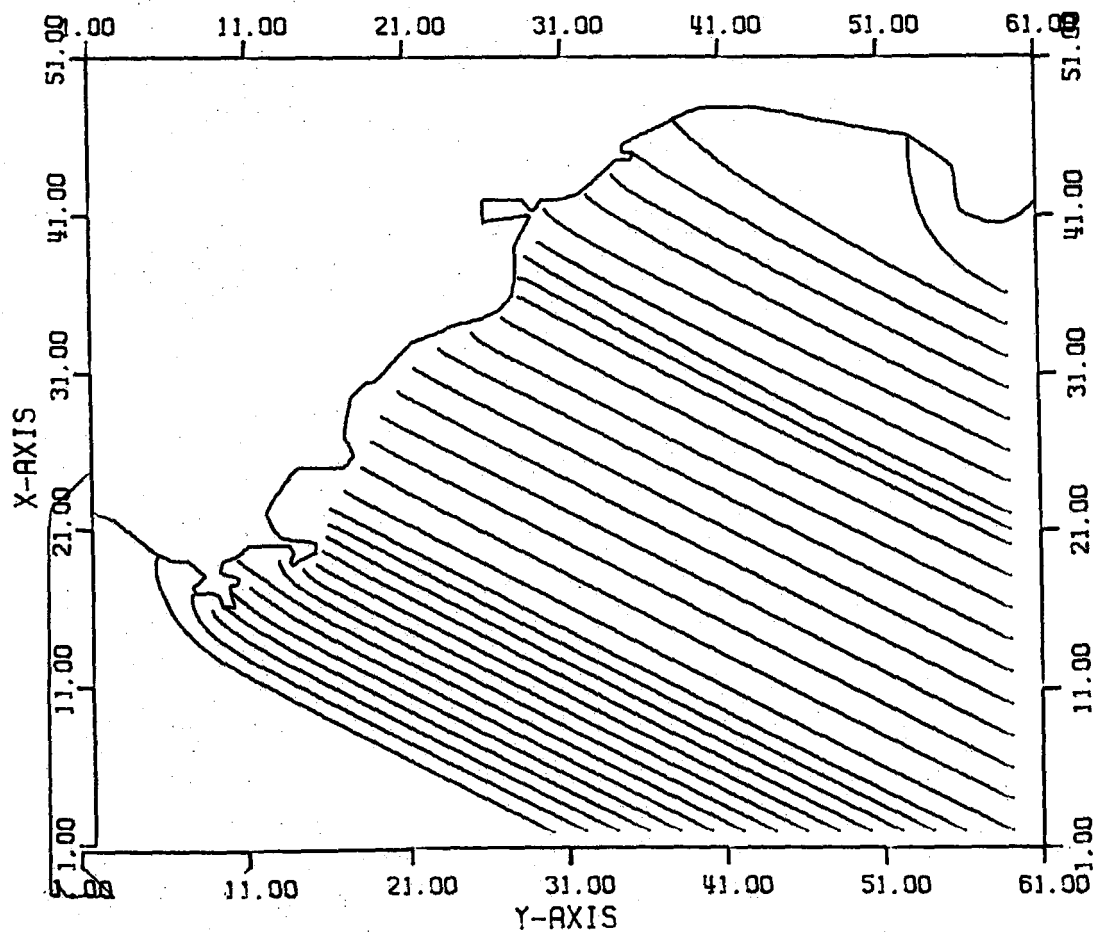


TRACED JMO
 CHECKED
 DATE DEC. '80
 REF C/SEA 8048

HANGKLIP DAM
 REFRACTION DIAGRAM T = 7s , DEEPWATER DIR.=150°

FIGURE
 17

REFRACTION (DUTOIT) - PALMIET
 WAVE DIRECTION AND PERIOD 150 - 9S
 SCALE OF PLOT = 1: 196850.00



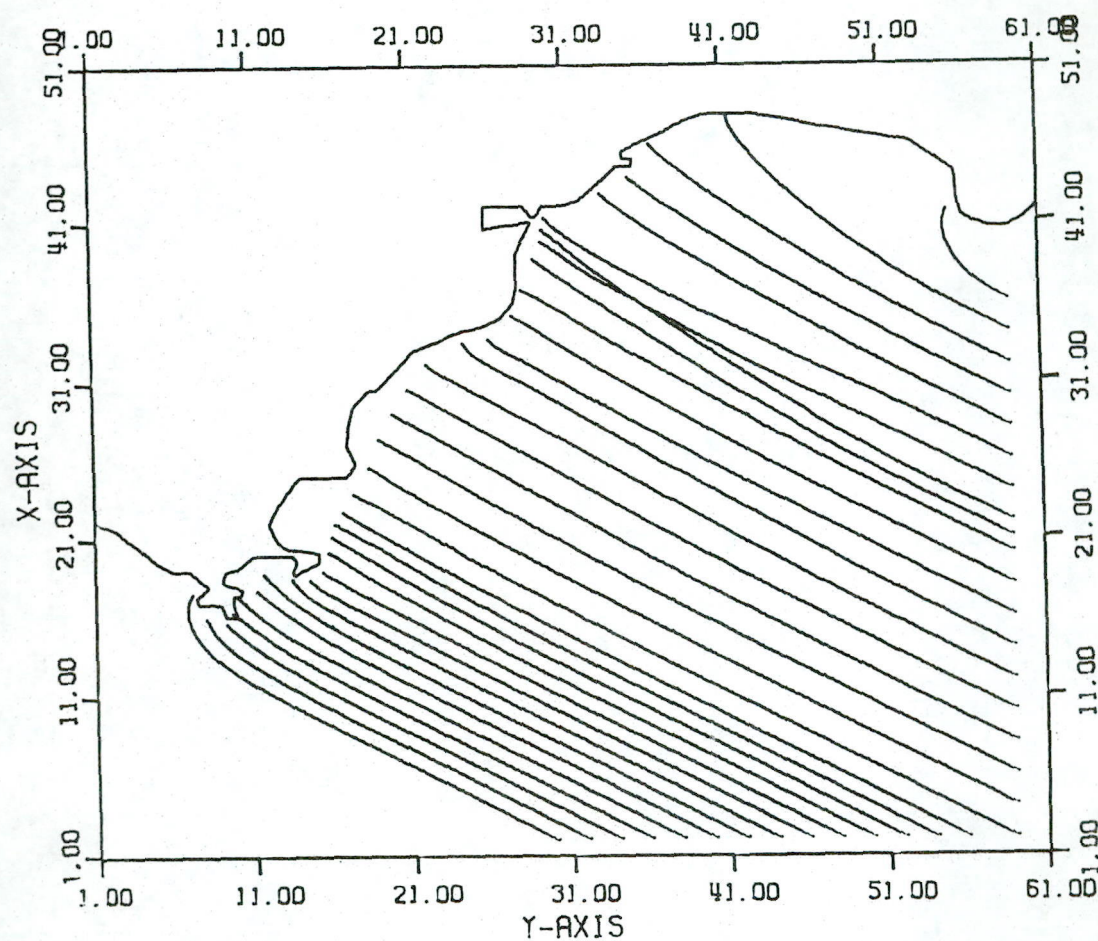
TRACED JMO
 CHECKED
 DATE DEC. '80
 REF C/SEA 8048

HANGKLIP DAM

REFRACTION DIAGRAM T = 9s , DEEPWATER DIR.=150°

FIGURE
 18

REFRACTION (DUTOIT) - PALMIET
 WAVE DIRECTION AND PERIOD 150 - 11S
 SCALE OF PLOT = 1: 196850.00



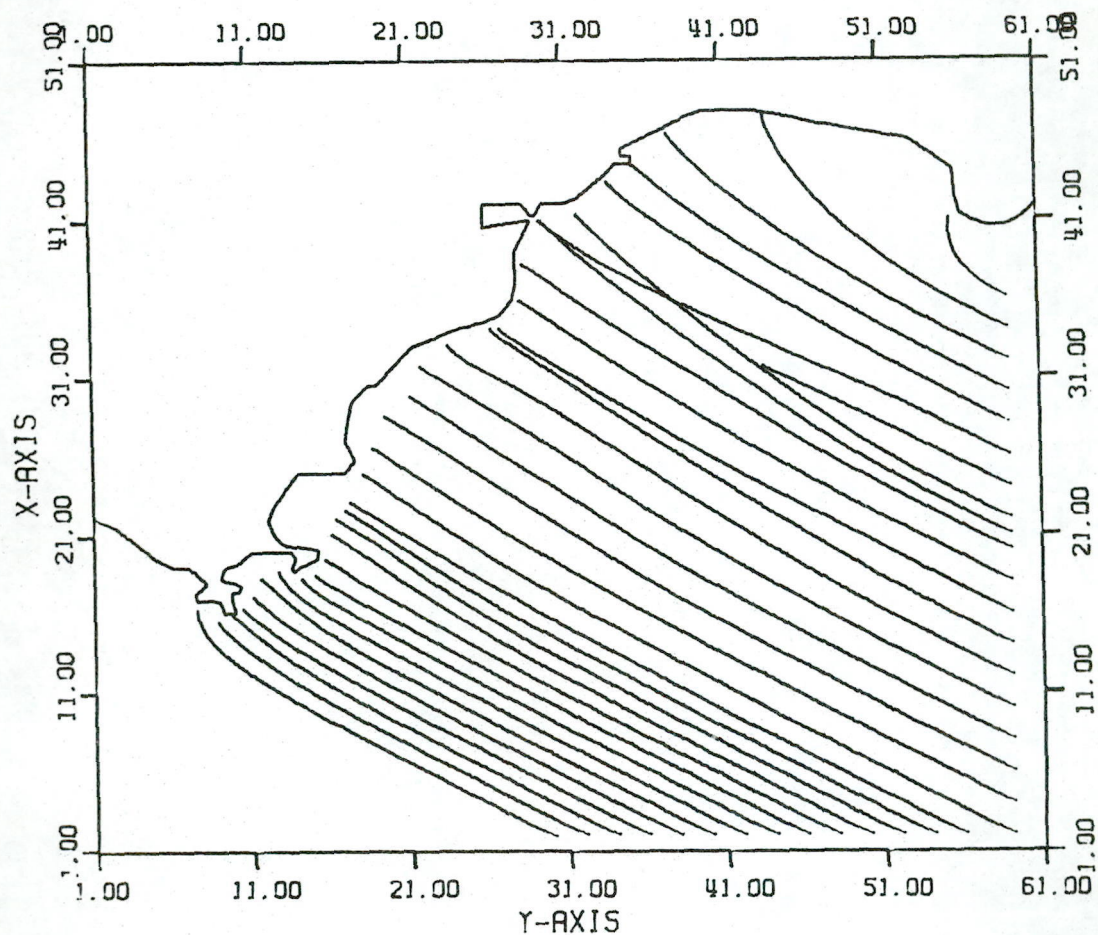
TRACED JMD
 CHECKED
 DATE DEC. 80
 REF C/SEA 8048

HANGKLIP DAM

REFRACTION DIAGRAM T = 11s , DEEPWATER DIR. = 150°

FIGURE
 19

REFRACTION (DUTOIT) - PALMIET
 WAVE DIRECTION AND PERIOD 150 - 13S
 SCALE OF PLOT = 1: 196850.00



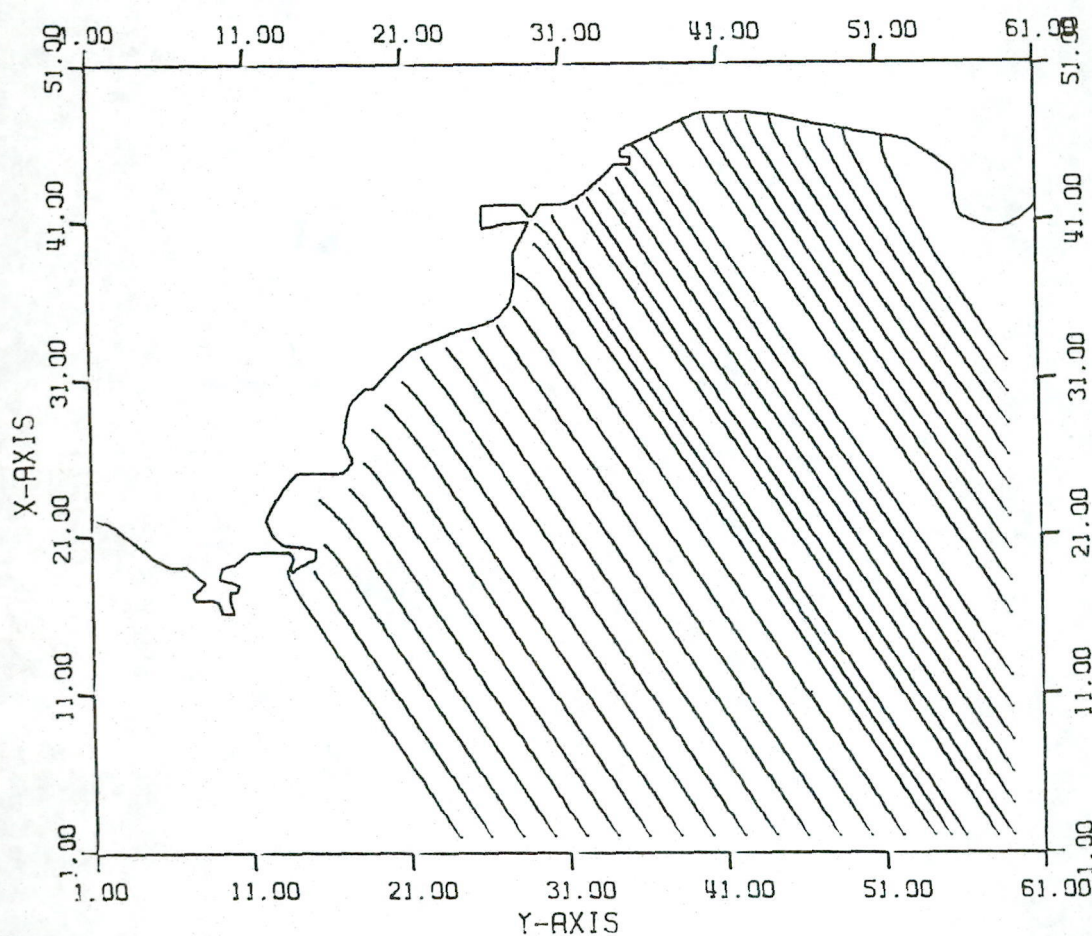
TRACED JMO
 CHECKED
 DATE DEC. '80
 REF C/SEA 8048

HANGKLIP DAM

REFRACTION DIAGRAM T = 13s , DEEPWATER DIR. = 150°

FIGURE
 20

REFRACTION (DUTOIT) - PALMIET
 WAVE DIRECTION AND PERIOD 180 -7S
 SCALE OF PLOT = 1: 196850.00



TRACED JMO

CHECKED

DATE DEC. '80

REF C/SEA 8048

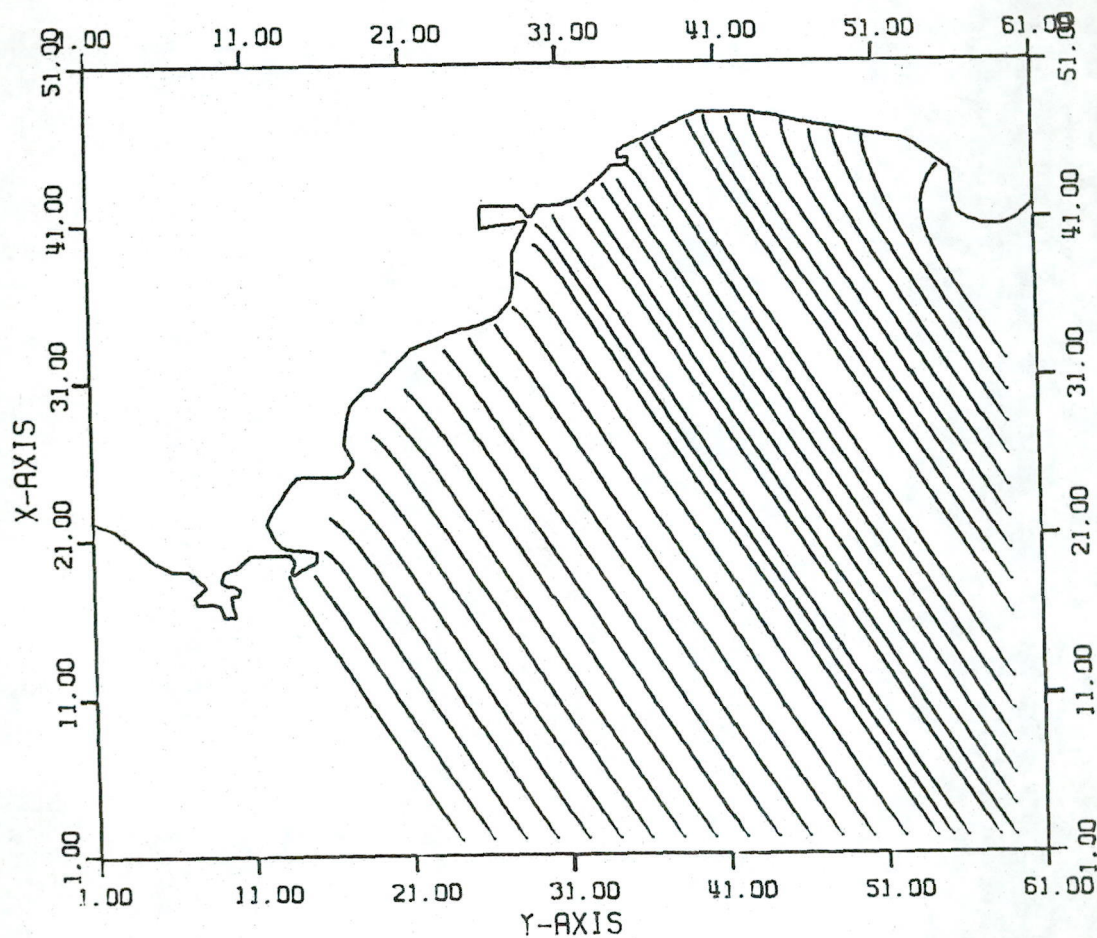
HANGKLIP DAM

REFRACTION DIAGRAM T = 7s , DEEPWATER DIR.=180°

FIGURE

21

REFRACTION (DUTOIT) - PALMIET
 WAVE DIRECTION AND PERIOD 180 - 9S
 SCALE OF PLOT = 1: 196850.00



TRACED JMD

CHECKED

DATE DEC. 90

REF C/SEA 8048

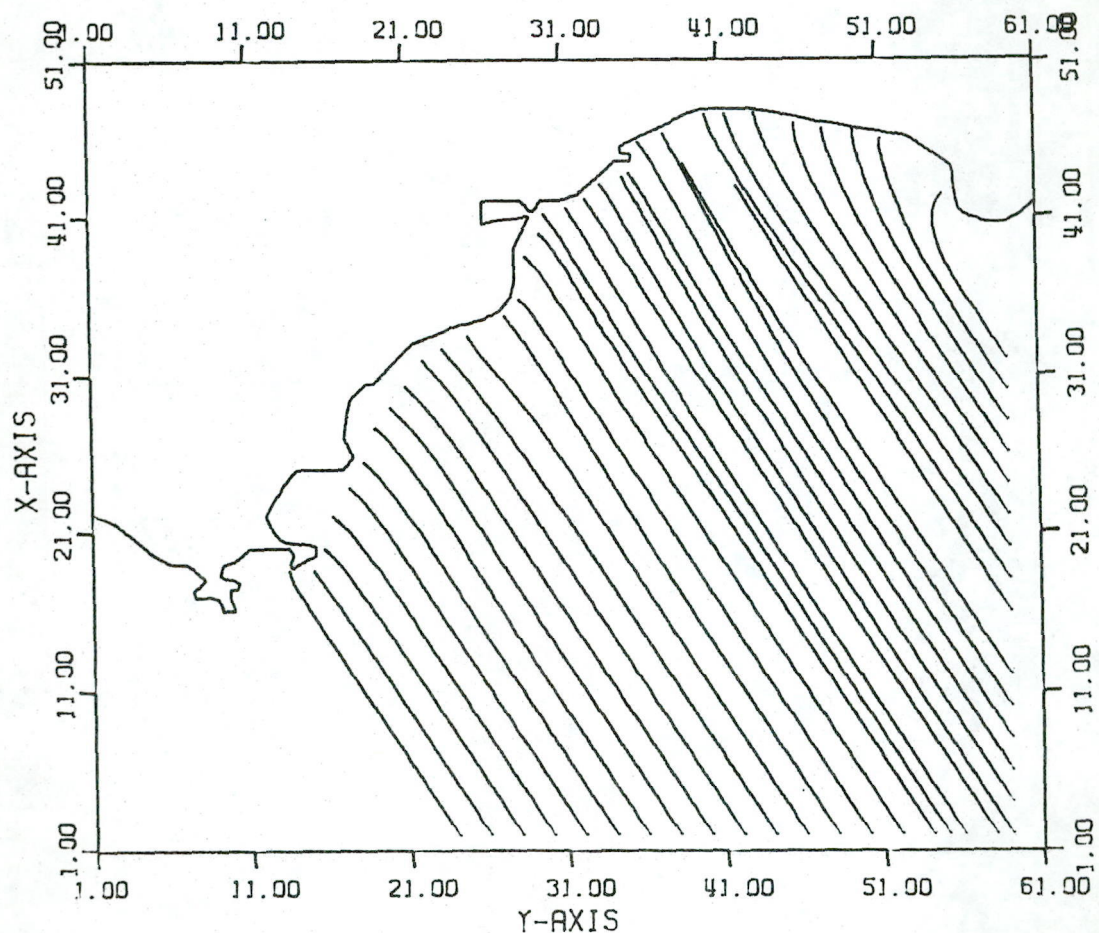
HANGKLIP DAM

REFRACTION DIAGRAM T = 9s , DEEPWATER DIR.=180°

FIGURE

22

REFRACTION (DUTOIT) - PALMIET
 WAVE DIRECTION AND PERIOD 180 -11S
 SCALE OF PLOT = 1: 196850.00



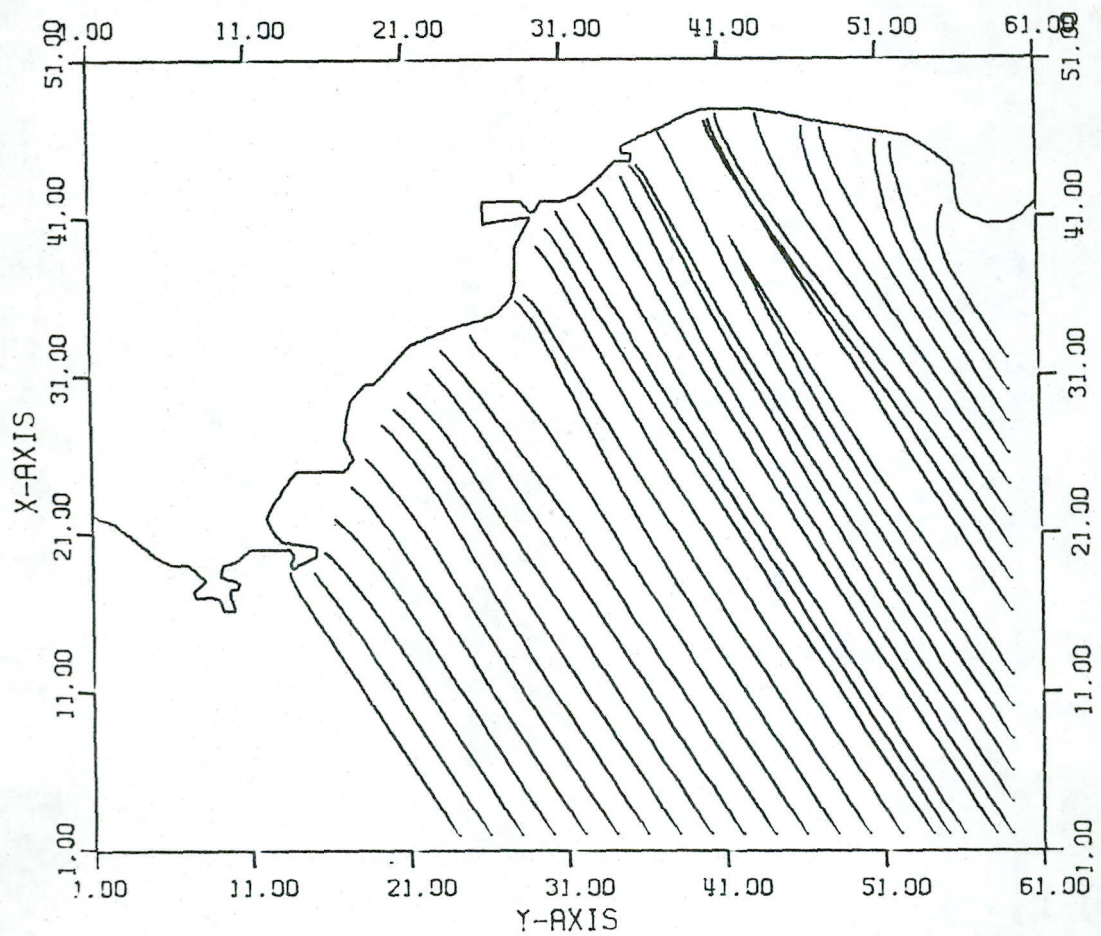
TRACED JMO
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 DATE DEC. '80
 REF C/SEA 8048

HANGKLIP DAM

REFRACTION DIAGRAM T = 11s , DEEPWATER DIR.=180°

FIGURE
 23

REFRACTION (DUTOIT) - PALMIET
 WAVE DIRECTION AND PERIOD 180 -13S
 SCALE OF PLOT = 1: 196850.00



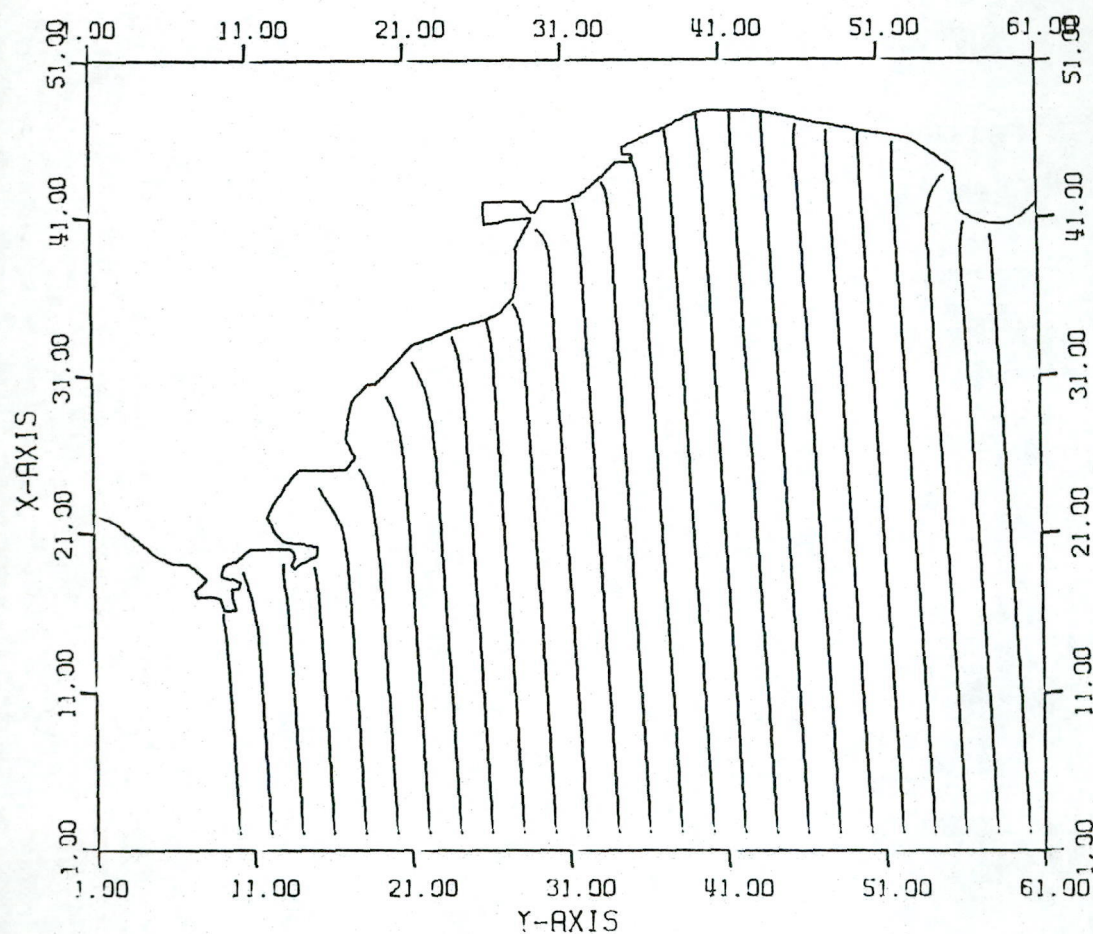
TRACED JMO
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 DATE DEC '80
 REF C/SEA 8048

HANGKLIP DAM

REFRACTION DIAGRAM T = 13s , DEEPWATER DIR.=180°

FIGURE
 24

REFRACTION (DUTOIT) - PALMIET
 WAVE DIRECTION AND PERIOD 210 - 7S
 SCALE OF PLOT = 1: 196850.00



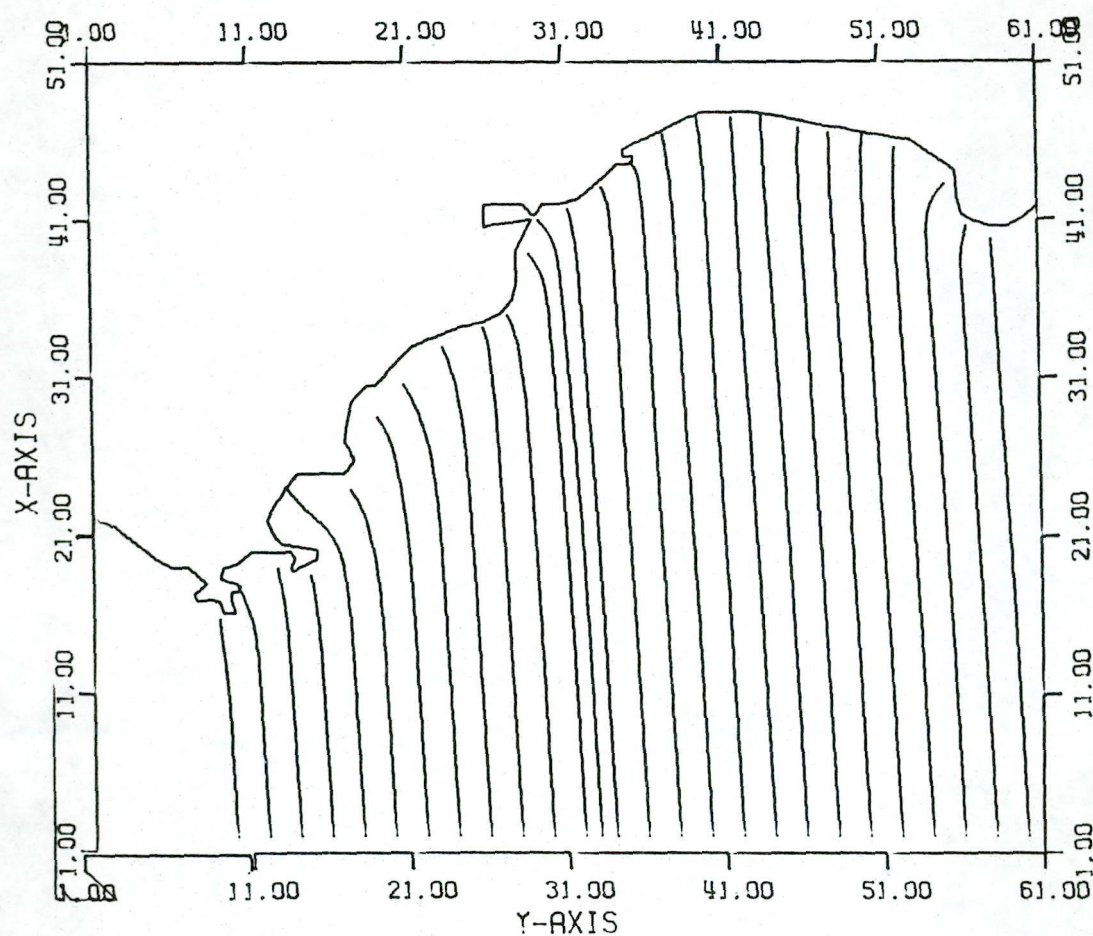
TRACED JMO
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 DATE DEC. '80
 RE: C/SEA 8048

HANGKLIP DAM

REFRACTION DIAGRAM $T = 7s$, DEEPWATER DIR. = 210°

FIGURE
 25

REFRACTION (DUTOIT) - PALMIET
 WAVE DIRECTION AND PERIOD 210 - 9S
 SCALE OF PLOT = 1: 196850.00



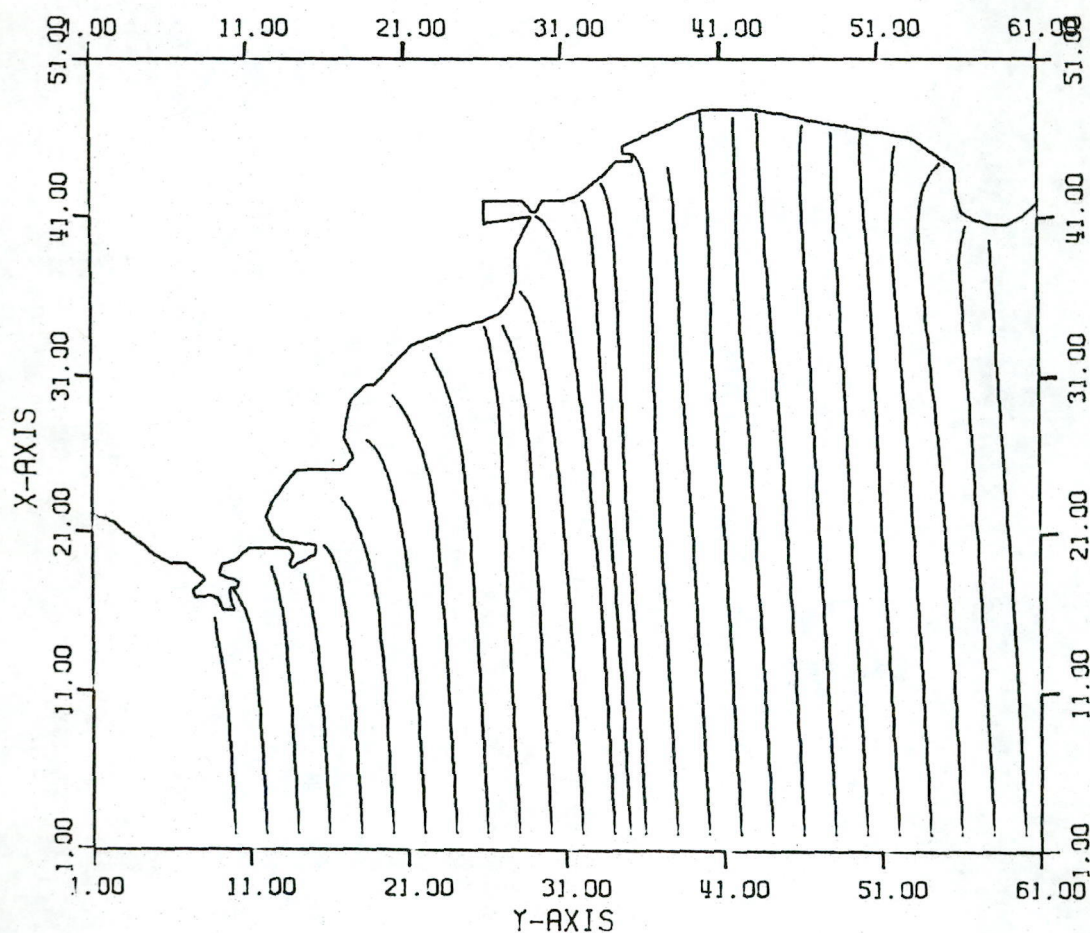
TRACED JMD
 CHECKED
 DATE DEC. 80
 REF C/SEA 8048

HANGKLIP DAM

REFRACTION DIAGRAM T = 9s , DEEPWATER DIR. = 210°

FIGURE
 26

REFRACTION (DUTOIT) - PALMIET
 WAVE DIRECTION AND PERIOD 210 - 11S
 SCALE OF PLOT = 1: 196850.00



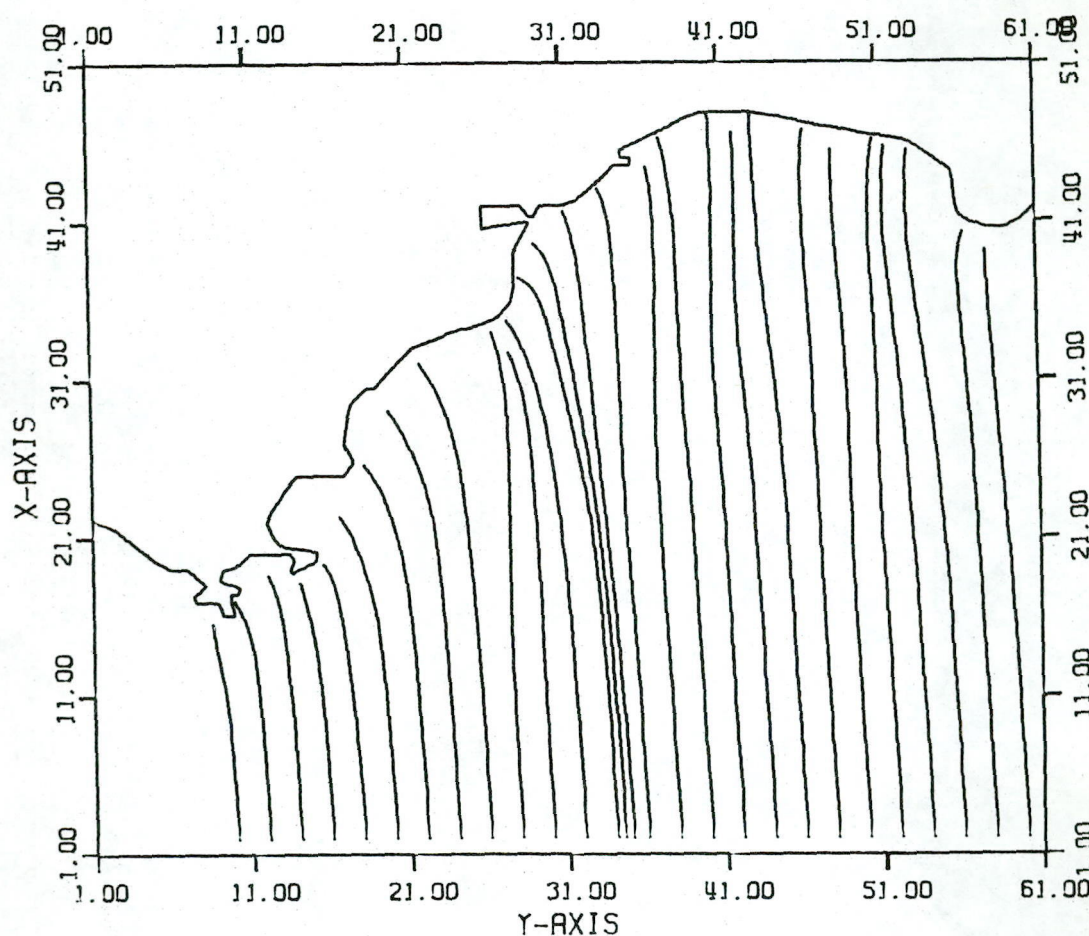
TRACED JMO
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 DATE DEC. 80
 REF C/SEA 8048

HANGKLIP DAM

REFRACTION DIAGRAM T = 11s , DEEPWATER DIR.=210°

FIGURE
 27

REFRACTION (DUTOIT) - PALMIET
 WAVE DIRECTION AND PERIOD 210 - 13S
 SCALE OF PLOT = 1: 196850.00



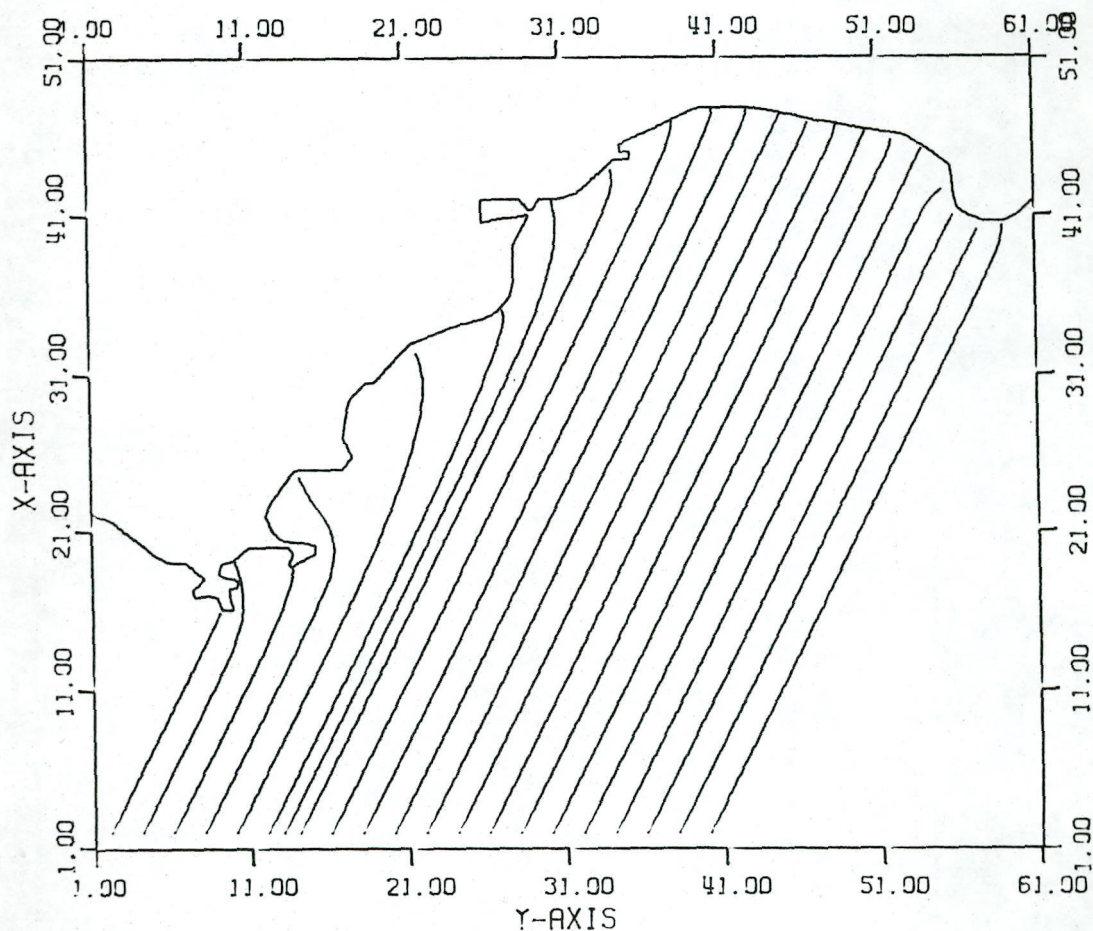
TRADED JMD
 CHECKED
 DATE DEC. 80
 REF C/SEA 8048

HANGKLIP DAM

REFRACTION DIAGRAM T = 13s , DEEPWATER DIR.=210°

FIGURE
 28

REFRACTION (DUTOIT) - PALMIET
 WAVE DIRECTION AND PERIOD 240 - 7S
 SCALE OF PLOT = 1: 196850.00



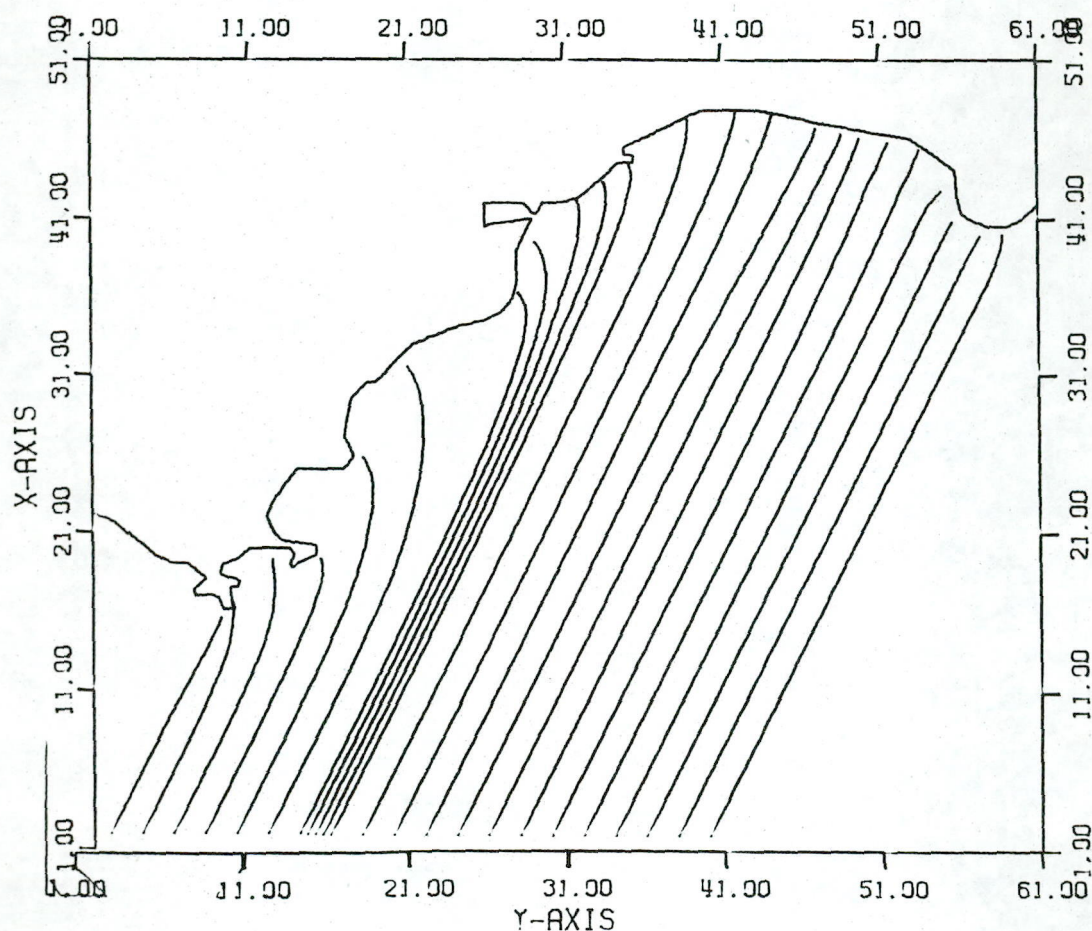
TRACED JMO
 CHECKED
 DATE DEC '80
 REF C/SEA 8048

HANGKLIP DAM

REFRACTION DIAGRAM T = 7s , DEEPWATER DIR.=240°

FIGURE
 29

REFRACTION (DUTOIT) - PALMIET
 WAVE DIRECTION AND PERIOD 240 - 9S
 SCALE OF PLOT = 1: 196850.00



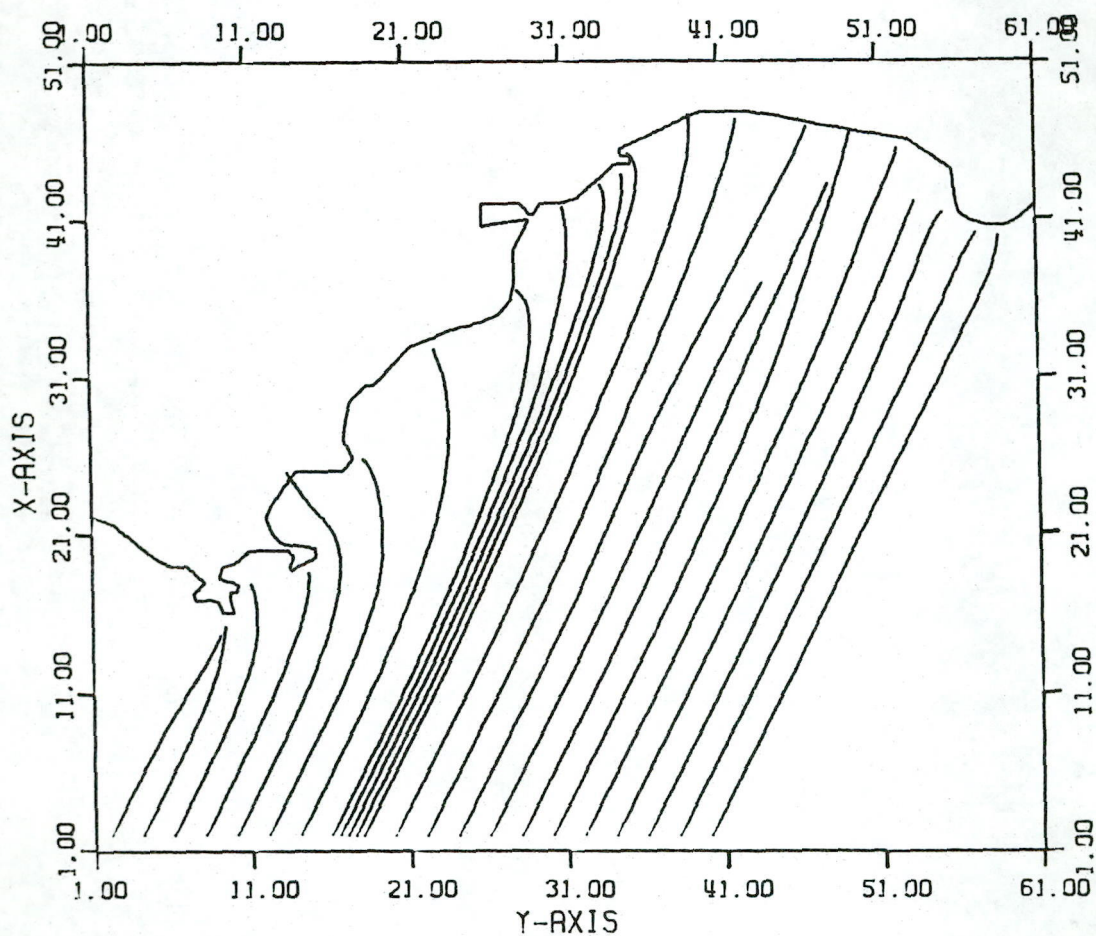
TRACED JMO
 CHECKED
 DATE DEC-80
 REF CSEA 8048

HANGKLIP DAM

REFRACTION DIAGRAM T = 9s , DEEPWATER DIR.=240°

FIGURE
 30

REFRACTION (DUTOIT) - PALMIET
 WAVE DIRECTION AND PERIOD 240 - 11S
 SCALE OF PLOT = 1: 196850.00



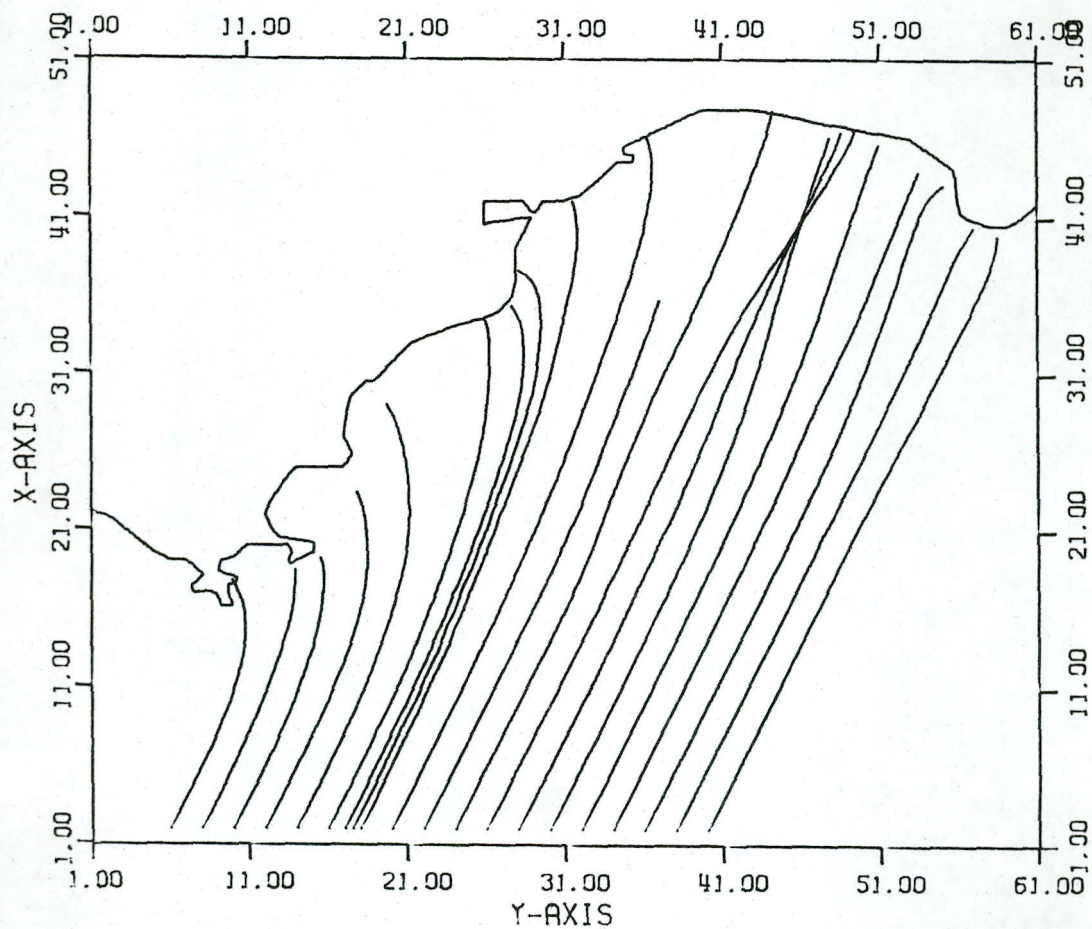
TRACED JMO
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 DATE DEC-80
 REF C/SEA 8048

HANGKLIP DAM

REFRACTION DIAGRAM T = 11s , DEEPWATER DIR.=240°

FIGURE
 31

REFRACTION (DUTOIT) - PALMIET
 WAVE DIRECTION AND PERIOD 240 - 13S
 SCALE OF PLOT = 1: 196850.00



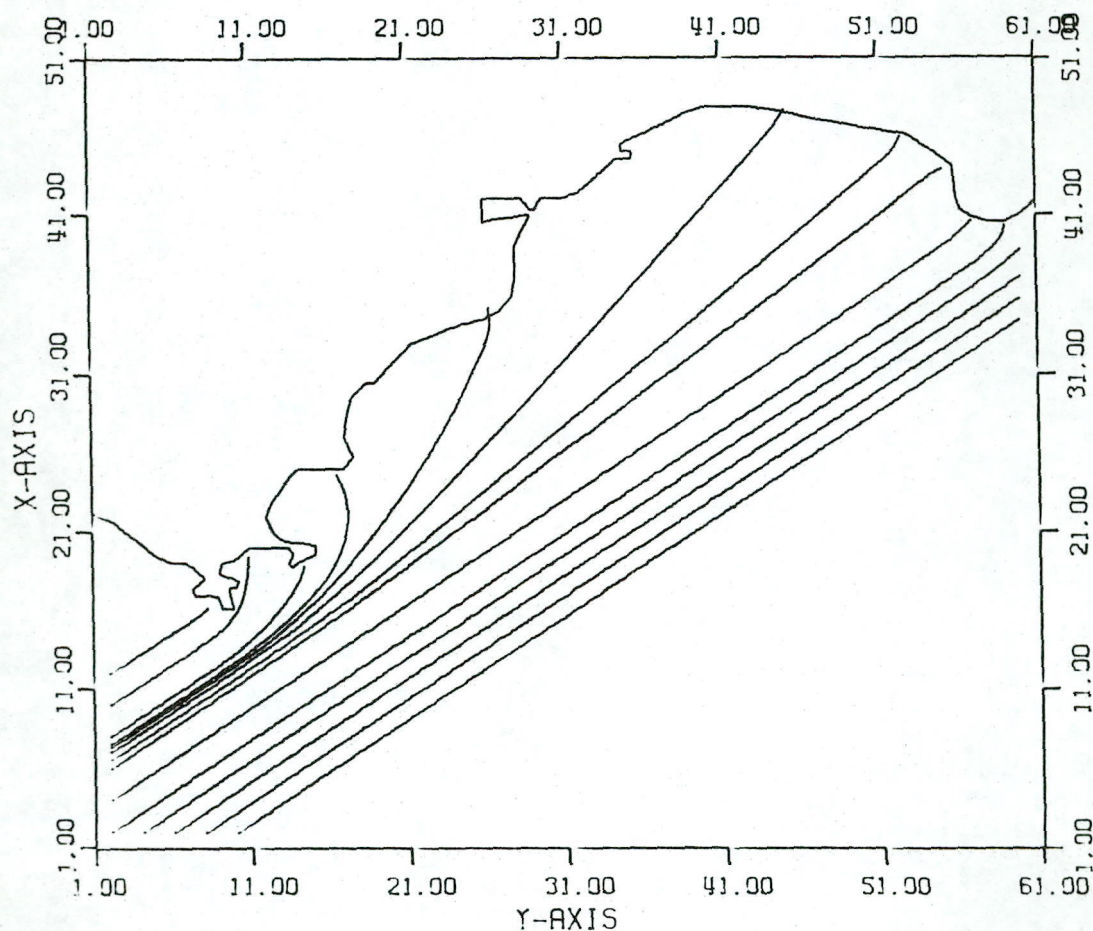
TRACED JMO
 CHECKED
 DATE DEC. 80
 REF. CSEA 8048

HANGKLIP DAM

REFRACTION DIAGRAM T = 13s , DEEPWATER DIR.=240°

FIGURE
 32

REFRACTION (DUTOIT) - PALMIET
 WAVE DIRECTION AND PERIOD 270 - 7S
 SCALE OF PLOT = 1: 196850.00

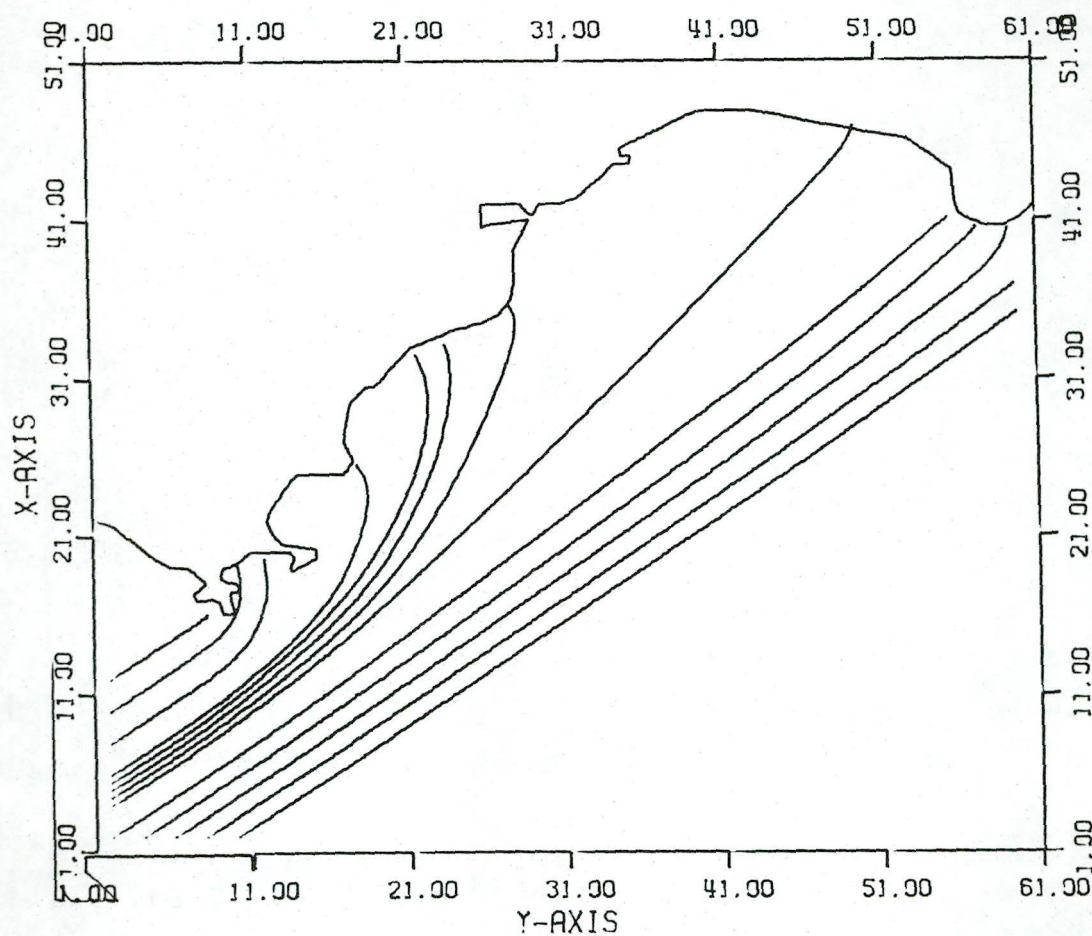


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 DATE DEC '80
 REF C/SEA F048

HANGKLIP DAM
 REFRACTION DIAGRAM T = 7s , DEEPWATER DIR.=270°

FIGURE
 33

REFRACTION (DUTOIT) - PALMIET
 WAVE DIRECTION AND PERIOD 270 - 9S
 SCALE OF PLOT = 1: 196850.00



TRACED JMO
 CHECKED
 DATE DEC. '80
 REF C/SEA 8048

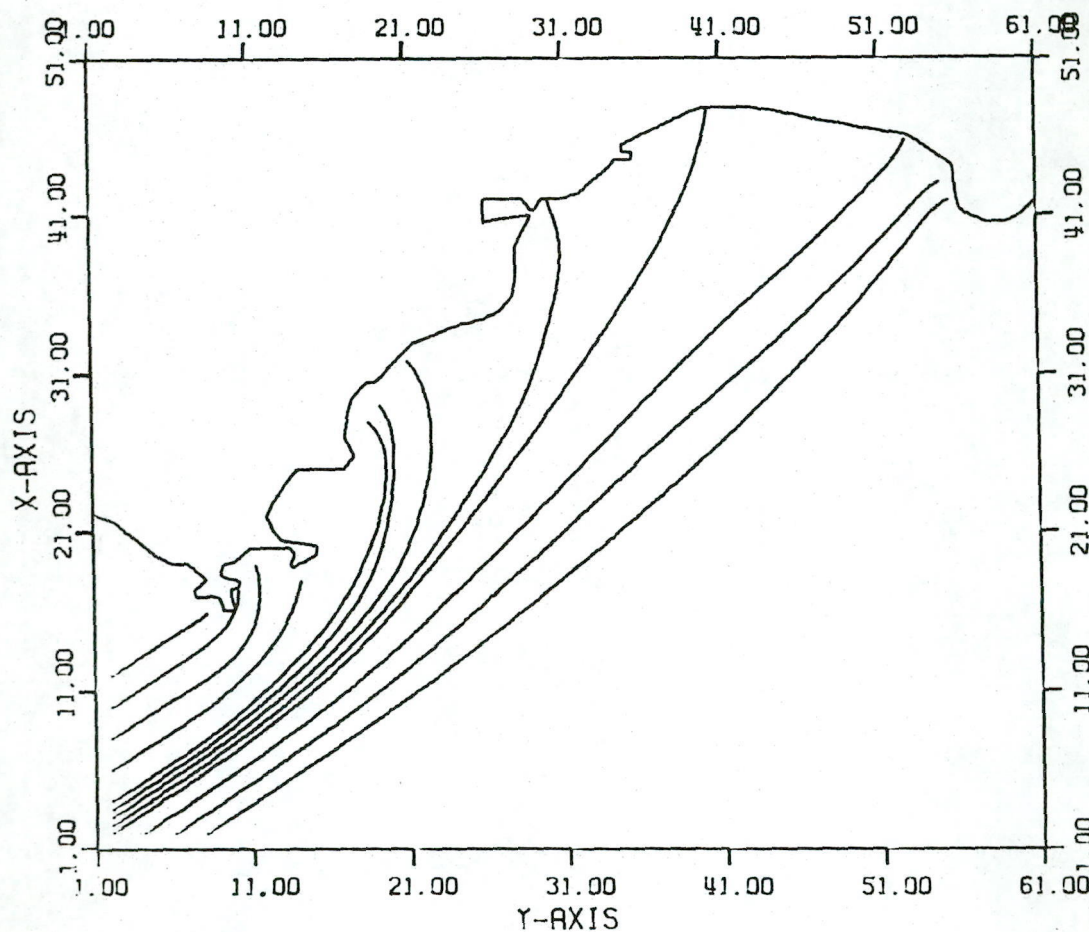
HANGKLIP DAM

REFRACTION DIAGRAM T = 9s , DEEPWATER DIR.=270°

FIGURE

34

REFRACTION (DUTOIT) - PALMIET
 WAVE DIRECTION AND PERIOD 270 -11S
 SCALE OF PLOT = 1: 196850.00



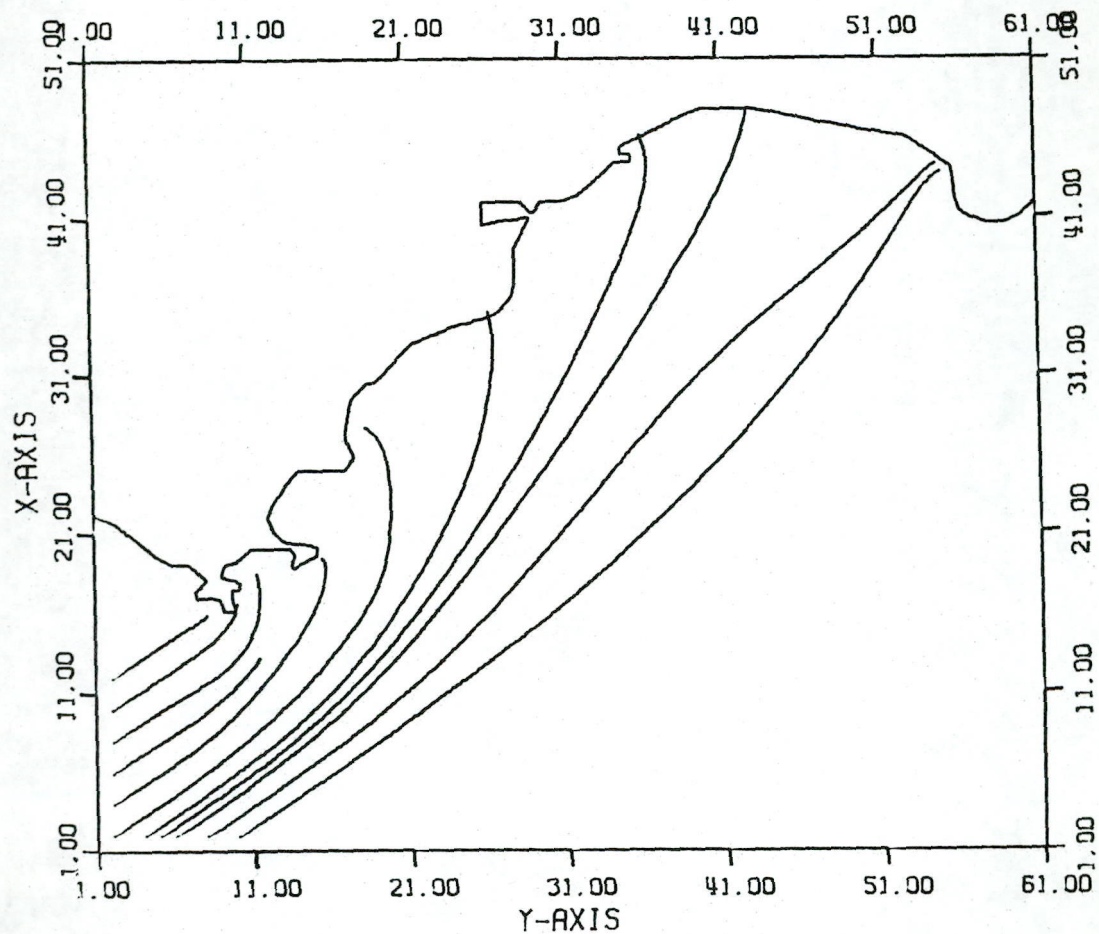
TRACED JMO
 CHECKED
 DATE DEC. '80
 REF C/SEA 8048

HANGKLIP DAM

REFRACTION DIAGRAM T = 11s , DEEPWATER DIR.=270°

FIGURE
 35

REFRACTION (DUTOIT) - PALMIET
 WAVE DIRECTION AND PERIOD 270 -13S
 SCALE OF PLOT = 1: 196850.00



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 CHECKED
 DATE DEC-80
 REF CREN 8349

HANGKLIP DAM

REFRACTION DIAGRAM T = 13s , DEEPWATER DIR.=270°

FIGURE
 36

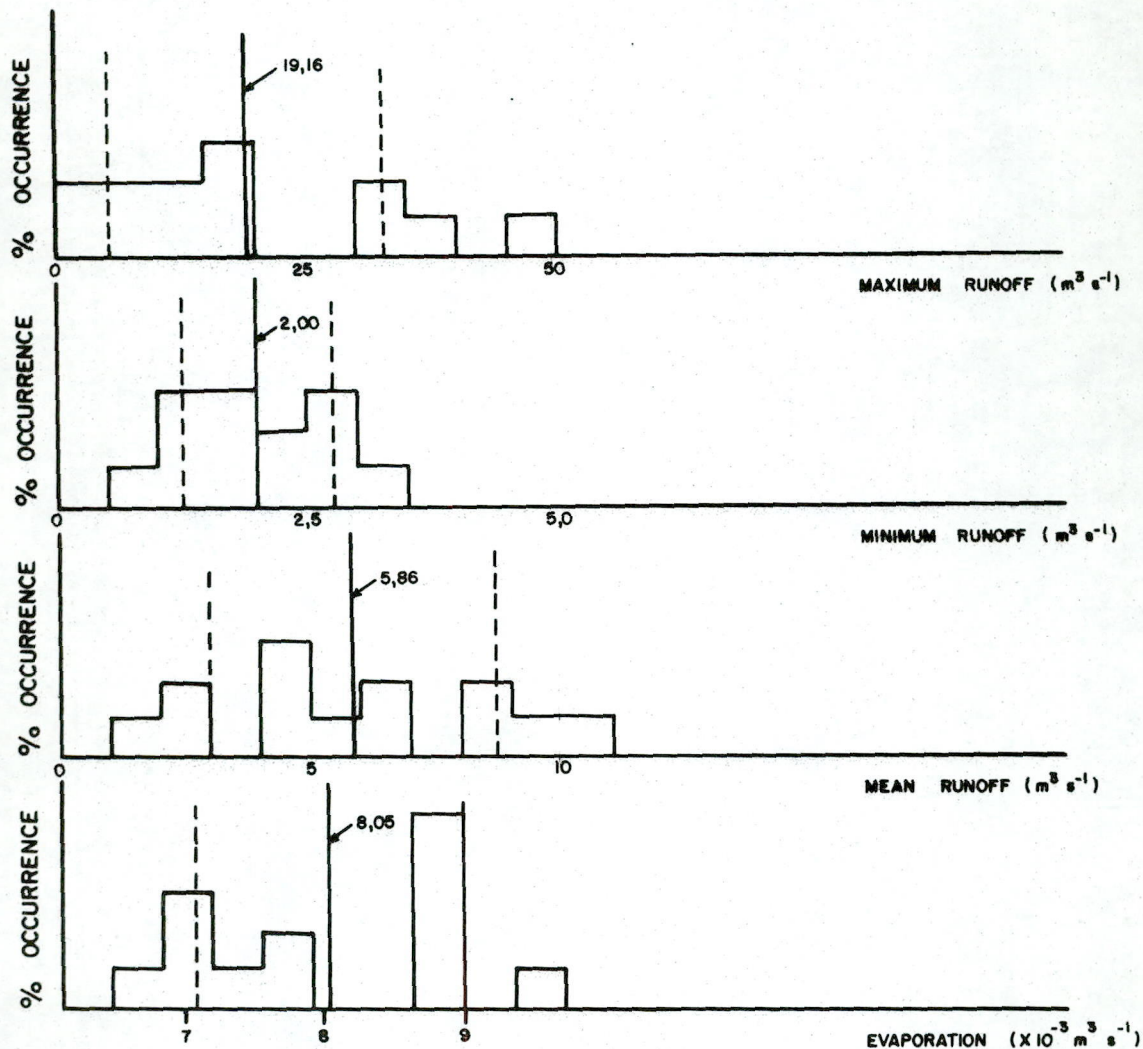
APPENDIX A

MONTHLY SUMMARY OF HYDROLOGICAL PARAMETERS

LIST OF FIGURES

Figure A 1	Hydrological parameters for October
A 2	Hydrological parameters for November
A 3	Hydrological parameters for December
A 4	Hydrological parameters for January
A 5	Hydrological parameters for February
A 6	Hydrological parameters for March
A 7	Hydrological parameters for April
A 8	Hydrological parameters for May
A 9	Hydrological parameters for June
A10	Hydrological parameters for July
A11	Hydrological parameters for August
A12	Hydrological parameters for September

OCTOBER



ALL RUNOFF DATA BASED ON DAILY OBSERVATIONS AT G4M07 FOR
1963/64 TO 1975/76

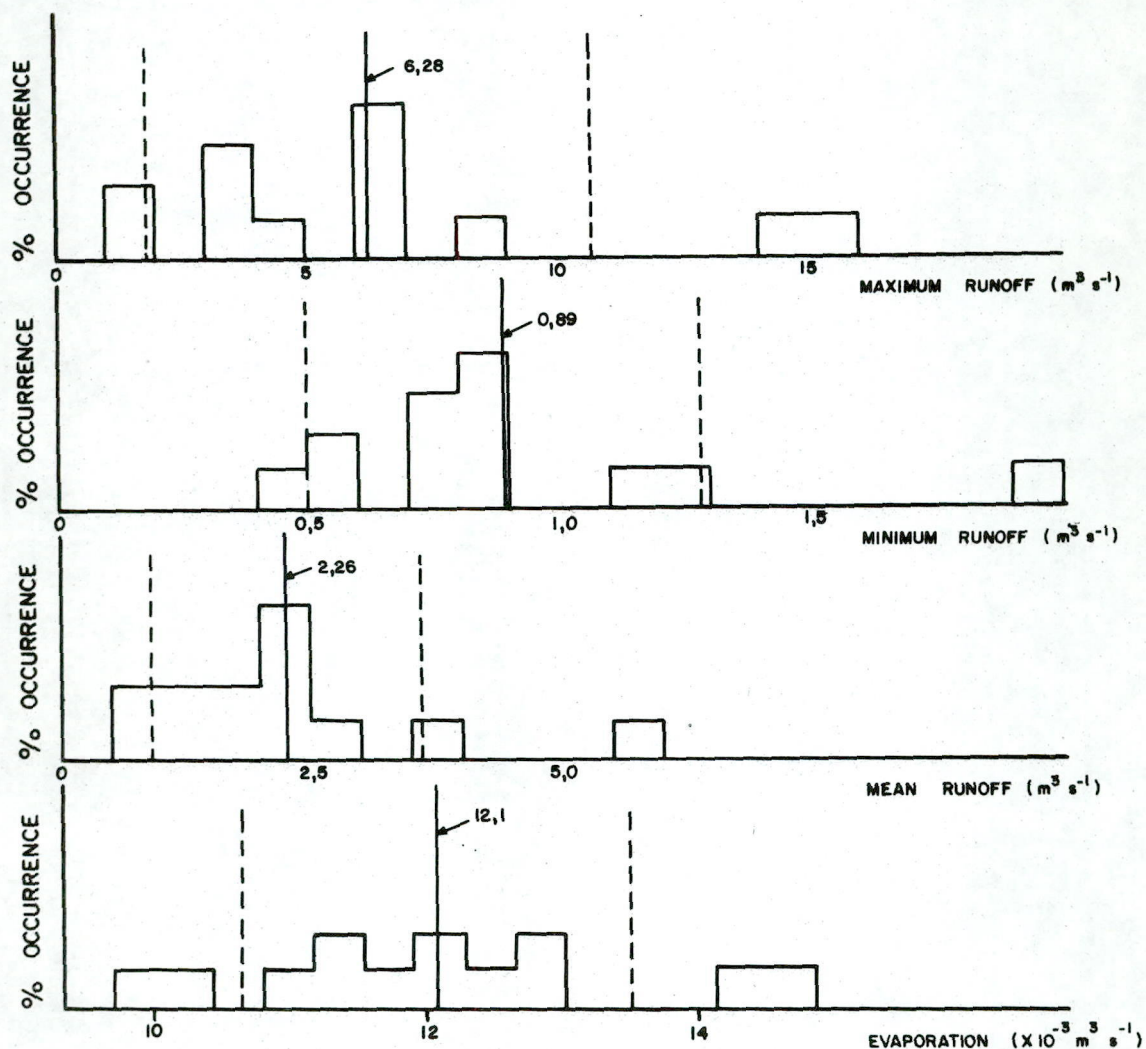
————— MEAN
----- ± ONE STANDARD DEVIATION

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HANGKLIP DAM HYDROLOGICAL PARAMETERS FOR OCTOBER

FIGURE
A1

NOVEMBER



ALL RUNOFF DATA BASED ON DAILY OBSERVATIONS AT G4M07 FOR
1963/64 TO 1975/76

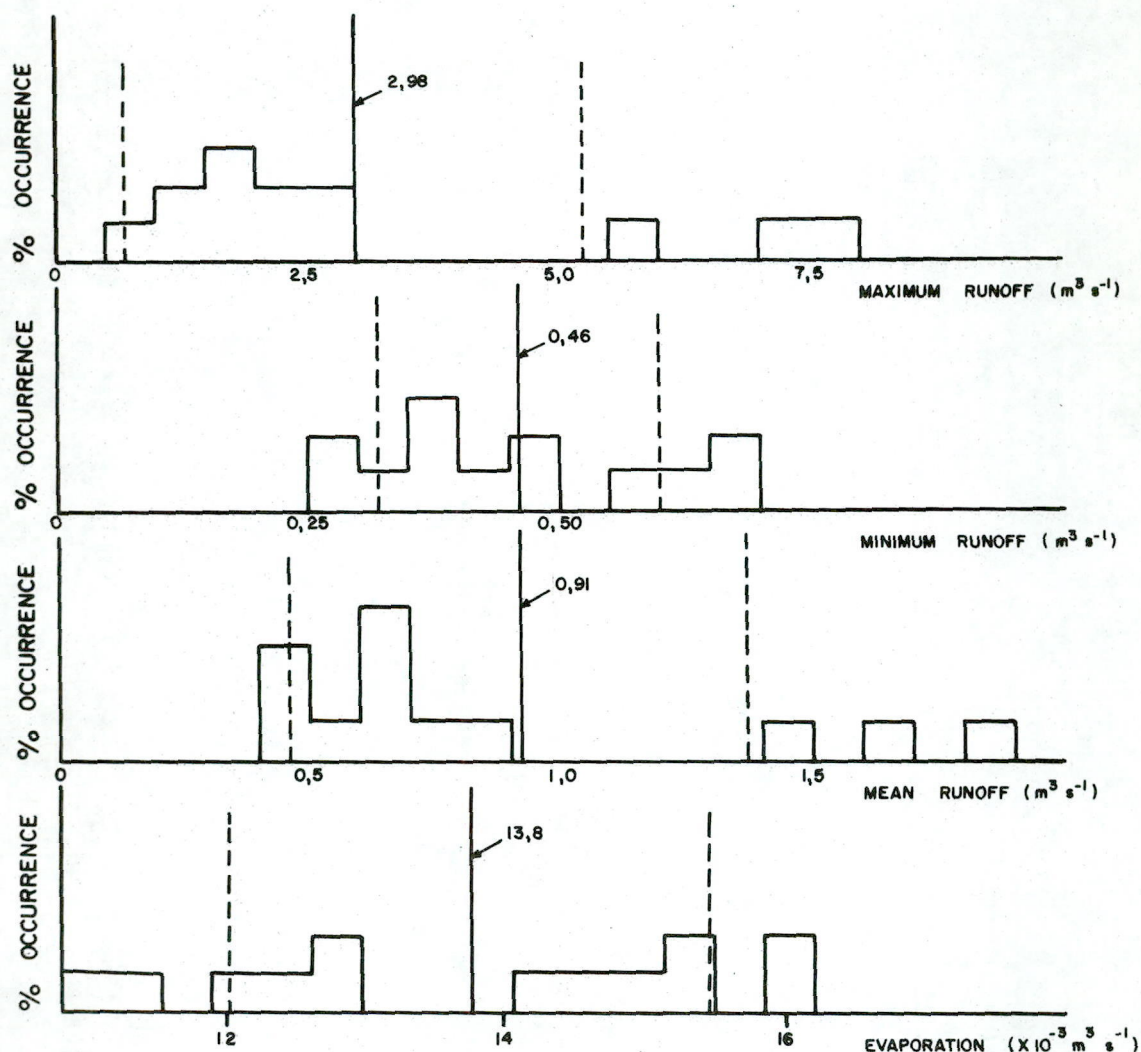
—— MEAN
- - - - - $\pm$ ONE STANDARD DEVIATION

TRACED JMO
CHECKED
DATE DEC. '80
REF C/SEA 8048

HANGKLIP DAM HYDROLOGICAL PARAMETERS FOR NOVEMBER

FIGURE
A2

DECEMBER



ALL RUNOFF DATA BASED ON DAILY OBSERVATIONS AT G4M07 FOR 1963/64 TO 1975/76

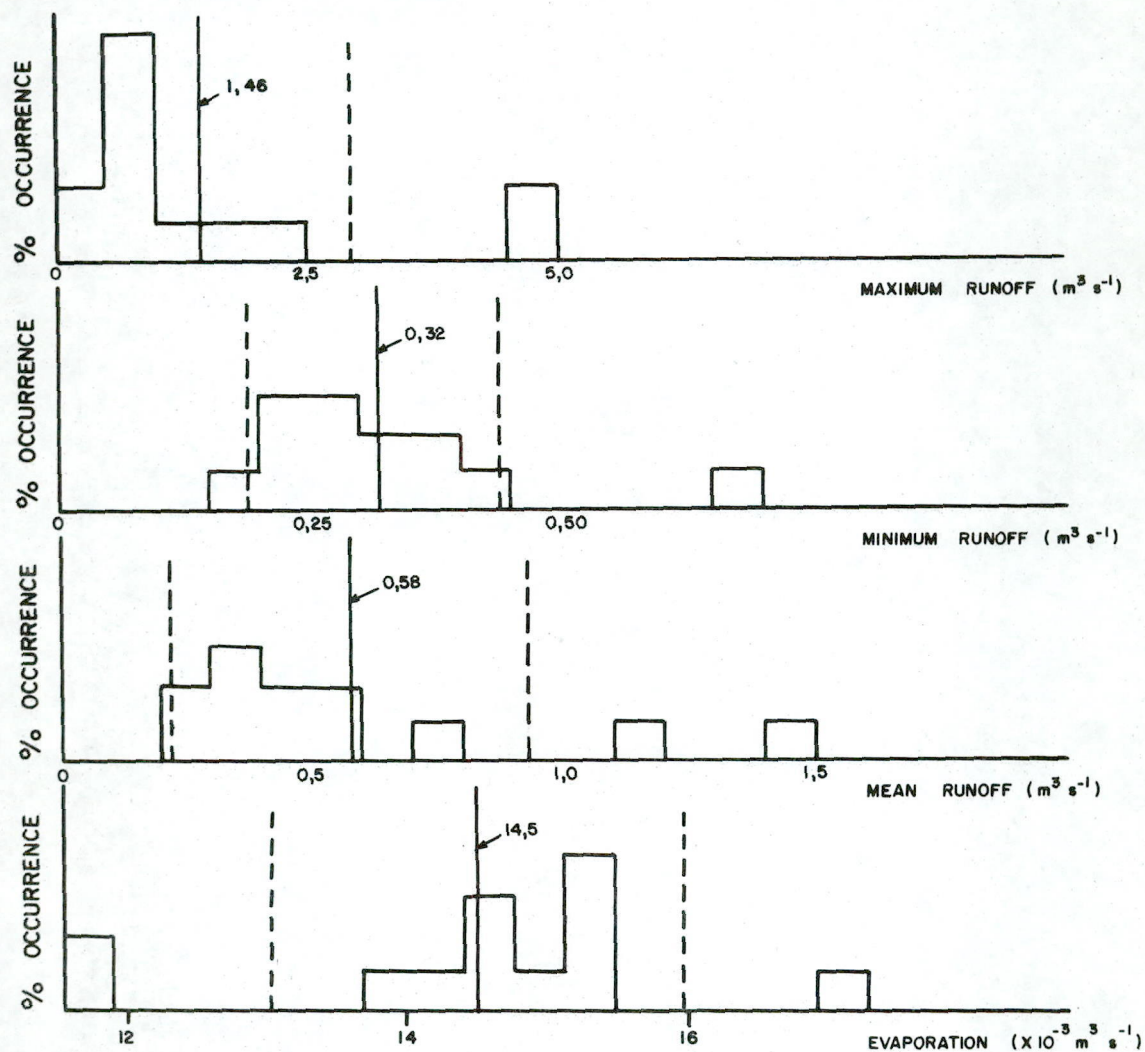
—— MEAN
 - - - - $\pm$ ONE STANDARD DEVIATION

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 CHECKED
 DATE DEC. 80
 REF C/REA 8048

HANGKLIP DAM HYDROLOGICAL PARAMETERS FOR DECEMBER

FIGURE
 A3

JANUARY



ALL RUNOFF DATA BASED ON DAILY OBSERVATIONS AT G4M07 FOR
1963/64 TO 1975/76

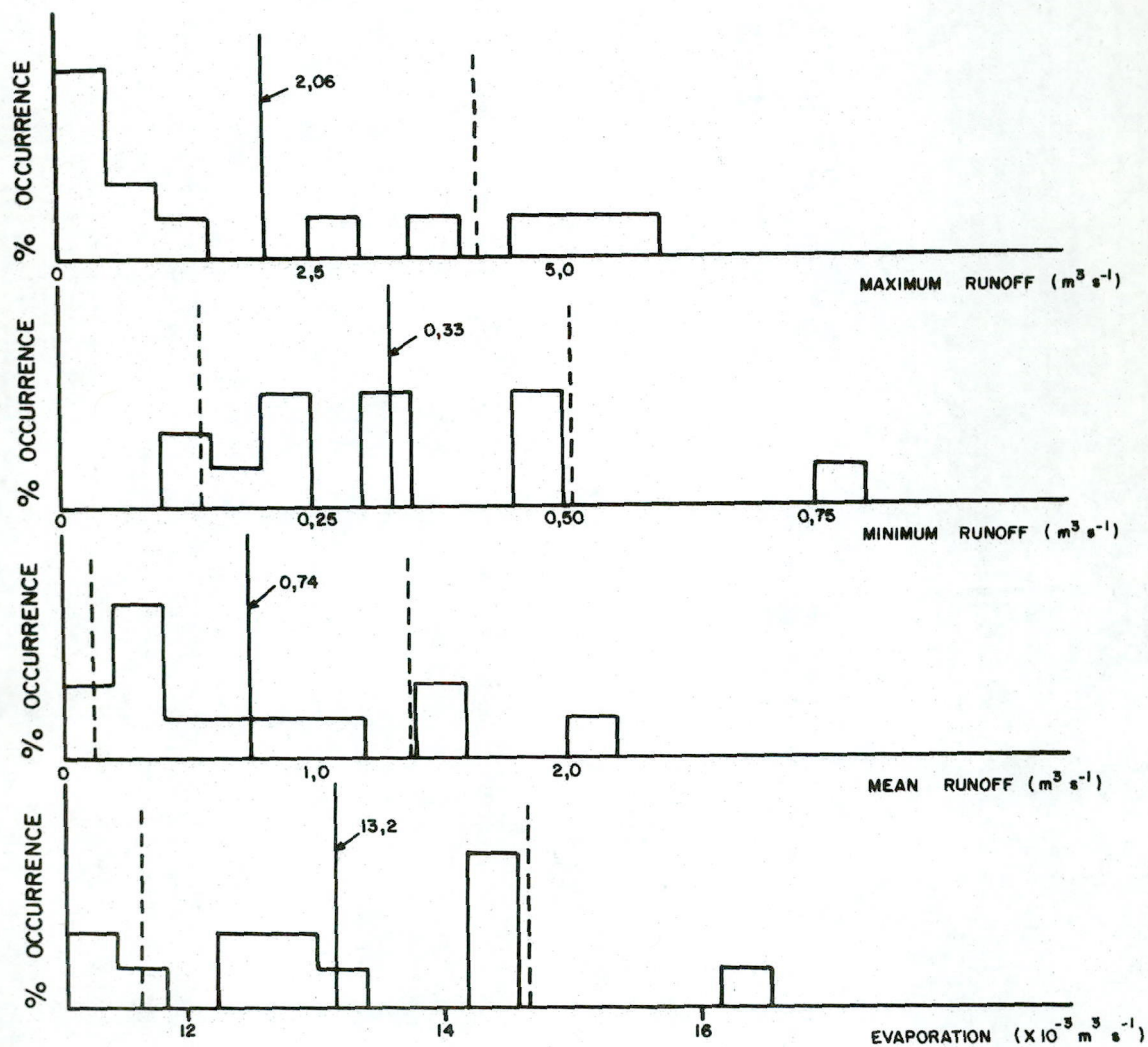
————— MEAN
----- ± ONE STANDARD DEVIATION

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CHECKED
DATE DEC '80
REF C/SEA 8048

HANGKLIP DAM HYDROLOGICAL PARAMETERS FOR JANUARY

FIGURE
A4

FEBRUARY



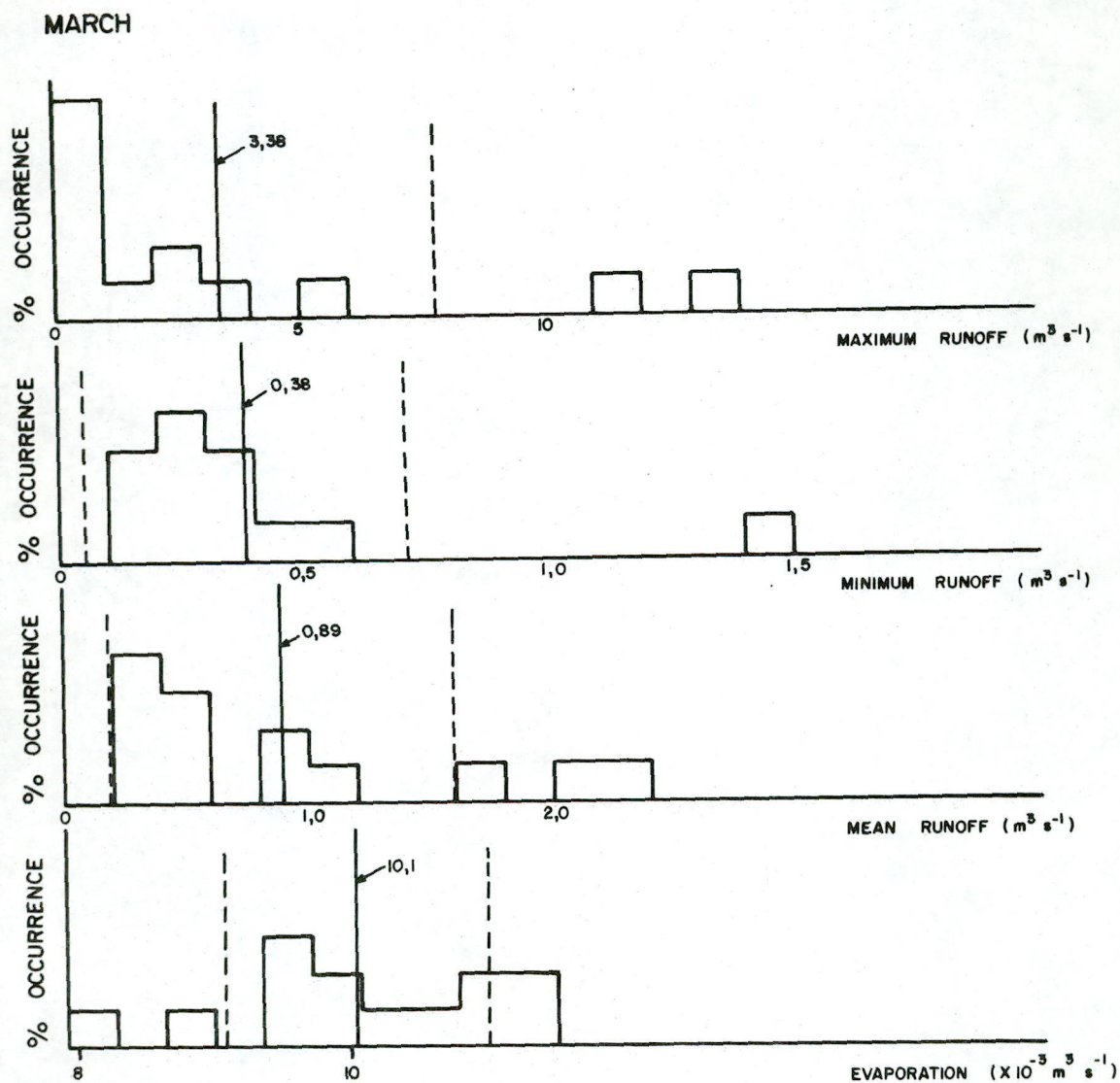
ALL RUNOFF DATA BASED ON DAILY OBSERVATIONS AT G4M07 FOR
1963/64 TO 1975/76

— MEAN
- - - ± ONE STANDARD DEVIATION

TRACED JMD
CHECKED
DATE DEC '90
REF C/SEA 8048

HANGKLIP DAM HYDROLOGICAL PARAMETERS FOR FEBRUARY

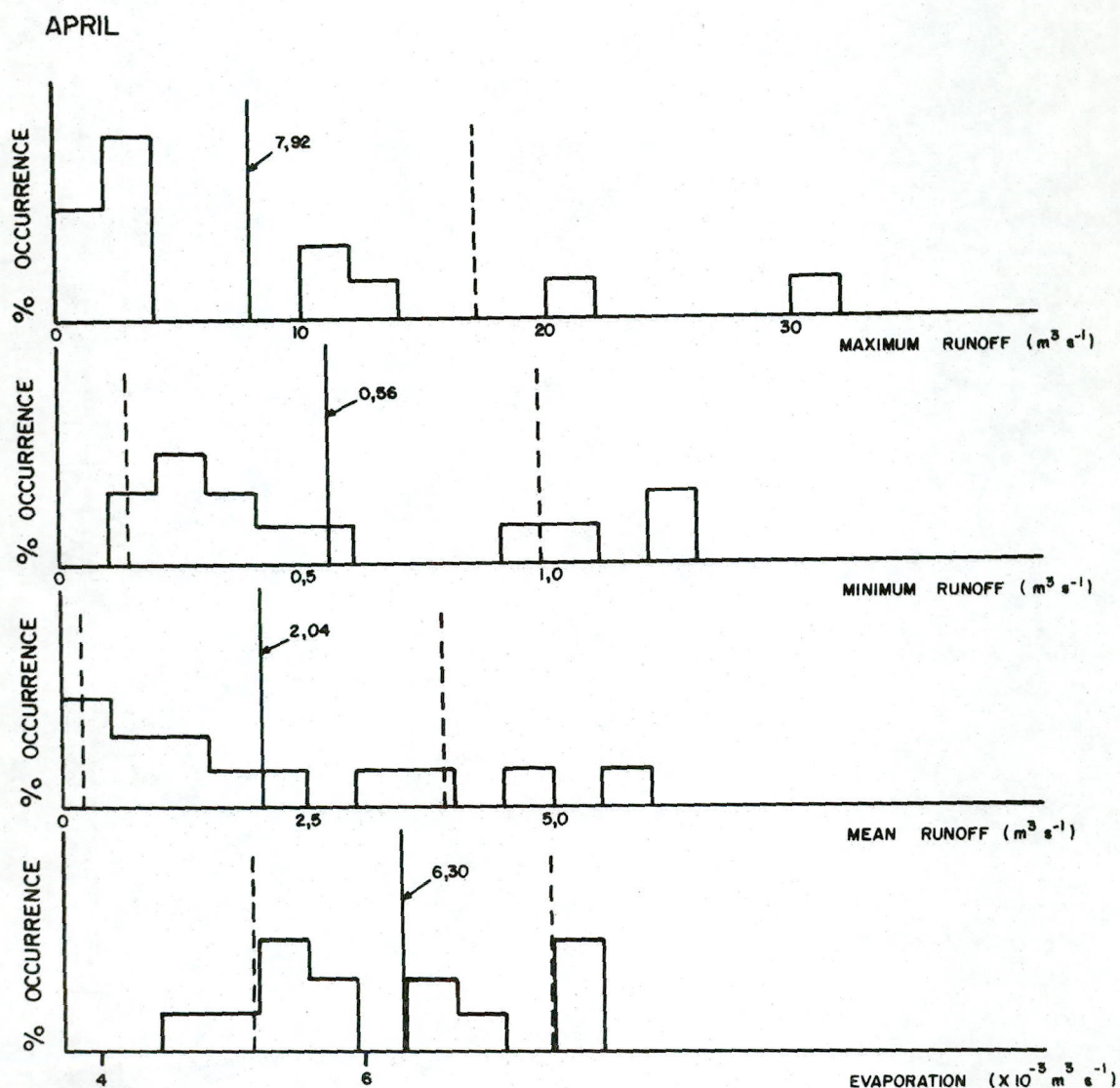
FIGURE
A5



TRACED JMO
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 DATE DEC. '80
 REF C/SEA 8048

HANGKLIP DAM
 HYDROLOGICAL PARAMETERS
 FOR MARCH

FIGURE
 A6



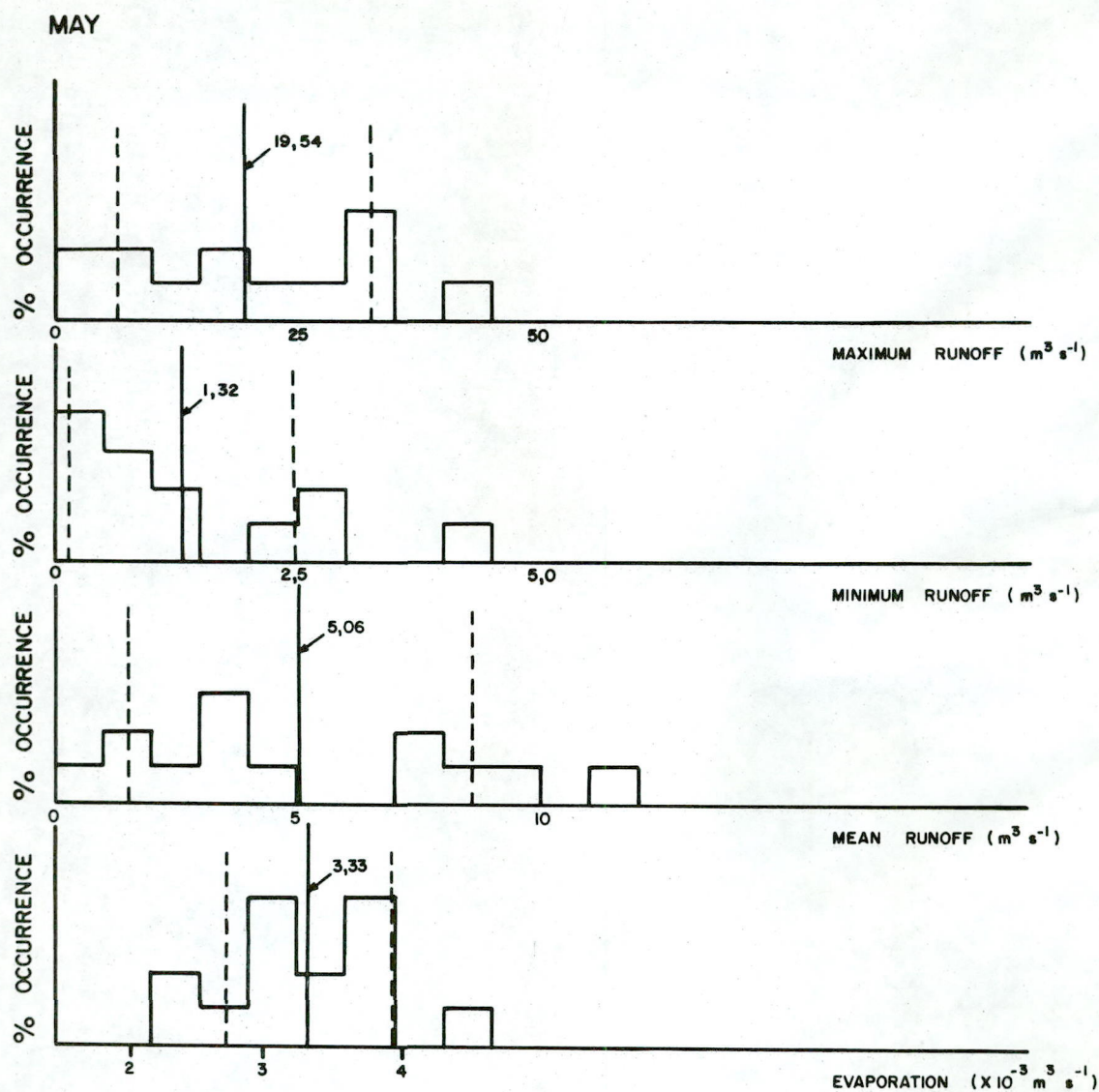
ALL RUNOFF DATA BASED ON DAILY OBSERVATIONS AT G4M07 FOR
1963/64 TO 1975/76

——— MEAN
- - - - - $\pm$ ONE STANDARD DEVIATION

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DATE DEC. '80
REF C/SEA 8048

HANGKLIP DAM
HYDROLOGICAL PARAMETERS
FOR APRIL

FIGURE
A7



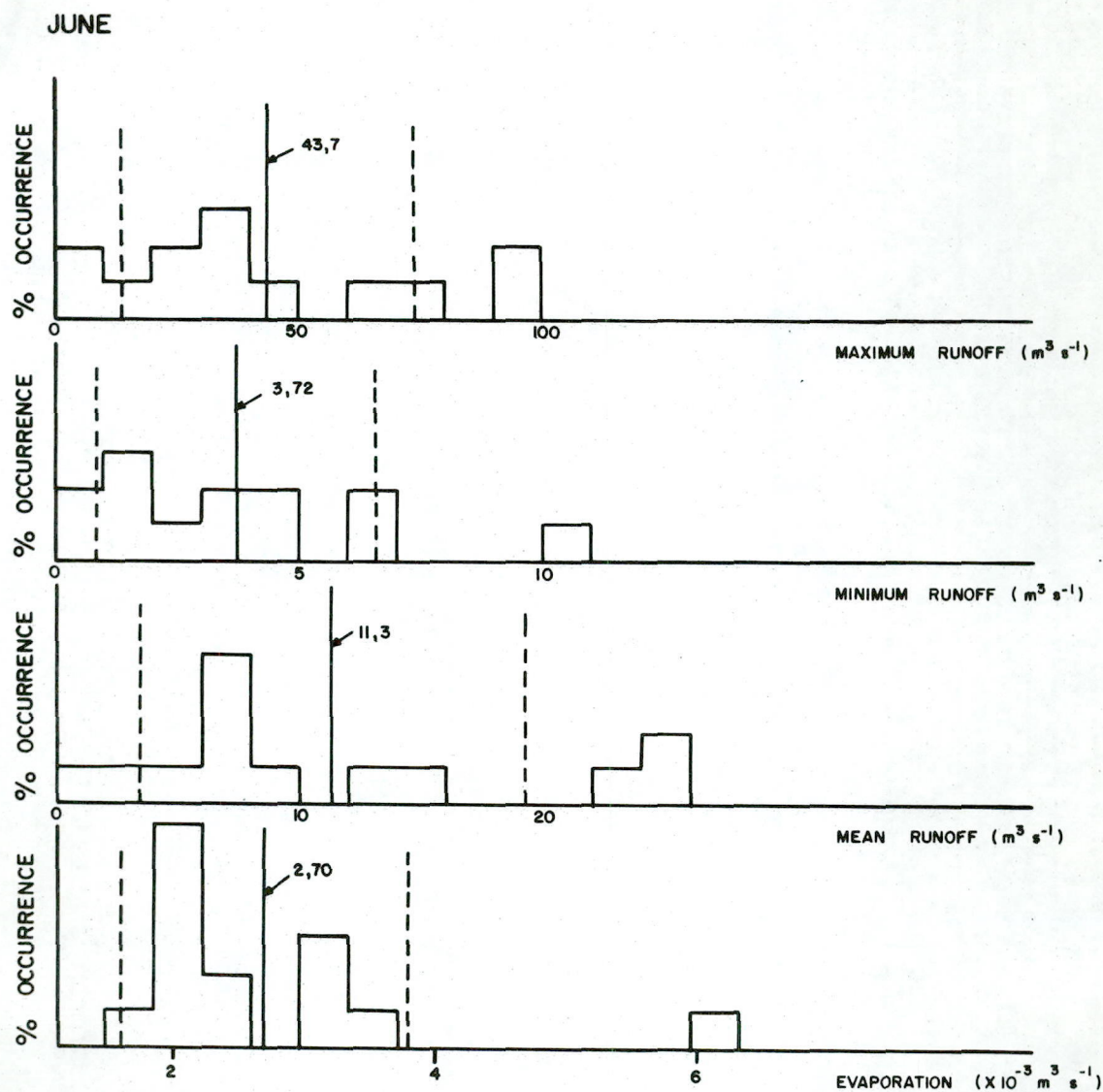
ALL RUNOFF DATA BASED ON DAILY OBSERVATIONS AT G4M07 FOR
1963/64 TO 1975/76

— MEAN
- - - ± ONE STANDARD DEVIATION

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DATE DEC. '80
REF C/SEA 8048

HANGKLIP DAM
HYDROLOGICAL PARAMETERS
FOR MAY

FIGURE
A8



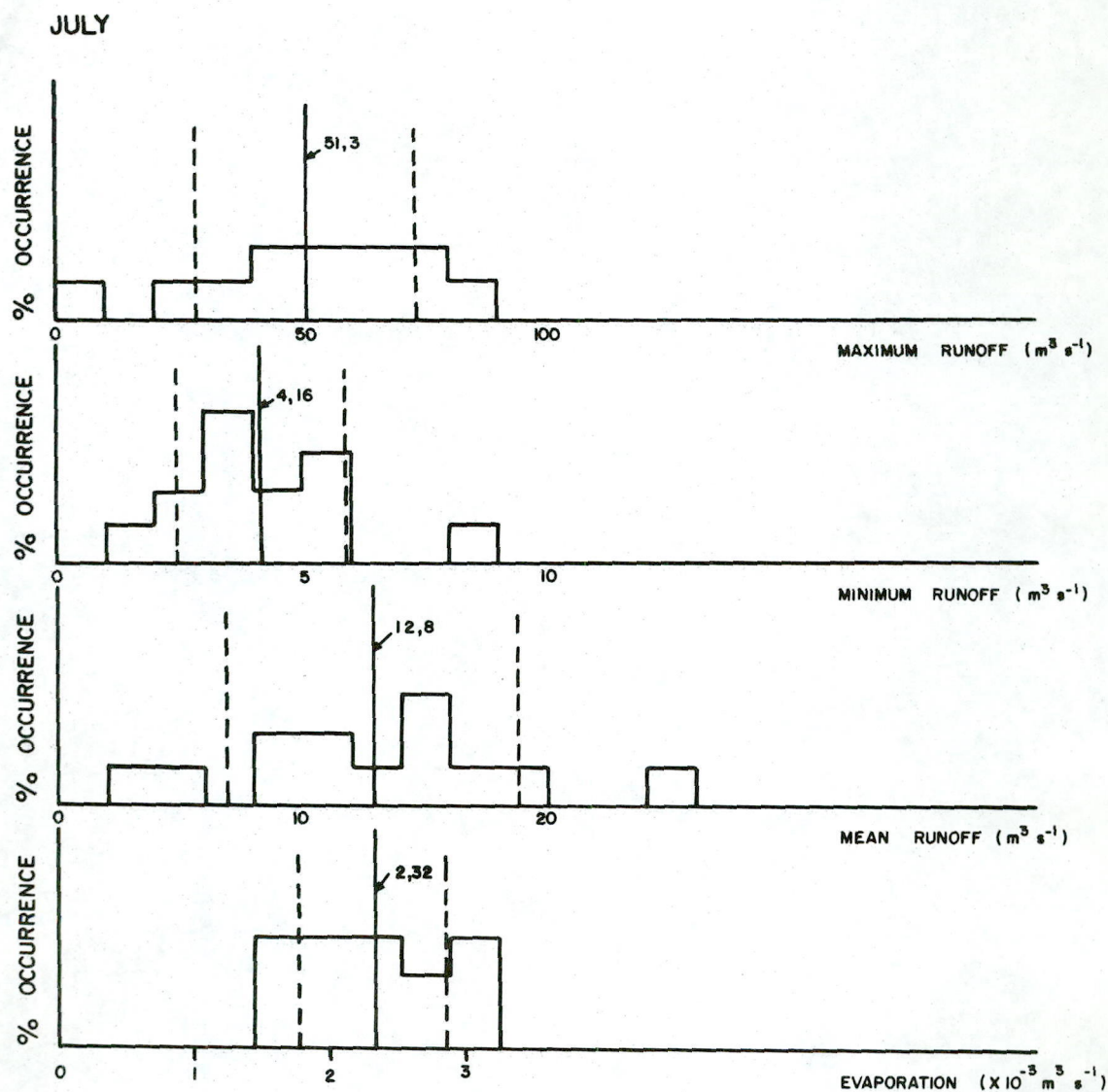
ALL RUNOFF DATA BASED ON DAILY OBSERVATIONS AT G4M07 FOR
1963/64 TO 1975/76

—— MEAN
- - - - ± ONE STANDARD DEVIATION

TRACED JMO
CHECKED
DATE DEC. 80
REF C/SEA 8048

HANGKLIP DAM
HYDROLOGICAL PARAMETERS
FOR JUNE

FIGURE
A9



ALL RUNOFF DATA BASED ON DAILY OBSERVATIONS AT G4M07 FOR
1963/64 TO 1975/76

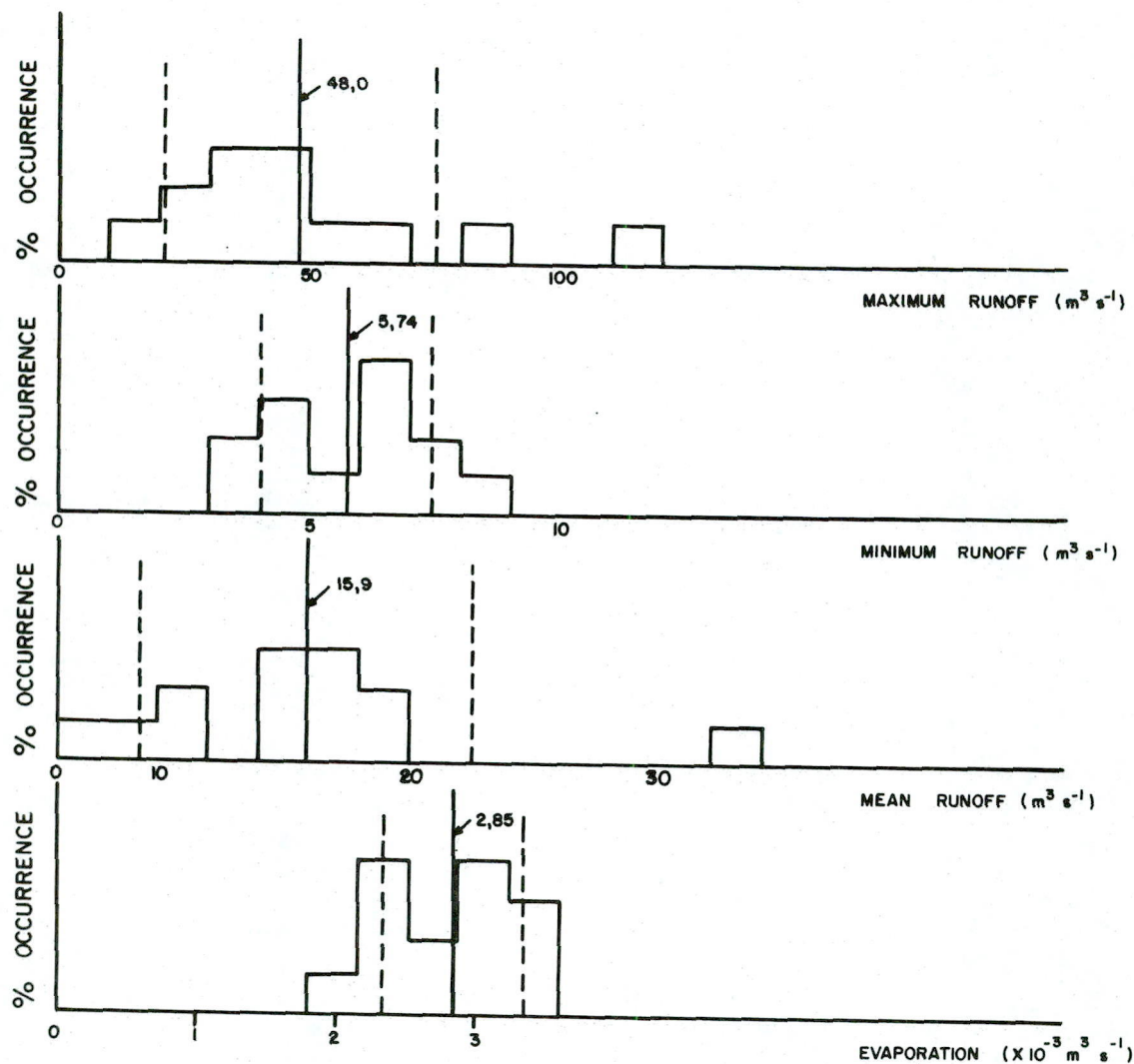
— MEAN
- - - ± ONE STANDARD DEVIATION

TRACED JMO
CHECKED
DATE DEC-80
REF C/SEA 8048

HANGKLIP DAM
HYDROLOGICAL PARAMETERS
FOR JULY

FIGURE
A10

AUGUST



ALL RUNOFF DATA BASED ON DAILY OBSERVATIONS AT G4M07 FOR 1963/64 TO 1975/76

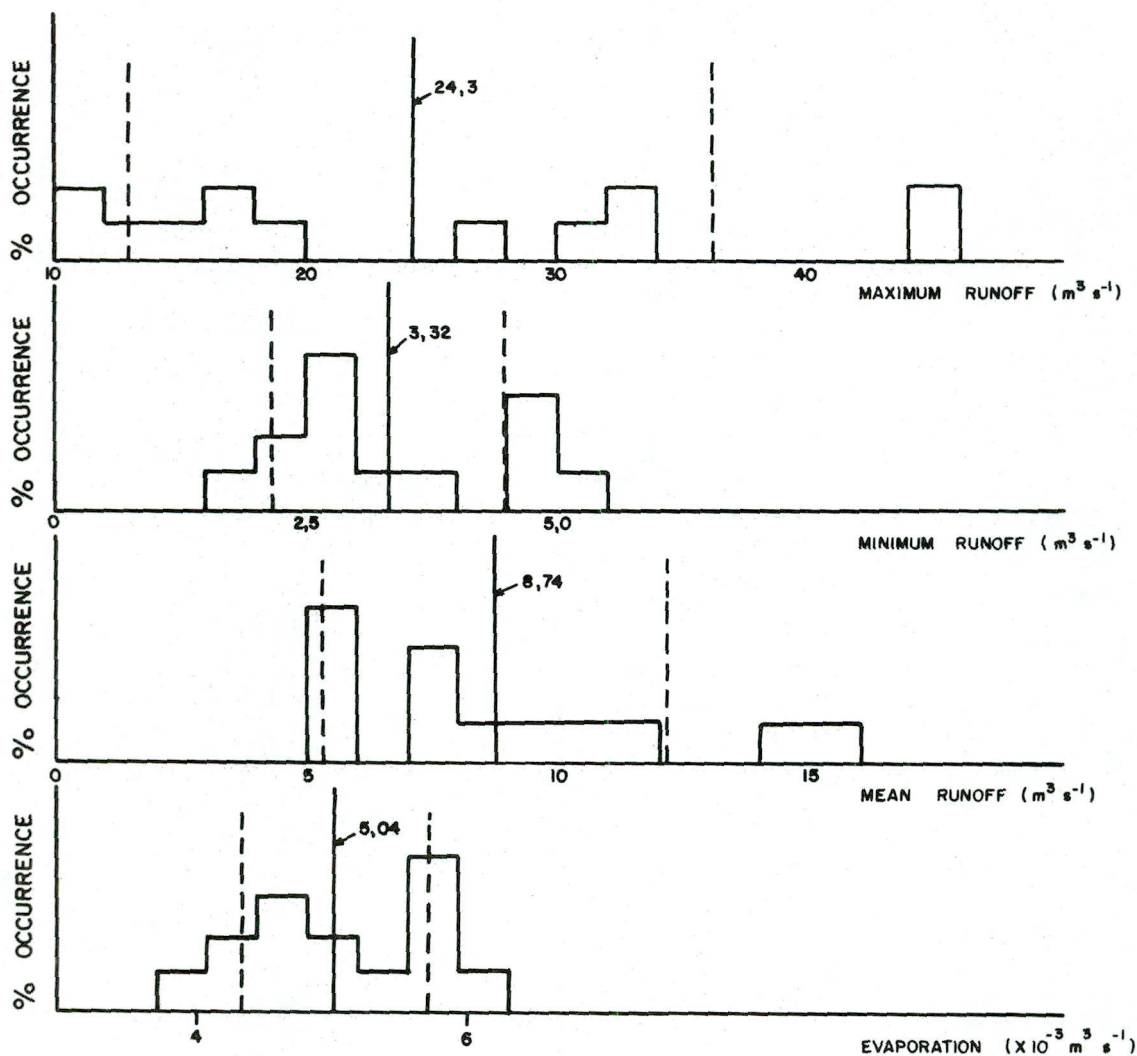
— MEAN
 - - - ± ONE STANDARD DEVIATION

TRACED JMD
 CHECKED
 DATE DEC. '80
 REF C/SEA 8048

HANGKLIP DAM HYDROLOGICAL PARAMETERS FOR AUGUST

FIGURE
 All

SEPTEMBER



ALL RUNOFF DATA BASED ON DAILY OBSERVATIONS AT G4M07 FOR 1963/64 TO 1975/76

—— MEAN
 - - - - ± ONE STANDARD DEVIATION

TRACED JMD
 CHECKED
 DATE DEC. 80
 BY C/SEA 8048

HANGKLIP DAM HYDROLOGICAL PARAMETERS FOR SEPTEMBER

FIGURE
 A12