

THE STORAGE OF WATER IN SAND.

An Investigation of the properties of Natural and Artificial  
Sand Reservoirs and of methods of developing such Reservoirs.

by

O. Wipplinger

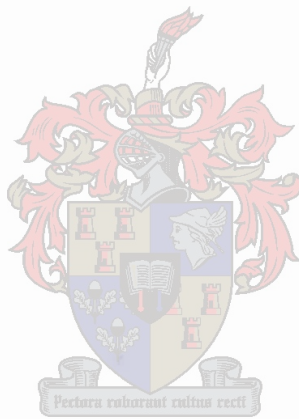




Fig.1. - An old Sand Storage Dam in the Kuiseb River,  
South West Africa.

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CHAPTER 1.  
INTRODUCTION

(1) The Importance of Water Supplies in a Semi-arid Region.

In a semi-arid country water supplies are amongst the most valuable assets. Supplies of fresh water contribute so fundamentally to economic development, that considerable expenditure is justified in improving on natural conditions in this respect.

Permanent water supplies in the form of springs or perennial streams are rare. Normally the development of a semi-arid region can only follow upon the creation of water supplies by artificial means.

A common method of providing water is by means of wells or boreholes, tapping underground supplies. Wells in unconsolidated alluvial deposits frequently yield important supplies for agricultural and urban requirements. Comparatively high rates of extraction are possible; but the volume of the deposits will determine the safe rate of extraction, where a permanent yield is desired. Boreholes in consolidated deposits and crystalline rock usually have smaller yields due to the lower permeability of such formations. In certain regions where geological conditions are unfavourable, water boring is a highly speculative venture and the possibilities of development by this method are limited.

The creation of water supplies by conserving surface runoff is an equally important aid to development. Failure to procure adequate underground water supplies or the weakening of existing sources of supplies are common reasons for embarking on such schemes. Surface runoff can be stored in open dams or reservoirs or it may be fed into underground water bearers from which it is subsequently extracted by pumping from wells or boreholes. An effective solution to a water supply problem is frequently to be found in a combination of surface and underground storage (page 184).

It is desirable to make a close study of storage losses at an early stage; since development, depending as it does on water supplies, will ultimately be limited by the efficiency of storage schemes.

Runoff in a semi-arid region is erratic. Years with practically no runoff at all and series of years with low runoff have to be allowed for (page 20 ). Evaporation from open water is of the order of nine feet per annum and open storage from one good runoff season to the next will therefore be accompanied by very substantial water losses (page 106 ).

## (2) Storage Efficiency in Open Storage Reservoirs.

An analytical study of the storage efficiency in open reservoirs will now be made:-

Definition of terms:-

Service  $T_p$  :- The duration of a rainless period which will result in the dam, originally full, being completely emptied through drawoff and evaporation.

Storage efficiency  $F$  :- The total drawoff during the period of service divided by the total storage capacity of the dam.

The shape of the reservoir will be assumed to be an inverted pyramid,\*

$$\begin{aligned} \text{i.e. } a_r &= c h_s^2 \\ v_s &= \frac{1}{3} c h_s^3 \end{aligned}$$

Where  $h_s$  is the maximum water depth at any particular stage of depletion and  $a_r$  and  $v_s$  the corresponding water surface and storage capacity respectively.

During any time interval  $\delta t$  and corresponding change in water depth  $\delta h_s$ , the volume which evaporates is  $E c h_s^2 \delta t$ , where  $E$  is the depth of evaporation per unit time.

The volume drawn off is  $F V_s \frac{\delta t}{T_D}$  where  $V_s$  is the capacity of the dam when full.

\* In the V-shaped valleys which occur in the central portion of South West Africa, inverted pyramids have been found to represent the shape of the basins with sufficient accuracy (see Figs. 14, 15 and 17 for comparison with actual capacity curves). If other types of topography prevail, such as the box canyons of the lower Fish River Area in South West Africa, special analysis is required.

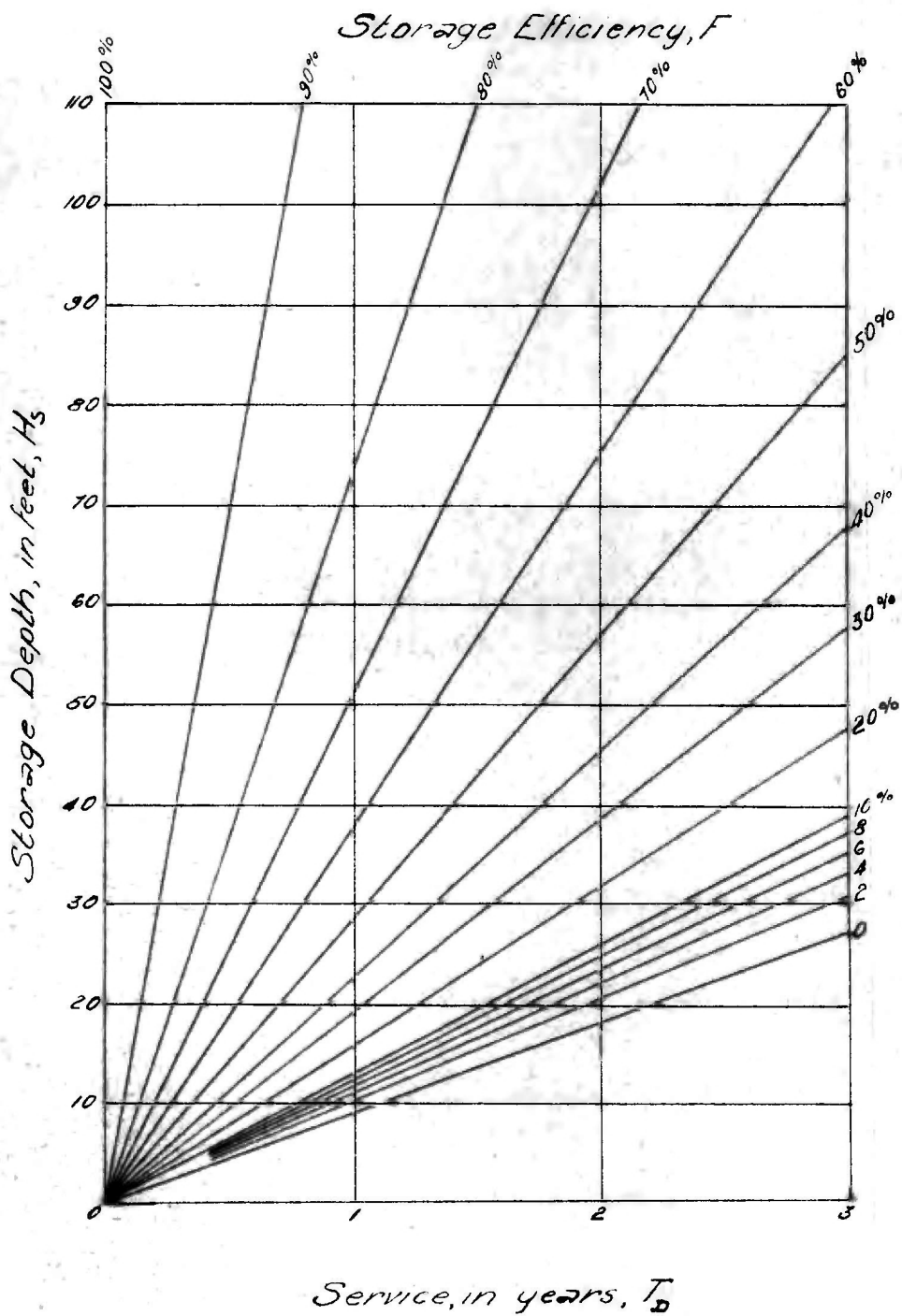


Fig.2.

Storage Efficiency Chart - Open Storage.

(based on 9feet evaporation per annum and formula

$$\sqrt{\frac{3ET_D}{FH_S}} \left(1 - \frac{ET_D}{H_S}\right) = \tan^{-1} \sqrt{\frac{3ET_D}{FH_S}} \quad )$$



According to the chart a dam with 110 feet of storage will yield approximately 59% of its capacity at the dam outlet if drawoff is spread over 3 years.

A dam with 35 feet of storage will only yield 6% at the outlet with drawoff spread over 3 years. With a dam of 10 million cubic feet capacity this percentage will be sufficient for the watering of 400 head of cattle. If, however, a drawoff of 50% is desired the service will be reduced to fifteen months. The dam could then only form part of a permanent water scheme, where it will serve as a means of relieving the strain on ground water supplies or where part of the drawoff can be stored underground, for use when the dam is empty.

In general the means of overcoming evaporation loss in dam schemes are to adopt large storage depths or to store water underground.

Apart from evaporation, silting constitutes an important factor in the planning of storage dams. The problem created by silting is largely financial. Storage capacity destroyed by silting has to be restored, which means additional, ever recurring expenditure. Only dam schemes where benefits derived are sufficient to warrant the cost of making good the effect of silting, can be regarded as permanent assets. Making good the effect of silting can consist of raising the dam wall,\* building a new dam or removing the silt which is deposited. The cost of silt removal is generally considered to be prohibitive, when compared with the value of the storage capacity regained. Where the maintenance of a water supply is a matter of extreme importance, silt removal may have to be considered.

In practice it has been found that sites for open storage dams which will yield a permanent supply and have a reasonable "life" as limited by silting, are difficult to find under semi-arid conditions.

\*Raising the dam wall can restore the capacity, but the depth for equal storage capacity will be reduced and evaporation losses increased.<sup>16</sup>

(3) Construction of Depletion Charts - Open Storage.

The capacity curve  $v_s = \frac{1}{3} c h_s^3$ , the rate of extraction  $Q_A$  and the rate of evaporation  $\bar{E}$  are given. For constructing the depletion curve the time  $t_D$  during which any residual storage  $h_s$  will be depleted must be found (Fig.13).

Let  $h$  and  $t$  be any depth and corresponding residual time during the depletion period; and  $\frac{\delta h}{\delta t}$  the rate of depletion at  $h$  &  $t$ .

$$\text{Volume evaporated in interval } \delta t = -Ech^2 \delta t$$

$$\text{Volume drawn off " " " " } = -Q_A \delta t$$

$$\text{Total depletion " " " " } = -ch^2 \delta h$$

$$\text{i.e. } Q_A \delta t + Ech^2 \delta t = ch^2 \delta h$$

$$\delta t = \frac{ch^2 \delta h}{Q_A + Ech^2}$$

Integrating between the limits  $t = t_D$  to  $t = 0$  and  $h = h_s$  to  $h = 0$

$$\begin{aligned} t_D &= \int_0^{h_s} \frac{ch^2 dh}{Q_A + Ech^2} \\ &= \int_0^{h_s} \frac{h^2 dh}{\frac{Q_A}{c} + Eh^2} \\ &= \int_0^{h_s} \left( \frac{1}{E} - \frac{Q_A}{cE} \frac{1}{\frac{Q_A}{c} + Eh^2} \right) dh \\ &= \frac{h_s}{E} - \frac{Q_A}{cE} \int_0^{h_s} \frac{dh}{\frac{Q_A}{c} + Eh^2} \end{aligned}$$

$$\text{but } \int \frac{dx}{a + bx^2} = \frac{1}{\sqrt{ab}} \tan^{-1} \left( x \sqrt{\frac{b}{a}} \right)$$

$$\begin{aligned} \therefore t_D &= \frac{h_s}{E} - \left[ \frac{Q_A}{cE} \frac{1}{\sqrt{\frac{Q_A}{c} E}} \tan^{-1} \left( \sqrt{\frac{EC}{Q_A}} h \right) \right]_0^{h_s} \\ &= \frac{h_s}{E} - \frac{1}{E} \sqrt{\frac{Q_A}{EC}} \tan^{-1} \left( \sqrt{\frac{EC}{Q_A}} h_s \right) \\ &= \frac{1}{E} \left( h_s - \sqrt{\frac{Q_A}{EC}} \tan^{-1} \left( \sqrt{\frac{EC}{Q_A}} h_s \right) \right) \quad (2) \end{aligned}$$

By substituting  $c = 726$  &  $Q_A = 200,000$ , the constant and draw-off for Otjimahona dam (Fig.14), the following equation for the depletion curve of this dam is obtained:-

$$t_D = \frac{1}{E} \left[ h_s - \sqrt{\frac{200,000}{726E}} \tan^{-1} \left( h_s \sqrt{\frac{726E}{200,000}} \right) \right]$$

$$= \frac{1}{E} \left[ h_s - \sqrt{\frac{276}{E}} \tan^{-1} \left( h_s \sqrt{\frac{E}{276}} \right) \right]$$

$$\text{if } E = 9 \text{ ft} \quad t_D = \frac{1}{9} \left( h_s - 5.53 \tan^{-1} \frac{h_s}{5.53} \right)$$

$$\text{if } E = 8 \text{ ft} \quad t_D = \frac{1}{8} \left( h_s - 5.88 \tan^{-1} \frac{h_s}{5.88} \right)$$

$$\text{if } E = 7 \text{ ft} \quad t_D = \frac{1}{7} \left( h_s - 6.23 \tan^{-1} \frac{h_s}{6.23} \right)$$

Depletion curves calculated by means of these formulae are plotted in Fig.13.

#### (4) Storage Efficiency in Sand Storage Reservoirs.

An interesting storage practice which promises to eliminate the silt problem, and largely also the evaporation problem, consists of storing water in sand filled dams. Sometimes conditions are such, that dams fill with material which is capable of absorbing water from floods and again yielding it to wells or infiltration galleries in sufficient quantity. Several instances of this description are on record.

For instance:-

G.G.Sykes<sup>1</sup> describes sand filled dams which are situated in the semi-arid region of the United States of America. Some of the structures are more than a hundred years old. The water yield is described as "small but dependable".

In 1907 the Government Water Engineer in South West Africa, von Zwergern<sup>2</sup>, submitted a report on water conservation methods. He described the method of conserving water in sand filled dams, which was successfully applied in certain mountainous parts of Germany (Schwaebische Alb) and suggested that if modified to suit local conditions, this method promised to solve the problem of watering places for stock.

To test this method a weir 16.5 feet high was constructed in 1907 in a 40 square mile catchment at the Bacteriological Station, Gamams, near Windhoek. A large portion of the catchment area is situated in the mountainous quartzite country south of Windhoek, and

a particularly clean and comparatively coarse sand is transported. In 1913 the basin was reported to be completely filled with sediments, mostly sand. The yield was estimated\* to be as high as 33% of the 630,000 cubic feet of fill, retained in the dam.

The dam solved the water supply problem of the Bacteriological Station. A well was sunk at the edge of the dam and drains the basin by means of a tunnel passing below original river bed level. As far as can be ascertained the well has yielded a continuous supply for gardening and stock ever since. The water contains iron salts in solution, which are precipitated in the reservoir in which the water is stored after pumping.

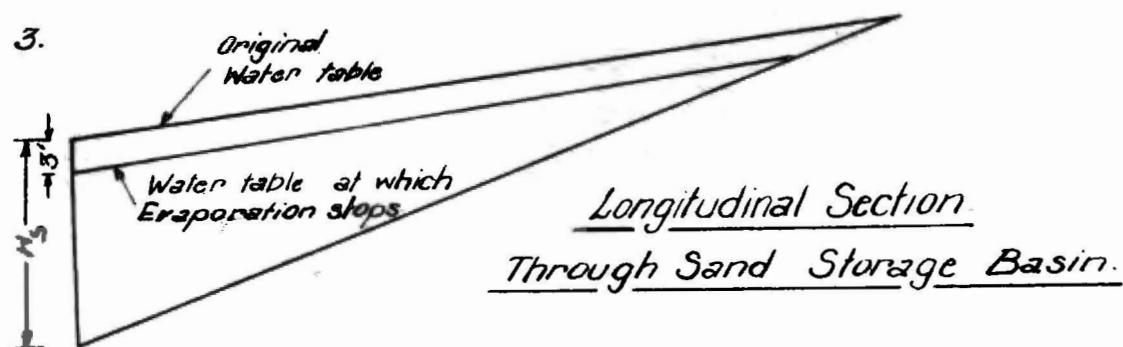
The results achieved at Gamams must be regarded as favourable. The products of weathering of mica schist catchments, for instance, are much finer than those of a quartzite catchment and less favourable results must therefore be expected; but even here success can be achieved by adopting special methods as described in chapters 3 and 4.

It is also of interest that the Drought Investigation Commission<sup>3</sup> which reported on the water problems in South West Africa in 1924 advocated the storage of water in sand filled dams, which are referred to in their report as "sponge dams"

Evaporation losses are reduced but not eliminated by adopting sand storage, as the following analytical study of storage efficiency will demonstrate:-

The water table will be assumed to recede in parallel planes sloping upwards, away from the dam wall; and the saturated volume below water tables to be of the form of an inverted pyramid (page 2)

Figure 3.



\* Estimate made by von Zwergern. Evidence on which this figure was based, not available.

Observations indicate that evaporation from water stored in sand ceases for all practical purposes when the water level drops to about three feet below the surface of the sand. With no drawoff this drop in water level occurs in about three months (page 103).

The following is a detailed analysis of storage efficiency in a sand reservoir:-

Notation:-

$n$  = Porosity of the sand.

$n_1$  = Average water content of the sand at the end of a flood season.  $n_1$  is only equal to  $n$  after exceptionally good flood seasons.

$n_2$  = Specific yield of the sand reservoir with an original water content  $n_1$

$V_s$  = Sand volume of reservoir (cubic feet)

$H_s$  = Maximum depth of sand (feet).

$Q_A$  = Rate of useful drawoff (cubic feet per annum).

$T_D$  = Total time required for depletion of reservoir (years).

$T_E$  = Time required for depletion of top 3 feet (years).

$R_E$  =  $\frac{\text{Volume of top three feet of sand}}{\text{Total volume of sand}}$

$$= 1 - \left( \frac{H_s - 3}{H_s} \right)^3$$

$F$  = Efficiency of Storage =  $\frac{\text{Useful drawoff}}{\text{Volume of water absorbed}}$

Evaporation will be assumed to be confined to the top three feet and to lower the water table by that amount in three months or 0.25 year, if no water is extracted from the reservoir. It is furthermore assumed that the top three feet are completely dried out between rainy seasons.

Analysis:-

The maximum yield of the basin in the theoretical case of instantaneous extraction ( $T_D = 0$  and  $T_E = 0$ ) will be  $V_s n_2$ .

If  $T_D \rightarrow \infty$ , however,  $T_E$  will equal 0.25 years, and the useful yield will be reduced by  $R_E V_s n_2$ .

For any other value of  $T_E$  it will be assumed that the reduction in useful yield due to evaporation will be proportional to  $T_E$ , i.e. equal to  $\frac{T_E}{0.25} R_E V_s n_2$

i.e.  $Q_A T_E =$  useful yield of upper three feet  
 $= R_E V_S n_2 - \frac{T_E}{0.25} R_E V_S n_2$   
 $= R_E V_S n_2 \left( 1 - \frac{T_E}{0.25} \right) \dots\dots\dots(3)$

also  $Q_A(T_D - T_E) =$  useful yield of lower 4-3 feet  
 $= (1 - R_E) V_S n_2 \dots\dots\dots(4)$

$$\frac{T_E}{T_D - T_E} = \frac{R_E \left( 1 - \frac{T_E}{0.25} \right)}{1 - R_E} \quad (\text{equation (3)} \div \text{equation (4)})$$

i.e.  $T_D - T_E = T_E \frac{1 - R_E}{R_E} \left( \frac{1}{1 - \frac{T_E}{0.25}} \right)$

$$= \frac{1 - R_E}{R_E} \frac{1}{\frac{1}{T_E} - 4}$$

$$T_D = T_E + \frac{1 - R_E}{R_E} \frac{1}{\frac{1}{T_E} - 4}$$

$$= T_E + \frac{\left( \frac{H_S - 3}{H_S} \right)^3}{1 - \left( \frac{H_S - 3}{H_S} \right)^3} \frac{1}{\frac{1}{T_E} - 4}$$

$$= T_E + \frac{1}{\left( \frac{H_S - 3}{H_S} \right)^3 - 1} \frac{1}{\frac{1}{T_E} - 4} \dots (5)$$

$F =$  efficiency of storage  
 $= \frac{\text{Total useful drawoff}}{\text{Total volume of water absorbed}}$   
 $= \frac{Q_A T_D}{\text{Total volume of water absorbed.}}$

Note:- The volume of water lost by evaporation is equal to  $R_E V_S n_2$  when there is no drawoff and less than this amount when drawoff occurs. Loss due to evaporation is never greater than  $R_E V_S n_2$ .

Consider

a) The state of the basin before replenishment:-

It will be assumed that the basin was depleted from an original capacity  $V n_1$  by drawoff and evaporation i.e. that the top three feet were completely dry and the basin below this zone contained  $n_1 - n_2$  by volume of retained moisture.

b) Replenishment:-

It will be assumed that a flood season causes replenishment to a water content  $n_1$ . A fraction  $n_1$  will be absorbed in the upper

three feet and a fraction  $n_2$  in the portion of the basin below this zone, to bring the water content throughout the basin to  $n_1$ .

The total volume absorbed will thus be equal to

$$R_E V_S n_1 + (1 - R_E) V_S n_2$$

$$\text{i.e. } F = \frac{Q_A T_D}{R_E V_S n_1 + (1 - R_E) V_S n_2}$$

$$\text{From equation (1) } R_E V_S = \frac{Q_A T_E}{n_2 \left(1 - \frac{T_E}{0.25}\right)}$$

$$\text{" " (2) } (1 - R_E) V_S n_2 = Q_A (T_D - T_E)$$

$$\begin{aligned} F &= \frac{Q_A T_D}{\frac{n_1}{n_2} \frac{Q_A T_E}{1 - \frac{T_E}{0.25}} + Q_A (T_D - T_E)} \\ &= \frac{T_D}{T_D - T_E + \frac{n_1}{n_2} \frac{T_E}{1 - \frac{T_E}{0.25}}} \\ &= \frac{T_D}{T_D - T_E + \frac{n_1}{n_2} \frac{1}{\frac{1}{T_E} - 4}} \quad (6) \end{aligned}$$

Equations (5) and (6) will now be solved for selected values of  $H_S$  and  $T_E$

| $H_S$ | $\frac{1}{\left(\frac{H_S}{H_S-3}\right)^3 - 1}$ | $T_E$ | $\frac{1}{\frac{1}{T_E} - 4}$ |
|-------|--|-------|-------------------------------|
| 10    | 0.52   | .25   | $\infty$                      |
| 20    | 1.59   | .24   | 6.0                           |
| 40    | 3.85   | .23   | 2.86                          |
| 60    | 6.25   | .22   | 1.82                          |
| 80    | 8.3  | .20   | 1.0                           |
| 100   | 10.0   | .18   | .645                          |
|       |  | .14   | .318                          |
|       |  | .10   | .167                          |
|       |  | .05   | .0625                         |
|       |  | .0    | 0                             |

If  $n_1 = 0.4$  and  $n_2 = 0.25$

$$\text{i.e. } \frac{n_1}{n_2} = 1.6$$

$$\text{and } H_S = 100,$$



for  $T_E = 0.14$ for  $T_E = 0.10$ for  $T_E = 0.05$ 

by equation(5):-

$$\begin{aligned} T_D &= T_E + 10 \times 0.318 \\ &= 0.14 + 3.18 \\ &= 3.32 \end{aligned}$$

$$\begin{aligned} T_D &= T_E + 10 \times 0.167 \\ &= 0.10 + 1.67 \\ &= 1.77 \end{aligned}$$

$$\begin{aligned} T_D &= T_E + 10 \times 0.0625 \\ &= 0.05 + 0.625 \\ &= 0.675 \end{aligned}$$

by equation (6):-

$$\begin{aligned} F &= \frac{3.32}{3.18 + 1.6 \times 0.318} \\ &= \frac{3.32}{3.18 + 0.51} \\ &= \frac{3.32}{3.69} \\ &= 0.90 \end{aligned}$$

$$\begin{aligned} F &= \frac{1.77}{1.67 + 1.6 \times 0.167} \\ &= \frac{1.77}{1.67 + 0.27} \\ &= \frac{1.77}{1.94} \\ &= 0.91 \end{aligned}$$

$$\begin{aligned} F &= \frac{0.675}{0.625 + 1.6 \times 0.0625} \\ &= \frac{0.675}{0.625 + 0.10} \\ &= \frac{0.675}{0.725} \\ &= 0.93 \end{aligned}$$

Similar computations were made for  $H_s = 80, 60, 40, 20$  etc.

The results were plotted and curves of equal storage efficiency drawn (Fig. 4 (a) ).

Similarly for  $n_1 = 0.25$  and  $n_2 = 0.10$  i.e.  $\frac{n_1}{n_2} = 2.5$ , the storage efficiencies shown in Fig. 4 (b) were computed and the corresponding curves of equal storage efficiency drawn.

By means of the curves in Figs. 4 (a) and 4 (b) an indication of storage efficiency for any given storage depth and depletion period, with two typical conditions of saturation can readily be obtained.

Fig. 4 (a)  
 $\eta_1 = 0.4$   
 $\eta_2 = 0.25$

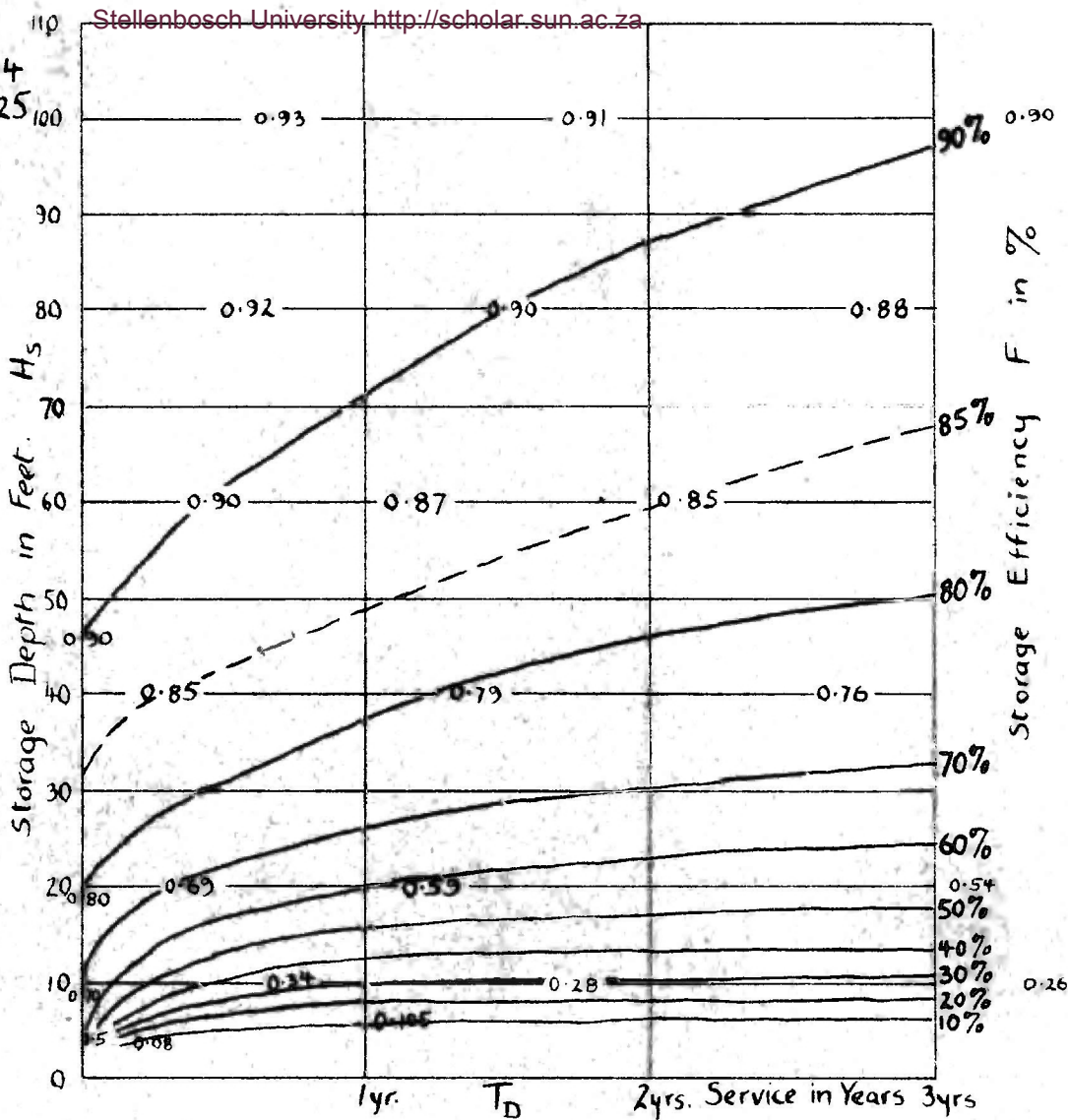
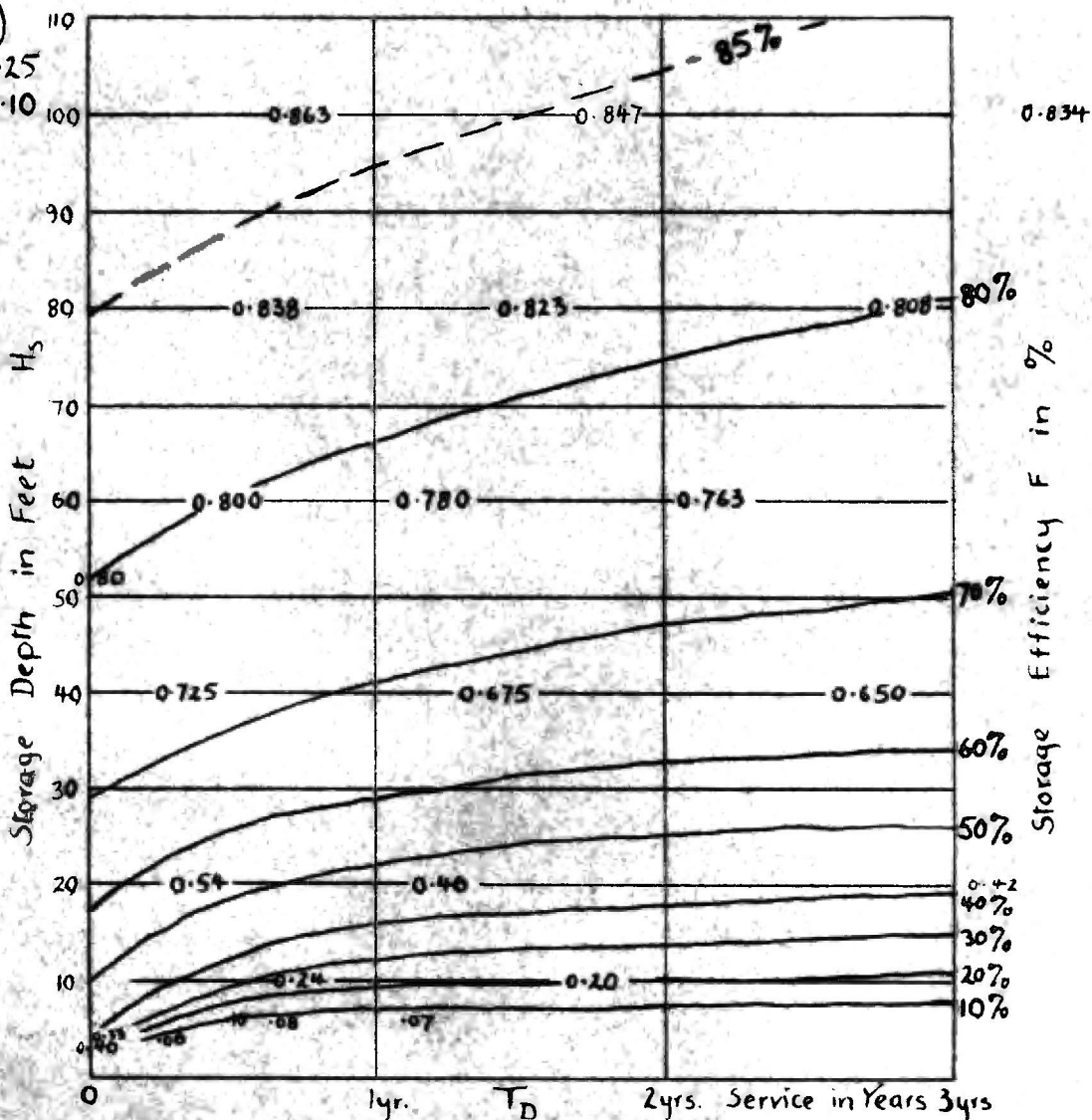


Fig. 4 (b)

$\eta_1 = 0.25$   
 $\eta_2 = 0.10$



Efficiency Charts. Sand Filled Dams.

Figs. 2 and 4 demonstrate one great advantage of storage in sand filled dams over that in open storage dams, namely the comparatively high storage efficiency of the former, even with low dam walls and long service periods. As already stated a further advantage of storage in sand filled dams is to be found in their immunity to silting.

Considering a practical example, a sand storage dam 35 feet deep will yield between 71% and 60% of the absorbed water with a service period of 3 years, compared with 6% in an open storage dam of equal depth (Figs. 2 and 4 ). If the total volume of sand is 10 million cubic feet, the following volumes of water will be absorbed and drawn off respectively:-

|                 | a) $n_1 = 0.40$ $n_2 = 0.25$  | b) $n_1 = 0.25$ $n_2 = 0.10$   |
|-----------------|---|--|
| Volume absorbed | $R_E V_S n_1 + (1 - R_E) V_S n_2$<br>$= V_S (.24 \times .4 + .76 \times .25)^*$<br>$= V_S (.096 + .190)$<br>$= .286 V_S$<br>$= 2.86 \text{ million cu.ft.}$ | $R_E V_S n_1 + (1 - R_E) V_S n_2$<br>$= V_S (.24 \times .25 + .76 \times .10)^*$<br>$= V_S (.060 + .076)$<br>$= .136 V_S$<br>$= 1.36 \text{ million cu.ft.}$ |
| Drawoff         | 71% of 2.86<br>$= 2.03 \text{ million cu.ft.}$  | 60% of 1.36<br>$= 0.82 \text{ million cu.ft.}$   |

\* For  $H_S = 35$      $1 - R_E = 0.76$     and     $R_E = 0.24$

An open storage dam of equal size will require 10 million cubic feet of water for replenishment and will only yield 0.6 million cubic feet during a subsequent service period of three years (page 5 ).

5) Construction of Depletion Charts - Sand Storage

The capacity curve  $v_s = \frac{1}{3} c h_s^3$  the rate of extraction  $Q_A$  and the specific yield  $n_2$  are given or estimated.

It follows from equation (3) that the time taken for depletion of the top three feet is given by

$$T_E = \frac{R_E V_S n_2}{Q_A + 4 R_E V_S n_2}$$

$$= \frac{1}{\frac{Q_A}{R_E V_S n_2} + 4}$$

Below this level the time  $t_D$  during which any residual storage  $V_S$  is depleted is given by

$$t_D = \frac{V_S n_2}{Q_A} = \frac{1}{3} \frac{n_2}{Q_A} c h_S^3$$

(6) Examples of Open and Sand Storage Reservoirs -  
Observed and computed Depletion.



Fig.5.

Avis Dam. An open storage reservoir with a capacity of 129 million cubic feet and a depth above outlet of 32 feet, forming part of the town water supply of Windhoek. When the reservoir is empty the town is supplied from boreholes.

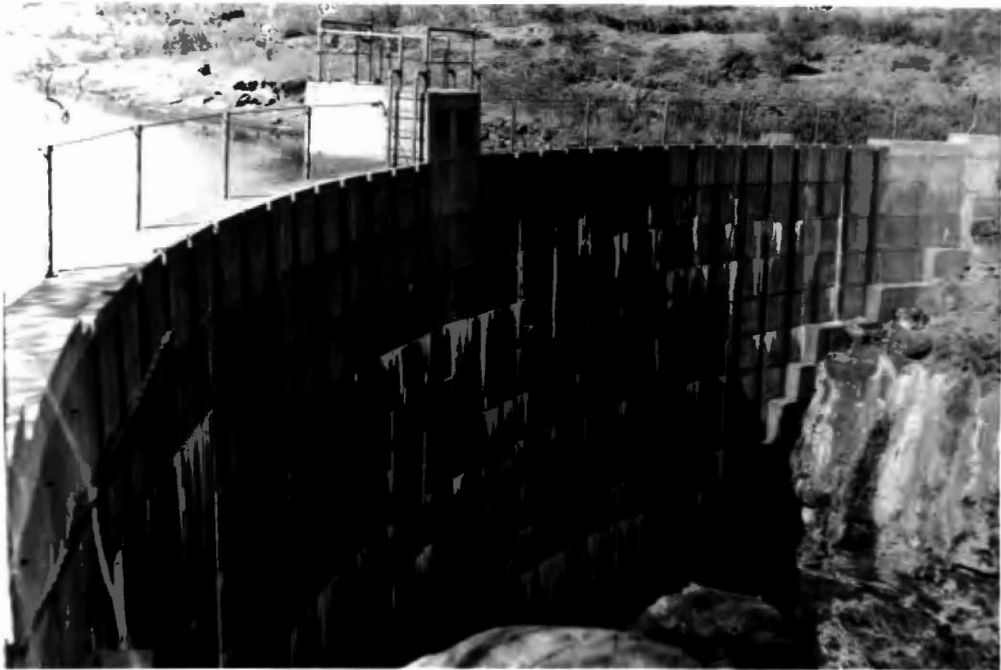


Fig.6.

Otjimahona Dam. An open storage reservoir with a capacity of 10 million cubic feet and a depth of 35 feet, constituting a water supply for cattle.



Figs. 7 & 8. Bulskop Dam. Third and fourth stage of a sand storage dam. The third stage is shown after it reached maturity (fully sanded up). The fourth stage, shortly after construction. The purpose of the siphon is explained in chapter 5.



Fig.9

First stage of the sand storage dam at Aukeigas, with the "sanding up" process not yet complete. The infiltration well in the early stages was closed by means of a timber cover at river bed level. In later stages the design shown in Fig.51 was adopted.



Fig.10.

Sand tongue entering the fifth stage of the sand storage dam at Aukeigas.

Fig.11.

Sand storage dam at Aukeigas. Fifth stage. Only 400 feet of the dam basin nearest the dam wall are visible. The upper 1200 feet are round the bend to the right.

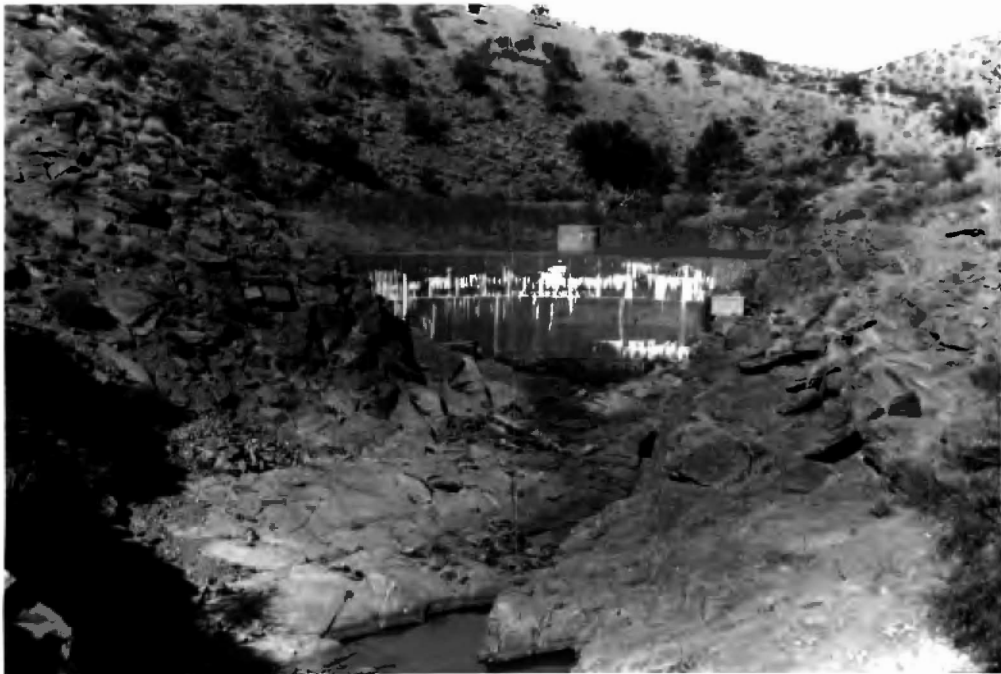




Figure 12.

Depletion Record Avis Dam

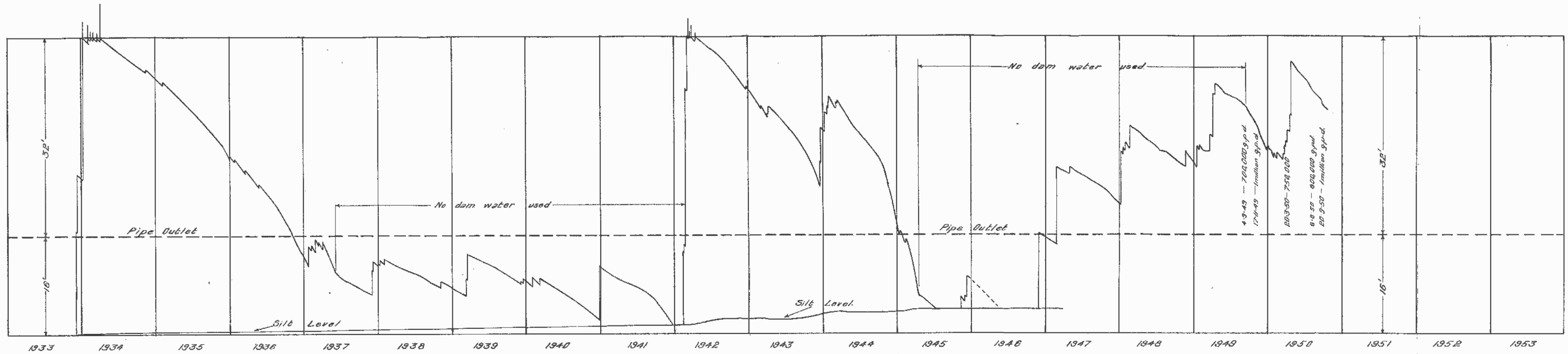


Figure 13.

Depletion Record Obijmahona Dam

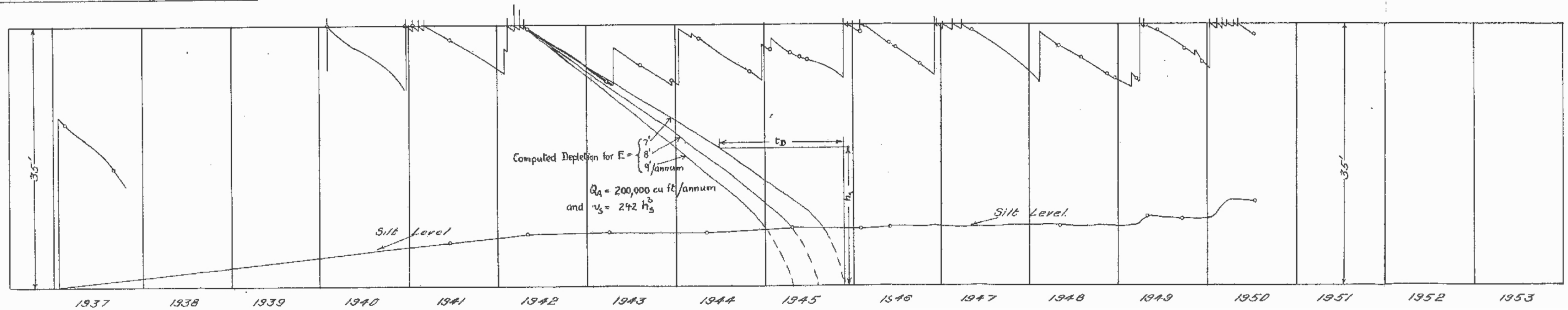
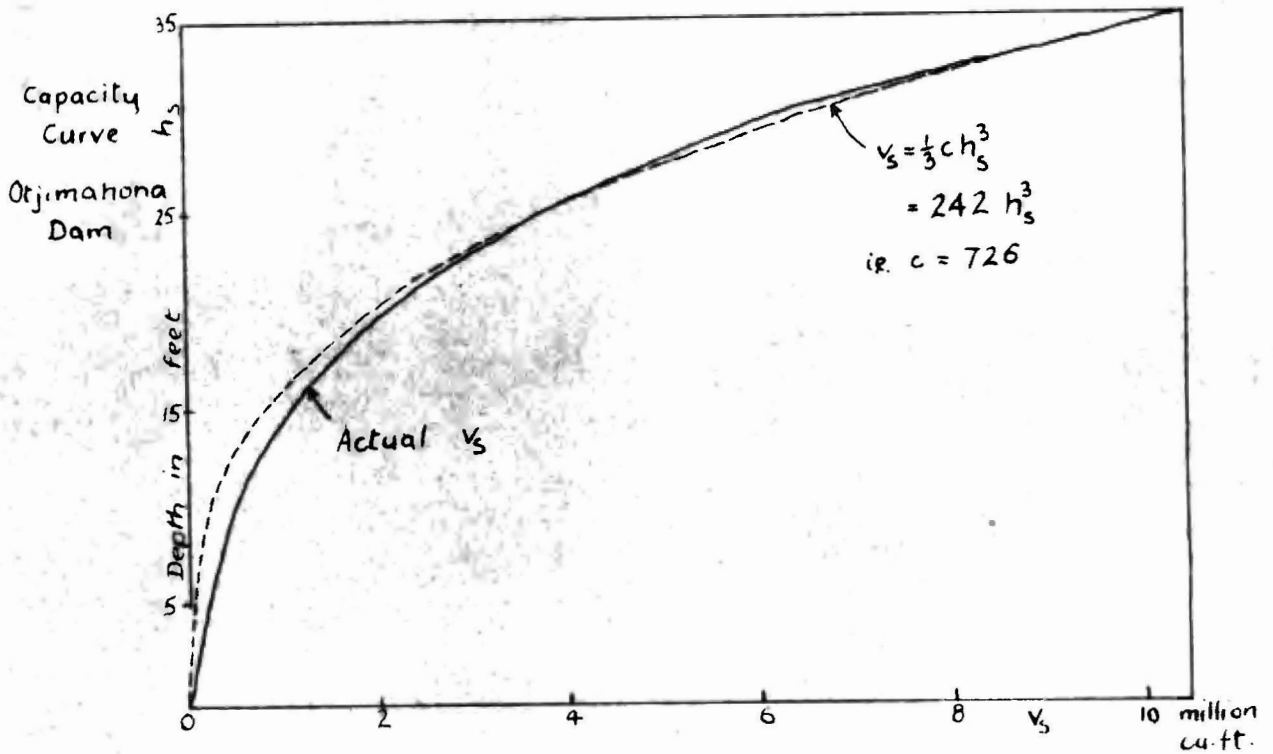


Fig. 14



Estimated consumption (400 head of cattle) = 200,000 cu.ft. per annum

Computations

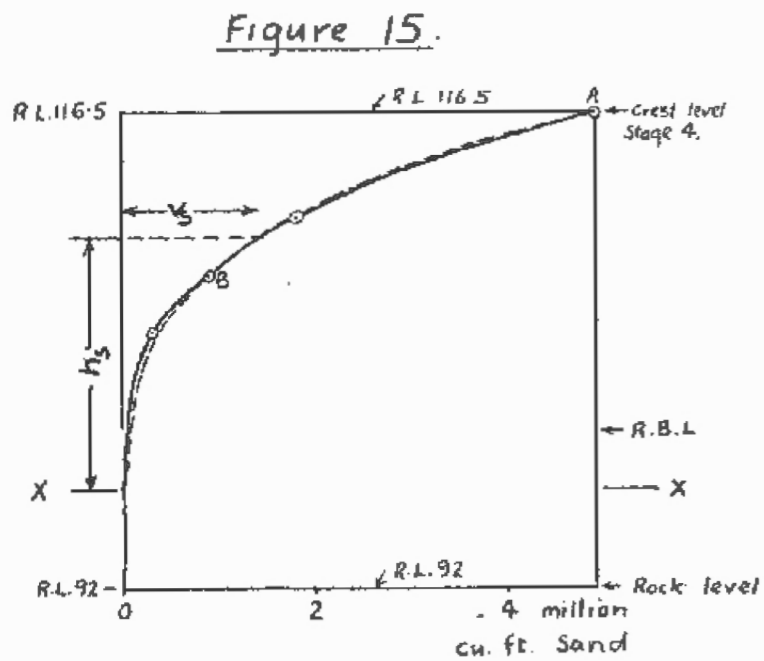
| $h_s$ | $E$ | $t_D = \frac{1}{E} \left( h_s - \sqrt{\frac{276}{E}} \tan^{-1} \left( \sqrt{\frac{E}{276}} h_s \right) \right)$<br>(equation 2.)        |
|-------|-----|---|
| 10'   | 9'  | $\frac{1}{9} (10 - 5.53 \tan^{-1} \frac{10}{5.53}) = \frac{1}{9} (10 - 5.53 \times 61.1 \times \frac{1}{57.3}) = \frac{4.1}{9} = 0.46$  |
|       | 8'  | $\frac{1}{8} (10 - 5.88 \tan^{-1} \frac{10}{5.88}) = \frac{1}{8} (10 - 5.88 \times 59.6 \times \frac{1}{57.3}) = \frac{3.9}{8} = 0.49$  |
|       | 7'  | $\frac{1}{7} (10 - 6.28 \tan^{-1} \frac{10}{6.28}) = \frac{1}{7} (10 - 6.28 \times 57.9 \times \frac{1}{57.3}) = \frac{3.7}{7} = 0.53$  |
| 20'   | 9'  | $\frac{1}{9} (20 - 5.53 \tan^{-1} \frac{20}{5.53}) = \frac{1}{9} (20 - 5.53 \times 74.5 \times \frac{1}{57.3}) = \frac{12.8}{9} = 1.43$ |
|       | 8'  | $\frac{1}{8} (20 - 5.88 \tan^{-1} \frac{20}{5.88}) = \frac{1}{8} (20 - 5.88 \times 73.6 \times \frac{1}{57.3}) = \frac{12.4}{8} = 1.55$ |
|       | 7'  | $\frac{1}{7} (20 - 6.28 \tan^{-1} \frac{20}{6.28}) = \frac{1}{7} (20 - 6.28 \times 72.6 \times \frac{1}{57.3}) = \frac{12.0}{7} = 1.72$ |
| 30'   | 9'  | $\frac{1}{9} (30 - 5.53 \tan^{-1} \frac{30}{5.53}) = \frac{1}{9} (30 - 5.53 \times 79.6 \times \frac{1}{57.3}) = \frac{22.3}{9} = 2.48$ |
|       | 8'  | $\frac{1}{8} (30 - 5.88 \tan^{-1} \frac{30}{5.88}) = \frac{1}{8} (30 - 5.88 \times 78.9 \times \frac{1}{57.3}) = \frac{21.9}{8} = 2.74$ |
|       | 7'  | $\frac{1}{7} (30 - 6.28 \tan^{-1} \frac{30}{6.28}) = \frac{1}{7} (30 - 6.28 \times 78.2 \times \frac{1}{57.3}) = \frac{21.4}{7} = 3.06$ |
| 35'   | 9'  | $\frac{1}{9} (35 - 5.53 \tan^{-1} \frac{35}{5.53}) = \frac{1}{9} (35 - 5.53 \times 81.0 \times \frac{1}{57.3}) = \frac{27.2}{9} = 3.02$ |
|       | 8'  | $\frac{1}{8} (35 - 5.88 \tan^{-1} \frac{35}{5.88}) = \frac{1}{8} (35 - 5.88 \times 80.5 \times \frac{1}{57.3}) = \frac{26.7}{8} = 3.34$ |
|       | 7'  | $\frac{1}{7} (35 - 6.28 \tan^{-1} \frac{35}{6.28}) = \frac{1}{7} (35 - 6.28 \times 79.8 \times \frac{1}{57.3}) = \frac{26.3}{7} = 3.73$ |

Depletion curves in accordance with these computations are plotted in Fig. 12.

① Sand storage capacity up to planes sloping 1:400 away from dam crest.

| Stage No. | Crest Level        | Total sand Capacity in Dam (million cu. ft.) |
|-----------|--------------------|--|
| 1         | 5 ft. above R.B.L. | 0.3  |
| 2         | 8 ft. " "          | 0.9  |
| 3         | 11 ft. " "         | 1.8  |
| 4         | 16'6" " "          | 4.9  |

② Equivalent Capacity Curve



Actual capacity, full line.  
Formula,  $V_s = 660 h_s^3$ , dotted line  
Note: XX is datum selected to fit cubic parabola through A and B.  
Capacity below XX neglected in depletion computations.  
If OO were selected as datum there would be considerable discrepancy between actual curve and cubic parabola.

③ Depletion Analysis:

$$V_s = 660 h_s^3$$

Estimated annual drawoff  $Q_A = 0.12$  million cu. ft.

From equation (4),  $T_D - T_E = \frac{(1 - R_E) V_s n_2}{Q_A}$

From equation (3),  $T_E = \frac{R_E V_s n_2}{Q_A + 4 R_E V_s n_2} = \frac{1}{\frac{Q_A}{R_E V_s n_2} + 4}$

| Stage No. | $H_s$ | $V_s$ | $R_E = 1 - \left(\frac{H_s - 3}{H_s}\right)^3$ | $T_D - T_E$ | $\frac{Q_A}{R_E V_s n_2}$ |
|-----------|-------|-------|--|-------------|---------------------------|
| 1         | 8     | .34   | .755   | .695 $n_2$  | .467 $\frac{1}{n_2}$      |
| 2         | 11    | .88   | .615   | 2.82 $n_2$  | .222 $\frac{1}{n_2}$      |
| 3         | 14    | 1.82  | .514   | 7.37 $n_2$  | .128 $\frac{1}{n_2}$      |
| 4         | 19.5  | 4.90  | .394   | 24.8 $n_2$  | .062 $\frac{1}{n_2}$      |

Ⓐ  $n_2 = .25$

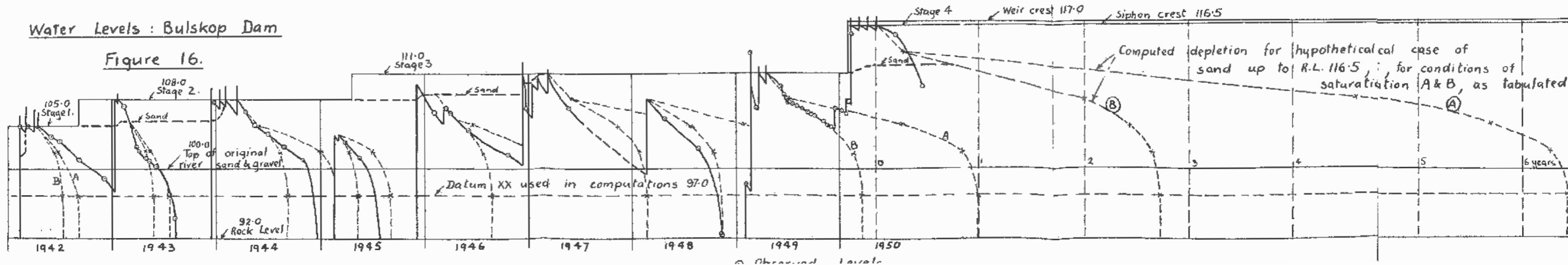
| Stage No. | $T_D - T_E$ | $\frac{Q_A}{R_E V_s n_2}$ | $T_E$ | $T_D$ |
|-----------|-------------|---------------------------|-------|-------|
| 1         | .17         | 1.868                     | .17   | .34   |
| 2         | .70         | .888                      | .20   | .90   |
| 3         | 1.84        | .512                      | .22   | 2.06  |
| 4         | 6.02        | .248                      | .24   | 6.26  |

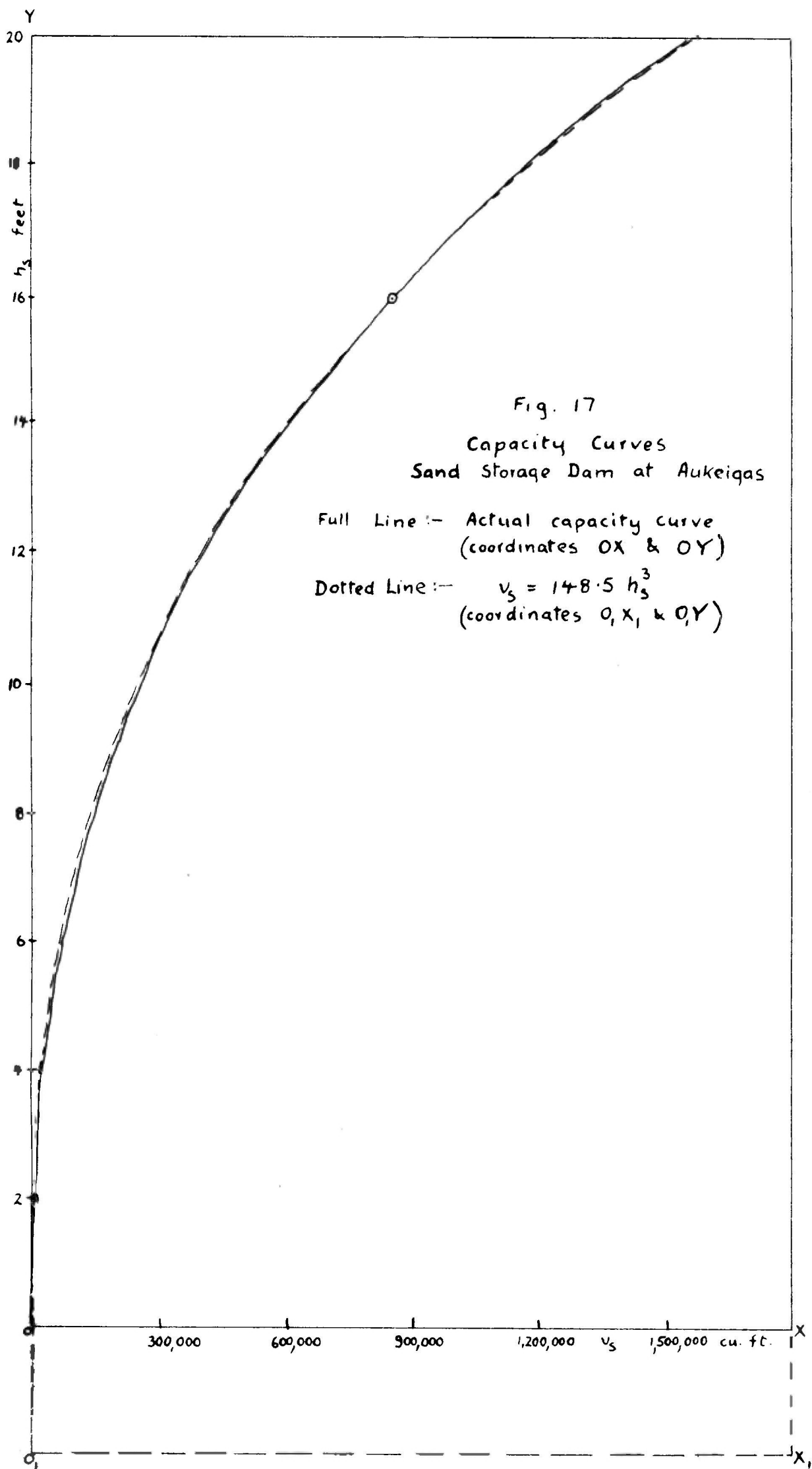
Ⓑ  $n_2 = .10$

| Stage No. | $T_D - T_E$ | $\frac{Q_A}{R_E V_s n_2}$ | $T_E$ | $T_D$ |
|-----------|-------------|---------------------------|-------|-------|
| 1         | .07         | 4.67                      | .11   | .18   |
| 2         | .28         | 2.22                      | .16   | .44   |
| 3         | .74         | 1.28                      | .19   | .93   |
| 4         | 2.48        | 0.62                      | .22   | 2.70  |

Water Levels: Bulskop Dam

Figure 16.





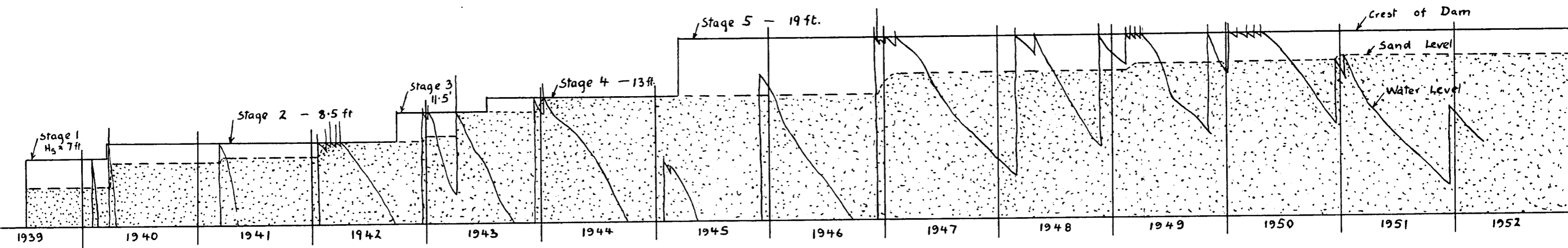


Fig. 18 — Depletion Curve — Sand Storage Dam at Aukeygas.  
Vert. Scale 1" = 10 feet — Hor. Scale 1/2" = 1 year

(7) Outline of Available Information and of the Additional Research undertaken by the Author.

It has already been indicated that the storage of water in sand filled dams is an old practice in some countries. The illustration, Fig. 1, shows an old sand storage dam in South West Africa. A number of examples of this nature are to be found in the Territory. The principle of stage construction to minimise the deposition of fine silt (see Chapter 4) was known to engineers and farmers in South West Africa; but the author came across no instance where it had been systematically applied. Van Reenen<sup>3</sup> does not mention stage construction in his discussion of "sponge dams".

The need for systematic observations and experiments, accompanied by the necessary analytical study was apparent. In this connection the author has undertaken the following work which is described in detail in the appropriate chapters of this treatise:-

a) Construction of stage construction dams at Aukeigas and Bulskop, followed by observations of sedimentation and water yield. In the dam at Aukeigas an exceptionally large stage was included to obtain data for estimating the maximum permissible height of stages under given conditions. In Bulskop Dam a siphon was installed in the final stage to obtain a practical test of this method of improving the effectiveness of a sand storage dam.

b) Analysis of the hydrographic data collected at the Nubuamis hydrographic station and Avis Dam near Windhoek. Construction of river gauges at Goreangab, Kranzplatz and other sites and systematic analysis of the records to obtain, inter alia, basic information for the design of sand storage dams.

c) Model experiments to determine the principles governing sedimentation in sand storage dams with and without siphons.

- d) Tests and experiments to determine the properties of sand deposits and the rate of evaporation of water stored in such deposits.
- e) Mathematical analysis of the depletion of open and sand storage reservoirs and comparison with actual depletion curves.
- f) Trial boreholes in the natural reservoirs at Omaruru, Swakopmund, Okombahe, etc., to determine the extent and properties of the water bearing deposits. Proposals for the elimination of brack water from the natural sand reservoir at Swakopmund, complete with the analyses on which the scheme is based.
- h) Investigation of the lower Fish River area in South West Africa and proposals to construct sand storage dams in that area.

#### (8) Acknowledgements

The author in the first instance wishes to acknowledge the interest shown by the South West Africa Administration in the development of subsurface storage. The Administration provided funds for works such as the dams at Aukeigas and Bulskop, the river gauges, the experimental ground water improvement scheme at Swakopmund, etc. Of equal importance in this connection was the interest shown and the assistance and encouragement given by the officers of the Administration; and the encouragement and criticism given by Professor R.Truter, Dr. D.F.Kokot and Mr. J.Gilmore in their capacity as an Examination Board of the University of Stellenbosch.

CHAPTER 2.BASIC DATA.(1) Catchment Yields.

The amount of water which a catchment will yield will depend on the rainfall and the nature of the catchment area. An examination of the records of runoff gauging stations will give a general picture of what may be expected of catchments in the region in which the stations are situated.

The following is a summary of runoff observed at four river gauges in the central mica schist area of South West Africa:-

Runoff in inches at four gauging stations

| Season                             | Swakop River<br>near<br>Okahandja | Gamams III | Gamams II | Gamams I |
|------------------------------------|-----------------------------------|------------|-----------|----------|
| 1942/43                            |                                   |            |           | 0.649    |
| 1943/44                            |                                   |            | 0.605     | 0.940    |
| 1944/45                            |                                   |            | 0.011     | 0.023    |
| 1945/46                            |                                   |            | 0.436     | 0.573    |
| 1946/47                            | 0.405                             | 0.531      | 0.455     | 0.820    |
| 1947/48                            | 0.121                             | 0.823      | 0.572     | 0.560    |
| 1948/49                            | 0.378                             | 1.280      | 0.901     | 0.835    |
| 1949/50                            | 0.275                             | 1.266      | 1.160     | 1.229    |
| 1950/51                            | 0.025                             | 0.032      | 0.019     | 0.037    |
| 1951/52                            | 0.049                             | 0.108      | 0.064     | 0.083    |
| Average<br>last six<br>seasons     | 0.209                             | 0.673      | 0.528     | 0.594    |
| Size of<br>catchment<br>(sq.miles) | 1156                              | 168        | 115       | 53       |



The gauging station, Gamams I, is situated in the Gamams River approximately eight miles north-west of Windhoek. Gamams II is situated below the confluence of the Gamams and the Aretareigas Rivers and Gamams III records the combined runoff of the Gamams, Aretareigas and Aukeigas Rivers.

The average annual rainfall in the Swakop catchment, according to the available isohyetal map is approximately 16 inches and in the catchments near Windhoek approximately 15 inches. In the catchments near Windhoek, mountainous and hilly country predominates, in the Swakop catchment only half the area has these characteristics, whereas the other half is inclined to be flat with appreciable soil cover.

It will be seen that there are considerable differences in average annual runoff. Estimates of runoff based on an examination of catchment characteristics and rainfall records alone, can be very misleading.

## (2) Maximum Probable Floods.

The rainfall throughout the Territory of South West Africa is of the continental, thunderstorm type; and floods of high intensity occur even in areas of low annual rainfall.

The table below and Fig.19 show how observed flood intensities approach the intensities given by G.B.Williams' formulae  $Q_1 = 1900 A^{0.75}$  and  $Q_2 = 3600 A^{0.45}$  for the Rocky Mountains Region of the United States of America<sup>11</sup>. (The formulae are for catchment areas below and above 10 square miles respectively). G.B.Williams comments as follows on flood intensities in the Rocky Mountains Region:- "In spite of the fact that the rainfall on these mountains is very scanty (only on a very small area does the mean reach 30 inches and in parts it is

less than 10 inches), a number of floods of high intensity have been recorded." In South West Africa the average annual rainfall varies from practically nothing in the desert region to 25 inches in small areas in the north of the Territory. Floods of high intensity have been recorded in several catchments not necessarily in the higher rainfall areas, as the following table will show:-

| Name of catchment            | Size (sq. miles) | Average Annual Rainfall (inches) | Observed Flood (cusecs) | Date of Occurrence             | Remarks  | Q in accordance with G.B. Williams' formula. (cusecs) |
|------------------------------|------------------|----------------------------------|-------------------------|--------------------------------|--|---|
| Nubua-mis Dam                | 1                | 15                               | 1,500<br>1,800          | 20.3.<br>1934<br>1.3.<br>1942  | - a)<br>Max.1931 <sup>a)</sup><br>to 1952            | 1,900   |
| Avis Dam                     | 40               | 15                               | 24,000                  | 1933/34                        | Max.1933 <sup>b)</sup><br>to 1952                    | 19,000  |
| Zamna-rib Dam                | 50               | 6                                | 15,000                  | 13.1.<br>1944                  | Max.1910 <sup>b)</sup><br>to 1944                    | 20,900  |
| Riet River                   | 70               | 6                                | 20,800                  | 13.1.<br>1944                  | - <sup>c)</sup>                                      | 24,400  |
| Voigts-grund Dam             | 148              | 6                                | 28,400                  | 1933/<br>1934                  | Max 1914 <sup>b)</sup><br>to 1934                    | 34,000  |
| Swakop River near Oka-handja | 1160             | 16                               | 30,000<br>70,000        | 1941/<br>1942<br>1930/<br>1931 | - <sup>c)</sup><br>Max 1930 <sup>d)</sup><br>to 1952 | 86,300  |

| Name of catchment        | Size (sq. miles) | Average Annual Rain-fall (Inches) | Observed Flood (cusecs) | Date of Occurrence | Remarks                    | in accordance with G.B. Williams' formula. (cusecs) |
|--------------------------|------------------|-----------------------------------|-------------------------|--------------------|----------------------------|---|
| Fish river at Hardap     | 5000             | 7                                 | 67,000                  | 1949/<br>1950      | - c)                       | 166,000   |
| Fish River at Kranzplatz | 5500             | 7                                 | 54,000                  | 1943/<br>1944      | - c)                       | 174,000   |
|                          |                  |                                   | 54,000                  | 1933/<br>1934      | - e)                       |   |
| Fish River at Gibeon     | 5550             | 7                                 | 100,000                 | 1922/<br>1923      | - f)                       | 176,000   |
|                          |                  |                                   | 112,000                 | 20.1.<br>1909      | g)<br>Max. 1908<br>to 1952 |   |
| Swakop River at Nudis    | 6000             | 12                                | 129,000                 | 1941/<br>1942      | - h)                       | 180,000   |
|                          |                  |                                   | 166,000                 | 1933/<br>1934      | h)<br>Max 1922<br>to 1952  |   |
|                          |                  |                                   | 90,000                  | 1922/23            | - h)                       |   |

a) Computed from automatic water level recorder data.

b) Flood marks in dam spillway. Approximate allowance made for absorption by storage capacity in the case of Avis Dam. Spillway discharge only in Zammarib Dam.

c) Flood marks on river bank.

d) Deduced from a flood report by road foreman.

e) Automatic water level recorder at gauging station.

f) High water mark painted on steps at school property, Gibeon.

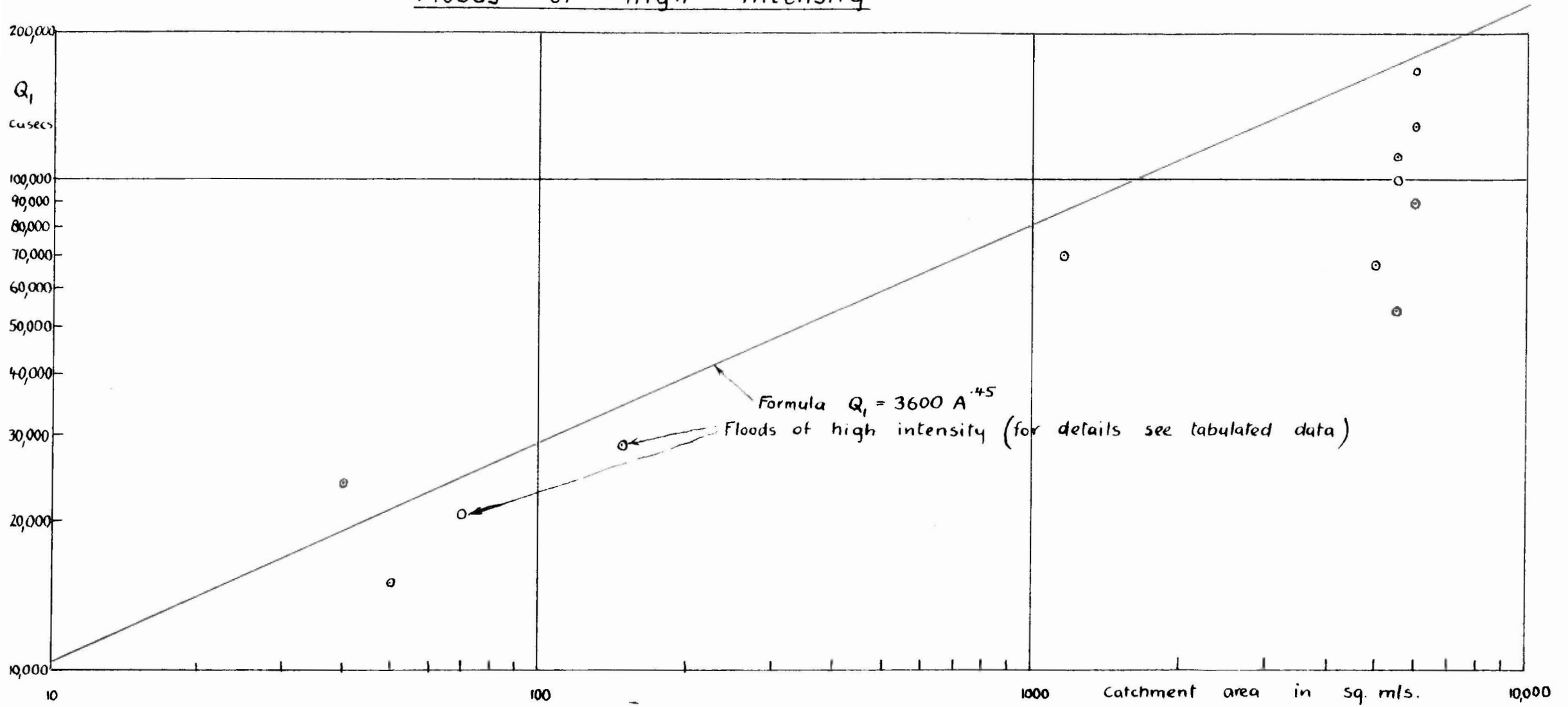
g) Observation and computation by P. Range<sup>18</sup>.

h) Observation and computation by R. Seydel<sup>17</sup>.

Until further information becomes available it is suggested that G.B. Williams' formulae be adopted for catchments in

Fig. 19.

Floods of High Intensity



South West Africa, regardless of the average annual rainfall of the area concerned. A reduction factor should, however, be applied where catchments are extremely absorbant (sand-veld, dolomite, etc.).

### (3) Detailed Study of Flood Intensities.

In investigating problems of sediment transportation special attention will be given to the relation between intensity of flow and transporting capacity of a stream. Floods of different peak intensity will have different transporting capacity. For this reason the records of the four river gauges in the central mica schist area will be considered in greater detail. The following tables give the intensity and yield of recorded floods, 1) in chronological order and 2) in order of magnitude of the peak intensity. From the latter classification the frequency of occurrence of floods with a peak intensity equal to or greater than any particular intensity  $Q$  or relative intensity  $Q/Q_1$  can be read, as well as the fraction of the total runoff  $F_R$  produced by floods with a peak intensity equal to or greater than any particular relative intensity  $Q/Q_1$ . Figs. 20 and 21 give this information in a convenient form.

Definitions:-

$Q$  = peak intensity of a flood.

$Q_1$  = maximum probable flood.

$Q/Q_1$  = relative intensity of a flood.

$F_R$  = fraction of total runoff produced by floods of relative intensity equal to or greater than  $Q/Q_1$ .

Swakop River - Recorded Floods in Chronological Order.

| Season  | Runoff No. | Amount of flow<br>(million cu. ft.) | Peak intensity<br>(cusecs) |
|---------|------------|-------------------------------------|----------------------------|
| 1946/47 | 1          | 1.30                                | 50                         |
|         | 2          | 14.58                               | 1750                       |
|         | 3          | 2.40                                | 275                        |
|         | 4          | 28.40                               | 1000                       |
|         | 5          | 53.10                               | 2600                       |
|         | 6          | 81.56                               | 2500                       |
|         | 7          | 39.14                               | 1950                       |
|         | 8          | 122.00                              | 5800                       |
|         | 9          | 73.70                               | 3700                       |
|         | 10         | 102.10                              | 3400                       |
|         | 11         | 7.80                                | 300                        |
|         | 12         | 87.70                               | 4700                       |
|         | 13         | 8.90                                | 300                        |
|         | 14         | 4.60                                | 200                        |
|         | 15         | 16.70                               | 500                        |
|         | 16         | 50.20                               | 2550                       |
|         | 17         | 28.20                               | 1675                       |
|         | 18         | 20.00                               | 1025                       |
|         | 19         | 145.10                              | 2250                       |
|         | 20         | <u>200.20</u>                       | 10,400                     |
|         |            | 1087.68                             |                            |

Swakop River - Recorded Floods in Chronological Order.

| Season  | Runoff No | Amount of flow<br>(million cu.ft) | Peak intensity<br>(cusecs) |
|---------|-----------|-----------------------------------|----------------------------|
| 1947/48 | 1         | 1.82                              | 450                        |
|         | 2         | 4.79                              | 300                        |
|         | 3         | 46.65                             | 3650                       |
|         | 4         | 108.89                            | 3900                       |
|         | 5         | 86.77                             | 1800                       |
|         | 6         | 10.03                             | 600                        |
|         | 7         | 5.97                              | 200                        |
|         | 8         | 3.93                              | 400                        |
|         | 9         | 5.46                              | <b>750</b>                 |
|         | 10        | 3.16                              | 100                        |
|         | 11        | 19.20                             | 1200                       |
|         | 12        | <u>29.10</u>                      | 1225                       |
|         |           | 325.77                            |                            |

Swakop River - Recorded Floods in Chronological Order.

| Season  | Runoff No. | Amount of flow<br>(million cu.ft) | Peak intensity.<br>(cusecs) |
|---------|------------|-----------------------------------|-----------------------------|
| 1948/49 | 1          | 6.54                              | 600                         |
|         | 2          | 12.42                             | 900                         |
|         | 3          | 27.78                             | 1900                        |
|         | 4          | 10.68                             | 1100                        |
|         | 5          | 56.25                             | 2200                        |
|         | 6          | 1.64                              | 100                         |
|         | 7          | 5.01                              | 550                         |
|         | 8          | 5.67                              | 700                         |
|         | 9          | 22.67                             | 1250                        |
|         | 10         | 144.97                            | 3300                        |
|         | 11         | 114.86                            | 1800                        |
|         | 12         | 9.89                              | 600                         |
|         | 13         | 7.30                              | 200                         |
|         | 14         | 12.21                             | 350                         |
|         | 15         | 35.10                             | 1550                        |
|         | 16         | 105.87                            | 4100                        |
|         | 17         | 26.72                             | 1300                        |
|         | 18         | 8.94                              | 550                         |
|         | 19         | 6.10                              | 200                         |
|         | 20         | 6.58                              | 150                         |
|         | 21         | 5.23                              | 500                         |
|         | 22         | 139.30                            | 2450                        |
|         | 23         | 27.90                             | 1600                        |
|         | 24         | 112.71                            | 3950                        |
|         | 25         | 30.96                             | 900                         |
|         | 26         | <u>71.72</u>                      | 1400                        |
|         |            | 1015.02                           |                             |



Swakop River - Recorded Floods in Chronological Order.

| Season  | Runoff No. | Amount of flow<br>(million cu.ft.) | Peak intensity.<br>(cusecs) |
|---------|------------|------------------------------------|-----------------------------|
| 1949/50 | 1          | 36.28                              | 850                         |
|         | 2          | 24.31                              | 3000                        |
|         | 3          | 11.76                              | 200                         |
|         | 4          | 35.28                              | 3400                        |
|         | 5          | 23.12                              | 800                         |
|         | 6          | 11.61                              | 1000                        |
|         | 7          | 26.46                              | 3150                        |
|         | 8          | 11.66                              | 200                         |
|         | 9          | 13.78                              | 800                         |
|         | 10         | 7.52                               | 250                         |
|         | 11         | 6.72                               | 400                         |
|         | 12         | 34.62                              | 400                         |
|         | 13         | 30.08                              | 1325                        |
|         | 14         | 48.69                              | 2900                        |
|         | 15         | 21.47                              | 1000                        |
|         | 16         | 21.58                              | 950                         |
|         | 17         | 10.50                              | 475                         |
|         | 18         | 11.34                              | 400                         |
|         | 19         | 24.42                              | 250                         |
|         | 20         | 13.84                              | 480                         |
|         | 21         | 34.77                              | 1200                        |
|         | 22         | 31.83                              | 1000                        |
|         | 23         | 15.26                              | 400                         |
|         | 24         | 23.33                              | 400                         |
|         | 25         | 20.27                              | 450                         |
|         | 26         | 13.18                              | 300                         |
|         | 27         | 13.89                              | 2075                        |
|         | 28         | 42.39                              | 1350                        |

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|  |    |              |      |
|--|----|--------------|------|
|  | 29 | 17.66        | 550  |
|  | 30 | 15.94        | 820  |
|  | 31 | 37.26        | 900  |
|  | 32 | 12.14        | 150  |
|  | 33 | 21.66        | 375  |
|  | 34 | <u>14.33</u> | 1575 |
|  |    | 738.96       |      |

Swakop River - Recorded Floods in Chronological Order.

| Season  | Runoff No. | Amount of flow<br>(million cu.ft.) | Peak intensity<br>(cusecs) |
|---------|------------|------------------------------------|----------------------------|
| 1950/51 | 1          | 21.83                              | 2140                       |
|         | 2          | 0.28                               | 40                         |
|         | 3          | 4.23                               | 500                        |
|         | 4          | 9.43                               | 700                        |
|         | 5          | 1.47                               | 70                         |
|         | 6          | 7.37                               | 350                        |
|         | 7          | <u>21.42</u>                       | 2140                       |
|         |            | 66.03                              |                            |

Swakop River - Recorded Floods in Chronological Order.

| Season  | Runoff No. | Amount of flow<br>(million cu.ft.) | Peak intensity<br>(cusecs) |
|---------|------------|------------------------------------|----------------------------|
| 1951/52 | 1          | 10.61                              | 700                        |
|         | 2          | 3.55                               | 560                        |
|         | 3          | 11.23                              | 860                        |
|         | 4          | 2.06                               | 500                        |
|         | 5          | 22.96                              | 560                        |
|         | 6          | 39.77                              | 1830                       |
|         | 7          | 6.71                               | 200                        |
|         | 8          | 8.38                               | 300                        |
|         | 9          | 5.50                               | 500                        |
|         | 10         | <u>20.97</u>                       | 900                        |
|         |            | 131.74                             |                            |

Swakop River - Recorded Floods in Order of Magnitude.

$$Q_1 = 3600 \times 1165^{.45} = 86,300 \text{ cusecs.}$$

| Flood No | Peak<br>cusecs<br>Q | Amount<br>mil.<br>cu.<br>ft. | Σ Amount | $F_R$ | $Q/Q_1$ |
|----------|---------------------|------------------------------|----------|-------|---------|
| 1        | 10400               | 200.20                       | 200.20   | 0.050 | 0.1206  |
| 2        | 5800                | 122.00                       | 322.20   | 0.096 | 0.0672  |
| 3        | 4700                | 87.70                        | 409.90   | 0.122 | 0.0545  |
| 4        | 4100                | 105.87                       | 515.77   | 0.153 | 0.0475  |
| 5        | 3950                | 112.71                       | 628.48   | 0.187 | 0.0458  |
| 6        | 3900                | 108.89                       | 737.37   | 0.219 | 0.0452  |
| 7        | 3700                | 73.70                        | 811.07   | 0.241 | 0.0429  |
| 8        | 3650                | 46.65                        | 857.72   | 0.255 | 0.0423  |
| 9        | 3400                | 102.10                       | 959.82   | 0.285 | 0.0392  |
| 10       | 3400                | 35.28                        | 995.10   | 0.296 | 0.0392  |
| 11       | 3300                | 144.97                       | 1140.07  | 0.339 | 0.0383  |
| 12       | 3150                | 26.46                        | 1166.53  | 0.347 | 0.0365  |
| 13       | 3000                | 24.31                        | 1190.84  | 0.354 | 0.0348  |
| 14       | 2900                | 48.69                        | 1239.53  | 0.369 | 0.0336  |
| 15       | 2600                | 53.10                        | 1292.63  | 0.385 | 0.0301  |
| 16       | 2550                | 50.20                        | 1342.83  | 0.400 | 0.0296  |
| 17       | 2500                | 81.56                        | 1424.39  | 0.423 | 0.0290  |
| 18       | 2450                | 139.30                       | 1563.69  | 0.466 | 0.0284  |
| 19       | 2250                | 145.10                       | 1708.79  | 0.508 | 0.0261  |
| 20       | 2200                | 56.25                        | 1765.04  | 0.525 | 0.0255  |
| 21       | 2140                | 21.83                        | 1786.87  | 0.531 | 0.0248  |
| 22       | 2140                | 21.42                        | 1808.29  | 0.538 | 0.0248  |
| 23       | 2075                | 13.89                        | 1822.18  | 0.542 | 0.0240  |
| 24       | 1950                | 39.14                        | 1861.32  | 0.554 | 0.0226  |
| 25       | 1900                | 27.78                        | 1889.10  | 0.560 | 0.0220  |
| 26       | 1830                | 39.77                        | 1928.87  | 0.573 | 0.0212  |
| 27       | 1800                | 114.86                       | 2043.73  | 0.608 | 0.0209  |
| 28       | 1800                | 86.77                        | 2130.50  | 0.633 | 0.0209  |

| Flood No. | Peak<br>cusecs<br>Q | Amount<br>mil.cu.<br>ft. | Σ Amount | $F_R$ | $\frac{Q}{Q_1}$ |
|-----------|---------------------|--------------------------|----------|-------|-----------------|
| 29        | 1750                | 14.58                    | 2145.08  | 0.638 | 0.0203          |
| 30        | 1675                | 28.20                    | 2173.28  | 0.647 | 0.0194          |
| 31        | 1600                | 27.90                    | 2201.18  |       |                 |
| 32        | 1575                | 14.33                    | 2215.51  |       |                 |
| 33        | 1550                | 35.10                    | 2250.61  |       |                 |
| 34        | 1425                | 29.10                    | 2279.71  |       |                 |
| 35        | 1400                | 19.20                    | 2298.91  | 0.683 | 0.0162          |
| 36        | 1400                | 71.72                    | 2370.63  |       |                 |
| 37        | 1350                | 42.39                    | 2413.02  |       |                 |
| 38        | 1325                | 30.03                    | 2443.10  |       |                 |
| 39        | 1300                | 26.72                    | 2469.82  |       |                 |
| 40        | 1250                | 22.67                    | 2492.49  | 0.740 | 0.0145          |
| 41        | 1200                | 34.77                    | 2527.26  |       |                 |
| 42        | 1100                | 10.68                    | 2537.94  |       |                 |
| 43        | 1025                | 20.00                    | 2557.94  |       |                 |
| 44        | 1000                | 21.47                    | 2579.41  |       |                 |
| 45        | 1000                | 11.61                    | 2591.02  |       |                 |
| 46        | 1000                | 31.83                    | 2622.85  |       |                 |
| 47        | 1000                | 28.40                    | 2651.25  |       |                 |
| 48        | 950                 | 21.58                    | 2672.83  |       |                 |
| 49        | 900                 | 37.26                    | 2710.09  |       |                 |
| 50        | 900                 | 30.96                    | 2741.05  | 0.816 | 0.0104          |
| 51        | 900                 | 20.97                    | 2762.02  |       |                 |
| 52        | 900                 | 12.42                    | 2774.44  |       |                 |
| 53        | 860                 | 11.23                    | 2785.67  |       |                 |
| 54        | 850                 | 36.28                    | 2821.95  |       |                 |
| 55        | 820                 | 15.94                    | 2837.89  |       |                 |
| 56        | 800                 | 23.12                    | 2861.01  |       |                 |
| 57        | 800                 | 13.78                    | 2874.79  |       |                 |
| 58        | 750                 | 5.46                     | 2880.25  |       |                 |
| 59        | 700                 | 10.61                    | 2890.86  |       |                 |

| Flood No. | Peak<br>cusecs<br>$Q$ | Amount<br>mil.cu.<br>ft. | $\Sigma$ Amount | $F_R$ | $\frac{Q}{Q_1}$ |
|-----------|-----------------------|--------------------------|-----------------|-------|-----------------|
| 60        | 700                   | 9.43                     | 2900.29         | 0.862 | 0.0081          |
| 61        | 700                   | 5.67                     | 2905.96         |       |                 |
| 62        | 600                   | 10.03                    | 2915.99         |       |                 |
| 63        | 600                   | 9.89                     | 2925.88         |       |                 |
| 64        | 600                   | 6.54                     | 2932.42         |       |                 |
| 65        | 560                   | 22.96                    | 2955.38         |       |                 |
| 66        | 560                   | 3.55                     | 2958.93         |       |                 |
| 67        | 550                   | 17.66                    | 2976.59         |       |                 |
| 68        | 550                   | 8.94                     | 2985.53         |       |                 |
| 69        | 550                   | 5.01                     | 2990.54         |       |                 |
| 70        | 500                   | 16.70                    | 3007.24         | 0.895 | 0.0058          |
| 71        | 500                   | 5.50                     | 3012.74         |       |                 |
| 72        | 500                   | 5.23                     | 3017.97         |       |                 |
| 73        | 500                   | 4.23                     | 3022.20         |       |                 |
| 74        | 500                   | 2.06                     | 3024.26         |       |                 |
| 75        | 480                   | 13.84                    | 3038.10         |       |                 |
| 76        | 475                   | 10.50                    | 3048.60         |       |                 |
| 77        | 450                   | 20.27                    | 3068.87         |       |                 |
| 78        | 450                   | 1.82                     | 3070.69         |       |                 |
| 79        | 400                   | 34.62                    | 3105.31         |       |                 |
| 80        | 400                   | 23.33                    | 3128.64         | 0.930 | 0.0046          |
| 81        | 400                   | 15.26                    | 3143.90         |       |                 |
| 82        | 400                   | 11.34                    | 3155.24         |       |                 |
| 83        | 400                   | 6.72                     | 3161.96         |       |                 |
| 84        | 400                   | 3.93                     | 3165.89         |       |                 |
| 85        | 375                   | 21.66                    | 3187.55         |       |                 |
| 86        | 350                   | 12.21                    | 3199.76         |       |                 |
| 87        | 350                   | 7.37                     | 3207.13         |       |                 |
| 88        | 300                   | 13.18                    | 3220.31         |       |                 |
| 89        | 300                   | 8.90                     | 3229.21         |       |                 |
| 90        | 300                   | 8.38                     | 3237.59         | 0.961 | 0.0035          |

Gamams III - Recorded Floods in Chronological Order.

| Season  | Runoff No | Amount of flow<br>(million cu.ft.) | Peak intensity<br>(cusecs) |
|---------|-----------|------------------------------------|----------------------------|
| 1946/47 | 1         | 0.47                               | 225                        |
|         | 2         | 0.07                               | 10                         |
|         | 3         | 4.49                               | 875                        |
|         | 4         | 5.78                               | 1050                       |
|         | 5         | 4.01                               | 500                        |
|         | 6         | 1.85                               | 100                        |
|         | 7         | 15.33                              | 1325                       |
|         | 8         | 8.39                               | 1200                       |
|         | 9         | 1.80                               | 75                         |
|         | 10        | 1.95                               | 150                        |
|         | 11        | 7.42                               | 700                        |
|         | 12        | 1.05                               | 450                        |
|         | 13        | 6.16                               | 40                         |
|         | 14        | 1.65                               | 300                        |
|         | 15        | 5.89                               | 775                        |
|         | 16        | 0.37                               | 50                         |
|         | 17        | 4.68                               | 650                        |
|         | 18        | 2.76                               | 450                        |
|         | 19        | 8.61                               | 1675                       |
|         | 20        | 2.57                               | 250                        |
|         | 21        | 30.21                              | 3400                       |
|         | 22        | 2.83                               | 275                        |
|         | 23        | 25.10                              | 2600                       |
|         | 24        | 16.13                              | 2300                       |
|         | 25        | 20.00                              | 3550                       |
|         | 26        | 2.93                               | 500                        |
|         | 27        | 7.16                               | 550                        |
|         | 28        | 13.22                              | 1400                       |
|         | 29        | 2.20                               | 150                        |
|         | 30        | 2.21                               | 200                        |
|         | 31        | 5.77                               | 1400                       |
|         |           | <u>207.05</u>                      |                            |

Gamams III - Recorded Floods in Chronological Order.

| Season  | Runoff No. | Amount of Flow<br>(million cu.ft) | Peak intensity.<br>(cusecs) |
|---------|------------|-----------------------------------|-----------------------------|
| 1947/48 | 1          | 8.28                              | 1775                        |
|         | 2          | 16.10                             | 1050                        |
|         | 3          | 0.63                              | 50                          |
|         | 4          | 22.35                             | 5100                        |
|         | 5          | 22.01                             | 3400                        |
|         | 6          | 36.62                             | 3200                        |
|         | 7          | 12.43                             | 1100                        |
|         | 8          | 17.88                             | 1500                        |
|         | 9          | 7.84                              | 1250                        |
|         | 10         | 14.05                             | 1550                        |
|         | 11         | 3.64                              | 190                         |
|         | 12         | 2.18                              | 250                         |
|         | 13         | 0.40                              | 40                          |
|         | 14         | 3.49                              | 400                         |
|         | 15         | 4.13                              | 300                         |
|         | 16         | 8.72                              | 2050                        |
|         | 17         | 3.27                              | 450                         |
|         | 18         | 135.80                            | 21000                       |
|         | 19         | 8.91                              | 550                         |
|         | 20         | <u>2.30</u>                       | 150                         |
|         |            | 321.03                            |                             |

Gamams III - Recorded Floods in Chronological Order.

| Season  | Runoff No | Amount of Flow<br>(million cu.ft.) | Peak intensity<br>(cusecs) |
|---------|-----------|------------------------------------|----------------------------|
| 1948/49 | 1         | 32.20                              | 4800                       |
|         | 2         | 15.48                              | 1600                       |
|         | 3         | 1.31                               | 100                        |
|         | 4         | 54.15                              | 6450                       |
|         | 5         | 2.29                               | 350                        |
|         | 6         | 0.97                               | 50                         |
|         | 7         | 34.00                              | 5650                       |
|         | 8         | 12.66                              | 2500                       |
|         | 9         | 30.08                              | 5500                       |
|         | 10        | 3.27                               | 700                        |
|         | 11        | 6.08                               | 600                        |
|         | 12        | 16.11                              | 775                        |
|         | 13        | 8.07                               | 1800                       |
|         | 14        | 3.26                               | 125                        |
|         | 15        | 1.16                               | 50                         |
|         | 16        | 7.62                               | 1000                       |
|         | 17        | 3.27                               | 300                        |
|         | 18        | 13.83                              | 1600                       |
|         | 19        | 14.39                              | 2450                       |
|         | 20        | 9.59                               | 1200                       |
|         | 21        | 18.75                              | 2050                       |
|         | 22        | 14.83                              | 1200                       |
|         | 23        | 10.46                              | 900                        |
|         | 24        | 2.83                               | 300                        |
|         | 25        | 28.12                              | 3100                       |
|         | 26        | 41.96                              | 2100                       |
|         | 27        | 59.08                              | 3200                       |
|         | 28        | <u>53.63</u>                       | 3075                       |
|         |           | 499.45                             |                            |



Gamams III - Recorded Floods in Chronological Order.

| Season  | Runoff No. | Amount of Flow<br>(million cu.ft.) | Peak intensity.<br>(cusecs) |
|---------|------------|------------------------------------|-----------------------------|
| 1949/50 | 1          | 1.27                               | 200                         |
|         | 2          | 0.32                               | 60                          |
|         | 3          | 0.25                               | 20                          |
|         | 4          | 18.20                              | 3460                        |
|         | 5          | 28.20                              | 2900                        |
|         | 6          | 23.13                              | 3650                        |
|         | 7          | 2.39                               | 130                         |
|         | 8          | .16                                | 15                          |
|         | 9          | 6.54                               | 520                         |
|         | 10         | .16                                | 10                          |
|         | 11         | 29.69                              | 1840                        |
|         | 12         | 27.57                              | 5660                        |
|         | 13         | 1.62                               | 100                         |
|         | 14         | 2.18                               | 110                         |
|         | 15         | 5.94                               | 440                         |
|         | 16         | 7.90                               | 900                         |
|         | 17         | 1.89                               | 150                         |
|         | 18         | 10.48                              | 1640                        |
|         | 19         | 5.12                               | 900                         |
|         | 20         | 10.88                              | 840                         |
|         | 21         | .31                                | 25                          |
|         | 22         | 8.31                               | 1200                        |
|         | 23         | .58                                | 40                          |
|         | 24         | 24.33                              | 2400                        |
|         | 25         | 5.71                               | 300                         |
|         | 26         | 3.91                               | 190                         |
|         | 27         | 4.14                               | 200                         |
|         | 28         | 4.95                               | 900                         |

| Flood No. | Peak<br>cusecs | Amount<br>mil.cu.<br>ft. | $\Sigma$ Amount | $F_R$ | $\frac{Q}{Q_1}$ |
|-----------|----------------|--------------------------|-----------------|-------|-----------------|
| 57        | 900            | 7.90                     | 1341.22         |       |                 |
| 58        | 900            | 7.62                     | 1348.84         |       |                 |
| 59        | 900            | 5.12                     | 1353.96         |       |                 |
| 60        | 900            | 4.95                     | 1358.91         | .862  | .0248           |
| 61        | 875            | 4.49                     | 1363.40         |       |                 |
| 62        | 840            | 10.88                    | 1374.28         |       |                 |
| 63        | 775            | 16.11                    | 1390.39         |       |                 |
| 64        | 775            | 5.89                     | 1396.28         |       |                 |
| 65        | 700            | 7.42                     | 1403.70         |       |                 |
| 66        | 700            | 3.27                     | 1406.97         |       |                 |
| 67        | 650            | 4.68                     | 1411.65         |       |                 |
| 68        | 600            | 6.08                     | 1417.73         |       |                 |
| 69        | 600            | 4.61                     | 1422.34         |       |                 |
| 70        | 550            | 8.91                     | 1431.25         | .910  | .0151           |
| 71        | 550            | 7.16                     | 1438.41         |       |                 |
| 72        | 550            | 3.71                     | 1442.12         |       |                 |
| 73        | 520            | 6.68                     | 1448.80         |       |                 |
| 74        | 520            | 6.54                     | 1455.34         |       |                 |
| 75        | 500            | 4.01                     | 1459.35         |       |                 |
| 76        | 500            | 2.93                     | 1462.28         |       |                 |
| 77        | 450            | 3.27                     | 1465.55         |       |                 |
| 78        | 450            | 2.76                     | 1468.31         |       |                 |
| 79        | 450            | 1.05                     | 1469.36         |       |                 |
| 80        | 440            | 5.94                     | 1475.30         | .937  | .0121           |
| 81        | 400            | 4.39                     | 1479.69         |       |                 |
| 82        | 400            | 3.49                     | 1483.18         |       |                 |
| 83        | 350            | 2.29                     | 1485.47         |       |                 |
| 84        | 330            | 2.63                     | 1488.10         |       |                 |
| 85        | 310            | 3.78                     | 1491.88         |       |                 |

| Flood No. | Peak<br>cusecs | Amount<br>mil.cu.<br>ft. | $\Sigma$ Amount | $F_R$ | $Q/Q_1$ |
|-----------|----------------|--------------------------|-----------------|-------|---------|
| 116       | 100            | 0.86                     | 1566.13         |       |         |
| 117       | 100            | 0.35                     | 1566.48         |       |         |
| 118       | 75             | 1.80                     | 1586.28         |       |         |
| 119       | 60             | 0.32                     | 1568.60         |       |         |
| 120       | 50             | 1.16                     | 1569.76         | .997  | .0014   |
| 121       | 50             | 0.97                     | 1570.73         |       |         |
| 122       | 50             | 0.63                     | 1571.36         |       |         |
| 123       | 50             | 0.37                     | 1571.73         |       |         |
| 124       | 40             | 0.58                     | 1572.31         |       |         |
| 125       | 40             | 0.40                     | 1572.71         |       |         |
| 126       | 40             | 0.16                     | 1572.87         |       |         |
| 127       | 30             | 0.98                     | 1573.85         |       |         |
| 128       | 25             | 0.31                     | 1574.16         |       |         |
| 129       | 20             | 0.25                     | 1574.41         |       |         |
| 130       | 15             | 0.16                     | 1574.57         |       |         |
| 131       | 10             | 0.16                     | 1574.73         |       |         |
| 132       | 10             | 0.07                     | 1574.80         | 1.00  | 0       |

## Total Runoff (million cubic feet)

|         |              |
|---------|--------------|
| 1946/47 | 207.05       |
| 1947/48 | 321.03       |
| 1948/49 | 499.45       |
| 1949/50 | 492.70       |
| 1950/51 | 12.61        |
| 1951/52 | <u>41.95</u> |
|         | 1574.79      |

|  |    |              |      |
|--|----|--------------|------|
|  | 29 | 17.66        | 550  |
|  | 30 | 15.94        | 820  |
|  | 31 | 37.26        | 900  |
|  | 32 | 12.14        | 150  |
|  | 33 | 21.66        | 375  |
|  | 34 | <u>14.33</u> | 1575 |
|  |    | 738.96       |      |

Swakop River - Recorded Floods in Chronological Order.

| Season  | Runoff No. | Amount of flow<br>(million cu.ft.) | Peak intensity<br>(cusecs) |
|---------|------------|------------------------------------|----------------------------|
| 1950/51 | 1          | 21.83                              | 2140                       |
|         | 2          | 0.28                               | 40                         |
|         | 3          | 4.23                               | 500                        |
|         | 4          | 9.43                               | 700                        |
|         | 5          | 1.47                               | 70                         |
|         | 6          | 7.37                               | 350                        |
|         | 7          | <u>21.42</u>                       | 2140                       |
|         |            | 66.03                              |                            |

Swakop River - Recorded Floods in Chronological Order.

| Season  | Runoff No. | Amount of flow<br>(million cu.ft.) | Peak intensity<br>(cusecs) |
|---------|------------|------------------------------------|----------------------------|
| 1951/52 | 1          | 10.61                              | 700                        |
|         | 2          | 3.55                               | 560                        |
|         | 3          | 11.23                              | 860                        |
|         | 4          | 2.06                               | 500                        |
|         | 5          | 22.96                              | 560                        |
|         | 6          | 39.77                              | 1830                       |
|         | 7          | 6.71                               | 200                        |
|         | 8          | 8.38                               | 300                        |
|         | 9          | 5.50                               | 500                        |
|         | 10         | <u>20.97</u>                       | 900                        |
|         |            | 131.74                             |                            |

Swakop River - Recorded Floods in Order of Magnitude.

$$Q_1 = 3600 \times 1165^{.45} = 86,300 \text{ cusecs.}$$

| Flood No | Peak<br>cusecs<br>Q | Amount<br>mil.<br>cu.<br>ft. | $\Sigma$ Amount | $F_R$ | $Q/Q_1$ |
|----------|---------------------|------------------------------|-----------------|-------|---------|
| 1        | 10400               | 200.20                       | 200.20          | 0.050 | 0.1206  |
| 2        | 5800                | 122.00                       | 322.20          | 0.096 | 0.0672  |
| 3        | 4700                | 87.70                        | 409.90          | 0.122 | 0.0545  |
| 4        | 4100                | 105.87                       | 515.77          | 0.153 | 0.0475  |
| 5        | 3950                | 112.71                       | 628.48          | 0.187 | 0.0458  |
| 6        | 3900                | 108.89                       | 737.37          | 0.219 | 0.0452  |
| 7        | 3700                | 73.70                        | 811.07          | 0.241 | 0.0429  |
| 8        | 3650                | 46.65                        | 857.72          | 0.255 | 0.0423  |
| 9        | 3400                | 102.10                       | 959.82          | 0.285 | 0.0392  |
| 10       | 3400                | 35.28                        | 995.10          | 0.296 | 0.0392  |
| 11       | 3300                | 144.97                       | 1140.07         | 0.339 | 0.0383  |
| 12       | 3150                | 26.46                        | 1166.53         | 0.347 | 0.0365  |
| 13       | 3000                | 24.31                        | 1190.84         | 0.354 | 0.0348  |
| 14       | 2900                | 48.69                        | 1239.53         | 0.369 | 0.0336  |
| 15       | 2600                | 53.10                        | 1292.63         | 0.385 | 0.0301  |
| 16       | 2550                | 50.20                        | 1342.83         | 0.400 | 0.0296  |
| 17       | 2500                | 81.56                        | 1424.39         | 0.423 | 0.0290  |
| 18       | 2450                | 139.30                       | 1563.69         | 0.466 | 0.0284  |
| 19       | 2250                | 145.10                       | 1708.79         | 0.508 | 0.0261  |
| 20       | 2200                | 56.25                        | 1765.04         | 0.525 | 0.0255  |
| 21       | 2140                | 21.83                        | 1786.87         | 0.531 | 0.0248  |
| 22       | 2140                | 21.42                        | 1808.29         | 0.538 | 0.0248  |
| 23       | 2075                | 13.89                        | 1822.18         | 0.542 | 0.0240  |
| 24       | 1950                | 39.14                        | 1861.32         | 0.554 | 0.0226  |
| 25       | 1900                | 27.78                        | 1889.10         | 0.560 | 0.0220  |
| 26       | 1830                | 39.77                        | 1928.87         | 0.573 | 0.0212  |
| 27       | 1800                | 114.86                       | 2043.73         | 0.608 | 0.0209  |
| 28       | 1800                | 86.77                        | 2130.50         | 0.633 | 0.0209  |

| Flood No. | Peak<br>cusecs<br>$Q$ | Amount<br>mil.cu.<br>ft. | $\leq$ Amount | $F_R$ | $\frac{Q}{Q_1}$ |
|-----------|-----------------------|--------------------------|---------------|-------|-----------------|
| 29        | 1750                  | 14.58                    | 2145.08       | 0.638 | 0.0203          |
| 30        | 1675                  | 28.20                    | 2173.28       | 0.647 | 0.0194          |
| 31        | 1600                  | 27.90                    | 2201.18       |       |                 |
| 32        | 1575                  | 14.33                    | 2215.51       |       |                 |
| 33        | 1550                  | 35.10                    | 2250.61       |       |                 |
| 34        | 1425                  | 29.10                    | 2279.71       |       |                 |
| 35        | 1400                  | 19.20                    | 2298.91       | 0.683 | 0.0162          |
| 36        | 1400                  | 71.72                    | 2370.63       |       |                 |
| 37        | 1350                  | 42.39                    | 2413.02       |       |                 |
| 38        | 1325                  | 30.08                    | 2443.10       |       |                 |
| 39        | 1300                  | 26.72                    | 2469.82       |       |                 |
| 40        | 1250                  | 22.67                    | 2492.49       | 0.740 | 0.0145          |
| 41        | 1200                  | 34.77                    | 2527.26       |       |                 |
| 42        | 1100                  | 10.68                    | 2537.94       |       |                 |
| 43        | 1025                  | 20.00                    | 2557.94       |       |                 |
| 44        | 1000                  | 21.47                    | 2579.41       |       |                 |
| 45        | 1000                  | 11.61                    | 2591.02       |       |                 |
| 46        | 1000                  | 31.83                    | 2622.85       |       |                 |
| 47        | 1000                  | 28.40                    | 2651.25       |       |                 |
| 48        | 950                   | 21.58                    | 2672.83       |       |                 |
| 49        | 900                   | 37.26                    | 2710.09       |       |                 |
| 50        | 900                   | 30.96                    | 2741.05       | 0.816 | 0.0104          |
| 51        | 900                   | 20.97                    | 2762.02       |       |                 |
| 52        | 900                   | 12.42                    | 2774.44       |       |                 |
| 53        | 860                   | 11.23                    | 2785.67       |       |                 |
| 54        | 850                   | 36.28                    | 2821.95       |       |                 |
| 55        | 820                   | 15.94                    | 2837.89       |       |                 |
| 56        | 800                   | 23.12                    | 2861.01       |       |                 |
| 57        | 800                   | 13.78                    | 2874.79       |       |                 |
| 58        | 750                   | 5.46                     | 2880.25       |       |                 |
| 59        | 700                   | 10.61                    | 2890.86       |       |                 |

| Flood No. | Peak<br>cusecs<br>$Q$ | Amount<br>mil.cu.<br>ft. | $\Sigma$ Amount | $F_R$ | $\frac{Q}{Q_1}$ |
|-----------|-----------------------|--------------------------|-----------------|-------|-----------------|
| 60        | 700                   | 9.43                     | 2900.29         | 0.862 | 0.0081          |
| 61        | 700                   | 5.67                     | 2905.96         |       |                 |
| 62        | 600                   | 10.03                    | 2915.99         |       |                 |
| 63        | 600                   | 9.89                     | 2925.88         |       |                 |
| 64        | 600                   | 6.54                     | 2932.42         |       |                 |
| 65        | 560                   | 22.96                    | 2955.38         |       |                 |
| 66        | 560                   | 3.55                     | 2958.93         |       |                 |
| 67        | 550                   | 17.66                    | 2976.59         |       |                 |
| 68        | 550                   | 8.94                     | 2985.53         |       |                 |
| 69        | 550                   | 5.01                     | 2990.54         |       |                 |
| 70        | 500                   | 16.70                    | 3007.24         | 0.895 | 0.0058          |
| 71        | 500                   | 5.50                     | 3012.74         |       |                 |
| 72        | 500                   | 5.23                     | 3017.97         |       |                 |
| 73        | 500                   | 4.23                     | 3022.20         |       |                 |
| 74        | 500                   | 2.06                     | 3024.26         |       |                 |
| 75        | 480                   | 13.84                    | 3038.10         |       |                 |
| 76        | 475                   | 10.50                    | 3048.60         |       |                 |
| 77        | 450                   | 20.27                    | 3068.87         |       |                 |
| 78        | 450                   | 1.82                     | 3070.69         |       |                 |
| 79        | 400                   | 34.62                    | 3105.31         |       |                 |
| 80        | 400                   | 23.33                    | 3128.64         | 0.930 | 0.0046          |
| 81        | 400                   | 15.26                    | 3143.90         |       |                 |
| 82        | 400                   | 11.34                    | 3155.24         |       |                 |
| 83        | 400                   | 6.72                     | 3161.96         |       |                 |
| 84        | 400                   | 3.93                     | 3165.89         |       |                 |
| 85        | 375                   | 21.66                    | 3187.55         |       |                 |
| 86        | 350                   | 12.21                    | 3199.76         |       |                 |
| 87        | 350                   | 7.37                     | 3207.13         |       |                 |
| 88        | 300                   | 13.18                    | 3220.31         |       |                 |
| 89        | 300                   | 8.90                     | 3229.21         |       |                 |
| 90        | 300                   | 8.38                     | 3237.59         | 0.961 | 0.0035          |

Gamams III - Recorded Floods in Chronological Order.

| Season  | Runoff No | Amount of flow<br>(million cu.ft.) | Peak intensity<br>(cusecs) |
|---------|-----------|------------------------------------|----------------------------|
| 1946/47 | 1         | 0.47                               | 225                        |
|         | 2         | 0.07                               | 10                         |
|         | 3         | 4.49                               | 875                        |
|         | 4         | 5.78                               | 1050                       |
|         | 5         | 4.01                               | 500                        |
|         | 6         | 1.85                               | 100                        |
|         | 7         | 15.33                              | 1325                       |
|         | 8         | 8.39                               | 1200                       |
|         | 9         | 1.80                               | 75                         |
|         | 10        | 1.95                               | 150                        |
|         | 11        | 7.42                               | 700                        |
|         | 12        | 1.05                               | 450                        |
|         | 13        | 0.16                               | 40                         |
|         | 14        | 1.65                               | 300                        |
|         | 15        | 5.89                               | 775                        |
|         | 16        | 0.37                               | 50                         |
|         | 17        | 4.68                               | 650                        |
|         | 18        | 2.76                               | 450                        |
|         | 19        | 8.61                               | 1675                       |
|         | 20        | 2.57                               | 250                        |
|         | 21        | 30.21                              | 3400                       |
|         | 22        | 2.83                               | 275                        |
|         | 23        | 25.10                              | 2600                       |
|         | 24        | 16.13                              | 2300                       |
|         | 25        | 20.00                              | 3550                       |
|         | 26        | 2.93                               | 500                        |
|         | 27        | 7.16                               | 550                        |
|         | 28        | 13.22                              | 1400                       |
|         | 29        | 2.20                               | 150                        |
|         | 30        | 2.21                               | 200                        |
|         | 31        |                                    | <u>5.77</u>                |
|         |           | 207.05                             |                            |



Gamams III - Recorded Floods in Chronological Order.

| Season  | Runoff No. | Amount of Flow<br>(million cu.ft) | Peak intensity.<br>(cusecs) |
|---------|------------|-----------------------------------|-----------------------------|
| 1947/48 | 1          | 8.28                              | 1775                        |
|         | 2          | 16.10                             | 1050                        |
|         | 3          | 0.63                              | 50                          |
|         | 4          | 22.35                             | 5100                        |
|         | 5          | 22.01                             | 3400                        |
|         | 6          | 36.62                             | 3200                        |
|         | 7          | 12.43                             | 1100                        |
|         | 8          | 17.88                             | 1500                        |
|         | 9          | 7.84                              | 1250                        |
|         | 10         | 14.05                             | 1550                        |
|         | 11         | 3.64                              | 190                         |
|         | 12         | 2.18                              | 250                         |
|         | 13         | 0.40                              | 40                          |
|         | 14         | 3.49                              | 400                         |
|         | 15         | 4.13                              | 300                         |
|         | 16         | 8.72                              | 2050                        |
|         | 17         | 3.27                              | 450                         |
|         | 18         | 135.80                            | 21000                       |
|         | 19         | 8.91                              | 550                         |
|         | 20         | <u>2.30</u>                       | 150                         |
|         |            | 321.03                            |                             |

Gamams III - Recorded Floods in Chronological Order.

| Season  | Runoff No | Amount of Flow<br>(million cu.ft.) | Peak intensity<br>(cusecs) |
|---------|-----------|------------------------------------|----------------------------|
| 1948/49 | 1         | 32.20                              | 4800                       |
|         | 2         | 15.48                              | 1600                       |
|         | 3         | 1.31                               | 100                        |
|         | 4         | 54.15                              | 6450                       |
|         | 5         | 2.29                               | 350                        |
|         | 6         | 0.97                               | 50                         |
|         | 7         | 34.00                              | 5650                       |
|         | 8         | 12.66                              | 2500                       |
|         | 9         | 30.08                              | 5500                       |
|         | 10        | 3.27                               | 700                        |
|         | 11        | 6.08                               | 600                        |
|         | 12        | 16.11                              | 775                        |
|         | 13        | 8.07                               | 1800                       |
|         | 14        | 3.26                               | 125                        |
|         | 15        | 1.16                               | 50                         |
|         | 16        | 7.62                               | 1000                       |
|         | 17        | 3.27                               | 300                        |
|         | 18        | 13.83                              | 1600                       |
|         | 19        | 14.39                              | 2450                       |
|         | 20        | 9.59                               | 1200                       |
|         | 21        | 18.75                              | 2050                       |
|         | 22        | 14.83                              | 1200                       |
|         | 23        | 10.46                              | 900                        |
|         | 24        | 2.83                               | 300                        |
|         | 25        | 28.12                              | 3100                       |
|         | 26        | 41.96                              | 2100                       |
|         | 27        | 59.08                              | 3200                       |
|         | 28        | <u>53.63</u>                       | 3075                       |
|         |           | 499.45                             |                            |

| Flood No. | Peak<br>cusecs | Amount<br>mil.cu.<br>ft. | $\Sigma$ Amount | $F_R$ | $\frac{Q}{Q_1}$ |
|-----------|----------------|--------------------------|-----------------|-------|-----------------|
| 57        | 900            | 7.90                     | 1341.22         |       |                 |
| 58        | 900            | 7.62                     | 1348.84         |       |                 |
| 59        | 900            | 5.12                     | 1353.96         |       |                 |
| 60        | 900            | 4.95                     | 1358.91         | .862  | .0248           |
| 61        | 875            | 4.49                     | 1363.40         |       |                 |
| 62        | 840            | 10.88                    | 1374.28         |       |                 |
| 63        | 775            | 16.11                    | 1390.39         |       |                 |
| 64        | 775            | 5.89                     | 1396.28         |       |                 |
| 65        | 700            | 7.42                     | 1403.70         |       |                 |
| 66        | 700            | 3.27                     | 1406.97         |       |                 |
| 67        | 650            | 4.68                     | 1411.65         |       |                 |
| 68        | 600            | 6.08                     | 1417.73         |       |                 |
| 69        | 600            | 4.61                     | 1422.34         |       |                 |
| 70        | 550            | 8.91                     | 1431.25         | .910  | .0151           |
| 71        | 550            | 7.16                     | 1438.41         |       |                 |
| 72        | 550            | 3.71                     | 1442.12         |       |                 |
| 73        | 520            | 6.68                     | 1448.80         |       |                 |
| 74        | 520            | 6.54                     | 1455.34         |       |                 |
| 75        | 500            | 4.01                     | 1459.35         |       |                 |
| 76        | 500            | 2.93                     | 1462.28         |       |                 |
| 77        | 450            | 3.27                     | 1465.55         |       |                 |
| 78        | 450            | 2.76                     | 1468.31         |       |                 |
| 79        | 450            | 1.05                     | 1469.36         |       |                 |
| 80        | 440            | 5.94                     | 1475.30         | .937  | .0121           |
| 81        | 400            | 4.39                     | 1479.69         |       |                 |
| 82        | 400            | 3.49                     | 1483.18         |       |                 |
| 83        | 350            | 2.29                     | 1485.47         |       |                 |
| 84        | 330            | 2.63                     | 1488.10         |       |                 |
| 85        | 310            | 3.78                     | 1491.88         |       |                 |

| Flood No. | Peak<br>cusecs | Amount<br>mil.cu.<br>ft. | $\Sigma$ Amount | $F_R$ | $Q/Q_1$ |
|-----------|----------------|--------------------------|-----------------|-------|---------|
| 116       | 100            | 0.86                     | 1566.13         |       |         |
| 117       | 100            | 0.35                     | 1566.48         |       |         |
| 118       | 75             | 1.80                     | 1586.28         |       |         |
| 119       | 60             | 0.32                     | 1568.60         |       |         |
| 120       | 50             | 1.16                     | 1569.76         | .997  | .0014   |
| 121       | 50             | 0.97                     | 1570.73         |       |         |
| 122       | 50             | 0.63                     | 1571.36         |       |         |
| 123       | 50             | 0.37                     | 1571.73         |       |         |
| 124       | 40             | 0.58                     | 1572.31         |       |         |
| 125       | 40             | 0.40                     | 1572.71         |       |         |
| 126       | 40             | 0.16                     | 1572.87         |       |         |
| 127       | 30             | 0.98                     | 1573.85         |       |         |
| 128       | 25             | 0.31                     | 1574.16         |       |         |
| 129       | 20             | 0.25                     | 1574.41         |       |         |
| 130       | 15             | 0.16                     | 1574.57         |       |         |
| 131       | 10             | 0.16                     | 1574.73         |       |         |
| 132       | 10             | 0.07                     | 1574.80         | 1.00  | 0       |

## Total Runoff (million cubic feet)

|         |              |
|---------|--------------|
| 1946/47 | 207.05       |
| 1947/48 | 321.03       |
| 1948/49 | 499.45       |
| 1949/50 | 492.70       |
| 1950/51 | 12.61        |
| 1951/52 | <u>41.95</u> |
|         | 1574.79      |

Gamams II - Recorded Floods in Chronological Order.

| Season  | Runoff No. | Amount of Flow<br>(million cu.ft.) | Peak Intensity<br>(cusecs) |
|---------|------------|------------------------------------|----------------------------|
| 1943/44 | 1          | 4.16                               | 820                        |
|         | 2          | 5.54                               | 1050                       |
|         | 3          | 18.83                              | 4200                       |
|         | 4          | 1.38                               | 150                        |
|         | 5          | 11.31                              | 1300                       |
|         | 6          | 43.00                              | 5600                       |
|         | 7          | 27.64                              | 1200                       |
|         | 8          | 9.48                               | 1500                       |
|         | 9          | 2.17                               | 190                        |
|         | 10         | 0.53                               | 34                         |
|         | 11         | 11.38                              | 700                        |
|         | 12         | 4.93                               | 130                        |
|         | 13         | 5.06                               | 230                        |
|         | 14         | 1.59                               | 50                         |
|         | 15         | 3.84                               | 390                        |
|         | 16         | 6.35                               | 450                        |
|         | 17         | 1.23                               | 180                        |
|         | 18         | 2.18                               | 125                        |
|         | 19         | <u>0.78</u>                        | 70                         |
|         |            | 161.38                             |                            |

Gamams III - Recorded Floods in Chronological Order.

| Season  | Runoff No. | Amount of Flow<br>(million cu.ft.) | Peak intensity.<br>(cusecs) |
|---------|------------|------------------------------------|-----------------------------|
| 1949/50 | 1          | 1.27                               | 200                         |
|         | 2          | 0.32                               | 60                          |
|         | 3          | 0.25                               | 20                          |
|         | 4          | 18.20                              | 3460                        |
|         | 5          | 28.20                              | 2900                        |
|         | 6          | 23.13                              | 3650                        |
|         | 7          | 2.39                               | 130                         |
|         | 8          | .16                                | 15                          |
|         | 9          | 6.54                               | 520                         |
|         | 10         | .16                                | 10                          |
|         | 11         | 29.69                              | 1840                        |
|         | 12         | 27.57                              | 5660                        |
|         | 13         | 1.62                               | 100                         |
|         | 14         | 2.18                               | 110                         |
|         | 15         | 5.94                               | 440                         |
|         | 16         | 7.90                               | 900                         |
|         | 17         | 1.89                               | 150                         |
|         | 18         | 10.48                              | 1640                        |
|         | 19         | 5.12                               | 900                         |
|         | 20         | 10.88                              | 840                         |
|         | 21         | .31                                | 25                          |
|         | 22         | 8.31                               | 1200                        |
|         | 23         | .58                                | 40                          |
|         | 24         | 24.33                              | 2400                        |
|         | 25         | 5.71                               | 300                         |
|         | 26         | 3.91                               | 190                         |
|         | 27         | 4.14                               | 200                         |
|         | 28         | 4.95                               | 900                         |

| Flood No. | Peak<br>cusecs | Amount<br>mil.cu.ft. | Σ Amount | $F_R$ | $\frac{Q}{Q_1}$ |
|-----------|----------------|----------------------|----------|-------|-----------------|
| 27        | 2100           | 41.96                | 996.97   | .633  | .0593           |
| 28        | 2050           | 18.75                | 1015.72  | .645  | .0580           |
| 29        | 2050           | 8.72                 | 1024.44  | .650  | .0580           |
| 30        | 1900           | 21.11                | 1045.55  | .663  | .0537           |
| 31        | 1840           | 29.69                | 1075.24  |       |                 |
| 32        | 1810           | 17.31                | 1092.55  |       |                 |
| 33        | 1800           | 8.07                 | 1100.62  |       |                 |
| 34        | 1775           | 8.28                 | 1108.90  |       |                 |
| 35        | 1675           | 8.61                 | 1117.51  | .708  | .0473           |
| 36        | 1640           | 10.48                | 1127.99  |       |                 |
| 37        | 1600           | 15.48                | 1143.47  |       |                 |
| 38        | 1600           | 13.83                | 1157.30  |       |                 |
| 39        | 1550           | 14.05                | 1171.35  |       |                 |
| 40        | 1500           | 17.88                | 1189.23  | .755  | .0412           |
| 41        | 1400           | 13.22                | 1202.45  |       |                 |
| 42        | 1400           | 5.77                 | 1208.22  |       |                 |
| 43        | 1325           | 15.33                | 1223.55  |       |                 |
| 44        | 1250           | 7.84                 | 1231.39  |       |                 |
| 45        | 1200           | 14.83                | 1246.22  |       |                 |
| 46        | 1200           | 9.59                 | 1255.81  |       |                 |
| 47        | 1200           | 8.39                 | 1264.20  |       |                 |
| 48        | 1200           | 8.31                 | 1272.51  |       |                 |
| 49        | 1100           | 12.43                | 1284.94  |       |                 |
| 50        | 1100           | 7.74                 | 1292.68  | .821  | .0302           |
| 51        | 1050           | 6.10                 | 1298.78  |       |                 |
| 52        | 1050           | 5.78                 | 1304.56  |       |                 |
| 53        | 1000           | 7.62                 | 1312.18  |       |                 |
| 54        | 1000           | 6.86                 | 1319.04  |       |                 |
| 55        | 950            | 3.82                 | 1322.86  |       |                 |
| 56        | 900            | 10.46                | 1333.32  |       |                 |

Gamams II - Recorded Floods in Chronological Order.

| Season  | Runoff No. | Amount of Flow<br>(million cu.ft.) | Peak Intensity<br>(cusecs) |
|---------|------------|------------------------------------|----------------------------|
| 1944/45 | 1          | 0.05                               | 10                         |
|         | 2          | 0.08                               | 35                         |
|         | 3          | 0.99                               | 110                        |
|         | 4          | 0.14                               | 15                         |
|         | 5          | 0.85                               | 100                        |
|         | 6          | 0.43                               | 25                         |
|         | 7          | <u>0.33</u>                        | 20                         |
|         |            | 2.87                               |                            |

Gamams II - Recorded Floods in Chronological Order.

| Season  | Runoff No. | Amount of Flow<br>(million cu.ft.) | Peak Intensity<br>(cusecs) |
|---------|------------|------------------------------------|----------------------------|
| 1945/46 | 1          | 2.52                               | 740                        |
|         | 2          | 2.47                               | 330                        |
|         | 3          | 22.34                              | 1600                       |
|         | 4          | 67.46                              | 9000                       |
|         | 5          | 6.33                               | 875                        |
|         | 6          | 3.27                               | 570                        |
|         | 7          | 6.21                               | 450                        |
|         | 8          | <u>1.05</u>                        | 150                        |
|         |            | 111.65                             |                            |



|    |             |      |
|----|-------------|------|
| 29 | 9.68        | 1250 |
| 30 | 2.35        | 350  |
| 31 | 2.66        | 350  |
| 32 | 1.39        | 125  |
| 33 | 7.95        | 1000 |
| 34 | 1.50        | 75   |
| 35 | 1.55        | 100  |
| 36 | <u>4.72</u> | 1100 |
|    | 121.37      |      |

## Gamams II - Recorded Floods in Chronological Order.

| Season  | Runoff No. | Amount of Flow<br>(million cu.ft.) | Peak Intensity.<br>(cusecs) |
|---------|------------|------------------------------------|-----------------------------|
| 1947/48 | 1          | 0.79                               | 75                          |
|         | 2          | 7.55                               | 1525                        |
|         | 3          | 4.33                               | 650                         |
|         | 4          | 12.44                              | 3950                        |
|         | 5          | 10.25                              | 2200                        |
|         | 6          | 17.02                              | 2200                        |
|         | 7          | 2.83                               | 375                         |
|         | 8          | 8.90                               | 800                         |
|         | 9          | 2.25                               | 500                         |
|         | 10         | 6.75                               | 1200                        |
|         | 11         | 2.07                               | 85                          |
|         | 12         | 0.94                               | 120                         |
|         | 13         | 0.32                               | 40                          |
|         | 14         | 1.57                               | 250                         |
|         | 15         | 1.35                               | 100                         |
|         | 16         | 6.54                               | 1600                        |
|         | 17         | 1.74                               | 275                         |
|         | 18         | 4.00                               | 900                         |
|         | 19         | 2.26                               | 200                         |
|         | 20         | <u>58.86</u>                       | 8000                        |
|         | 152.76     |                                    |                             |

Gamams II - Recorded Floods in Chronological Order.

| Season  | Runoff No. | Amount of Flow<br>(million cu.ft.) | Peak Intensity.<br>(cusecs) |
|---------|------------|------------------------------------|-----------------------------|
| 1948/49 | 1          | 20.05                              | 3750                        |
|         | 2          | 8.14                               | 1100                        |
|         | 3          | 0.50                               | 40                          |
|         | 4          | 39.56                              | 6000                        |
|         | 5          | 1.05                               | 50                          |
|         | 6          | 26.23                              | 5300                        |
|         | 7          | 5.94                               | 1700                        |
|         | 8          | 21.94                              | 4550                        |
|         | 9          | 2.40                               | 600                         |
|         | 10         | 0.22                               | 40                          |
|         | 11         | 1.05                               | 300                         |
|         | 12         | 2.13                               | 150                         |
|         | 13         | 4.19                               | 875                         |
|         | 14         | 2.48                               | 900                         |
|         | 15         | 0.23                               | 15                          |
|         | 16         | 0.20                               | 10                          |
|         | 17         | 2.12                               | 400                         |
|         | 18         | 0.92                               | 50                          |
|         | 19         | 6.08                               | 1000                        |
|         | 20         | 18.97                              | 1700                        |
|         | 21         | 2.43                               | 1100                        |
|         | 22         | 14.33                              | 1200                        |
|         | 23         | 5.32                               | 300                         |
|         | 24         | 17.44                              | 2700                        |
|         | 25         | 13.07                              | 1250                        |
|         | 26         | 7.40                               | 1200                        |
|         | 27         | 1.26                               | 100                         |
|         | 28         | <u>15.22</u>                       | 2300                        |
|         |            | 240.87                             |                             |

|    |              |      |
|----|--------------|------|
| 30 | 3.13         | 300  |
| 31 | 14.75        | 2800 |
| 32 | 3.76         | 680  |
| 33 | 1.95         | 90   |
| 34 | 1.69         | 40   |
| 35 | 0.78         | 40   |
| 36 | 0.89         | 45   |
| 37 | 11.76        | 1400 |
| 38 | 1.46         | 90   |
| 39 | 3.26         | 280  |
| 40 | 8.18         | 4500 |
| 41 | 95.26        | 5600 |
| 42 | 6.35         | 380  |
| 43 | <u>11.22</u> | 1380 |
|    | 309.75       |      |

## Gamams II - Recorded Floods in Chronological Order.

| Season  | Runoff No. | Amount of Flow<br>(million cu.ft.) | Peak intensity.<br>(cusecs) |
|---------|------------|------------------------------------|-----------------------------|
| 1950/51 | 1          | 1.02                               | 150                         |
|         | 2          | 1.81                               | 450                         |
|         | 3          | 1.49                               | 300                         |
|         | 4          | <u>0.73</u>                        | 125                         |
|         |            | 5.05                               |                             |

## Gamams II - Recorded Floods in Chronological Order.

| Season  | Runoff No. | Amount of Flow<br>(million cu.ft.) | Peak Intensity<br>(cusecs) |
|---------|------------|------------------------------------|----------------------------|
| 1951/52 | 1          | 2.48                               | 450                        |
|         | 2          | 4.65                               | 600                        |
|         | 3          | 1.70                               | 130                        |
|         | 4          | 6.27                               | 800                        |
|         | 5          | 11.72                              | 330                        |
|         | 6          | <u>0.21</u>                        | 100                        |
|         |            | 17.03                              |                            |

Gamams II - Recorded Floods in Order Of Magnitude.

$$Q_1 = 3600 \times 115^{.45} = 30,500$$

| Flood No | Peak<br>cusecs | Amount<br>mil.cu.<br>ft. | $\leq$ Amount | $F_R$ | $\frac{Q}{Q_1}$ |
|----------|----------------|--------------------------|---------------|-------|-----------------|
| 1        | 9000           | 67.46                    | 67.46         | 0.060 | .296            |
| 2        | 8000           | 58.86                    | 126.32        | 0.113 | .262            |
| 3        | 6000           | 39.56                    | 165.88        | 0.148 | .197            |
| 4        | 5600           | 95.26                    | 261.14        | 0.232 | .184            |
| 5        | 5600           | 43.00                    | 304.14        | 0.271 | .184            |
| 6        | 5300           | 26.23                    | 330.37        | 0.294 | .174            |
| 7        | 5050           | 19.98                    | 350.35        | 0.312 | .166            |
| 8        | 4550           | 21.94                    | 372.29        | 0.331 | .149            |
| 9        | 4500           | 8.18                     | 380.47        | 0.338 | .1475           |
| 10       | 4200           | 18.83                    | 399.30        | 0.355 | .1378           |
| 11       | 3950           | 12.44                    | 411.74        | 0.367 | .1295           |
| 12       | 3750           | 20.05                    | 431.79        | 0.385 | .1230           |
| 13       | 3510           | 13.72                    | 445.51        | 0.397 | .1150           |
| 14       | 3180           | 13.81                    | 459.32        | 0.408 | .1044           |
| 15       | 2800           | 14.75                    | 474.07        | 0.422 | .0919           |
| 16       | 2700           | 17.44                    | 491.51        | 0.437 | .0885           |
| 17       | 2500           | 17.32                    | 508.83        | 0.453 | .0820           |
| 18       | 2300           | 15.22                    | 524.05        | 0.466 | .0755           |
| 19       | 2200           | 17.02                    | 541.07        | 0.482 | .0722           |
| 20       | 2200           | 10.25                    | 551.32        | 0.490 | .0722           |
| 21       | 2050           | 18.80                    | 570.12        | 0.508 | .0672           |
| 22       | 1900           | 13.22                    | 583.34        | 0.519 | .0623           |
| 23       | 1700           | 18.97                    | 602.31        | 0.537 | .0558           |
| 24       | 1700           | 5.94                     | 608.25        | 0.541 | .0558           |
| 25       | 1640           | 13.57                    | 621.82        | 0.554 | .0538           |
| 26       | 1600           | 22.34                    | 644.16        | 0.573 | .0525           |
| 27       | 1600           | 7.42                     | 651.58        | 0.580 | .0525           |
| 28       | 1600           | 6.54                     | 658.12        | 0.585 | .0525           |
| 29       | 1560           | 16.17                    | 674.29        | 0.600 | .0512           |

| Flood No | Peak<br>cusecs | Amount<br>mil.<br>cu.<br>ft. | $\Sigma$ Amount | $F_R$ | $\frac{Q}{Q_1}$ |
|----------|----------------|------------------------------|-----------------|-------|-----------------|
| 30       | 1525           | 7.55                         | 681.84          | 0.608 | .0500           |
| 31       | 1500           | 9.48                         | 691.32          | 0.615 | .0492           |
| 32       | 1450           | 6.61                         | 697.93          | 0.621 | .0476           |
| 33       | 1400           | 11.76                        | 709.69          | 0.632 | .0460           |
| 34       | 1380           | 11.22                        | 720.91          | 0.642 | .0453           |
| 35       | 1300           | 11.31                        | 732.22          | 0.652 | .0427           |
| 36       | 1250           | 13.07                        | 745.29          | 0.663 | .0410           |
| 37       | 1250           | 9.68                         | 754.97          |       |                 |
| 38       | 1200           | 27.64                        | 782.61          |       |                 |
| 39       | 1200           | 14.33                        | 796.94          |       |                 |
| 40       | 1200           | 7.40                         | 804.34          |       |                 |
| 41       | 1200           | 6.75                         | 811.09          |       |                 |
| 42       | 1150           | 7.29                         | 818.38          |       |                 |
| 43       | 1100           | 8.47                         | 826.85          |       |                 |
| 44       | 1100           | 8.14                         | 834.99          |       |                 |
| 45       | 1100           | 5.34                         | 840.33          | 0.748 | .0361           |
| 46       | 1100           | 4.72                         | 845.05          |       |                 |
| 47       | 1100           | 2.43                         | 847.48          |       |                 |
| 48       | 1050           | 6.91                         | 854.39          |       |                 |
| 49       | 1050           | 5.54                         | 859.93          |       |                 |
| 50       | 1000           | 7.95                         | 867.88          |       |                 |
| 51       | 1000           | 6.08                         | 873.96          |       |                 |
| 52       | 900            | 4.00                         | 877.96          |       |                 |
| 53       | 900            | 2.48                         | 880.44          |       |                 |
| 54       | 875            | 6.33                         | 886.77          | 0.790 | .0287           |
| 55       | 875            | 4.19                         | 890.96          |       |                 |
| 56       | 820            | 4.16                         | 895.12          |       |                 |
| 57       | 800            | 8.90                         | 904.02          |       |                 |
| 58       | 800            | 6.27                         | 910.29          |       |                 |

| Flood No | Peak<br>cusecs | Amount<br>mil.<br>cu.<br>ft. | $\Sigma$ Amount | $F_R$ | $\frac{Q}{Q_1}$ |
|----------|----------------|------------------------------|-----------------|-------|-----------------|
| 118      | 100            | 1.35                         | 1082.34         |       |                 |
| 119      | 100            | 1.26                         | 1083.60         |       |                 |
| 120      | 100            | 0.85                         | 1084.45         |       |                 |
| 121      | 100            | 0.47                         | 1084.92         |       |                 |
| 122      | 100            | 0.21                         | 1085.13         |       |                 |
| 123      | 90             | 1.94                         | 1087.07         |       |                 |
| 124      | 90             | 1.46                         | 1088.53         |       |                 |
| 125      | 90             | 1.13                         | 1089.66         |       |                 |
| 126      | 90             | 0.86                         | 1090.52         | 0.971 | .0030           |
| 127      | 90             | 0.81                         | 1091.33         |       |                 |
| 128      | 85             | 2.07                         | 1093.40         |       |                 |
| 129      | 80             | 1.11                         | 1094.51         |       |                 |
| 130      | 80             | 0.60                         | 1095.11         |       |                 |
| 131      | 75             | 1.50                         | 1096.61         |       |                 |
| 132      | 75             | 0.79                         | 1097.40         |       |                 |
| 133      | 70             | 2.72                         | 1100.12         |       |                 |
| 134      | 70             | 1.39                         | 1101.51         |       |                 |
| 135      | 70             | 0.78                         | 1102.29         | 0.980 | .0023           |
| 136      | 50             | 1.80                         | 1104.09         |       |                 |
| 137      | 50             | 1.59                         | 1105.68         |       |                 |
| 138      | 50             | 1.05                         | 1106.73         |       |                 |
| 139      | 50             | 0.93                         | 1107.66         |       |                 |
| 140      | 50             | 0.92                         | 1108.58         |       |                 |
| 141      | 50             | 0.46                         | 1109.04         |       |                 |
| 142      | 45             | 1.02                         | 1110.06         |       |                 |
| 143      | 45             | 0.89                         | 1110.95         |       |                 |
| 144      | 45             | 0.69                         | 1111.64         | 0.990 | .0015           |
| 145      | 45             | 0.58                         | 1112.22         |       |                 |
| 146      | 40             | 1.69                         | 1113.91         |       |                 |
| 147      | 40             | 0.78                         | 1114.69         |       |                 |
| 148      | 40             | 0.50                         | 1115.19         |       |                 |
| 149      | 40             | 0.32                         | 1115.51         |       |                 |
| 150      | 40             | 0.30                         | 1115.81         |       |                 |

| Flood No | Peak<br>cusecs | Amount<br>mil.<br>cu.<br>ft. | $\Sigma$ Amount | $F_R$  | $\frac{Q}{Q_1}$ |
|----------|----------------|------------------------------|-----------------|--------|-----------------|
| 151      | 40             | 0.22                         | 1116.03         |        |                 |
| 152      | 35             | 0.54                         | 1116.57         |        |                 |
| 153      | 35             | 0.50                         | 1117.07         | 0.995  | .0011           |
| 154      | 35             | 0.08                         | 1117.15         |        |                 |
| 155      | 34             | 0.53                         | 1117.68         |        |                 |
| 156      | 30             | 0.32                         | 1118.00         |        |                 |
| 157      | 30             | 0.27                         | 1118.27         |        |                 |
| 158      | 25             | 1.04                         | 1119.31         |        |                 |
| 159      | 25             | 0.43                         | 1119.74         |        |                 |
| 160      | 25             | 0.32                         | 1120.06         |        |                 |
| 161      | 25             | 0.31                         | 1120.37         |        |                 |
| 162      | 20             | 0.43                         | 1120.80         | 0.998  | .0007           |
| 163      | 20             | 0.33                         | 1121.13         |        |                 |
| 164      | 20             | 0.25                         | 1121.38         |        |                 |
| 165      | 20             | 0.22                         | 1121.60         |        |                 |
| 166      | 15             | 0.29                         | 1121.89         |        |                 |
| 167      | 15             | 0.23                         | 1122.12         |        |                 |
| 168      | 15             | 0.14                         | 1122.26         |        |                 |
| 169      | 10             | 0.22                         | 1122.48         |        |                 |
| 170      | 10             | 0.20                         | 1122.68         |        |                 |
| 171      | 10             | 0.05                         | 1122.73         | 1.0000 | .0003           |

Total Runoff (million cubic feet)

|         |              |
|---------|--------------|
| 1943/44 | 161.38       |
| 1944/45 | 2.87         |
| 1945/46 | 111.65       |
| 1946/47 | 121.37       |
| 1947/48 | 152.76       |
| 1948/49 | 240.87       |
| 1949/50 | 309.77       |
| 1950/51 | 5.05         |
| 1951/52 | <u>17.03</u> |
|         | 1122.75      |

Gamams I - Recorded Floods in Chronological Order.

| Season  | Runoff No | Amount of Flow<br>(million cu.ft.) | Peak Intensity.<br>(cusecs) |
|---------|-----------|------------------------------------|-----------------------------|
| 1942/43 | 1         | 8.86                               | 1050                        |
|         | 2         | 3.00                               | 520                         |
|         | 3         | 0.40                               | 17                          |
|         | 4         | 1.34                               | 145                         |
|         | 5         | 0.31                               | 64                          |
|         | 6         | 1.65                               | 280                         |
|         | 7         | 0.39                               | 57                          |
|         | 8         | 14.28                              | 1650                        |
|         | 9         | 5.79                               | 600                         |
|         | 10        | 33.19                              | 6100                        |
|         | 11        | <u>10.61</u>                       | 1900                        |
|         |           | 79.82                              |                             |
| 1943/44 | 1         | 5.02                               | 1060                        |
|         | 2         | 5.17                               | 1150                        |
|         | 3         | 9.77                               | 2800                        |
|         | 4         | 1.80                               | 164                         |
|         | 5         | 6.21                               | 1500                        |
|         | 6         | 22.90                              | 3090                        |
|         | 7         | 22.50                              | 1070                        |
|         | 8         | 7.34                               | 1450                        |
|         | 9         | 1.97                               | 190                         |
|         | 10        | .58                                | 38                          |
|         | 11        | 9.37                               | 800                         |
|         | 12        | 5.02                               | 200                         |
|         | 13        | 4.36                               | 250                         |
|         | 14        | 1.35                               | 49                          |
|         | 15        | 3.35                               | 447                         |
|         | 16        | 4.99                               | 430                         |
|         | 17        | 1.50                               | 257                         |
|         | 18        | 1.93                               | 103                         |
|         | 19        | <u>.54</u>                         | 85                          |
|         |           | 115.67                             |                             |



Gamams I - Recorded Floods in Chronological Order.

| Season  | Runoff No | Amount of Flow<br>(million cu.ft.) | Peak Intensity<br>(cusecs) |
|---------|-----------|------------------------------------|----------------------------|
| 1946/47 | 1         | 2.34                               | 650                        |
|         | 2         | 3.82                               | 375                        |
|         | 3         | 1.63                               | 240                        |
|         | 4         | 0.94                               | 125                        |
|         | 5         | 9.81                               | 1275                       |
|         | 6         | 0.66                               | 20                         |
|         | 7         | 0.98                               | 150                        |
|         | 8         | 1.80                               | 200                        |
|         | 9         | 0.92                               | 300                        |
|         | 10        | 2.12                               | 450                        |
|         | 11        | 4.88                               | 1250                       |
|         | 12        | 2.69                               | 650                        |
|         | 13        | 8.86                               | 1900                       |
|         | 14        | 1.52                               | 300                        |
|         | 15        | 1.46                               | 170                        |
|         | 16        | 15.83                              | 1300                       |
|         | 17        | 14.28                              | 2200                       |
|         | 18        | 6.98                               | 1150                       |
|         | 19        | 2.38                               | 600                        |
|         | 20        | 4.25                               | 500                        |
|         | 21        | 6.90                               | 1400                       |
|         | 22        | 0.94                               | 100                        |
|         | 23        | <u>4.81</u>                        | 1450                       |
|         |           | 100.80                             |                            |

Gamams I - Recorded Floods in Chronological Order.

| Season  | Runoff No | Amount of Flow<br>(million cu.ft.) | Peak Intensity<br>(cusecs) |
|---------|-----------|------------------------------------|----------------------------|
| 1947/48 | 1         | 4.40                               | 1320                       |
|         | 2         | 1.96                               | 350                        |
|         | 3         | 4.71                               | 1100                       |
|         | 4         | 16.75                              | 1400                       |
|         | 5         | 2.28                               | 500                        |
|         | 6         | 3.46                               | 430                        |
|         | 7         | 0.39                               | 50                         |
|         | 8         | 4.19                               | 850                        |
|         | 9         | 0.71                               | 200                        |
|         | 10        | 1.60                               | 300                        |
|         | 11        | 1.41                               | 200                        |
|         | 12        | 4.11                               | 1050                       |
|         | 13        | 1.60                               | 200                        |
|         | 14        | 15.05                              | 1800                       |
|         | 15        | 5.08                               | 700                        |
|         | 16        | <u>1.10</u>                        | 150                        |
|         |           | 68.80                              |                            |

Gamams I - Recorded Floods in Chronological Order.

| Season  | Runoff No | Amount of Flow | Peak Intensity |
|---------|-----------|----------------|----------------|
| 1948/49 | 1         | 11.75          | 2650           |
|         | 2         | 4.84           | 600            |
|         | 3         | 0.44           | 100            |
|         | 4         | 21.39          | 3750           |
|         | 5         | 0.31           | 80             |
|         | 6         | 5.50           | 870            |
|         | 7         | 0.52           | 325            |
|         | 8         | 8.77           | 1825           |
|         | 9         | 1.55           | 400            |

|    |             |      |
|----|-------------|------|
| 10 | 1.15        | 360  |
| 11 | 3.14        | 260  |
| 12 | 5.31        | 850  |
| 13 | 2.36        | 1000 |
| 14 | 2.15        | 450  |
| 15 | 0.92        | 100  |
| 16 | 1.05        | 125  |
| 17 | 7.44        | 550  |
| 18 | 3.64        | 440  |
| 19 | 2.49        | 300  |
| 20 | 6.65        | 800  |
| 21 | 5.55        | 450  |
| 22 | 1.49        | 240  |
| 23 | <u>4.11</u> | 800  |
|    | 102.52      |      |

Gamams I - Recorded Floods in Chronological Order.

| Season  | Runoff No | Amount of Flow<br>(million cu. ft.) | Peak Intensity.<br>(cusecs) |
|---------|-----------|-------------------------------------|-----------------------------|
| 1949/50 | 1         | 4.58                                | 400                         |
|         | 2         | 8.19                                | 1930                        |
|         | 3         | 9.24                                | 1250                        |
|         | 4         | 3.59                                | 950                         |
|         | 5         | 0.47                                | 100                         |
|         | 6         | 2.41                                | 220                         |
|         | 7         | 5.55                                | 500                         |
|         | 8         | 4.42                                | 1100                        |
|         | 9         | 0.21                                | 100                         |
|         | 10        | 0.21                                | 100                         |
|         | 11        | 1.94                                | 200                         |
|         | 12        | 2.38                                | 260                         |

|    |             |      |
|----|-------------|------|
| 13 | 0.26        | 160  |
| 14 | 2.99        | 600  |
| 15 | 2.28        | 600  |
| 16 | 4.82        | 300  |
| 17 | 2.88        | 450  |
| 18 | 0.10        | 50   |
| 19 | 8.90        | 1100 |
| 20 | 2.46        | 150  |
| 21 | 0.92        | 100  |
| 22 | 0.99        | 250  |
| 23 | 0.73        | 50   |
| 24 | 1.89        | 275  |
| 25 | 8.69        | 1700 |
| 26 | 0.99        | 300  |
| 27 | 0.81        | 100  |
| 28 | 0.52        | 60   |
| 29 | 0.50        | 60   |
| 30 | 0.40        | 60   |
| 31 | 9.62        | 600  |
| 32 | 0.76        | 100  |
| 33 | 1.44        | 200  |
| 34 | 7.59        | 2150 |
| 35 | 38.14       | 2650 |
| 36 | 2.62        | 240  |
| 37 | <u>6.34</u> | 800  |
|    | 150.83      |      |

Gamams I - Recorded Floods in Chronological Order.

| Season  | Runoff No | Amount of Flow<br>(million cu.ft.) | Peak Intensity.<br>(cusecs) |
|---------|-----------|------------------------------------|-----------------------------|
| 1950/51 | 1         | 1.02                               | 150                         |
|         | 2         | 1.65                               | 425                         |
|         | 3         | 1.28                               | 250                         |
|         | 4         | <u>.63</u>                         | 125                         |
|         |           | 4.58                               |                             |

Gamams I - Recorded Floods in Chronological Order.

| Season  | Runoff No | Amount of Flow<br>(million cu.ft.) | Peak Intensity<br>(cusecs) |
|---------|-----------|------------------------------------|----------------------------|
| 1951/52 | 1         | 1.47                               | 330                        |
|         | 2         | 3.33                               | 470                        |
|         | 3         | 1.87                               | 150                        |
|         | 4         | 1.80                               | 240                        |
|         | 5         | 1.44                               | 300                        |
|         | 6         | <u>0.25</u>                        | 140                        |
|         |           | 10.16                              |                            |

Gamams I - Recorded Floods in Order of Magnitude.

$$53 \text{ sq.miles.} \quad Q_1 = 3600 \times 53^{0.45} = 21,500$$

| Flood No. | Peak<br>cusecs | Amount<br>mil.<br>cu.<br>ft. | $\frac{Q}{Q_1}$ | $\Sigma$ Amount | $F_R$ |
|-----------|----------------|------------------------------|-----------------|-----------------|-------|
| 1         | 7400           | 52.46                        | 0.344           | 52.46           | 0.074 |
| 2         | 6100           | 33.19                        | 0.285           | 85.65           | 0.121 |
| 3         | 3750           | 21.39                        | 0.174           | 107.04          | 0.152 |
| 4         | 3090           | 22.90                        | 0.144           | 129.94          | 0.184 |
| 5         | 2800           | 9.77                         | 0.130           | 139.71          | 0.198 |
| 6         | 2650           | 38.14                        | 0.123           | 177.85          | 0.252 |

| Flood No | Peak<br>cusecs | Amount<br>mil.<br>cu.<br>ft. | $\frac{Q}{Q_1}$ | ΣAmount | $F_R$ |
|----------|----------------|------------------------------|-----------------|---------|-------|
| 7        | 2650           | 11.75                        | 0.123           | 189.60  | 0.269 |
| 8        | 2200           | 14.28                        | 0.102           | 203.88  | 0.288 |
| 9        | 2150           | 7.59                         | 0.100           | 211.47  | 0.299 |
| 10       | 1930           | 8.19                         | 0.0898          | 219.66  | 0.311 |
| 11       | 1900           | 10.61                        | 0.0883          | 230.27  | 0.326 |
| 12       | 1900           | 8.86                         | 0.0883          | 239.13  | 0.339 |
| 13       | 1825           | 8.77                         | 0.0849          | 247.90  | 0.351 |
| 14       | 1800           | 15.05                        | 0.0837          | 262.95  | 0.372 |
| 15       | 1700           | 8.69                         | 0.0791          | 271.64  | 0.385 |
| 16       | 1650           | 14.28                        | 0.0768          | 285.92  | 0.405 |
| 17       | 1500           | 6.21                         | 0.0698          | 292.13  | 0.415 |
| 18       | 1450           | 7.34                         | 0.0675          | 299.47  | 0.424 |
| 19       | 1450           | 4.81                         | 0.0675          | 304.28  | 0.432 |
| 20       | 1400           | 16.75                        | 0.0652          | 321.03  | 0.455 |
| 21       | 1400           | 6.90                         | 0.0652          | 327.93  | 0.465 |
| 22       | 1320           | 4.40                         | 0.0615          | 332.33  | 0.471 |
| 23       | 1300           | 15.83                        | 0.0605          | 348.16  | 0.495 |
| 24       | 1275           | 9.81                         | 0.0593          | 357.97  | 0.507 |
| 25       | 1250           | 9.24                         | 0.0582          | 367.21  | 0.520 |
| 26       | 1250           | 4.88                         | 0.0582          | 372.09  | 0.527 |
| 27       | 1150           | 6.98                         | 0.0535          | 379.07  | 0.537 |
| 28       | 1150           | 5.17                         | 0.0535          | 384.24  | 0.543 |
| 29       | 1100           | 8.90                         | 0.0512          | 393.14  | 0.557 |
| 30       | 1100           | 4.71                         | 0.0512          | 397.85  | 0.563 |
| 31       | 1100           | 4.42                         | 0.0512          | 402.27  | 0.570 |
| 32       | 1070           | 22.50                        | 0.0498          | 424.77  | 0.601 |
| 33       | 1060           | 5.02                         | 0.0493          | 429.79  | 0.609 |
| 34       | 1050           | 8.86                         | 0.0489          | 438.65  | 0.622 |
| 35       | 1050           | 4.11                         | 0.0489          | 442.76  | 0.628 |

| Flood No | Peak<br>cusecs | Amount<br>mil.<br>cu.<br>ft. | $Q/Q_1$ | $\Sigma$ Amount | $F_R$ |
|----------|----------------|------------------------------|---------|-----------------|-------|
| 36       | 1000           | 2.36                         | 0.0465  | 445.12          | 0.630 |
| 37       | 950            | 3.59                         | 0.0442  | 448.71          | 0.636 |
| 38       | 925            | 6.92                         | 0.0430  | 455.63          |       |
| 39       | 870            | 5.50                         | 0.0405  | 461.13          |       |
| 40       | 850            | 5.31                         | 0.0395  | 466.44          | 0.660 |
| 41       | 850            | 4.19                         | 0.0395  | 470.63          |       |
| 42       | 800            | 9.37                         | 0.0372  | 480.00          |       |
| 43       | 800            | 6.65                         | 0.0372  | 486.65          |       |
| 44       | 800            | 6.34                         | 0.0372  | 492.99          |       |
| 45       | 800            | 4.11                         | 0.0372  | 497.10          |       |
| 46       | 700            | 5.08                         | 0.0326  | 502.18          |       |
| 47       | 650            | 3.66                         | 0.0302  | 505.84          |       |
| 48       | 650            | 2.69                         | 0.0302  | 508.53          |       |
| 49       | 650            | 2.34                         | 0.0302  | 510.87          |       |
| 50       | 600            | 9.62                         | 0.0279  | 520.49          | 0.736 |
| 51       | 600            | 5.79                         | 0.0279  | 526.28          | 2.    |
| 52       | 600            | 4.84                         | 0.0279  | 531.12          |       |
| 53       | 600            | 2.99                         | 0.0279  | 534.11          |       |
| 54       | 600            | 2.38                         | 0.0279  | 536.49          |       |
| 55       | 600            | 2.28                         | 0.0279  | 538.77          |       |
| 56       | 550            | 7.44                         | 0.0256  | 546.21          |       |
| 57       | 520            | 3.00                         | 0.0242  | 549.21          |       |
| 58       | 520            | 2.50                         | 0.0242  | 551.71          |       |
| 59       | 500            | 4.25                         | 0.0233  | 555.96          |       |
| 60       | 500            | 5.55                         | 0.0233  | 561.51          | 0.795 |
| 61       | 500            | 2.28                         | 0.0233  | 563.79          |       |
| 62       | 470            | 3.33                         | 0.0219  | 567.12          |       |
| 63       | 450            | 5.55                         | 0.0210  | 572.67          |       |
| 64       | 450            | 2.88                         | 0.0210  | 575.55          |       |
| 65       | 450            | 2.15                         | 0.0210  | 577.70          |       |

| Flood No | Peak<br>cusecs | Amount<br>mil.<br>cu.<br>ft. | $\frac{Q}{Q_1}$ | $\Sigma$ Amount | $F_R$ |
|----------|----------------|------------------------------|-----------------|-----------------|-------|
| 66       | 450            | 2.12                         | 0.0210          | 579.82          |       |
| 67       | 447            | 3.35                         | 0.0208          | 583.17          |       |
| 68       | 440            | 3.64                         | 0.0205          | 586.81          |       |
| 69       | 430            | 4.99                         | 0.0200          | 591.80          |       |
| 70       | 430            | 3.46                         | 0.0200          | 595.26          | 0.843 |
| 71       | 425            | 1.65                         | 0.0198          | 596.91          |       |
| 72       | 400            | 4.58                         | 0.0186          | 601.49          |       |
| 73       | 400            | 1.55                         | 0.0186          | 603.04          |       |
| 74       | 375            | 3.82                         | 0.0175          | 606.86          |       |
| 75       | 360            | 1.15                         | 0.0168          | 608.01          |       |
| 76       | 350            | 1.96                         | 0.0163          | 609.97          |       |
| 77       | 330            | 1.47                         | 0.0154          | 611.44          |       |
| 78       | 325            | 0.52                         | 0.0151          | 611.96          |       |
| 79       | 305            | 2.11                         | 0.0142          | 614.07          |       |
| 80       | 300            | 4.82                         | 0.0140          | 618.89          | 0.876 |
| 81       | 300            | 1.52                         | 0.0140          | 620.41          |       |
| 82       | 300            | 2.49                         | 0.0140          | 622.90          |       |
| 83       | 300            | 1.60                         | 0.0140          | 624.50          |       |
| 84       | 300            | 1.44                         | 0.0140          | 625.94          |       |
| 85       | 300            | 0.99                         | 0.0140          | 626.93          |       |
| 86       | 300            | 0.92                         | 0.0140          | 627.85          |       |
| 87       | 280            | 1.65                         | 0.0130          | 629.50          |       |
| 88       | 275            | 1.89                         | 0.0128          | 631.39          |       |
| 89       | 270            | 1.34                         | 0.0126          | 632.73          |       |
| 90       | 260            | 3.14                         | 0.0121          | 635.87          | 0.900 |
| 91       | 260            | 2.38                         | 0.0121          | 638.25          |       |
| 92       | 257            | 1.50                         | 0.0120          | 639.75          |       |
| 93       | 250            | 4.36                         | 0.0116          | 644.11          |       |



| Flood No | Peak<br>cusecs | Amount<br>mil.<br>cu.<br>ft. | $\frac{Q}{Q_1}$ | $\Sigma$ Amount | $F_R$ |
|----------|----------------|------------------------------|-----------------|-----------------|-------|
| 94       | 250            | 1.28                         | 0.0116          | 645.39          |       |
| 95       | 250            | 0.99                         | 0.0116          | 646.38          |       |
| 96       | 240            | 2.62                         | 0.0112          | 649.00          |       |
| 97       | 240            | 1.80                         | 0.0112          | 650.80          |       |
| 98       | 240            | 1.63                         | 0.0112          | 652.43          |       |
| 99       | 240            | 1.49                         | 0.0112          | 653.92          |       |
| 100      | 220            | 2.41                         | 0.0102          | 656.33          | 0.928 |
| 101      | 200            | 5.02                         | 0.0093          | 661.35          |       |
| 102      | 200            | 1.94                         | 0.0093          | 663.29          |       |
| 103      | 200            | 1.80                         | 0.0093          | 665.09          |       |
| 104      | 200            | 1.60                         | 0.0093          | 666.69          |       |
| 105      | 200            | 1.44                         | 0.0093          | 668.13          |       |
| 106      | 200            | 1.41                         | 0.0093          | 669.54          |       |
| 107      | 200            | 0.71                         | 0.0093          | 670.25          |       |
| 108      | 190            | 1.97                         | 0.0088          | 672.22          |       |
| 109      | 180            | 0.71                         | 0.0084          | 672.93          |       |
| 110      | 170            | 1.46                         | 0.0079          | 674.39          | 0.955 |
| 111      | 164            | 1.80                         | 0.0076          | 676.19          |       |
| 112      | 160            | 0.26                         | 0.0074          | 676.45          |       |
| 113      | 150            | 2.46                         | 0.0070          | 678.91          |       |
| 114      | 150            | 1.87                         | 0.0070          | 680.78          |       |
| 115      | 150            | 1.10                         | 0.0070          | 681.88          |       |
| 116      | 150            | 1.02                         | 0.0070          | 682.90          |       |
| 117      | 150            | 0.98                         | 0.0070          | 683.88          |       |
| 118      | 145            | 1.34                         | 0.0067          | 685.22          |       |
| 119      | 140            | 0.25                         | 0.0065          | 685.47          |       |
| 120      | 125            | 0.94                         | 0.0058          | 686.41          | 0.971 |
| 121      | 125            | 0.63                         | 0.0058          | 687.04          |       |
| 122      | 125            | 1.05                         | 0.0058          | 688.09          |       |

| Flood No | Peak<br>cusecs | Amount<br>mil.<br>cu.<br>ft. | $\frac{Q}{Q_1}$ | $\Sigma$ Amount | $F_R$ |
|----------|----------------|------------------------------|-----------------|-----------------|-------|
| 123      | 118            | 1.09                         | 0.0055          | 689.18          |       |
| 124      | 118            | 0.91                         | 0.0055          | 690.09          |       |
| 125      | 103            | 1.93                         | 0.0048          | 692.02          |       |
| 126      | 100            | 0.92                         | 0.0046          | 692.94          |       |
| 127      | 100            | 0.92                         | 0.0046          | 693.86          |       |
| 128      | 100            | 0.81                         | 0.0046          | 694.67          |       |
| 129      | 100            | 0.76                         | 0.0046          | 695.43          |       |
| 130      | 100            | 0.47                         | 0.0046          | 695.90          | 0.985 |
| 131      | 100            | 0.21                         | 0.0046          | 696.11          |       |
| 132      | 100            | 0.21                         | 0.0046          | 696.32          |       |
| 133      | 100            | 0.44                         | 0.0046          | 696.76          |       |
| 134      | 100            | 0.94                         | 0.0046          | 697.70          |       |
| 135      | 85             | 0.75                         | 0.0040          | 698.45          |       |
| 136      | 85             | 0.54                         | 0.0040          | 698.99          |       |
| 137      | 80             | 0.31                         | 0.0040          | 699.30          |       |
| 138      | 68             | 0.10                         | 0.0032          | 699.40          |       |
| 139      | 64             | 0.31                         | 0.0030          | 699.71          |       |
| 140      | 60             | 0.52                         | 0.0028          | 700.23          | 0.991 |
| 141      | 60             | 0.50                         | 0.0028          | 700.73          |       |
| 142      | 60             | 0.40                         | 0.0028          | 701.13          |       |
| 143      | 57             | 0.39                         | 0.0026          | 701.52          |       |
| 144      | 50             | 0.73                         | 0.0023          | 702.25          |       |
| 145      | 50             | 0.39                         | 0.0023          | 702.64          |       |
| 146      | 50             | 0.10                         | 0.0023          | 702.74          |       |
| 147      | 49             | 1.35                         | 0.0023          | 704.09          |       |
| 148      | 40             | 0.31                         | 0.0019          | 704.40          |       |
| 149      | 38             | 0.58                         | 0.0018          | 704.98          |       |
| 150      | 35             | 0.30                         | 0.0016          | 705.28          | 0.998 |
| 151      | 20             | 0.66                         | 0.0009          | 705.94          |       |
| 152      | 17             | 0.40                         | 0.0008          | 706.34          |       |
| 153      | 10             | 0.06                         | 0.0005          | 706.40          |       |
| 154      | 10             | 0.05                         | 0.0005          | 706.45          | 1.000 |

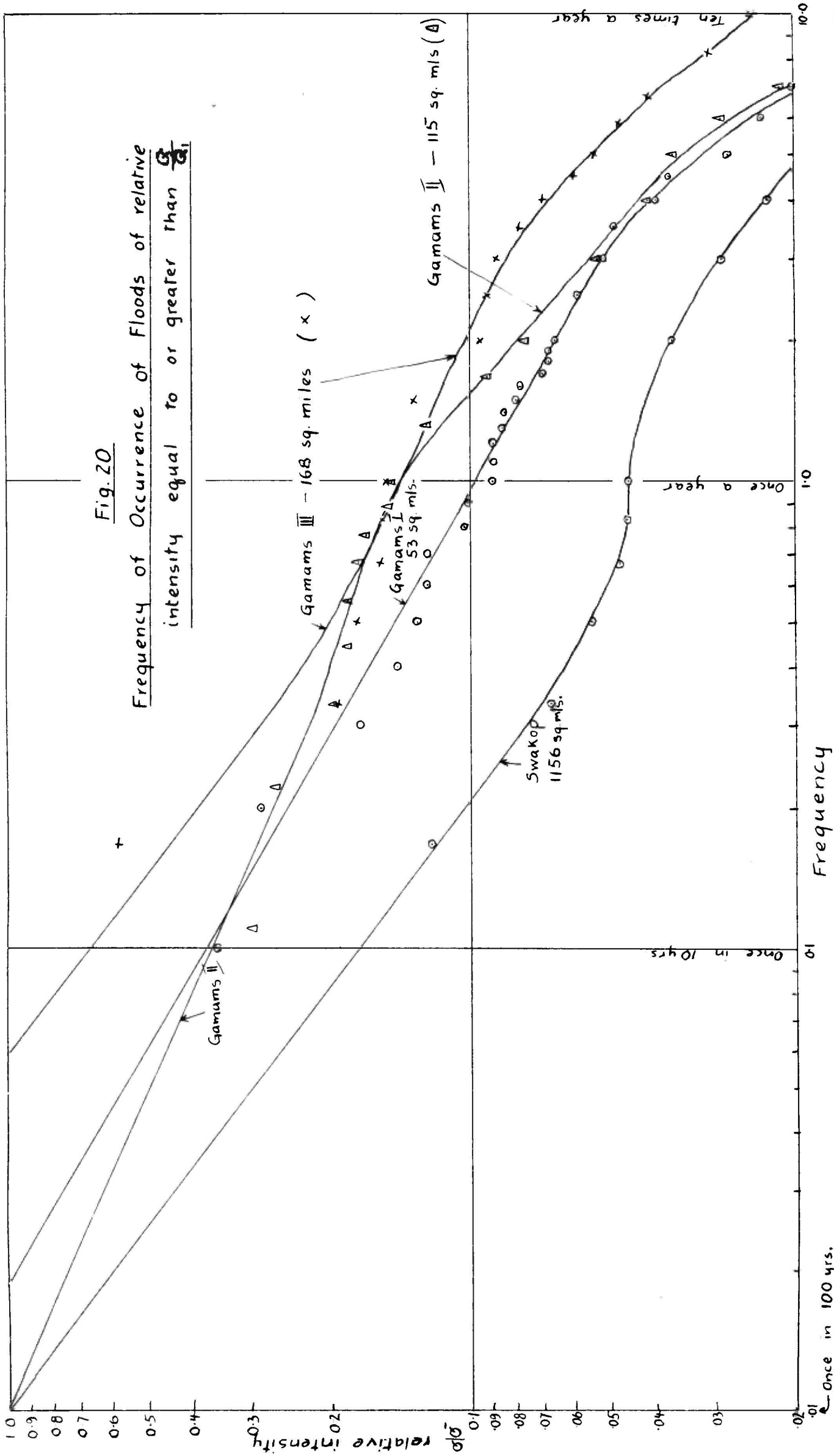
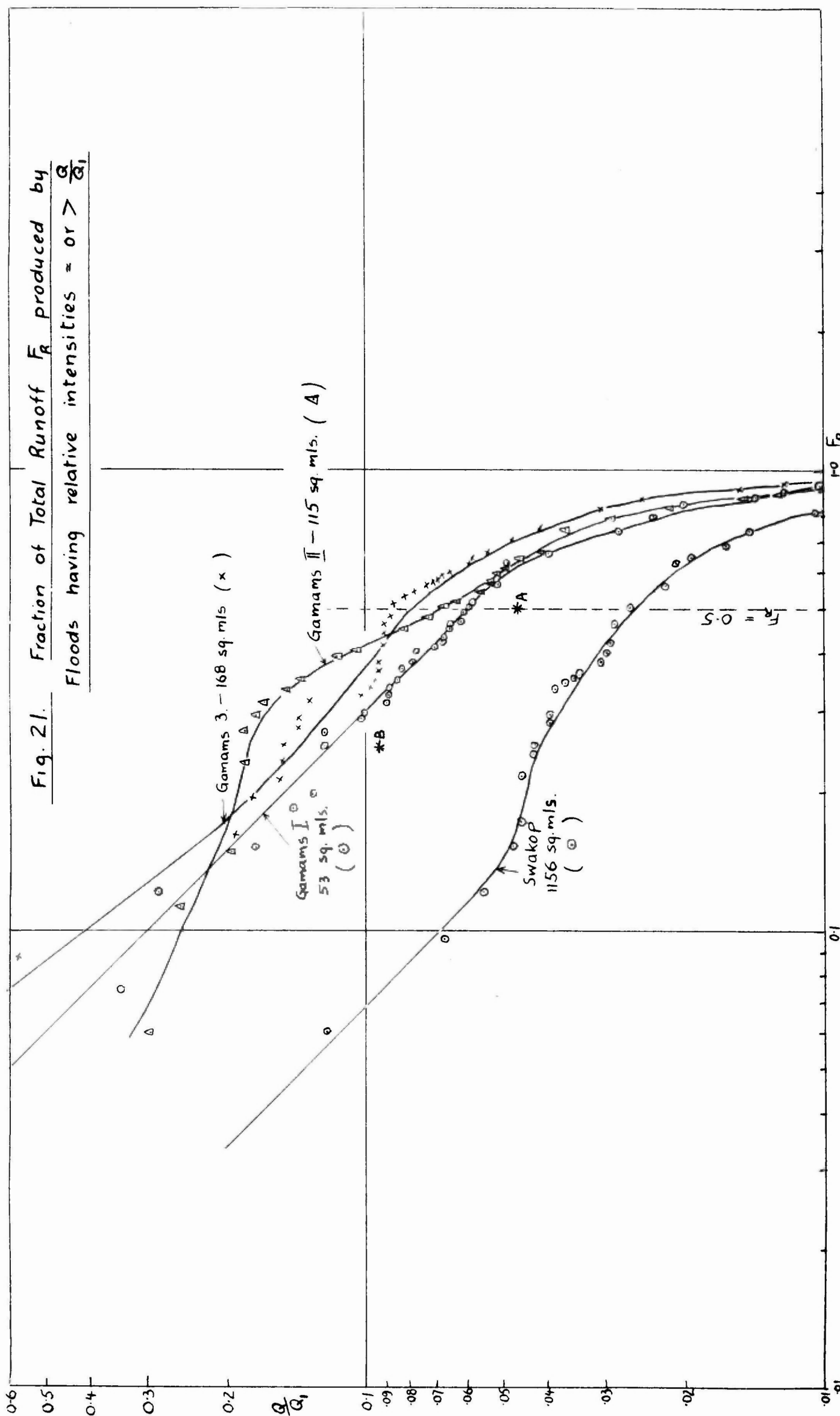


Fig. 21. Fraction of Total Runoff  $F_R$  produced by  
Floods having relative intensities  $\geq$  or  $> \frac{Q}{Q_1}$



An examination of the data given will reveal that just as average annual runoff varies greatly from catchment to catchment, so does the frequency with which floods of various relative intensity occur. Certain catchments yield floods of appreciable relative intensity more frequently than others, resulting in a corresponding difference in total runoff.

The following table of relative intensities corresponding to  $F_R = 0.5$  was derived from Fig. 21:-

| Catchment Area | Average Annual Runoff (inches)<br>(page 27 ) | $\frac{Q}{Q_1}$ for $F_R = 0.5$ |
|----------------|--|---------------------------------|
| Swakop River   | .209   | .025                            |
| Gamams III     | .673   | .08                             |
| Gamams II      | .528   | .07                             |
| Gamams I       | .594   | .06                             |

It will be seen that in the instances on record  $\frac{Q}{Q_1}$  for  $F_R = 0.5$  is roughly proportional to the average annual runoff. This may be used as a guide where information is available concerning total runoff but no gauging of flood intensities has been undertaken.

In sand transportation and sedimentation problems it will be convenient to consider a series of equivalent floods, all of the same peak intensity, instead of the full range of peak intensities occurring in practice. The author suggests that  $\frac{Q}{Q_1}$  for  $F_R = 0.5$  be accepted as the relative intensity of such equivalent floods, as half the total flow will consist of floods of smaller intensity and the other half of greater intensity

On page 59 of his publication, G.B. Williams<sup>11</sup> defines "probable maximum" floods as those which are

likely to occur not more often than once in 100 years.

He classifies floods into the following categories:-

| Category           | Interval at which floods can be expected to recur | Ratios of Intensities |
|--------------------|---|-----------------------|
| "Frequent"         | 3 to 5 years                                      | 0.50                  |
| "Unusual"          | 20 years  | 0.80                  |
| "Probable Maximum" | 100 years   | 1.00                  |
| "Catastrophic"     | -   | 1.5 to 2.5            |

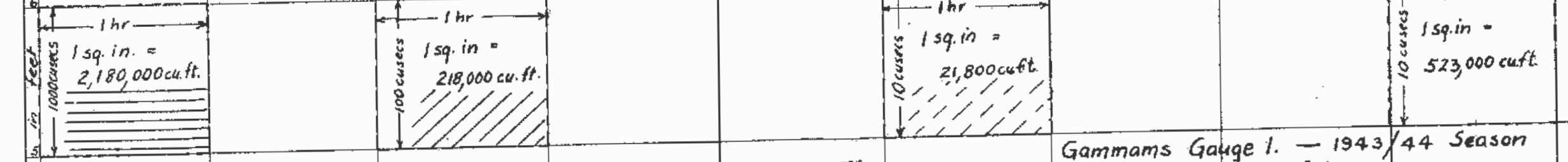
Fig. 19 indicates that the high floods observed in South West Africa during the past 20 to 50 years are, with one exception, still somewhat below the curve given by G.B.Williams' formula for the Rocky Mountains, and that the formula could be used as a practical guide in estimating probable maxima. The detailed analysis of the records of four river gauges plotted in Fig.20 gives the following information of the intervals between the occurrence of floods of various intensities.

| Relative Intensity<br>( $Q_1 = 1.0$ ) | Interval in Years |           |            |        |
|---------------------------------------|-------------------|-----------|------------|--------|
|                                       | Gamams I          | Gamams II | Gamams III | Swakop |
| 0.50                                  | 17                | 24        | 7          | 40     |
| 0.80                                  | 37                | 63        | 13         | 75     |
| 1.00                                  | 50                | 100       | 17         | 100    |

It should be noted that all these values are extrapolated and therefore somewhat indefinite. There is every indication that the curve for Gamams III has been deflected upwards by one flood of very high intensity and that in the course of time a curve more consistent with those of the other catchments will result. The following general conclusions can, however, be drawn:-

Fig. 22

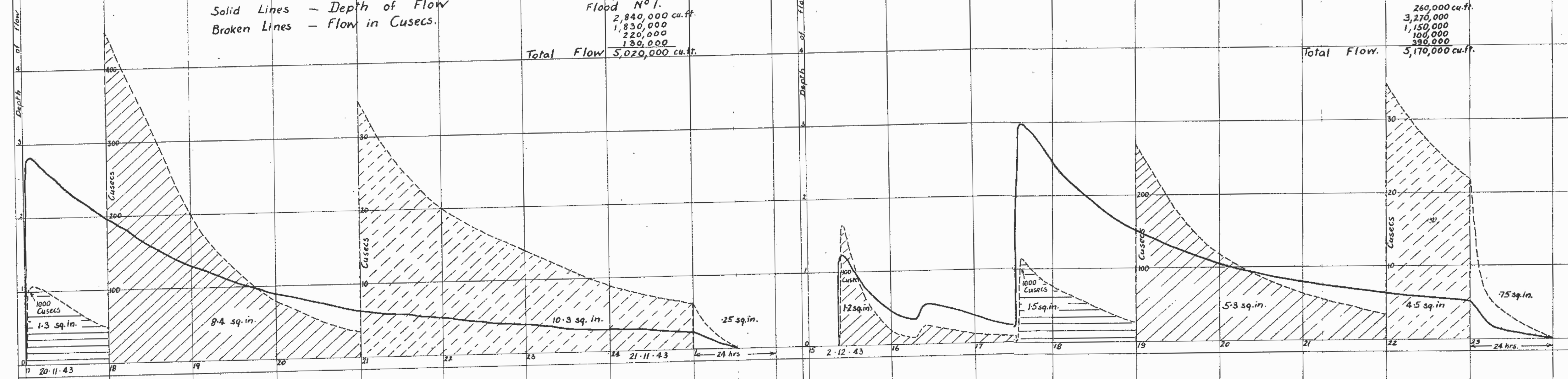
Scales



Solid Lines - Depth of Flow  
Broken Lines - Flow in Cusecs.

Gammams Gauge I. - 1943/44 Season  
Flood No 1.  
2,840,000 cu. ft.  
1,830,000  
220,000  
130,000  
Total Flow 5,020,000 cu. ft.

Gammams Gauge I. - 1943/44 Season  
Flood No 2.  
260,000 cu. ft.  
3,270,000  
1,150,000  
100,000  
390,000  
Total Flow 5,170,000 cu. ft.



Gammams Gauge I. - 1943/44 Season  
Flood No 3  
7,630,000 cu. ft.  
1,630,000  
510,000  
Total Flow 9,770,000 cu. ft.

Gammams Gauge I. - 1943/44 Season  
Flood No 4  
1,350,000 cu. ft.  
190,000  
260,000  
Total Flow 1,800,000 cu. ft.

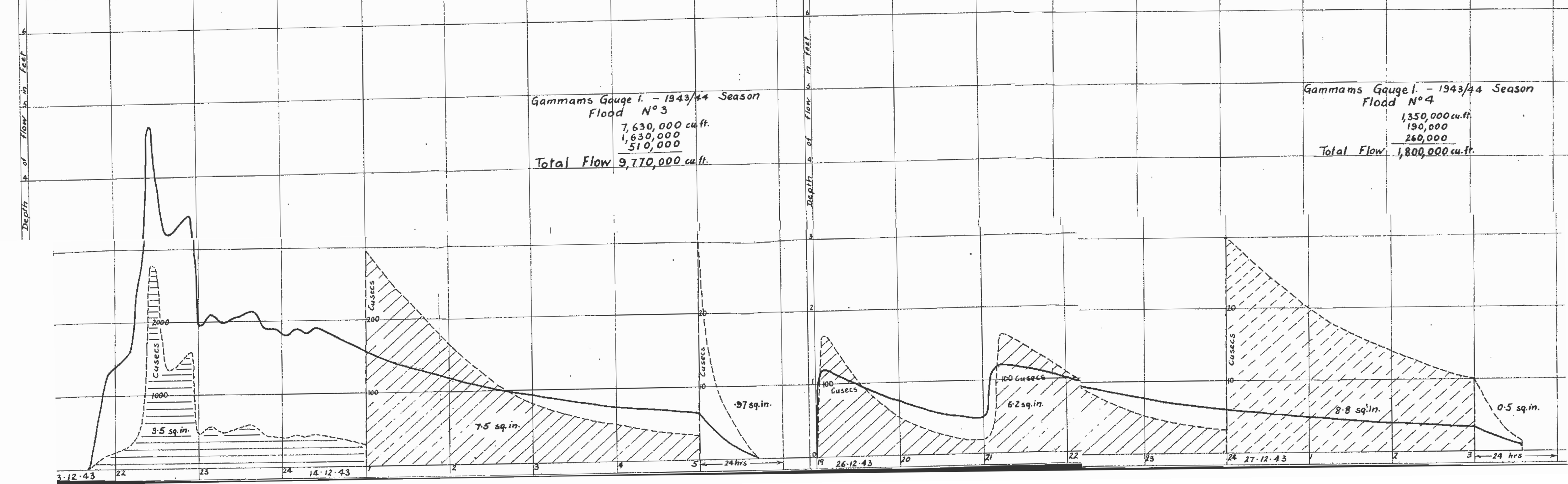
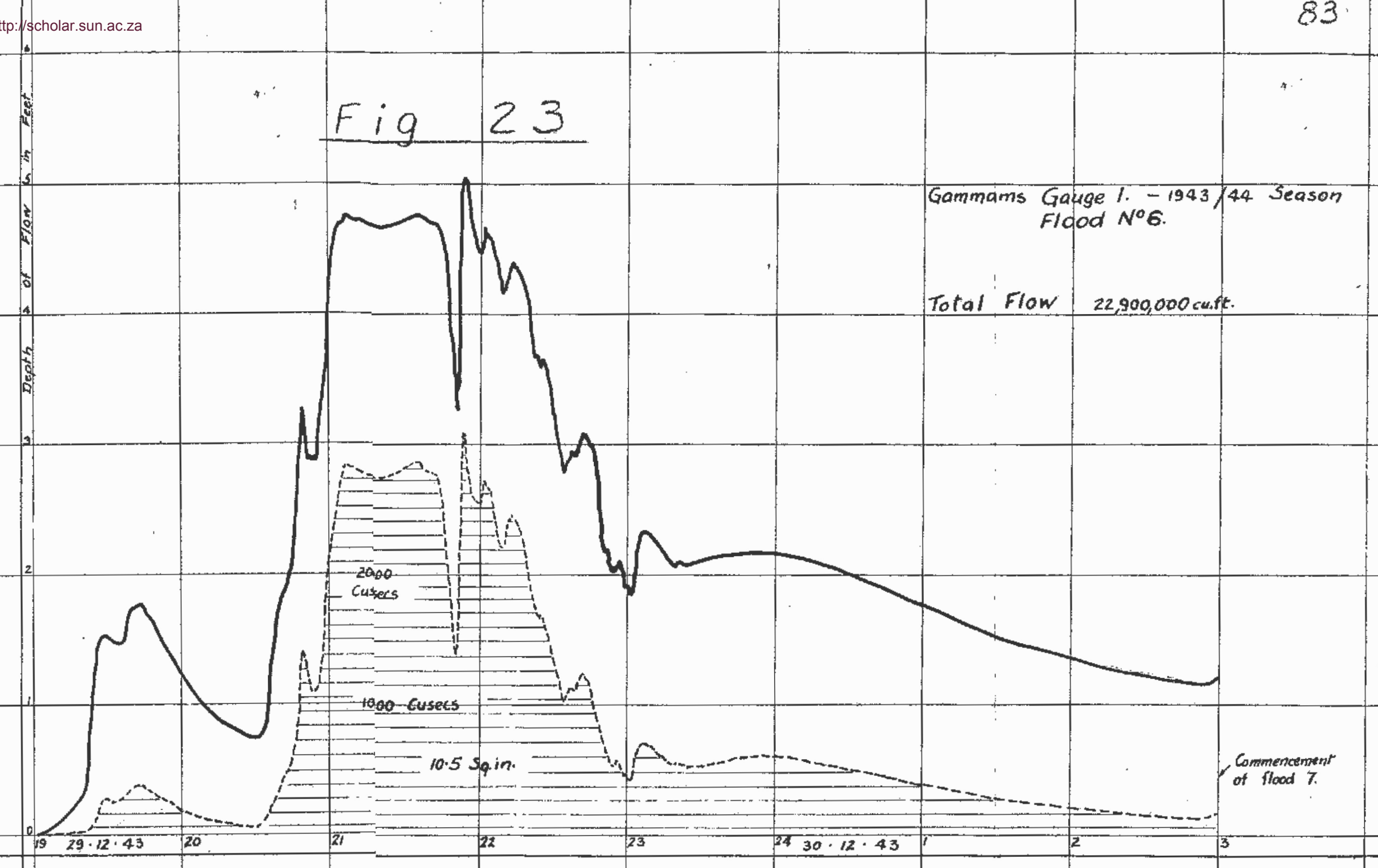
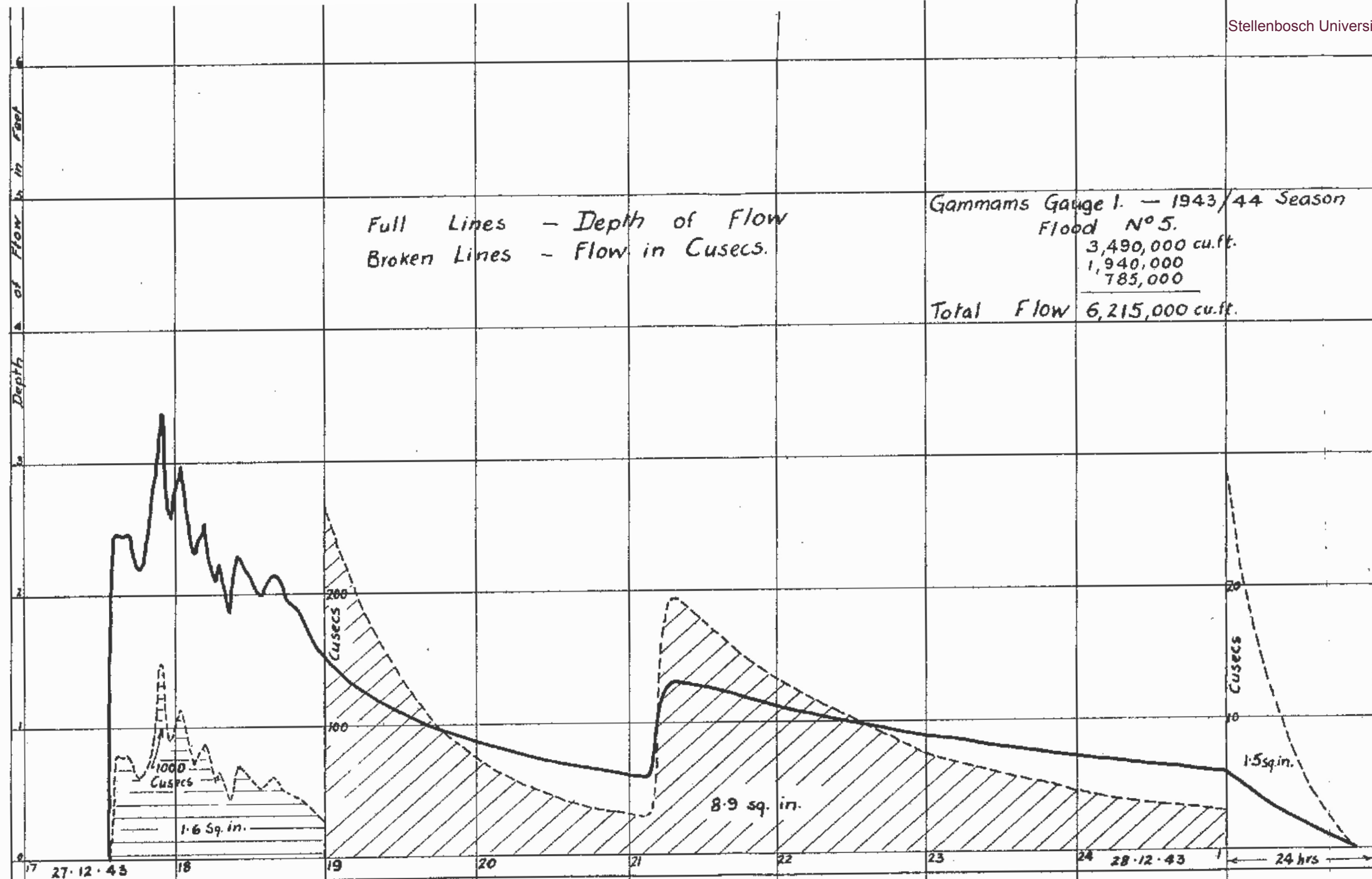


Fig 23

Full Lines - Depth of Flow  
Broken Lines - Flow in Cusecs.

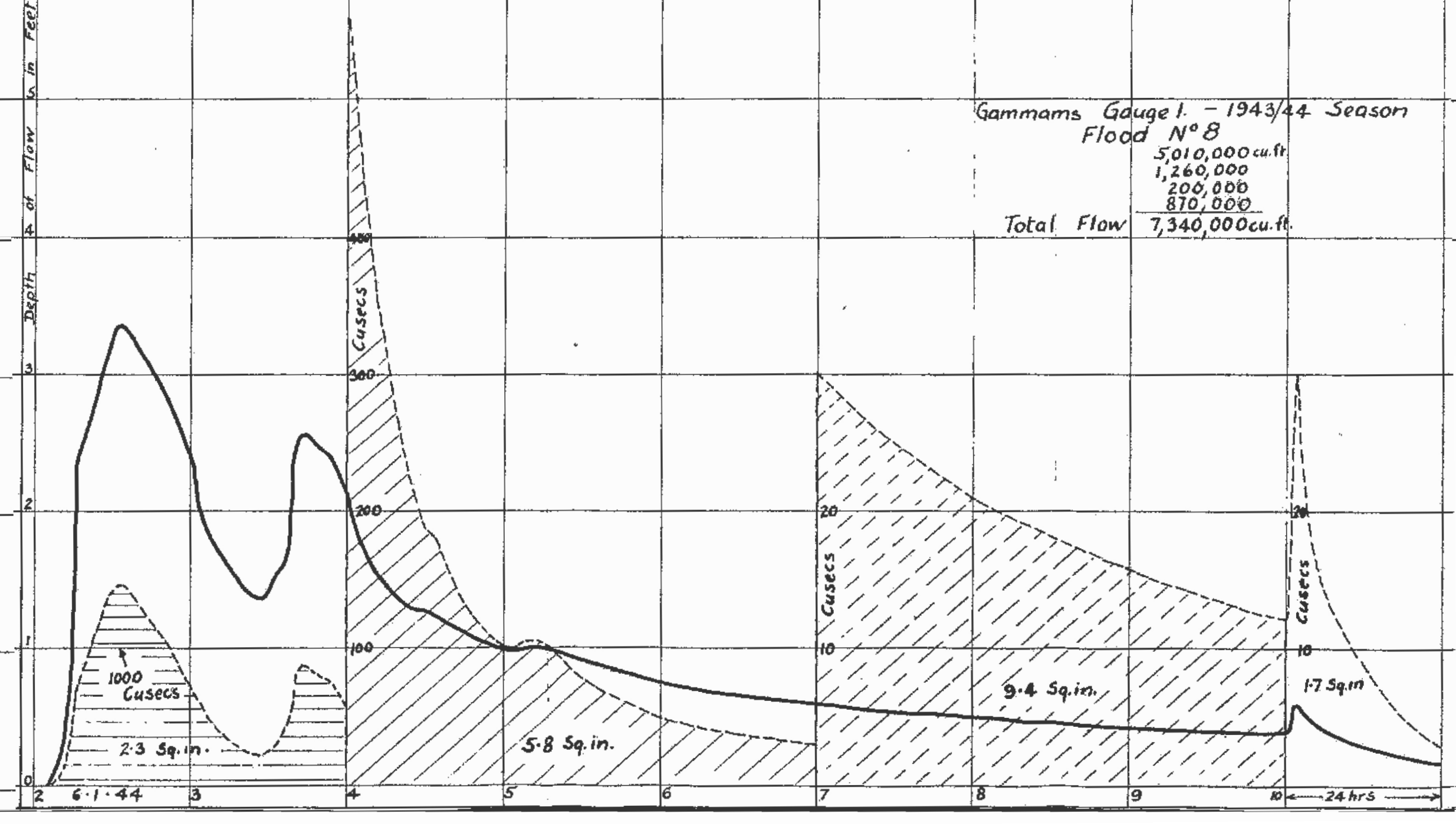
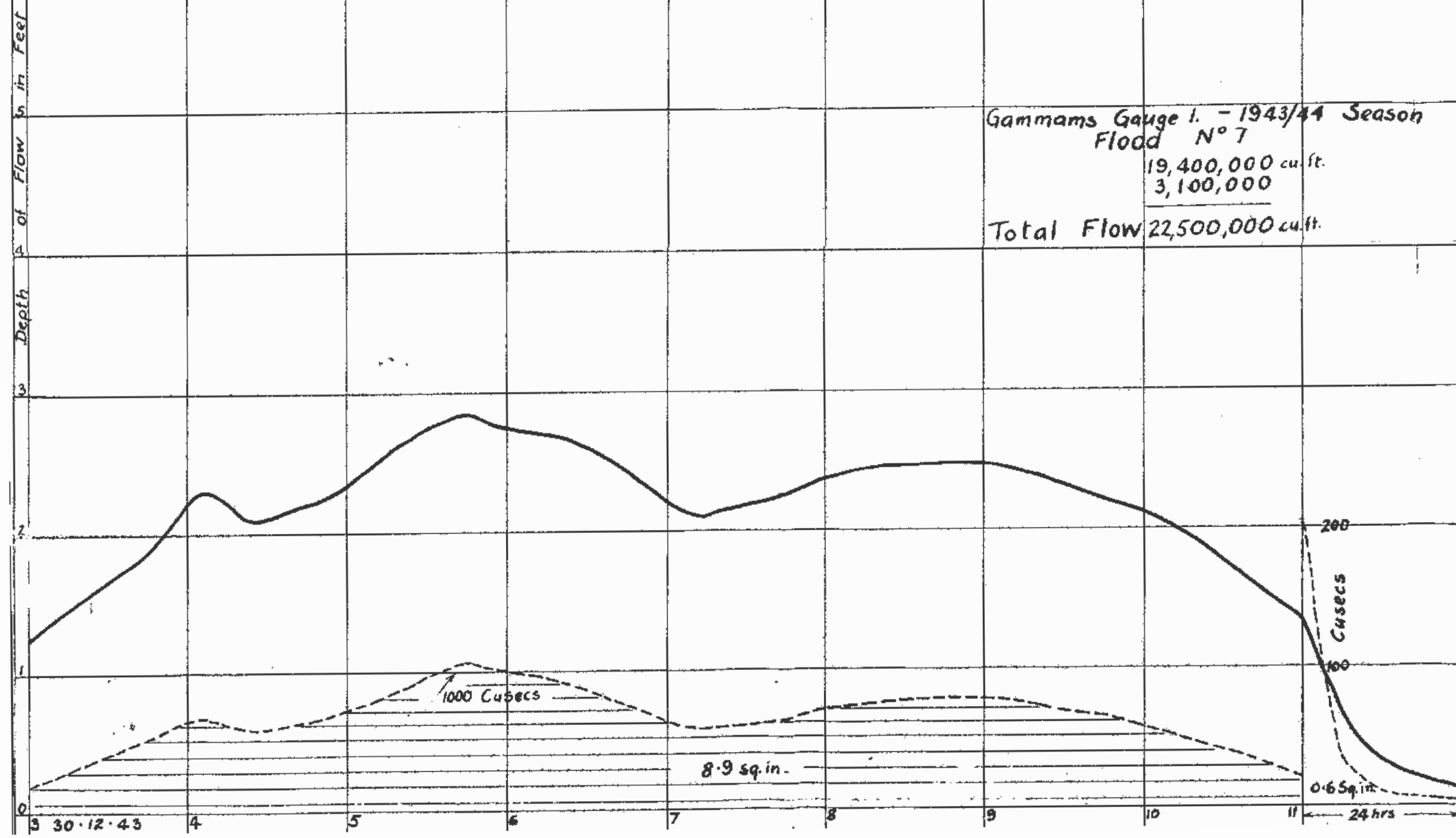
Gammams Gauge I. - 1943/44 Season  
Flood N° 5.  
3,490,000 cu.ft.  
1,940,000  
785,000  
Total Flow 6,215,000 cu.ft.

Gammams Gauge I. - 1943/44 Season  
Flood N° 6.  
Total Flow 22,900,000 cu.ft.



Gammams Gauge I. - 1943/44 Season  
Flood N° 7.  
19,400,000 cu.ft.  
3,100,000  
Total Flow 22,500,000 cu.ft.

Gammams Gauge I. - 1943/44 Season  
Flood N° 8.  
5,010,000 cu.ft.  
1,260,000  
200,000  
870,000  
Total Flow 7,340,000 cu.ft.





(5) Sediments transported by Floods.

In the sand storage dam at Aukeigas the total volume of sediments deposited in thirteen seasons amounted to 1,102,500 cu.ft. of which 152,300 cu.ft. are classified as silt and the remaining 950,200 cu.ft. as fine sand with some medium and coarse sand and gravel (See page 138). The volume of silt transported by the river by far exceeded 152,300 cu.ft. as the stage construction method was specially devised to minimise silt deposition and most of the silt was carried over the dam wall. The volume of fine sand with some medium and coarse sand and gravel (referred to collectively as material coarser than silt in the discussion which follows) transported by the river, also exceeded the volume which was deposited, as an appreciable volume of such material was carried over the dam wall by floods after the complete "sanding up" of stage 2 in the 1941/42 season. Smaller volumes were lost in this way in 1942/43 and 1943/44. The silt and material coarser than silt deposited in the dam amount to 835 and 5220 cubic feet per square mile per annum and are equivalent to a denudation of the catchment of .00036 and .00224 inches respectively.

The silt survey of the Oruaondo Dam (page 136) indicates a total denudation of .0153 inches per annum of which .0053 inches are in the form of sediment coarser than silt. In the Spitskoppies Dam the total denudation was .0185 inches per annum. Denudation in the small catchment Oruaondo and Spitskoppies was abnormally high due to overgrazing of the areas after construction of the dams. The sedimentation in the sand storage dam at Aukeigas on the

other hand represents only a portion of the denudation, as already explained. Denudation which may be expected under average conditions in the region concerned is estimated as follows:-

Total sediments 30,000 cubic feet per square mile per annum or .00129 inches per annum denudation.

Material coarser than silt 7,500 cubic feet per square mile per annum or .00032 inches per annum denudation.

The average grain size of sediments coarser than silt deposited in stages 1 to 4 of the sand storage dam at Aukeigas is 0.01 inches (see table on page 143 ).

#### (6) Slope of River Beds:

Mueller<sup>5</sup> in his publication on the principles underlying the regulation of river channels describes experiments which show that there is a very definite relationship between slope of deposits on the one hand and concentration, grain size, and flow on the other. He has deduced the formula,

$$q_a^{2/3} i = a_2 D + b_2 W^{2/3} \dots\dots\dots(7)$$

where  $a_2$  and  $b_2$  are constants,  $W$  the weights of the sediments transported per second per unit width of river channel,  $q_a$  the flow per second per unit width of channel,  $D$  the grain size of the transported material and  $i$  the slope of the channel. The experiments were conducted with uniform rates of discharge  $q_a$  maintained over long periods and with coarse material varying in grading from  $\frac{1}{4}$ " to 2".

As we are dealing with the much finer sediments of a semi-arid region and with flood waves of short duration and pronounced peak intensity it was necessary to conduct a series of experiments more in keeping with these conditions.

A concrete flume of triangular cross section was constructed. The typical <sup>flood</sup> characteristics were reproduced by allowing a vessel of capacity corresponding to the total volume of a flood, to discharge through an orifice. Carefully measured quantities of sand of an average grain size of .009 inches were added at the upper end of the flume and the slope of the deposits determined after repeated floods. A weir was fixed at the lower end of the flume. By varying the crest level of the weir, different river bed widths could be achieved. (Figs.24 & 56).

The following notation was adopted:-

$q_b$  the peak intensity of the "floods" in cusecs per foot width of flume

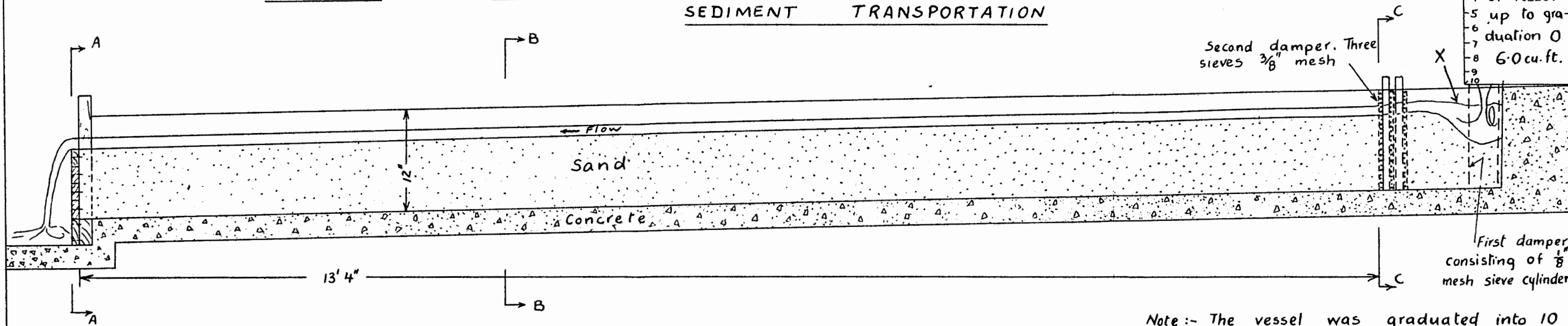
$D_{50}$  the average grain size of the sediment load in inches.

$\rho$  the weight of sediment load expressed as a percentage by weight of the flow.

$i$  the slope of the deposits.

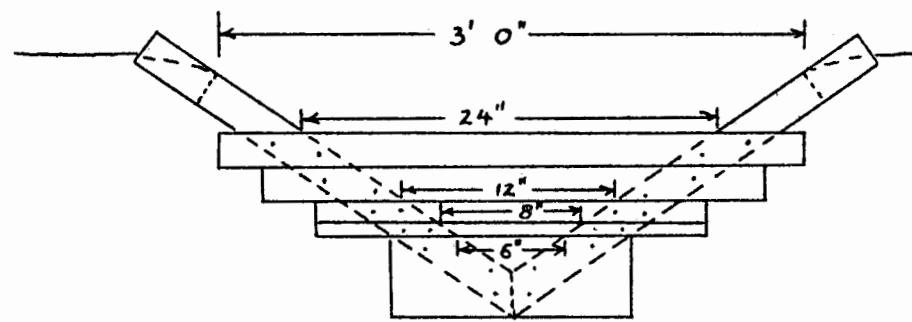
FIG. 24

DETAILS OF APPARATUS USED FOR INVESTIGATING  
SEDIMENT TRANSPORTATION

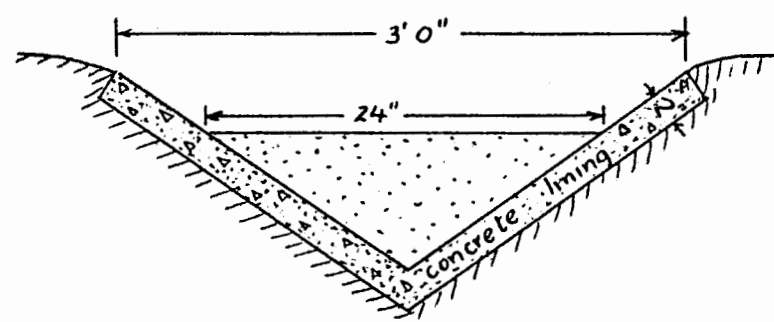


Longitudinal Section - Scale 1" = 1' 0"

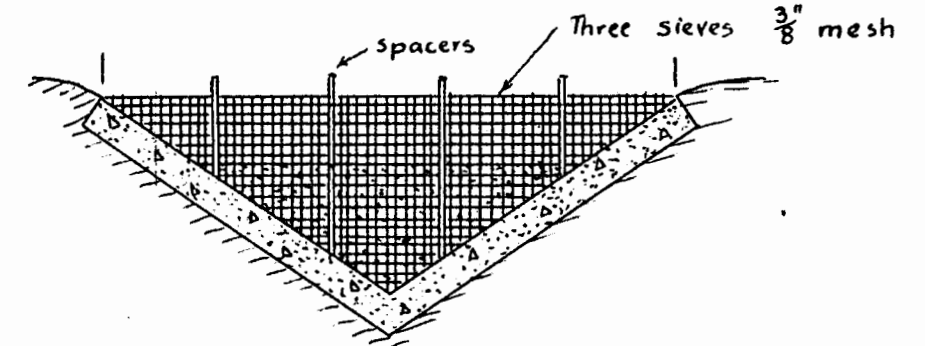
Note:- The vessel was graduated into 10 equal portions. As each portion was drained, a measured quantity of sediment to give the desired percentage by weight was added, as shown by the arrow X.



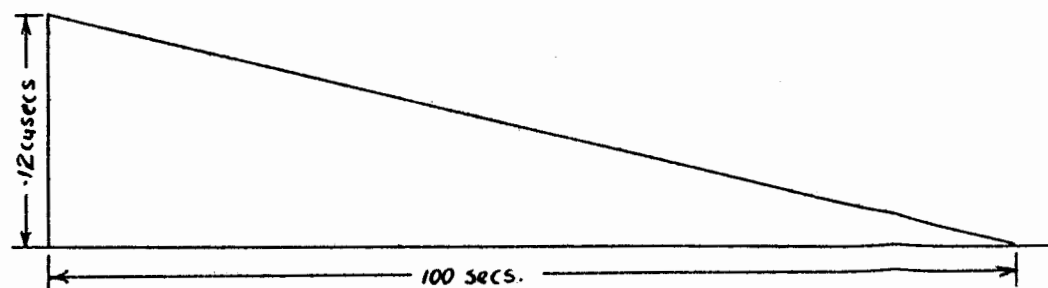
Section AA showing raising of weir in 2" stages



Section BB.



Section CC showing second damper for producing even flow



Discharge Curve produced by allowing the contents of the vessel to discharge through the Orifice.

Method of measuring the amount of sediment required for each graduation :-

The weight of sediment for the required sediment content P was first calculated. An amount of sediment (dry) of this weight was poured into a small metal cylinder, closed at one end. The cylinder was cut off at the surface of the sand and was then used for measuring. A different size of cylinder or a combination of cylinders was used for each percentage required.

The experiments yielded the following results:-

| Experiment No | Flume width | $q_b$ | $D_{50}$ | $P$  | $i$  |
|---------------|-------------|-------|----------|------|------|
| 1             | 6 inches    | .24   | .009     | 0.8% | .005 |
| 2             | 8 "         | .18   | "        | "    | .006 |
| 3             | 12          | .12   | "        | "    | .006 |
| 4             | 24          | .06   | "        | "    | .010 |
| 5             | 6 "         | .24   | "        | 1.6% | .009 |
| 6             | 8 "         | .18   | "        | "    | .010 |
| 7             | 12 "        | .12   | "        | "    | .010 |
| 8             | 24 "        | .06   | "        | "    | .011 |
| 9             | 6 "         | .24   | "        | 4.0% | .025 |
| 10            | 8 "         | .18   | "        | "    | .027 |
| 11            | 12 "        | .12   | "        | "    | .028 |
| 12            | 24 "        | .06   | "        | "    | .029 |

A formula of similar form to that derived by Mueller was now sought, to fit in with the results obtained.

Dividing throughout by  $q_a^{2/3}$  Mueller's formula, equation (7) becomes

$$i = a_2 D \left( \frac{1}{q_a} \right)^{2/3} + b_2 \left( \frac{W}{q_a} \right)^{2/3}$$

$$\text{but } P = 100 \frac{W}{q_a \gamma_{\text{water}}}$$

where  $\gamma_{\text{water}}$  = the density of water (adopting the same units of weight and volume as in  $W$  and  $q_a$ )

$q_b$ , the peak intensity per unit width of channel, will now be used instead of  $q_a$ , the uniform intensity of floods adopted by Mueller.

The general form of the formula to be investigated can therefore be written as follows:-

$$i = X_1 D \left( \frac{1}{q_b} \right)^u + X_2 P^r \dots\dots\dots(8)$$

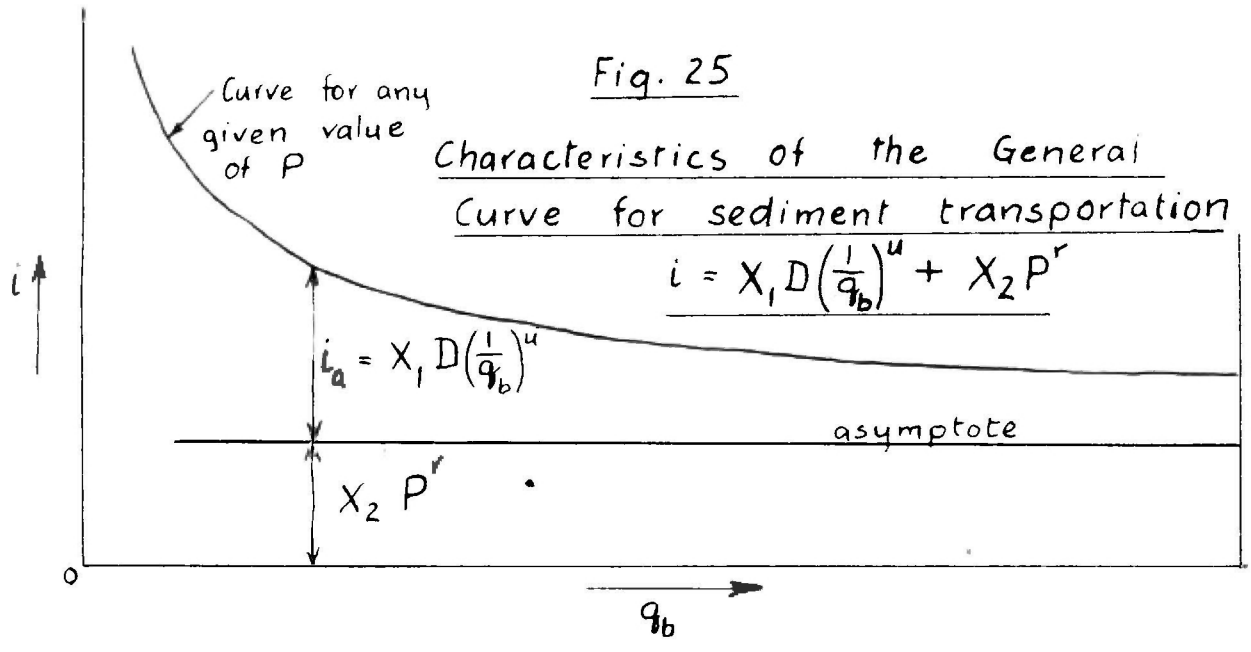
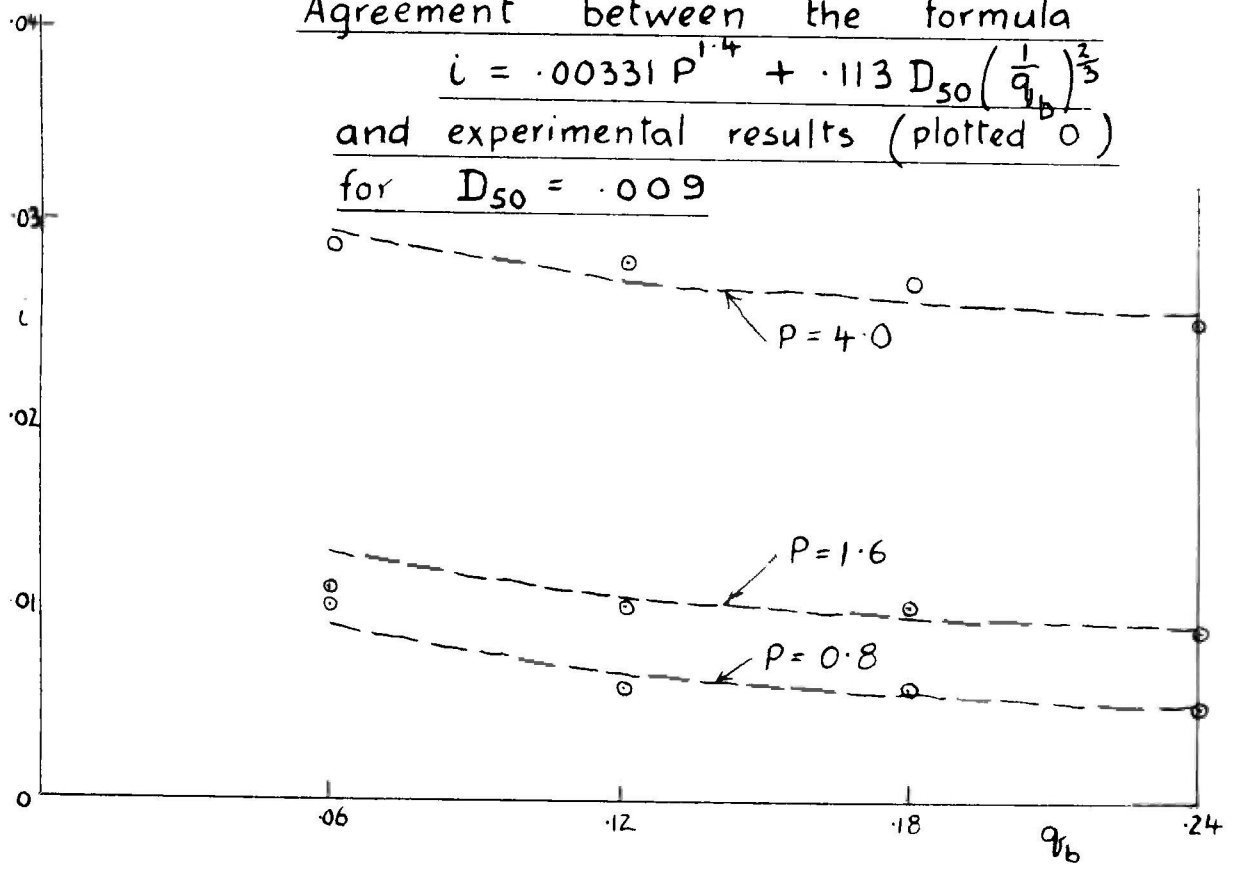


Fig. 26

Agreement between the formula  

$$i = .00331 P^{1.4} + .113 D_{50} \left( \frac{1}{q_b} \right)^{2/3}$$
  
and experimental results (plotted  $\circ$ )  
for  $D_{50} = .009$



$X_1$ ,  $X_2$ ,  $u$  and  $r$  are constants to be determined from experimental results.

The form of equation (8) is such that by plotting  $i$  against  $q_b$  for fixed values of  $P$ , a series of curves of identical form, asymptotic to straight lines  $i = X_2 P^r$  will be obtained. The identical portions above the asymptote are given by the formula  $i_a = X_1 D \left( \frac{1}{q_b} \right)^u$  (Fig. 25)

A value for  $u$  equal to  $\frac{2}{3}$  was tried in keeping with results obtained by Mueller. An inspection of the plotted experimental results showed that the difference between  $i_a$  when  $q_b = 0.06$  and  $i_a$  when  $q_b = 0.24$  should be approximately .004 with each percentage sediment loading tested.

$$\begin{aligned} \text{i.e. } .004 &= X_1 (.009) \left( \frac{1}{.06} \right)^{\frac{2}{3}} - X_1 (.009) \left( \frac{1}{.24} \right)^{\frac{2}{3}} \\ &= X_1 (.009) (6.52 - 2.59) \\ \therefore X_1 &= \frac{.004}{.009 \times 3.93} \\ &= .113 \end{aligned}$$

i.e. The portion of the ordinates above the asymptote for each of the three values of  $P$  is given by the curve  $i_a = .113 D_{50} \left( \frac{1}{q_b} \right)^{\frac{2}{3}}$

By moving this curve up and down on the plotted observations, the best agreement with experimental results and the corresponding height of the asymptote above zero were found.

For  $P = 0.8$ , the asymptote was found in this way to be  $i \doteq .0025$ , for  $P = 1.6$ ,  $i \doteq .0060$ , and for  $P = 4.0$ ,  $i = .0230$ .

Plotting the above figures in the form  $\log i$  against  $\log P$ . the points were found to lie very nearly on the straight line  $\log i = -2.48 + 1.4 \log P$

occurrence of floods than in the Gamams Area.

Information is available as to the quantities and nature of sediments yielded by catchments in this region (page 85 ). A total denudation of .0129 inches with .0032 inches in the form of sediments coarser than silt is expected,  $D_{50}$  of the material coarser than silt being approximately 0.01 inches.

The average annual runoff is assumed to be 0.5 inches and the width of the river bed 130 feet (Bulskop Dam, Stage 3).

The variables in equation (9) are as follows:-

$$P = \frac{.0032 \times 1.6}{0.5} \times 100 = 1.02\% \text{ (specific gravity of sediments = 1.6 )}$$

$$q_b = \frac{1050}{130} = 8.1 \text{ cusecs/ft width}$$

$$D_{50} = 0.01 \text{ inches.}$$

Substituting in equation (9),

$$\begin{aligned} i &= .0033 \times 1.02^{.4} + 0.113 \times 0.01 \times \left(\frac{1}{8.1}\right)^{\frac{2}{3}} \\ &= .00341 + .00028 \\ &= .00369 \end{aligned}$$

After complete sanding up of stage 3 of Bulskop Dam the sediments actually assumed a slope of .0033.

#### (7) (8) Water Storage in Sediments.

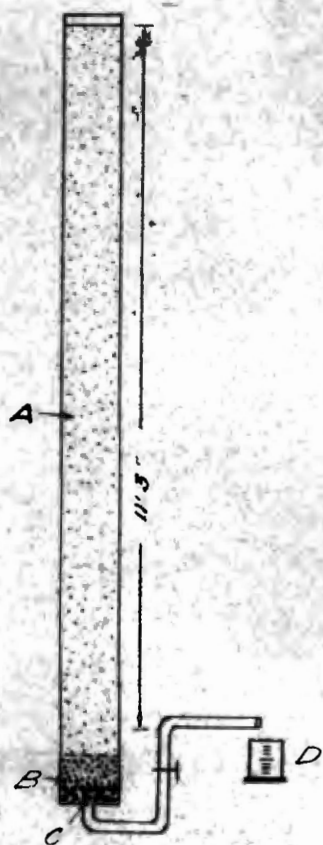
##### (a) Porosity.

The porosity depends largely on the size and shape of grains, uniformity of grain size and compaction.

The porosity of freshly deposited silt may be very high, figures of 0.8 and 0.9 being reported for freshly deposited alluvium of the Mississippi delta<sup>4</sup>. Silt deposits become consolidated in time through weight of overlying material, alternate drying out



| Diameter of Grain<br>(inches) | Porosity | Specific Yield |
|-------------------------------|----------|----------------|
| .002                          | 0.30     | 12%            |
| .003                          | 0.30     | 16%            |
| .004                          | 0.30     | 20%            |
| .010                          | 0.30     | 23%            |
| .040                          | 0.30     | 28%            |



*Figure 27.*  
*Apparatus used for determining*  
*Specific Yield & Permeability.*  
*A Sand being tested*  
*B Filter of coarser sand and fine gravel*  
*C Screen.*  
*D Graduated measuring cylinder*

Fig.27 illustrates the tests which the author conducted to determine the specific yield of the fine sand of the sand selected for the purpose is tabulated on from the Bulskop Dam. The sieve analysis<sup>page 180</sup> (sample 1) The sand was saturated with water and then dropped into the cylinder, small quantities at a time. By this procedure a certain amount of compaction was obtained. The porosity of an undisturbed sample taken one foot below the surface at the end of the experiments was found to be 0.45. On draining the sand the first time a specific yield of 0.342 was measured. During the process of draining a compaction of 1.5 inches

occurred (1% on the 12 ft. column of sand). The sand was again saturated with water by allowing a tap to drip on the surface over a prolonged period. The same degree of saturation as with the initial method of filling was not achieved, since the measured yield in this case only amounted to 0.286. No further compaction occurred during the second test.

The conclusions drawn from these tests are that with completely saturated sand of this grading a specific yield of approximately 0.25 and a porosity of 0.40 can be expected. Complete saturation of deposits of fine sand by surface application of water is by no means easy to achieve.

(c) Permeability.

The coefficients of permeability of samples of material taken in the bed of the Swakop River near Swakopmund varied from  $k = 58$  feet per diem for fine sand to 183 for medium and 1580 for very coarse sand (pages 110 & 111 ). Fine sands from sand storage dams were found to have a permeability of only 10 feet per diem (page 183, samples 1 & 2). The fine sands in a sand storage dam are, however, interbedded with coarser material and are deposited on top of a river bed or an artificial filter bed of coarser material. The average permeability in a sand storage dam is therefore considerably in excess of the figure for fine sand, as can be seen from the following observations at the sand storage dam at Aukeigas:-

The discharge from the dam on 27.6.52 was 90 gallons an hour or 346 cubic feet a day, the slope of the water table in the direction of flow was 1:300 and the cross section of the sediments at the dam wall 530 square feet.

(Fig.46) As explained in Appendix B, example 7, this is a case of flow with receding water table and the velocity is actually dependant on the slope of the base of the deposits which is 1:62.

The permeability in a horizontal direction is therefore given by  $k = \frac{q}{a} \frac{l}{\sin \theta} = \frac{346}{630} \times 62.0 = 34.0$  feet per day.

For a sand storage dam to be successful, it is not sufficient that the specific yield of the retained material should be adequate. The average permeability in a horizontal direction must also be large enough to permit the water being extracted at the desired rate.

(d) Capillarity.

The following figures are derived from a summary of observations of capillarity by various observers given by Tolman<sup>4</sup> :-

| Size of Sand grains (inches) | Extent of lift (inches) | Time required to reach limit (days) |
|------------------------------|-------------------------|-------------------------------------|
| .0063                        | 19.25                   | 171                                 |
| .0118                        | 13                      | 188                                 |
| .0197                        | 11                      | 138                                 |
| Sandy loam soil              | 52                      | 144                                 |

The initial rapid drop in water level observed in evaporation experiments (page 142) is due to the combined effect of capillarity and evaporation.

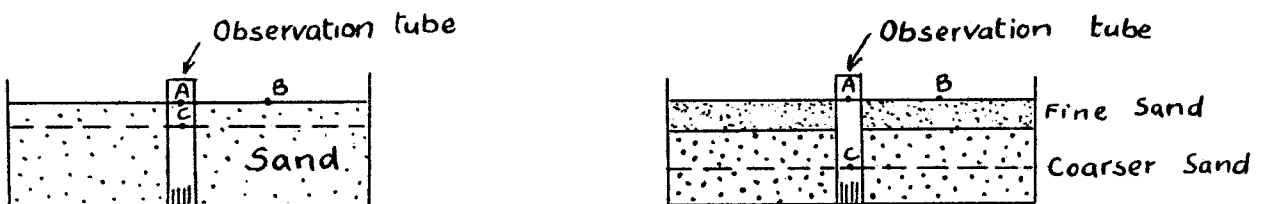


Fig. 28 Section through evaporation tanks (diagrammatic)

$25 \times \frac{12}{11} = 27\%$ . Columns of the same sand 1 foot and less in height will not yield water by gravity.

If the sand body were of the form of an inverted pyramid as in a sand storage dam, and only the upper 11 feet of a total depth of 12 feet were drained, there will be very little difference between the true specific yield of the material and the apparent specific yield, since the lower 1 foot only contains  $\frac{1}{1728}$  of the total water content, compared with  $\frac{1}{12}$  in the cylindrical container.

(e) Evaporation Losses.

In addition to the climatic factors the properties of the sand have to be considered. Capillarity and permeability affect the rate of evaporation. Van Reenen<sup>3</sup> gives the following information on evaporation:- "Sand, such as that usually found in the river beds of South West Africa does not favour the capillary movement of water in an upward direction against gravity, and when once the level of the underground water on the upstream side of the "ground-sill" commences to fall, a mulch of dry sand, through which very little evaporation can take place is soon formed". Van Reenen quotes the following comments by Widtsoe<sup>6</sup> on certain experiments with evaporation from soil:- "The rapid evaporation due to arid conditions so dried out the top soil that the loss of water in one year was only 11.2 inches as against 51.6 inches for a similar soil under humid conditions which permitted a slow but steady evaporation." Widtsoe's explanation for this phenomenon is that "the top soil (under arid conditions) is dried out so rapidly that the lower soil layers cannot send moisture upwards in time to supply the loss.

Under such conditions the evaporation is automatically decreased. The top dry soil thus induced is an effective check upon the upward movement of the water."

If water is stored in fine sand the high capillarity of the sand will bring about an initial high rate of evaporation, thereafter however, the low permeability will assist in forming an effective protection of dry sand above the capillary fringe. With coarse sand the capillary fringe will be only slightly above the water table but due to the greater permeability a greater thickness of dry sand will be required to afford effective protection against evaporation.

In the stage construction dams described in chapters 4 and 5, the material at the surface of any particular stage is coarser than that deposited lower down, resulting<sup>a</sup> in a reduction in capillary rise to the surface. Horizontal layers of silt, not necessarily continuous, are interbedded with the sand in the lower portion of the stage resulting in a reduction of permeability in a vertical direction. These conditions all tend to reduce evaporation losses.

To arrive at a basis of estimating evaporation losses from water stored in sand the author has undertaken the following experiments and analyses:-

(i) Determination of moisture loss by weighing:-

A cylinder six inches in diameter and eight inches high was filled with dry sand, moderately compacted. Material from the Bulskop Dam was used (Sample 1, page 180 ) The sand was saturated with water and the cylinder placed in an exposed position in the sun. The amount of water in the sand at any instant was determined by weighing the cylinder and subtracting the known dry weight.

The following results were obtained:-

| Time              | Average water content expressed as percentage by volume. | Rate of evaporation expressed in feet of water per month (Slope of water content-time curve) |
|-------------------|--|--|
| Initial condition | 40%  | 2.2feet  |
| after 0.5 days    | 36%  | 1.7 "  |
| " 1.5 "           | 28%  | 1.2 "  |
| " 2.5 "           | 22%  | 1.1 "  |
| " 3.5 "           | 17%  | 1.0 "  |
| " 4.5 "           | 12%  | 0.9 "  |
| " 5.5 "           | 9%   | 0.3 "  |
| " 9.5 "           | 7%   | 0.1 "  |
| " 27.5 "          | 4%   | 0.05"  |

(ii) Lowering of water table due to evaporation.

The apparatus used in determining specific yield was modified as shown in Fig.29.

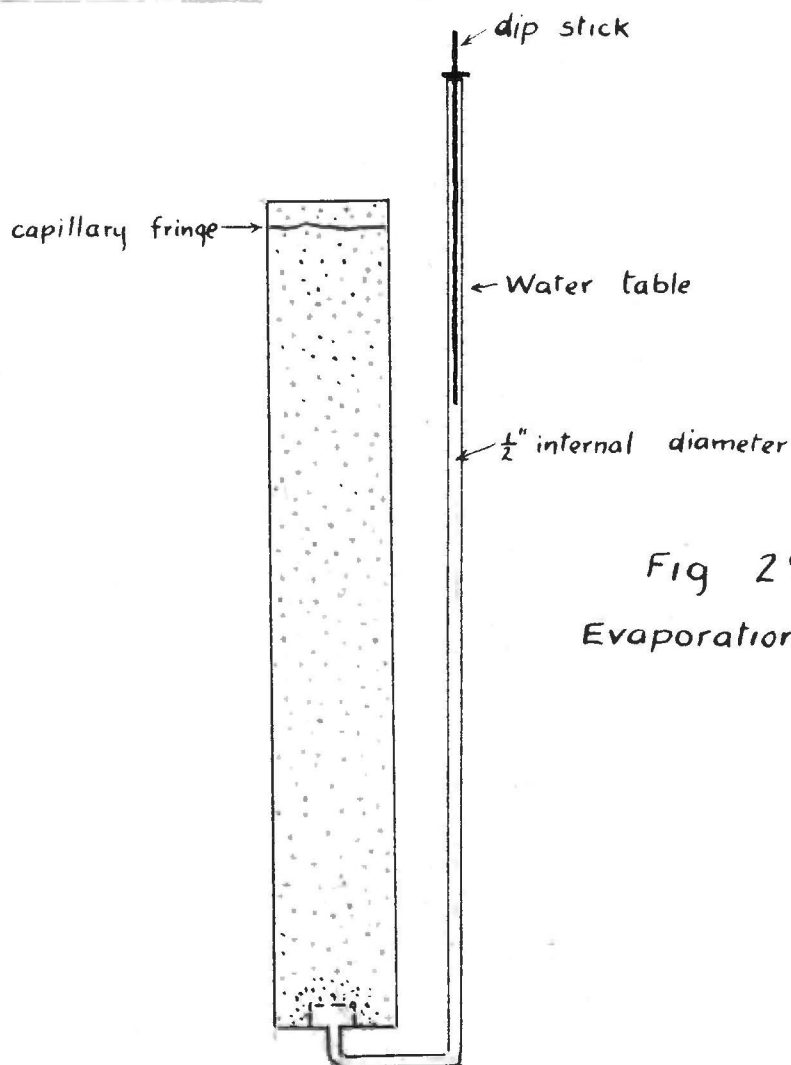


Fig 29  
Evaporation Tube

As the diameter of the indicator tube (Fig.29) was small compared with that of the tube containing saturated sand, the water table was lowered by an amount equal to the capillary rise, very rapidly. Thereafter the weighing tests just described would predict a decline in water table at an initial rate of  $\frac{2.2}{n} = 5.5$  feet per month. ( $n$  = porosity of the sand = 0.4). When the water table is lowered to such an extent that the sand near the surface contains about 7% by volume of moisture a rate of lowering of the water table by  $\frac{0.1}{n} = 0.25$  feet per month would be expected in accordance with the weighing tests.

The test with the apparatus shown in Fig.29 filled with typical sand from a sand storage dam, showed a lowering of water table of 1 foot after 4 days, 2 feet after 10 days and 3 feet after about 40 days (Fig.47).

The top of the sand tube was in the sun and the remainder in the shade. Water level readings were taken at 2 p.m. every day. The initial rate of lowering of the water table was approximately 8 feet/month, declining to almost zero in 40 days. It was found that the water level in the indicator tube was subject to fluctuations due to changes in temperature. At 2 p.m. on the first day the water level in the tube was at zero, i.e. at the level of the top of the stand pipe. Overnight it dropped to 18 inches below zero to return to 3 inches below zero by 2 p.m. the next day. Fluctuations also occurred on subsequent days. As the water table dropped the daily fluctuations became less, amounting to 4 to 6 inches for the greater part of the experiment. Daily temperature variations are

responsible for these fluctuations (expansion and contraction of the water and changes in surface tension). Similar fluctuations are not observed in wells in sand storage dams, no doubt due to smaller variations in temperature with the larger body of water. Due to the temperature fluctuations to which the apparatus was subjected, too much reliance should not be placed on the water level observations in the indicator tube.

The water table in the stand pipe (Fig 29) was lowered to 3'6" below surface after almost two years. No measurable rise in the water table due to rainfall occurred at any time in the intervening period. The lowering of the water table at the end of the experiment was measured first in the indicator tube (measurement 3'9 $\frac{1}{2}$ " at 2 p.m. and was then checked by excavating the sand down to the water table (3'6"). The latter figure is considered the more accurate and is taken as an indication of the limit to which the water table in fine sand will be lowered by evaporation over a prolonged period.

Similar results were obtained by means of a <sup>sand filled evaporation tank buried in the</sup> sand storage dam at Aukeigas. The initial rate of lowering of the water table was 5 feet per month and the rate became negligible after the water table had receded to 2'6" below the surface.(Fig.47)

In the sand storage dam at Aukeigas water table observations during 1942 showed a decline at the dam wall of three feet in 60 days, due to evaporation and extraction. The extraction in this dam was accurately determined by gauging as described on page 139. Making due allowance for extraction it was deduced that evaporation alone was at the rate of approximately 3 ft. in 90 days and that evaporation below the 3 feet level



was negligible (Fig 47). The causes for the difference between the rate of evaporation observed in experiments and under field conditions to be found in the more effective self-mulching of the sand as deposited in stages by flowing water (effect of silt layers etc) in the lower temperatures of the larger ground water bodies and in the presence of coarse sand in the upper portion of the sand reservoir.

Fig.47 gives complete data collected by the various methods in a form convenient for comparison of results.

Summing up the evidence, it would appear safe to assume the following when assessing the storage efficiency of sand reservoirs:-

If there is no extraction the lowering of the water table is at the rate of 3 ft. in 90 days. Evaporation from the saturated zone stops for practical purposes when the water table has receded three feet below the surface of the sand. Further recession of the water table is due to extraction alone but the drying out of the moisture above the capillary fringe continues up to a depth of 3 feet below the surface of the sand and then also stops for all practical purposes.

#### (8) Evaporation in Open Storage Reservoirs.

Evaporation from open water depends (a) upon the prevailing climate conditions (relative Humidity, temperature of water and air, wind velocities etc.) and (b) upon the size of the body of water and the nature of the surrounding topography. Analysis of available reservoir depletion data is considered the most practical approach to the problem of estimating evaporation losses.

From Fig.12 it will be seen that during two periods,

each of about 3 to 4 years duration, no water was drawn of from Avis Dam. The decline in water level during these periods was therefore entirely due to evaporation and seepage; and has been analysed as follows:-

First period. (Dam nearly empty)

| Year    | 1st Quarter | 2nd Quarter | 3rd Quarter | 4th Quarter |
|---------|-------------|-------------|-------------|-------------|
| 1937    |             |             | 1.7 ft.     | 1.8 ft.     |
| 1938    | 1.6 ft.     | 1.2 ft.     | 1.7 ft.     | 1.4 ft.     |
| 1939    | 1.5 ft.     | 1.5 ft.     | 1.6 ft.     | 1.8 ft.     |
| 1940    | 2.2 ft.     | 1.9 ft.     | 2.1 ft.     | 2.1 ft.     |
| 1941    | 2.0 ft.     | 1.4 ft.     | 2.1 ft.     | ?           |
| Average | 1.82 ft.    | 1.50 ft.    | 1.85 ft.    | 1.78 ft.    |

i.e average 6.95 ft. per annum.

Second period. (Dam about half full, water level fluctuating above and below 15 feet over outlet pipe).

| Year    | 1st Quarter | 2nd Quarter | 3rd Quarter | 4th Quarter |
|---------|-------------|-------------|-------------|-------------|
| 1947    |             | 1.4 ft.     | 2.0 ft.     | 2.6 ft.     |
| 1948    | ?           | 2.4 ft.     | 2.0 ft.     | 2.4 ft.     |
| 1949    | ?           | 2.0 ft.     | 2.0 ft.     |             |
| Average | ?           | 1.93 ft.    | 2.0 ft.     | 2.5 ft.     |

The record is incomplete in respect of the first quarter as runoff occurred in most of the intervals between water level observations during these months. The record of the first period, however, shows that evaporation of approximately equal order of magnitude can be expected in the 1st and 4th quarters. It will therefore be assumed that with the water level at approximately 15 feet over outlet pipe, the decline in water level will be as follows:-

|                |                |
|----------------|----------------|
| 1st Quarter    | 2.5 ft.        |
| 2nd Quarter    | 2.0 ft.        |
| 3rd Quarter    | 2.0 ft.        |
| 4th Quarter    | <u>2.5 ft.</u> |
| Total for year | 9.0 ft.        |

During low stages (first period referred to above) only negligible seepage takes place, as the lower portion of the reservoir has been rendered practically watertight by deposits of fine silt. The annual decline in water table of 6.95 feet or 7 feet in round figures will be assumed to be due to evaporation alone.

If this assumption is correct the water losses when the dam is half full will amount 7 feet evaporation and 2 feet seepage per annum, making up the total observed depletion of 9 feet per annum. From the capacity curve of the dam it is found that the corresponding seepage discharge will amount to 3700 gallons an hour, a figure which can unfortunately not be verified by gauging as it is apparent that a considerable portion of the seepage of this dam is transpired by vegetation or discharged into waterbearing strata.

The depletion curves of Otjimahona Dam (Fig.13) afforded a further opportunity of determining the annual evaporation from an open storage basin. Draw-off was assumed to be 200,000 cu.ft. per annum (400 head of cattle) and depletion was computed in accordance with equation (2) for annual evaporation of 7,8 and 9 feet. The computed depletion curves were plotted on the chart of observed depletion (Fig,13). By comparing computed and actual depletion it is concluded that evaporation and seepage losses in this dam amount to about 7.5 feet. As this is a particularly well constructed dam in a region of dense mica schist

rock (Fig.6). the depletion of 7.5 feet is considered to be due to evaporation alone.

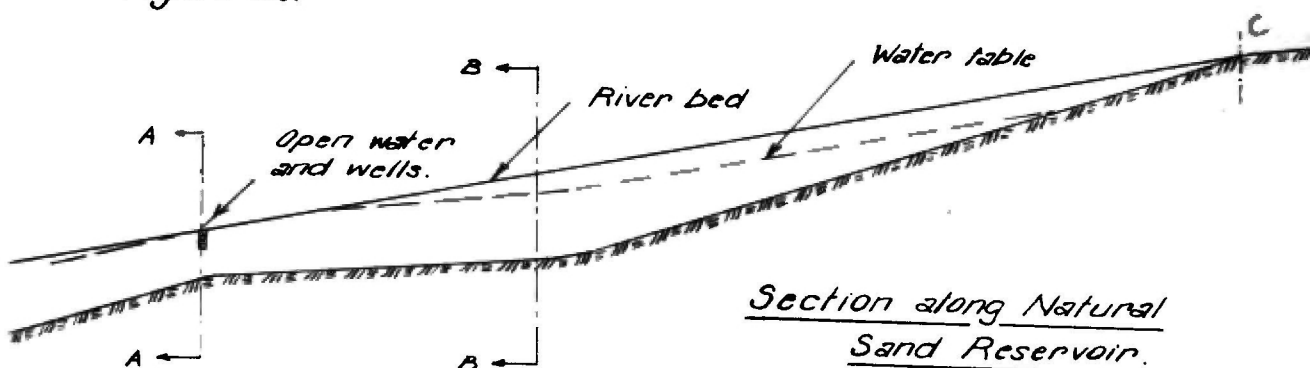
In planning new reservoirs in this region at reasonably impervious sites an assumed rate of 9 feet per annum for evaporation and seepage is suggested, so as to make ample provision for possible seepage losses.

CHAPTER 3.NATURAL ~~AND~~ SAND RESERVOIRS.(1) Water Movement in Natural Sand Reservoirs.

Sandy river beds with no surface flow, ~~except~~ in flood seasons, are a common feature of semi-arid regions. As already indicated, the water stored in sand is frequently an important source of supply. The most universal cause of natural sand reservoirs would appear to be erosion of valleys in a humid geological period and subsequent sand deposition in a less humid period, during which rivers are unable to cope with the sediment load. Systematic trial boring in river beds in such instances will reveal the original river course, buried beneath the sand.

The following sketch illustrates the author's conception of a natural sand reservoir and the position where open water occurs and where the first wells are usually brought down. In this example, all the water will eventually drain away in a drought of sufficient duration. The retention of water from one flood season to the next is due to the frictional resistance to flow through sand and not to the presence of any positive underground rock barrier extending above the normal rock level

Figure 30.



A brief description of a number of natural sand reservoirs follows:-

The sand bed of the Omaruru River at Omaruru varies in width from 200 to 1000 feet. The slope of the sand is approximately 1:280. The grading varies from medium grained granitic sand at the surface to coarse granitic sand at the base with  $D_{50}$  approximately  $\frac{1}{10}$ th. of an inch. At Omburo about 18 miles upstream of Omaruru the river bed consists of exposed rock throughout. Between Omburo and Omaruru the river bed consists of granitic sand but no information is available about rock levels as the existing wells are not taken down to rock. At the upper end of the town of Omaruru a complete cross section of the river was determined by trial boreholes. The sand fills a V-shaped valley and is 38 feet deep at the deepest point. The width of the sand fill at the top is just over 1000 feet. The existing town wells are at the side of the valley and are 10, 14.1 and 17.3 feet deep, respectively. The wells are drawn upon <sup>at</sup> a combined rate of 200,000 gallons a day in summer and this supply can be maintained except after very poor rainy seasons. The site of the present pumping scheme is equivalent to section B in Fig.30. At the lower end of the town (section A) 7800 feet downstream of B the river bed narrows down abruptly to 200 feet in width with a sand filling 20 feet deep. The supply can be rendered more permanent by sinking wells in the deepest portions of cross sections. A tube well was recently sunk for this purpose at a point 7000 feet downstream of B where the width of the sand fill is still approximately 400 feet. The slope of the river between the old and the new wells is 1:280 and <sup>of</sup> the deepest point of the rock bed 1:1000.

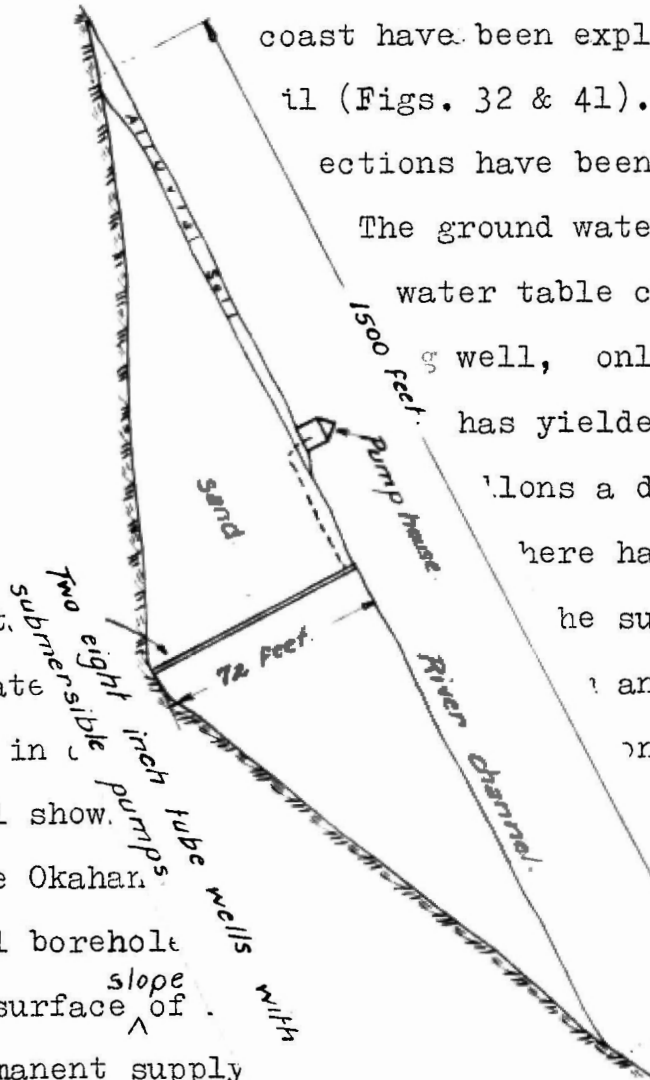
The low levels of the course of the Swakop River of considerable depth. The

coast have been explored by trial (Figs. 32 & 41). The sections have been tested. The ground water flow is water table cannot drop 1500 feet. A well, only ten feet deep, has yielded an amount of water per day. After the supply here is complicated and will be dealt with in a separate section.

Fig.31 shows the origin of the water through the Okaharipa borehole and has a surface slope of 72 feet. The water table at present (August 1952) is approximately 14 feet below the surface.

obtained from wells varying in depth from 10 to 15 feet next to the pump house. The wells, shown in Fig.31, were developed at the deepest point in the river section and submerged pumps installed. In 1950 the town consumption had risen to 132,000 gallons a day. Although there has been very little flow in the river since 1950 no water shortage has been experienced. The water table at present (August 1952) is approximately 14 feet below the surface.

Figure 31.



The region A in Fig.30 illustrates a very common cause of high ground water table namely shallow rock level, accompanied by narrowing of the river bed. Clay deposits at bends or junctions may also give rise to Shallow ground water tables. Where a river with coarse deposits joins one with finer sand, the latter will act as a partial ground water barrier, causing a shallow water table where the coarser deposits end.

As resistance to flow is a governing factor, it will be necessary in an investigation of sand storage scheme to determine the permeability of the sediments which store the water.

The full cross section of the river bed must be explored by trial boreholes and the permeability of samples taken at various depths determined in accordance with the method set out in Chapter 7.

Fig.32 shows the cross section of the Swakop River at a proposed pump station two miles from Swakopmund. It will be seen that the material could be classified into three grades, fine sand, medium sand and very coarse sand occupying 1360, 2200 and 4210 square feet, respectively.

The permeability of typical samples from the three zones was tested with the following results:-

Fine Sand:-

|           |     |   |           |      |     |      |
|-----------|-----|---|-----------|------|-----|------|
| Sample 1. | $k$ | = | 63        | feet | per | diem |
| 2         | $k$ | = | 55        | "    | "   | "    |
| 3         | $k$ | = | <u>57</u> | "    | "   | "    |
| Average   |     |   | 58        | "    | "   | "    |



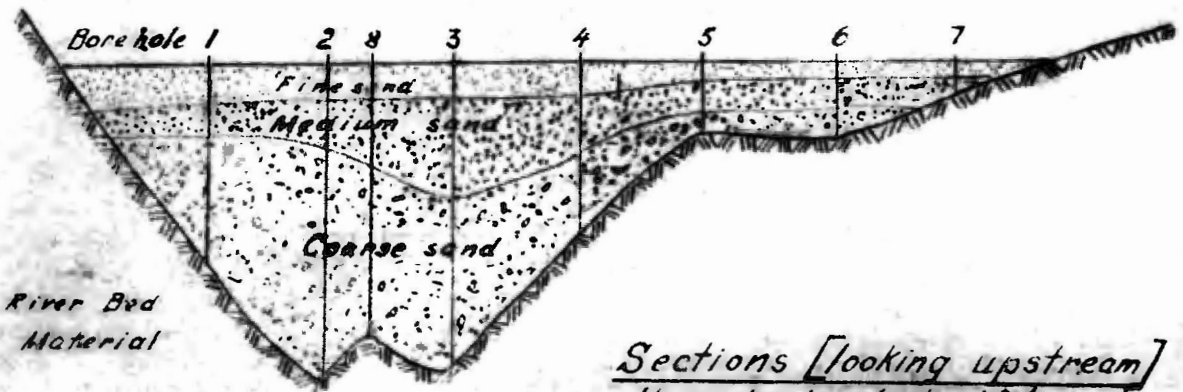
Medium Sand:-

|           |     |   |            |      |     |      |
|-----------|-----|---|------------|------|-----|------|
| Sample 1. | $k$ | = | 120        | feet | per | diem |
| 2.        | $k$ | = | 190        | "    | "   | "    |
| 3.        | $k$ | = | <u>238</u> | "    | "   | "    |
| Average   |     |   | 183        | "    | "   | "    |

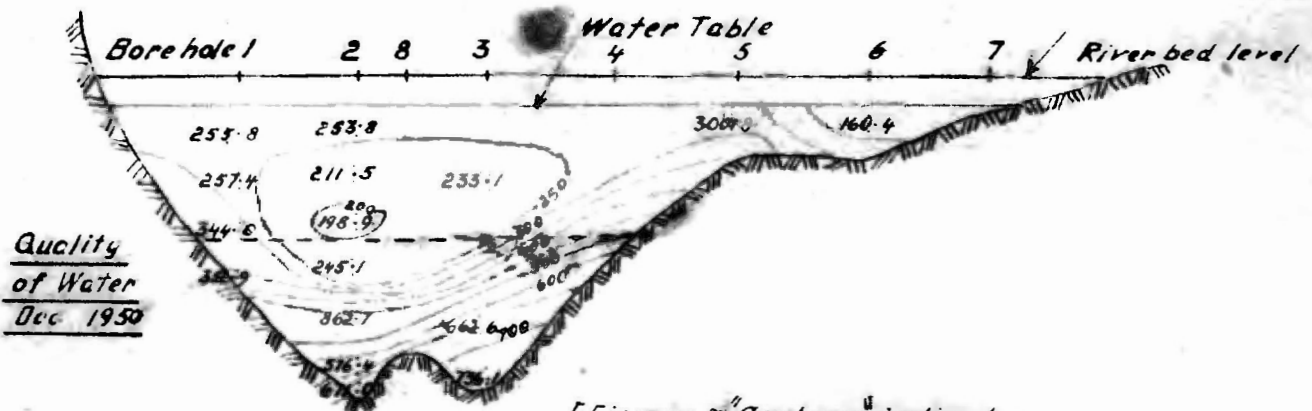
Very Coarse Sand:-

|          |     |   |             |      |     |      |
|----------|-----|---|-------------|------|-----|------|
| Sample 1 | $k$ | = | 1320        | feet | per | diem |
| 2.       | $k$ | = | 1210        | "    | "   | "    |
| 3.       | $k$ | = | <u>2200</u> | "    | "   | "    |
| Average  |     |   | 1580        | "    | "   | "    |

Fig. 32  
Quality of Sand & Water Swakop River



Sections [looking upstream]  
through river bed at A (Fig. 42)  
Hor. Scale: 1" = 100'  
Vert. Scale: 1" = 20'



[Figures & "Contours" indicate  
total dissolved solids in parts  
per 100,000]

The slope of the river bed at the site is 1:300. The effective velocities of flow as used in Darcy's formula are, therefore, 0.19, 0.61 and 5.27 feet per diem for each of the three classes of material, respectively. Multiplying the effective velocities by corresponding cross sectional areas, the following rates of seepage flow with water table at river bed level are found:-

|                   |               |                      |   |   |
|-------------------|---------------|----------------------|---|---|
| Fine sand,        | 258           | cubic feet per diem, |   |   |
| Medium sand,      | 1,343         | "                    | " | " |
| Very coarse sand, | <u>22,200</u> | "                    | " | " |
| Total             | 23,801        | "                    | " | " |

or 143,000 gallons per diem.

The lowest water level observed at this site after the river had not flown for two rainy seasons was 14 ft. below river bed level (August 1952). Assuming a further drop to 16 feet before the first floods, the minimum seepage flow will be 2176 (saturated area in square feet)

$\times 5.27$  (effective velocity) = 11,470 cubic feet per diem = 71,700 gallons per diem. Should the river be without runoff for three years in succession the rate of seepage will be reduced still further; but it is unlikely that such a succession will occur. In order to dam up some of the excess ground water flow during periods when a surplus is available and to prevent seepage past the site during times of deficiency a ground water cutoff consisting of steel sheet piling will be provided downstream of the proposed pump station.

## (2) Water Extraction by means of Tube Wells.

The problem of tapping the river cross section shown in Fig.32 by means of tube wells will now be considered.

The formula developed by Dupuit<sup>4</sup> for flow towards a tube well is

$$q_y = \frac{\pi k (D^2 - d^2)}{2.3 \log_{10} \frac{R}{r}}$$

where  $r$  = the radius of the well.

$R$  = the radius of the circle of influence. within which the ground water has been lowered by pumping.

$d$  = the depth of water at the tube well.

$D$  = the depth of water at and beyond the circle of influence.

$k$  = coefficient of permeability in a horizontal direction, since Dupuit's analysis assumes horizontal flow.

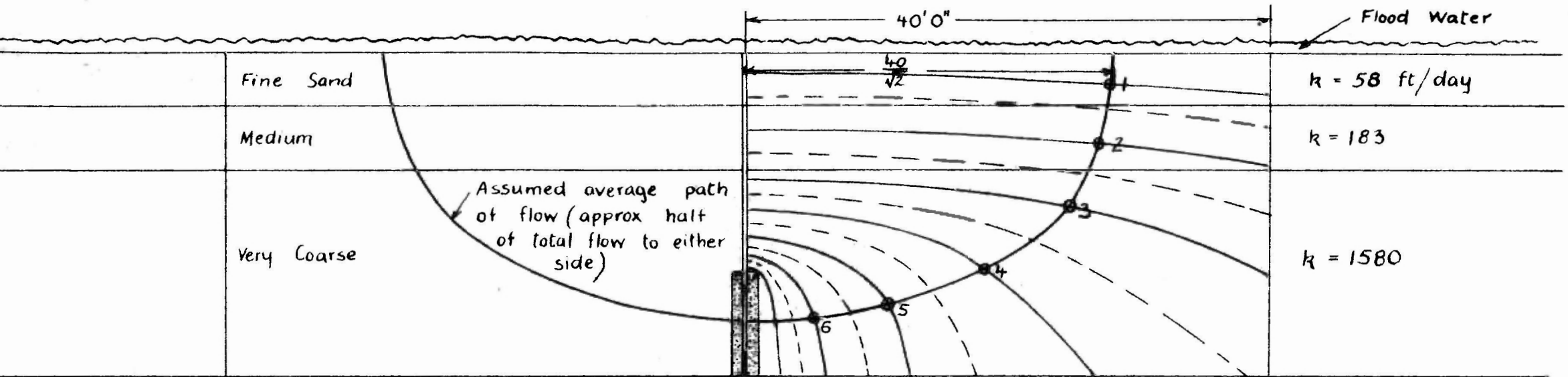
$q_y$  = yield of the tube well in cu.ft.per day.

An average value of  $k$  for the full cross section of the river will be deduced from the data that 23,800 cubic feet of seepage were yielded by the full cross sectional area of 7770 sq. feet with the slope of the water table 1:300. i.e Average value of  $k = \frac{23800}{7700} \times 300 = 920$  ft. a day.

With an extraction of 150,000 gallons a day (24,000 cubic feet a day) the lowering of the water table will be given by

$$\begin{aligned} D^2 - d^2 &= \frac{2.3 \log_{10} \frac{R}{r}}{\pi k} q_y \\ &= \frac{2.3}{\pi \times 920} \left( \log_{10} \frac{R}{r} \right) 24000 \\ &= 19.1 \log_{10} \frac{R}{r} \end{aligned}$$

With an extractn of 100,000 gallons a day (16,000 cubic feet a day) the lowering of the water table will



| Segment                                       | Length of seepage path $l$ (feet) | Area transverse to flow $a$ (sq. ft.) | $k$ (ft/day) | Loss of head $h = \frac{l}{a k} q$ |
|---|-----------------------------------|---------------------------------------|--------------|------------------------------------|
| 1   | 4.0                               | 5050                                  | 58           | 0.0000137 $q$                      |
| 2   | 5.0                               | 5100                                  | 183          | 0.0000054 $q$                      |
| 3   | 7.0                               | 5250                                  | 1580         | 0.0000008 $q$                      |
| 4   | 9.5                               | 3014                                  | "            | 0.0000020 $q$                      |
| 5   | 8.0                               | 795                                   | "            | 0.0000064 $q$                      |
| 6   | 4.0                               | 300                                   | "            | 0.0000084 $q$                      |
| 7   | 2.2                               | 118                                   | "            | 0.0000118 $q$                      |
| Total loss of head from surface to envelope = |                                   |                                       |              | 0.0000485 $q$                      |

Fig. 33

Loss of head with tube wells so spaced that each commands a radius of 40 feet

The gravel envelope, designed in accordance with the principles laid down on page 150, will consist of material which will pass through a  $\frac{3}{8}$  inch mesh sieve and will be retained on a  $\frac{1}{4}$  inch mesh sieve. The loss of head due to seepage through material of this grading will be negligible.

The tube well itself will be constructed of 6 inch asbest-cement tubing with 1720 holes  $\frac{1}{4}$  inch in diameter, giving a total area of perforations equal to three times the cross sectional area of the tube. With a porosity of the gravel envelope equal to  $\frac{1}{3}$ , the area of unobstructed perforations may be taken as being equal to the area of the tube. If  $v_t$  is the velocity of flow in the tube or through unobstructed perforations, then the loss<sub>of</sub> head at entrance may be taken as  $\frac{1}{2g} \left( \frac{v_t}{C} \right)^2$  where  $C$  = the coefficient of discharge, say 0.6.

$$v_t = \frac{q_f}{\pi \times 0.25^2} \times \frac{1}{24} \times \frac{1}{3600} \text{ ft/sec.}$$

$$= \frac{q_f}{17000} \text{ ft/sec.}$$

$$\text{Loss of head at entrance} = \frac{1}{2g} \left( \frac{q_f}{17000 \times 0.6} \right)^2$$

$$= \left( \frac{q_f}{81,500} \right)^2$$

With  $q_f = 16000$  cu.ft./day (100,000 gallons/day)

Loss of head at entrance = 0.039 feet.

The tube wells were purposely designed for very low friction losses so as to obtain as great a flow as possible with small loss of head (page 127 ).

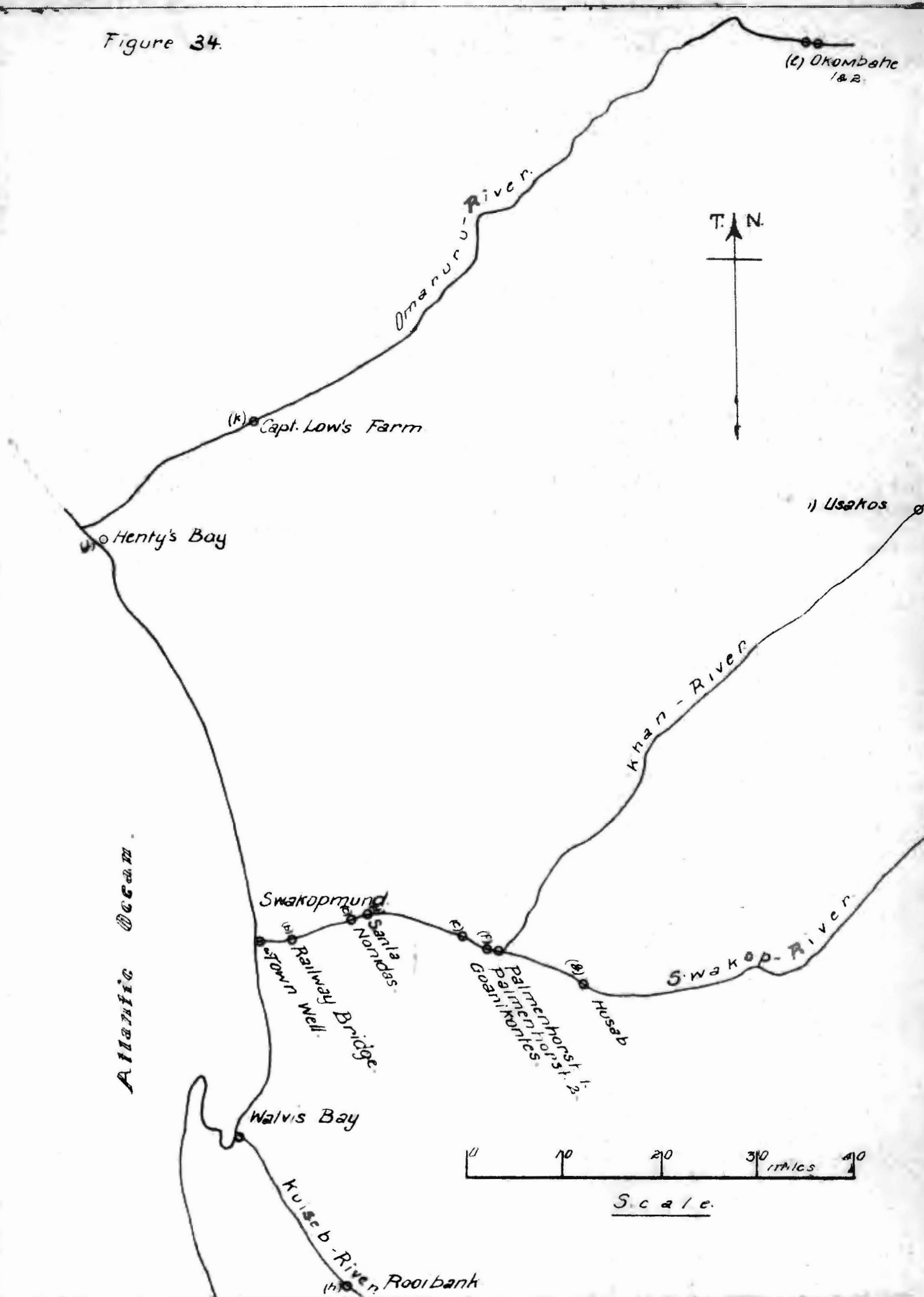
(5)

(3) The Brack Water Problem.

The ground waters of rivers in certain regions are high in total dissolved solids. In humid and semi-arid regions high salt content only occurs under unfavourable geological conditions. Granite areas in such regions are usually known for their supplies of soft water. In arid regions, however, high salt content occurs in most sand rivers, even in granite areas.

The following map and schedule illustrate the brack water problem at a number of points in the lower reaches of the Omaruru, Swakop and Kuiseb River Beds. The average annual rainfall in the area covered by the map varies from 1 inch at the coast to 6 inches near Usakos.

Figure 34.



| Locality Symbol on Map | Sample No | Date     | Total dissolved solids. | Analyst | Remarks  |
|------------------------|-----------|----------|-------------------------|---------|--|
| a                      | 17        | July'50  | 2000                    | H       |  |
| b                      | 1         | 25.9'48  | 18,040                  | G       | Water from shallow well near centre of river. There was still open seepage water in the river at this point and evaporation may account for the high saline content.   |
| c                      | 1         | March'45 | 2640                    | G       | Concrete cylinder in river bed - S.A.R. water supply.  |
|                        | 2         | 10.6.'49 | 2400                    | S.A.R.  |  |
|                        | 3         | 17.8.49  | 2270                    | S.A.R.  |  |
|                        | 4         | 20.9.49  | 2300                    | S.A.R.  |  |
|                        | 5         | 25.11.49 | 2300                    | S.A.R.  |  |
| d                      | 1         | March'45 | 3100                    | G       | Farm water supply  |
| ee                     | 1         | 25.9.48  | 700                     | G       | Low rate of extraction-drinking water only. Other wells at Goanikontes more saline.  |
| f                      | 1         | Oct.49   | 2020                    | G       | Seepage ditch drawing the upper ground water from the river bed.   |
|                        | 2         | Oct.49   | 2710                    | G       | Well partly fed by seepage ditch. Heavy extraction.  |
| g                      | 1         | Sept.49  | 3150                    | G       | Hand pump.   |
| h                      | 1         | 23.6.22  | 507                     | G       | Exploratory tube wells at various points at Rooibank. Water table at 0 to 3 ft; but samples taken at 10ft. Sample 1 taken near left bank of river where water table was well below surface and sample 5 near right bank where open water occurs. |
|                        | 2         | 23.6.22  | 821                     | G       |  |
|                        | 3         | 23.6.22  | 1561                    | G       |  |
|                        | 4         | 24.6.22  | 1817                    | G       |  |
|                        | 5         | 24.6.22  | 3840                    | G       |  |



| Locality Symbol on Map | Sample No | Date     | Total dissolved solids | Analyst | Remarks  |
|------------------------|-----------|----------|------------------------|---------|--|
| h                      | 6         | 25.9.48. | 860                    | G       | Town Supply from Concrete cylinder at Rooibank. Same site as sample l. |
| i                      | 1         | Oct.49   | 880                    | G       | Concrete cylinder in Khan River. Town Water Supply.                    |
| j                      | 1         | Oct.49   | 940                    | G       | Fresh water supply at coast used by campers.                           |
| k                      | 1         | Oct.49   | 1700                   | G       | Irrigation well.   |
| l                      | 1         | 11.10.47 | 750                    | G       | Concrete Cylinder, 14 ft. deep.  |
|                        | 2         | 11.10.47 | 1000                   | G       | Exploratory tube well, 40 ft. deep.                                    |

Note: G denotes analysis by Government Laboratory, H by Hansa Brewery, M - miscellaneous and S.A.R. South African Railway Laboratory.

An examination of all available data has lead to the following conclusions:-

- 1). Shortly after floods, water in shallow wells such as the existing municipal well at Swakopmund improves in quality but towards the end of the dry season the water is again highly mineralised. Apart from these seasonal effects the records of the municipal well at Swakopmund indicate a gradual deterioration from season to season during the period 1935 to 1948. The exceptional nature of the 1934 floods may be a possible explanation but the ever increasing consumption may have been the cause of the deterioration by drawing in more highly mineralised water from lower levels.
- 2). Attempts to obtain fresh water by drawing only the top layers as at Palmenhorst (Fig.35) will not neces-

sarily meet with success. The worst water sampled in these rivers was taken from a shallow well near the centre of the river bed (locality b in table). In general, however, deterioration in the quality of the water with increase in depth below river bed is expected. See section through the Swakop River two miles from the coast (Fig.32).

3) Runoff or seepage from the arid and desert regions would appear to be the primary cause of bad quality. The owner of the farm Palmenhorst, for instance, reported that an exceptional downpour occurred north of his farm in March 1949, causing one of the sandy tributaries of the Swakop River to flow. Extremely brack seepage percolated after the flood, killing prosopis trees (mesquite) which grew on the river edge at the confluence with the Swakop River. The sandy beds of tributaries have been observed to contain bands of rock salt (e.g. the tributary entering from the south at Husab). Weak springs with impotable salt water occur in some of the tributary valleys of the Khan, Swakop and Omaruru Rivers chiefly in the longitudes between Usakos and Palmenhorst. Further west the extremely low rainfall does not favour the occurrence of springs.

Summing up, the region east of the longitude of Usakos is without a serious brack water problem and it is suspected that most of the salt content is brought into the river in the arid region between the longitudes of Usakos and Palmenhorst. The contribution to the salt content by the desert region west of Palmenhorst is expected to be less due to the low rainfall, which is seldom strong enough to produce runoff and infiltration.

4). Floods in the main river are clearly the cause of improvement in quality. Consideration should be given to measures which will bring about a more effective mixing of fresh and brack water during floods or better still the ejection of brack water and infiltration of fresh water.

Fig.35.



Fig.36.



Palmenhorst. Seepage ditch in the bed of the Swakop River and pumped irrigation supply, illustrating the method employed by the owner, Mr. Poser, for drawing the upper seepage water for distribution to his lands.

Fig.37.



Fig.38



Vegetable growing on the farm Palmenhorst is a success, notwithstanding the comparatively high salt content of the water. It will be necessary, however, to guard against deterioration of the soil due to the continued use of water of this quality.

(4) Dilution of River Seepage during Floods by means of an interconnected System of Tube Wells.

Fig.39 illustrates the proposed system which will operate automatically as a result of the difference in head between the upper and lower groups of tube wells.

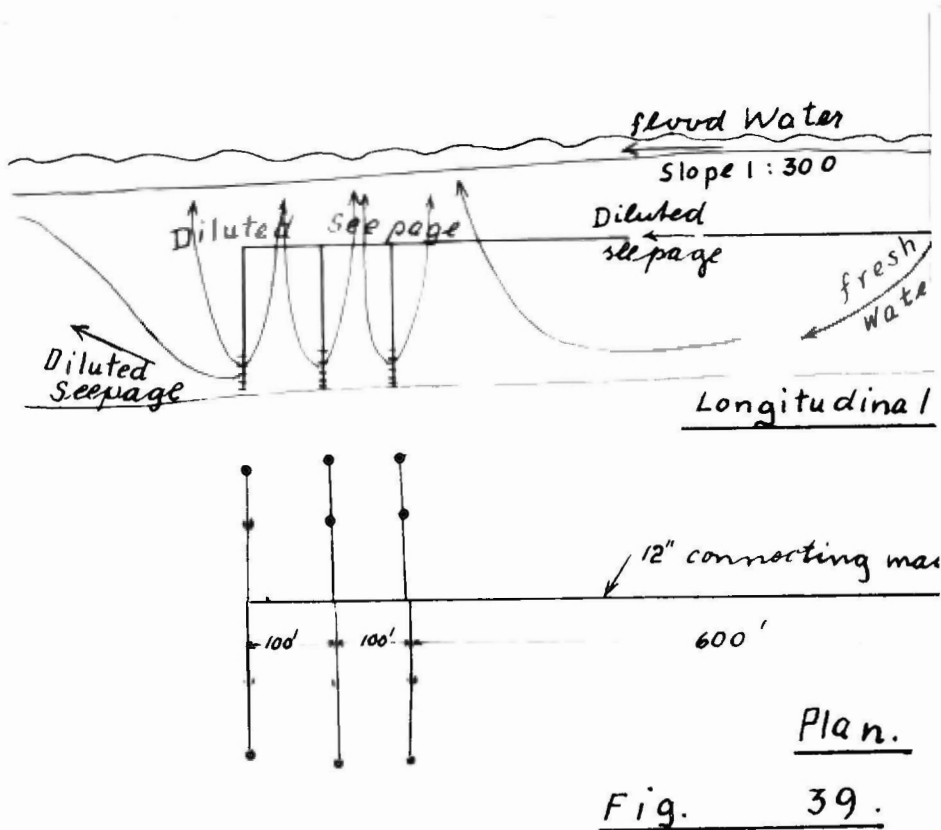


Fig. 39.

On the average the flow from the upper tube wells passes through the equivalent of 80 feet of 6 inch piping, then through 800 feet of 12 inch main, again through an equivalent of 80 feet of 6 inch piping and finally through the perforations of the lower group of tube wells.

The average difference of head causing flow is

$$\frac{800}{300} = 2.67 \text{ feet.}$$

Losses of head are as follows:-

Friction loss in sand from surface to perforations of upper group of tube wells plus a similar loss between perforations of lower group of tube wells and

surface of sand. From Fig.33 it follows that this loss in head =  $2 \times .0000485 q_f = .000097 q_f$ .

$$\text{Entrance and exit losses at tube wells} = 2 \times \left( \frac{q_f}{81,500} \right)^2 = .078 \left( \frac{q_f}{16,000} \right)^2 \quad (\text{page 117}).$$

Friction loss due to a flow  $q_f$  through 160 feet of 6 inch class C everite piping = 0.12 feet for a flow of 16,000 cubic feet a day or in general

$$0.12 \left( \frac{q_f}{16,000} \right)^2 \quad \text{according to the Everite Catalogue}^{14}.$$

Similarly the friction loss due to a flow of  $12q_f$  through 800 feet of 12 inch piping =  $2.0 \left( \frac{q_f}{16,000} \right)^2$

The total static head of 2.67 feet will therefore be accounted for as follows:-

Friction in sand  $0.000097 q_f$

Entrance and exit losses  $0.078 \left( \frac{q_f}{16,000} \right)^2$

Pipe friction losses 6 inch  $0.12 \left( \frac{q_f}{16,000} \right)^2$

" " " 12 "  $2.0 \left( \frac{q_f}{16,000} \right)^2$

$$\text{i.e. } 2.67 = 0.000097 q_f + 2.3 \left( \frac{q_f}{16,000} \right)^2$$

The solution of this quadratic equation is

$$q_f = 12700 \text{ cu.ft./day}$$

$$\text{i.e. } 12 q_f = 152,400 \text{ cu.ft./day}$$

Therefore the estimated natural seepage flow of the river of 23,800 cubic feet a day (page 112) will be mixed with 128,600 cubic feet of fresh water to result in a total flow of 152,400 cubic feet a day in the 12 inch connecting pipe. If the average salt content of the river seepage is 500 parts in 100,000 the salt content in the connecting tube will be  $\frac{23,800}{152,400} \times 500 = 78$  parts per 100,000.

Dilution to 100 parts per 100,000 would still produce a supply of very acceptable quality.

Fig. 40

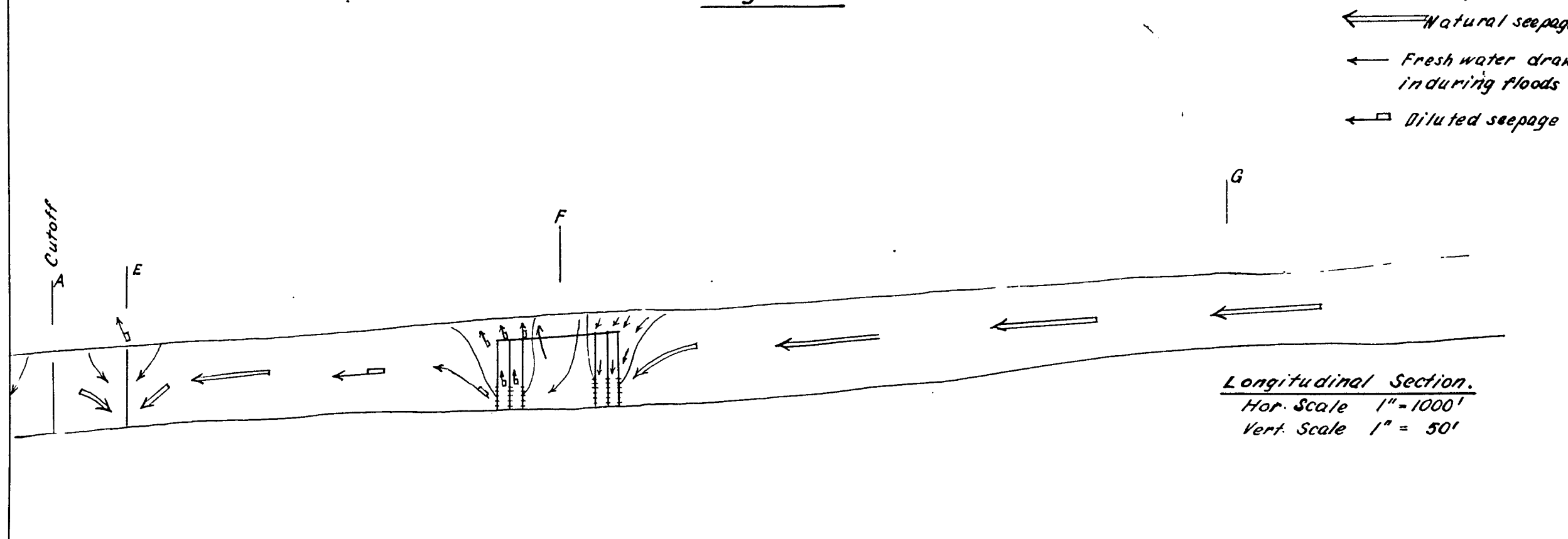


Fig.40 shows the action of the proposed tube well system F in relation to the pump station E and ground water cutoff A.

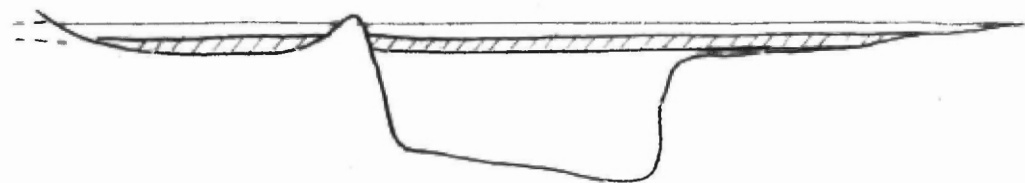
(5) Complete Diversion of River Seepage so that Flood Waters alone will recharge the Sand Bed.

The pump station and cutoff two miles from the coast at Swakopmund will result in a complete diversion of natural seepage from the lower two miles of river, in which the existing pump station is situated. The sand body from which the existing pump station draws its supply will be deprived of the replenishment by natural seepage which occurs at a comparatively steady rate; and will then depend on the erratic floods of the river only for its replenishment. Much greater fluctuations in water table must now be expected; but an improvement in quality will compensate for this.

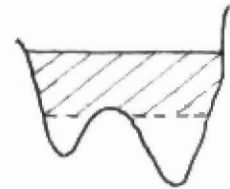
A comparison will be made between present and estimated future fluctuations in water table and yields.

Fig. 41

Stellenbosch University <http://scholar.sun.ac.za>  
Swakop Water Supply - Present Conditions.

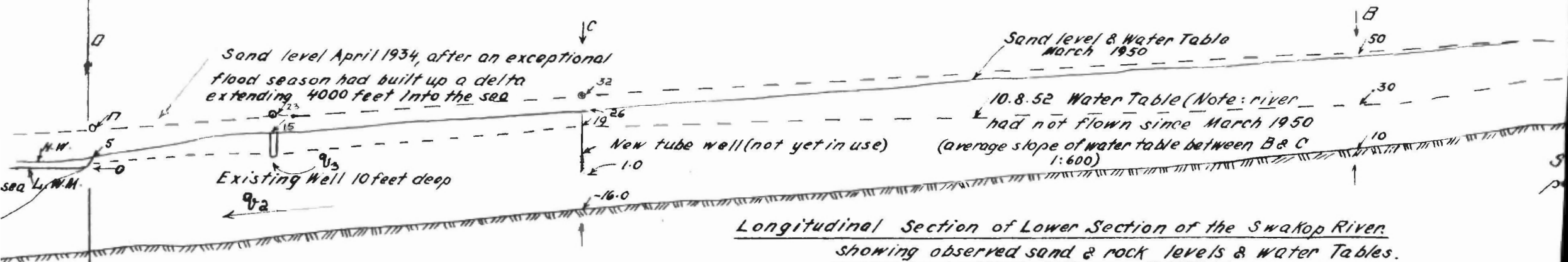


Section C



Section B

Sections:  
Vert. Scale 1" = 50 feet  
Hor. Scale 1" = 500 feet



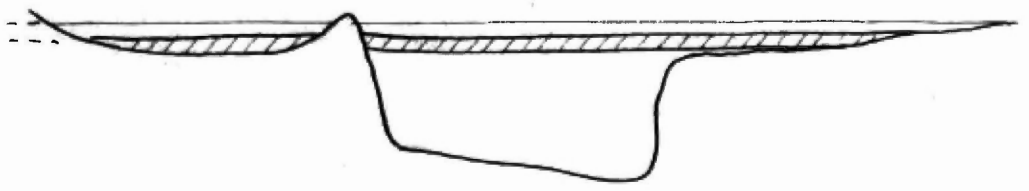
Longitudinal Section of Lower Section of the Swakop River.  
showing observed sand & rock levels & water Tables.

Vert. Scale 1" = 50 feet  
Hor. Scale 1" = 1000 feet.

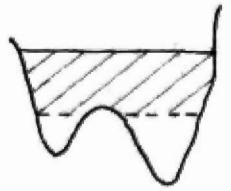


Fig. 41

Stellenbosch University <http://scholar.sun.ac.za>  
Swakop Water Supply - Present Conditions.



Section C

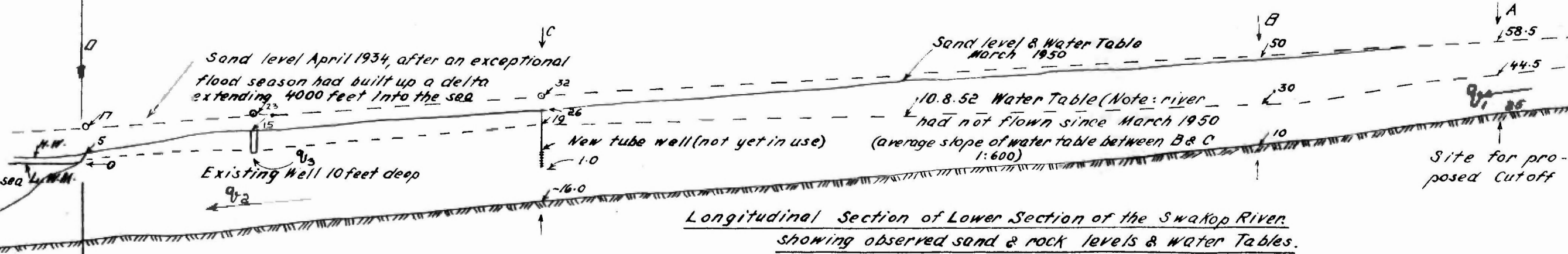


Section B



Section A

Sections.  
Vert. Scale 1" = 50 feet  
Hor. Scale 1" = 500 feet



Longitudinal Section of Lower Section of the Swakop River.  
showing observed sand & rock levels & water Tables.

Vert. Scale 1" = 50 feet  
Hor. Scale 1" = 1000 feet.

(a) Present conditions as demonstrated by observations, 31 March 1950 to 10 August 1952.

The sand of the river bed was completely saturated in March 1950 and no runoff occurred in the 1950/51 and 1951/52 seasons.

On the 10th of August 1952 the water table had receded to the levels shown in Fig.41. The length of the period of depletion, 31.3.50 to 10.8.52 was 863 days.

It will be assumed that the top 3 feet of river sand were depleted by evaporation and that the drained area below this zone produced a specific yield of 25% by volume. (Note: the sand is clean and of comparatively high permeability)

The computation of the total volume of water drained away is as follows, using the notation for sections and flow shown in Fig.41:-

| Section | Drained Area<br>(excluding<br>evaporation<br>zone) | Distance from<br>coast | Estimated volume<br>of water drained. |
|---------|--|------------------------|---------------------------------------|
| A       | nil  | 0                      | 2.5 million cu.ft.                    |
| B       | 5000 sq.ft.  | 4000 ft.               | 10.2 million cu.ft.                   |
| C       | 8000 sq.ft.  | 10250 ft.              | <u>3.0</u> million cu.ft.             |
| D       | 3300 sq.ft.  | 12350 ft.              |                                       |
|         |  | Total                  | 15.7 million cu.ft.                   |

In accordance with the computations on page 112.

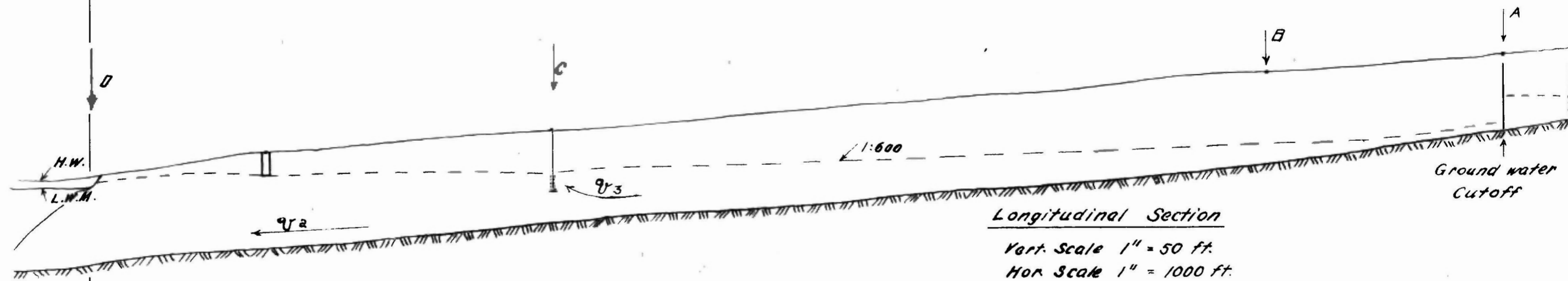
$q_1$  in March 1950 is estimated at 24,000 cu.ft./day and  $q_1$  in August 1952 is estimated at 12,000 cu.ft./day

Average value of  $q_1$  throughout the period say 18,000 cubic feet/day.

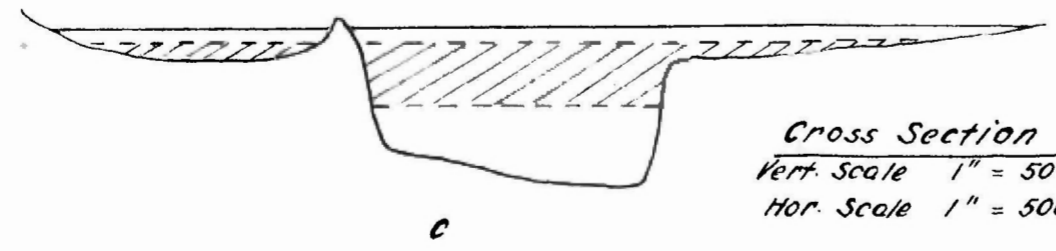
$q_2$  = town consumption = 16,000 cubic feet a day.

Equating the inflow into and efflux and extraction from the basin and allowing for the lowering in water

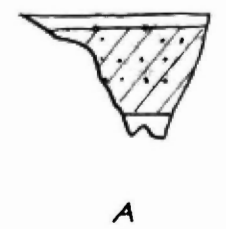
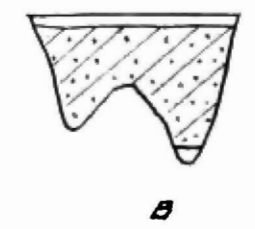
Swakop Water Supply - Operation of Lower Basin  
after installation of proposed tube well & cutoff.



Longitudinal Section  
Vert. Scale 1" = 50 ft.  
Hor. Scale 1" = 1000 ft.



Cross Section  
Vert. Scale 1" = 50'0"  
Hor. Scale 1" = 500'



table,

$$863 q_1 + 15,700,000 = 863 q_2 + 863 q_3$$

$$\text{i.e. } 863 q_2 = 15,700,000 + 863 (q_1 - q_3)$$

$$q_2 = 20,200 \text{ cubic feet/day.}$$

(b) Future conditions after installation of cutoff at A and tube well at C.

Following the notation of Fig.42, an analysis of results which are expected is as follows:-

The flow at A i.e.  $q_1$  will be reduced to zero.

Due to the low intake level of the tube well the discharge into the sea  $q_2$  will be greatly reduced, especially towards the end of the depletion period when the water table between sections D and C becomes practically level.

It will be assumed that during the first half of the depletion period the discharge  $q_2$  has the same value as computed for the 1950/52 period. Thereafter, however, it will be assumed to be reduced at a uniform rate to zero i.e.  $q_2$  will have an average value of  $\frac{3}{4} \times 20,200 = 15,150$  cubic feet/day.

Assuming again that the upper three feet are depleted by evaporation, the computation of the total volume drained is as follows:-

| Section | Drained Area<br>(excluding<br>evaporation<br>zone) | Distance from<br>coast | Estimated Volume<br>of Water Drained. |
|---------|--|------------------------|---------------------------------------|
| D       | nil  | 0                      | 6.4 million cu.ft.                    |
| C       | 12750  | 4000                   | 18.2 million cu.ft.                   |
| B       | 10500  | 10,250                 | <u>4.3</u> million cu.ft.             |
| A       | 6000   | 12,350                 |                                       |
|         |  | Total                  | 28.9 million cu.ft.                   |

Sand and gravel drains were laid on the floor of the dam basin and extend from the deeper portions of the basin to the area where inflowing floods will deposit the coarser sediments. The dam was so designed that the full supply level can readily be raised by six feet at a future date. The diagram (Fig. 44) shows how it is hoped eventually to retain a permanent asset even after complete silting of the reservoir.

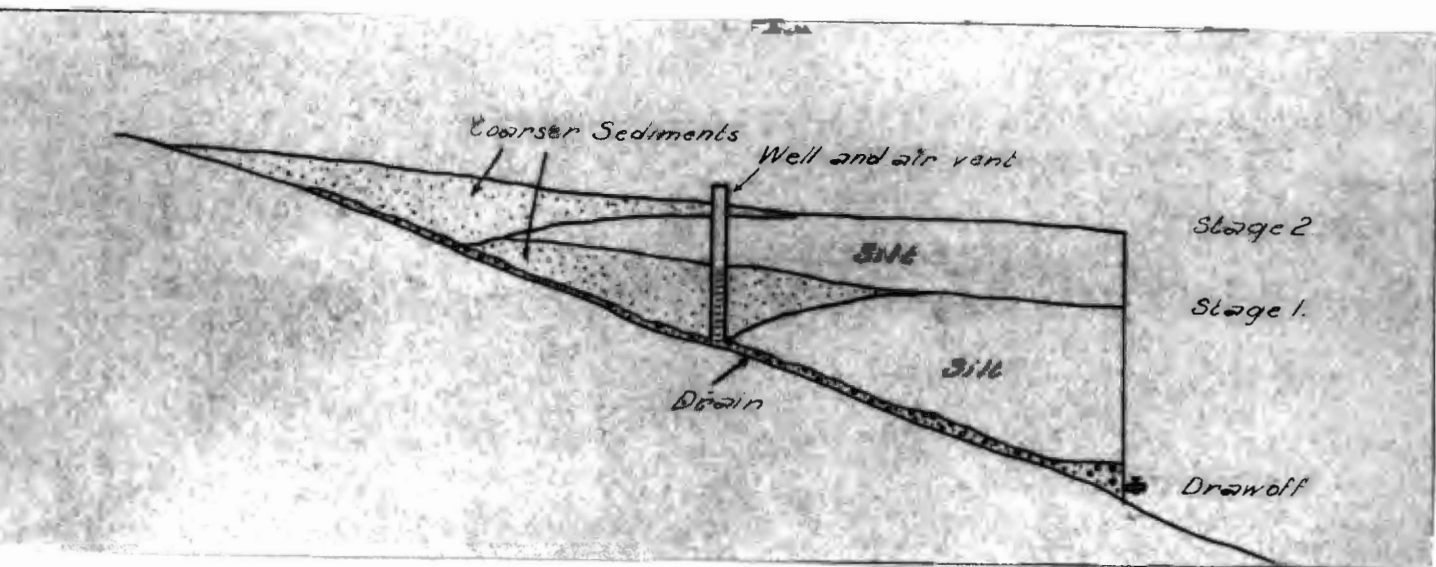


Fig. 44.

Ultimate development of a small storage reservoir  
Longitudinal section through the basin.

Insufficient permeability, one of the chief difficulties of water storage in sand filled dams, will be accentuated by the large silt bodies deposited. The water yielded by the coarser sediments may prove to be sufficient for a small herd of cattle only. It may be necessary in addition to maintain part of the storage capacity by silt removal at regular intervals. This method of constructing in two large stages cannot be recommended as a general solution until more information becomes available.

The storage basins of two small dams in this vicinity were surveyed with the object of determining the rate of silting.

The results are as follows:-

| Name of Dam                                    | Spitskoppies | Oruaondo                  |
|--|--------------|---------------------------|
| Catchment Area                                 | 1.0          | 1.0 square miles          |
| Period   | 1941 to 1944 | 1940 to 1944              |
| Number of rainy seasons                        | 3            | 4                         |
| Volume of sediments deposited                  | 0.13         | 0.14 million cu.ft.       |
| Rate of Sedimentation                          | 43,000       | 35,000 cu.ft./sq.ml/annum |
| Equivalent denudation over the catchment area. | .018         | .015 inches per annum     |
| Original capacity of Dam                       | 0.92         | 1.00 million cu.ft.       |

It is of interest to compare <sup>these</sup> results with the rates of silting observed in the Karroo. Kokot <sup>15</sup> quotes the following mean annual rates of denudation over the catchment areas of certain reservoirs in that region:-

| Reservoir          | Mean rate of denudation<br>(inches per annum) |
|--------------------|---|
| Lake Mentz         | 0.011   |
| van Rhyneveldspass | 0.016   |
| Lake Arthur        | 0.022   |
| Grass Ridge        | 0.012   |

A more detailed survey during 1952 of the silt deposits in Oruaondo dam yielded the following results:-

In the twelve years, 1940 to 1952, the storage capacity was reduced by 422,800 cubic feet. Actually 427,160 cubic feet of sediments were deposited, some of which above full supply level.

The classification of the materials deposited is as follows:-

|             |                   |
|-------------|-------------------|
| Silt        | 279,000 cu.ft.    |
| Fine Sand   | 133,880 "         |
| Medium Sand | 13,280 "          |
| Coarse Sand | 1,000 "           |
| Gravel      | <u>negligible</u> |
| Total       | 427,160 cu.ft.    |

The average sedimentation during the twelve years can be expressed as follows:-

|       |                                     |
|-------|-------------------------------------|
| Silt  | 23,250 cu.ft.per sq.mile per annum. |
| Sand  | <u>12,347</u> " " " " " "           |
| Total | 35,597 " " " " " "                  |

or as 0.0100, 0.0053 and 0.0153 inches denudation over the catchment area for silt, sand and total sediments, respectively.

The maximum depth of the deposits in Oruaondo Dam is only about five feet. Wells in the dam basin are already of some use as they yield water for cattle for approximately three months after the open water has been depleted.

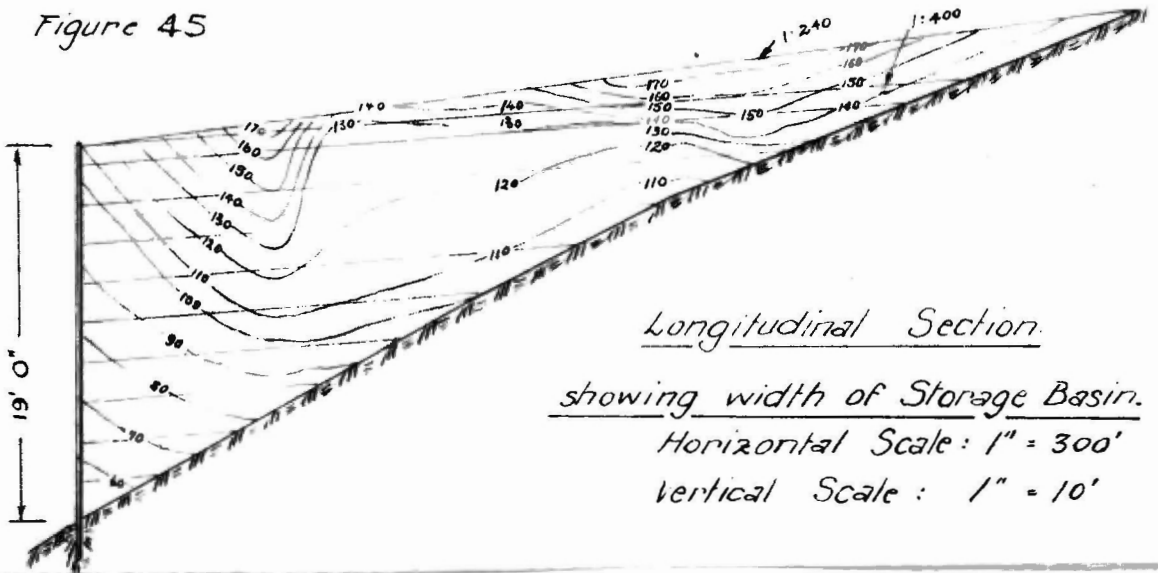
A method more certain to lead to useful results is stage construction. A weir is raised in stages so that velocities of flow through the basin during floods are appreciable and most of the fine silt is carried over the dam wall, whereas the coarser fractions are retained. Coarser sediments will absorb flood water more readily and possess higher yields and permeability, than the finer material deposited where stage construction is not adopted.

## (2) Sand Storage Dam in Aukeigas.

A sand storage dam was developed at Aukeigas near Windhoek, by the stage construction method. (first stage in 1939 - in all five stages). Sedimentation and water yield were observed in detail.

The catchment area of the dam is 14 square miles in extent. An elevation of the basin with "contours" indicating the width at any point <sup>is</sup> given in the following diagram:-

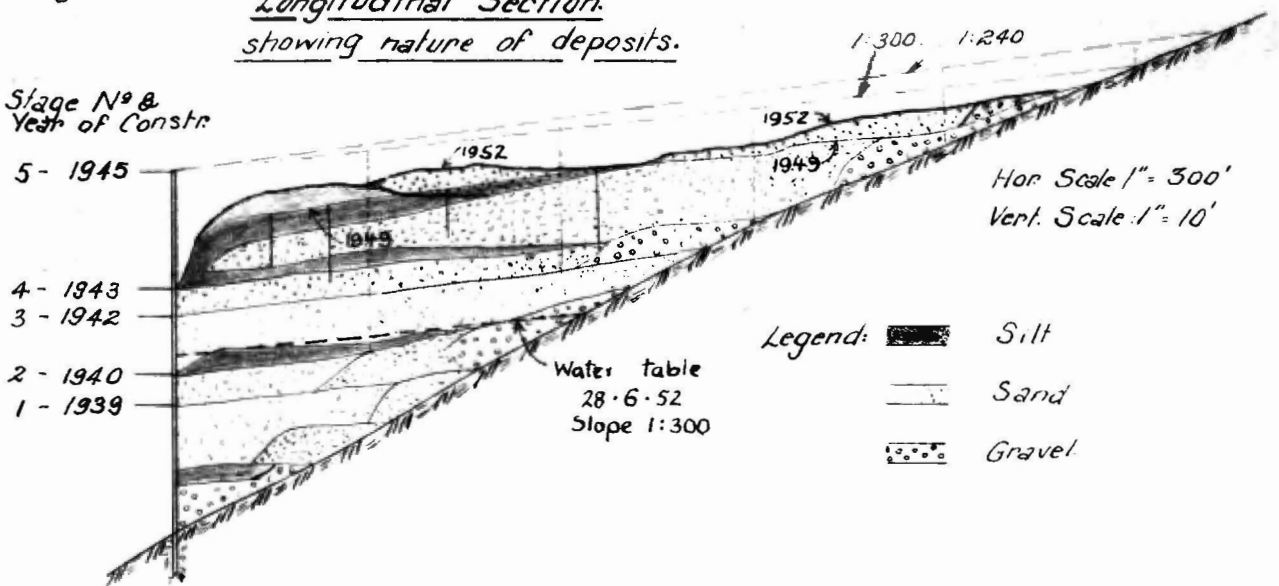
Figure 45



The following diagrams and table show the nature of the material deposited in the various construction stages:-

In Figure 46

Longitudinal Section.  
showing nature of deposits.





| Stage No             | Season                                | Material deposited(cu.ft.) |           |         | Total for season | Total for stage |
|----------------------|---------------------------------------|----------------------------|-----------|---------|------------------|-----------------|
|                      |                                       | Coarse Sand & Gravel       | Fine Sand | Silt    |                  |                 |
| 1                    | Carted in during construction 1939/40 | 10,300                     | 8,000     | nil     | 18,300           | 102,300         |
|                      |                                       | nil                        | 80,000    | 4,000   | 84,000           |                 |
| 2                    | 1940/41                               | nil                        | 14,400    | nil     | 14,400           | 140,500         |
|                      | 1941/42                               | 30,000                     | 96,000    | nil     | 126,100          |                 |
| 3                    | 1942/43                               | nil                        | 158,700   | 16,900  | 175,600          | 175,600         |
| 4                    | 1943/44                               | 23,100                     | 106,900   | nil     | 130,000          | 130,000         |
|                      | 1944/45                               | nil                        | nil       | nil     | nil              |                 |
| 5                    | 1945/46                               | nil                        | nil       | nil     | nil              | 572,400         |
|                      | 1946/47                               | 40,500                     | 291,500   | 105,400 | 437,400          |                 |
|                      | 1947/48                               |                            |           |         |                  |                 |
|                      | 1948/49                               |                            |           |         |                  |                 |
|                      | 1949/50                               |                            |           |         |                  |                 |
|                      | 1950/51                               | 14,500                     | 94,500    | 26,000  | 135,000          |                 |
| 1951/52              |                                       |                            |           |         |                  |                 |
| Total for 13 seasons |                                       | 118,400                    | 850,100   | 152,300 | 1,120,800        | 1,120,800       |

Subtracting the 18,300 cu.ft. which were carted in, it will be seen that the 1,102,500 cu.ft. of material deposited by floods consists of 100,100 cu.ft. of coarse sand and gravel, 850,100 cu.ft. of fine sand and 152,300 cu.ft. of silt.

The sedimentation may be expressed as follows:-

|                      |            |                      |   |   |
|----------------------|------------|----------------------|---|---|
| Silt                 | 835        | cu.ft./sq.mile/annum |   |   |
| Fine Sand            | 4670       | "                    | " | " |
| Coarse Sand & Gravel | <u>550</u> | "                    | " | " |
| Total                | 6055       | "                    | " | " |

It will be observed that fine sand predominates. If stages were to be constructed so that only coarse sand and gravel were retained, progress would be very slow indeed. Due to the low permeability of the fine material complete saturation is only obtained after copious floods. In this

connection it is of interest to compare the fine grading of the material in Aukeigas Dam with the coarser grading of the material in the Swakop River (page 143 ).

The foundations of the sand storage dam at Aukeigas were cement grouted until the seepage below the dam was more or less equivalent to the drawoff required. The seepage can be accurately gauged at a rock bar a short distance below the dam; and by observing the yield from time to time it was possible to arrive at reliable figures for the water yield of different horizons of the sand fill at different times.

The following table gives the percentage yield of sediments from three feet below the surface downwards, throughout the history of the scheme:-

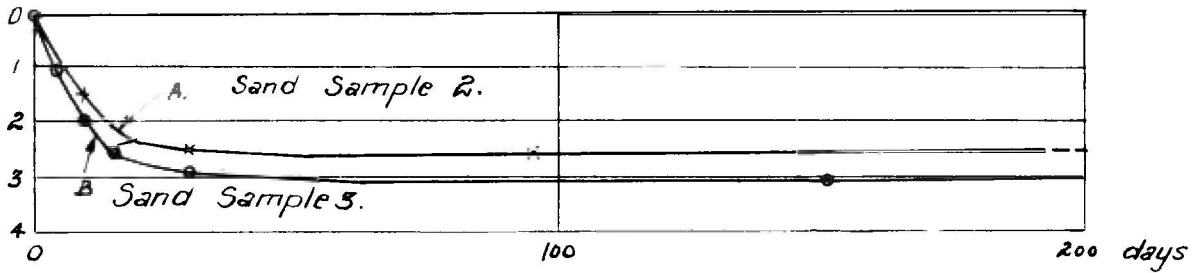
| Season  | Percentage yield from three feet below the surface downwards.  | Remarks.                   |
|---------|--|----------------------------|
| 1940/42 | 25.0%  | Exceptionally good season. |
| 1942/43 | 9.6%   | Normal.                    |
| 1943/44 | 8.0%   | Normal.                    |
| 1944/45 | 2.5% of total sand volume below the three foot level.<br>20% of sand volume below water table (see Fig.18) | Exceptionally poor season. |
| 1945/46 | 12.0%  | Normal.                    |
| 1946/47 | 17.2%  | Normal.                    |
| 1947/48 | 14.4%  | Normal.                    |
| 1948/49 | 14.0%  | Normal.                    |
| 1949/50 | ?  | Poor runoff season.        |
| 1950/51 | 16.9%  | Poor runoff season.        |

The yield after the 1941/42, 1943/44 and 1950/51 seasons will now be examined in greater detail:-

Evaporation from saturated Sand.

Water Level - Depth below sand in feet.

1 In tank and stand pipe.



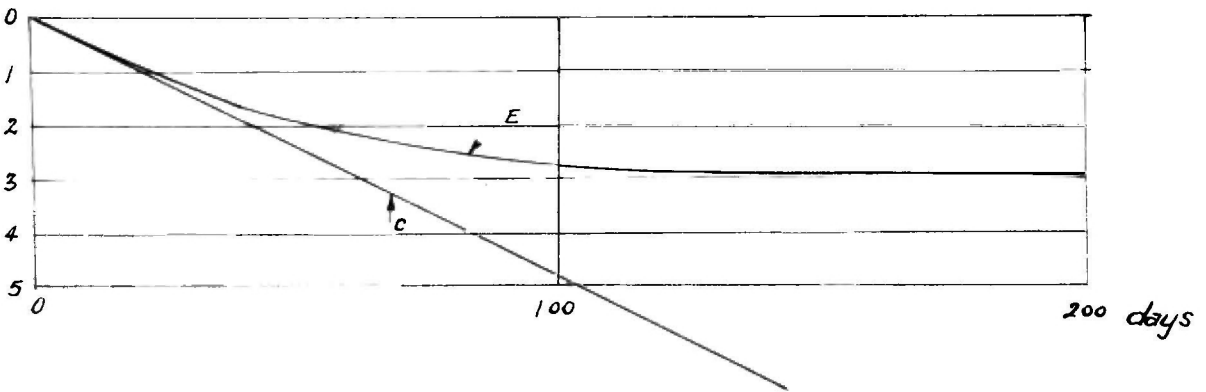
Curve A. Evaporation in tank in the Sand Storage Dam at Aukeygas (Aug. to Dec. 1943)

Curve B. Evaporation in stand pipe, 6" diameter. (June to Nov. 1943)

Note: A description of the stand pipe and samples are given on pages 100. & 143 to 144 respectively

Water Level at dam wall - Feet below crest.

2 In Sand Filled Dam.

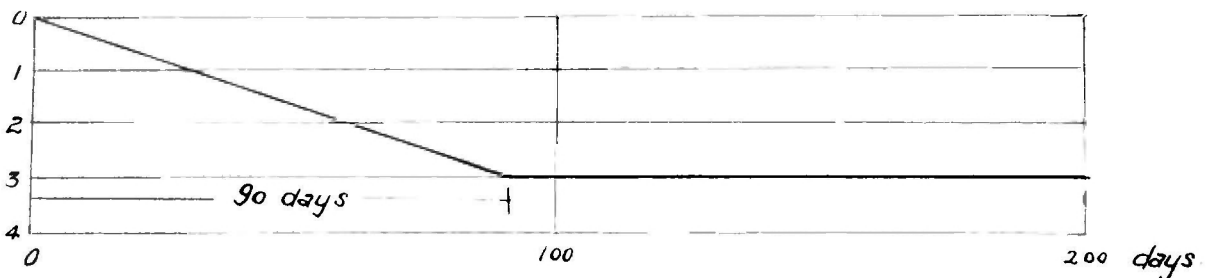


Curve C Depletion curve observed in the Sand Storage Dam at Aukeygas (April to July 1942)

Curve E: Evaporation alone. (Deduced from curve C and the known extraction and specific yield.)

3. Simplified Curve adopted in Analysis.

Water Level at Dam Wall - Feet below crest.



Some typical analyses of the grain size of sand and silt are given below:-

Sample 1. River bed material in the Swakop River near Okahandja.

| Grain Size (inches) | Percentage coarser than this grain size | Method of Determination. |
|---------------------|---|--------------------------|
| .188                | 7.4%                                    | Sieve U.S.A.No.4         |
| .094                | 14.6%                                   | No.8                     |
| .047                | 22.6%                                   | No.16                    |
| .037                | 28%                                     | No.20                    |
| .023                | 33.0%                                   | No.30                    |
| .018                | 48%                                     | No.40                    |
| .012                | 53.0%                                   | No.50                    |
| .009                | 73.0%                                   | No.80                    |
| .006                | 94.0%                                   | No.100                   |
| .003                | 100.0%                                  | No.200                   |

Sample 2. Representative mixture of samples from stages 1 to 4 of the sand storage dam at Aukeigas.(Relatively small stages).

| Grain Size (inches) | Percentage coarser than this grain size | Method of Determination.  |
|---------------------|---|---|
| .188                | 1.6%                                    | Sieve U.S.A.No.4  |
| .094                | 4.0%                                    | 8   |
| .047                | 6.3%                                    | 16  |
| .037                | 10.7%                                   | 20  |
| .023                | 12.0%                                   | 30  |
| .018                | 15.1%                                   | 40  |
| .012                | 17.0%                                   | 50  |
| .009                | 62.5%                                   | 80  |
| .006                | 96.0%                                   | 100   |
| .003                | 98.6%                                   | 200   |
| .001                | 99.6%                                   | Settling velocity for .001" is taken as 1" a minute.(see footnote page 144) |
| 0                   | 100%                                    |   |

Sample 3. Typical material from a large sand bank in Stage 3 of the Bulskop Dam near Okahandja. (Relatively small stage).

| Grain Size | Percentage coarser than this grain size. | Method of Determination.                                |
|------------|--|---|
| .188       | 0%                                       | Sieve U.S.A.No 4  |
| .094       | 0.3%                                     | 8   |
| .047       | 0.4%                                     | 16  |
| .037       | 0.8%                                     | 20  |
| .023       | 1.5%                                     | 30  |
| .018       | 2.3%                                     | 40  |
| .012       | 4.5                                      | 50  |
| .009       | 50.5                                     | 80  |
| .006       | 76.5                                     | 100   |
| .003       | 98.5                                     | 200   |
| .001       | 100.0                                    | Settling velocity for .001" is taken as 1" per Minute.* |
| 0          | 100.0%                                   |   |

Sample 4. Representative mixture of samples from Stage 5 of the sand storage Dam at Aukeigas. (Exceptionally large stage in which about 50% of all transported material brought down by floods are retained). In the case of Samples 2 and 3 only about 25% of all transported material were retained and 75% were carried with the flood water over the dam walls.

| Grain Size | Percentage coarser than this grain size. | Method of Determination.                                |
|------------|--|---|
| .188       | 1.1%                                     | Sieve U.S.A.No. 4                                       |
| .094       | 2.5%                                     | 8   |
| .047       | 4.6%                                     | 16  |
| .037       | 8.1%                                     | 20  |
| .023       | 9.0%                                     | 30  |
| .018       | 10.1%                                    | 40  |
| .012       | 14.7                                     | 50  |
| .009       | 47.6                                     | 80  |
| .006       | 72.3                                     | 100   |
| .003       | 74.8                                     | 200   |
| .001       | 89.3                                     | Settling velocity for .001" is taken as 1" per minute.* |
| 0          | 100%                                     |   |

\* With the viscosity of water = .014 poises and specific gravity of grains = 2.65, the settling velocity in inches per second, in accordance with Stokes Law<sup>7</sup>, is equal to  $16,300 \times (\text{diameter of particles in inches})^2$ . With grains .001 inches in diameter, the settling velocity is therefore .0163 inches/second or approximately 1 inch/minute.

### (3) Design of Sand Storage Dams.

The basic principle involved is to limit the size of stages so that velocities of flow through the basin are sufficiently high to transport most of the fine sediments over the dam crest. It is obvious that 100% perfection cannot be achieved as with the smallest floods the velocity of flow will always be low enough for the deposition of fine silt. Subsequent floods of greater magnitude may scour part or all of the fine material and deposit sand. In practice silt beds and lenses occur in successful sand storage dams and the object of design is not to eliminate silt entirely, but to reduce the extent of silt bodies so as not to interfere unduly with the proper functioning of the reservoir.

In the sand storage dam in Aukeigas, four stages which were small enough to retain satisfactory deposits were followed by a stage about seven times their average capacity. The object of constructing the fifth stage was to observe the nature of the sedimentation from year to year and thus to determine the maximum residual capacity which will still retain satisfactory sediments.

The original height of the stage was six feet with a capacity of 1,038,100 cubic feet. During the four rainy seasons 1945/46 to 1948/49, 437,400 cubic feet of sediments were deposited of which 105,400 cubic feet were silt. The silt occurred mainly in two beds, each an average thickness of about nine to twelve inches, extending over the whole of the lower half of the basin. This type of deposition is to be avoided because the silt takes up valuable space which could have been available for water storage and interferes with infiltration during floods.

After the 1948/49 season the residual capacity amounted to 600,700 cubic feet with a residual height near the dam wall of 3 feet. In the three seasons which followed 14,500 cubic feet of coarse sand and gravel, 94,500 cubic feet of fine sand and 26,000 cubic feet of silt were deposited. Some of the silt in the central portion of the basin was scoured away by floods thereby increasing the infiltration area (page 137). There was a marked improvement in the grading of deposits and the position in which they were deposited compared with the previous three seasons. Velocities of flow through the storage basin had increased due to the reduction in the depth of water. It is considered that the velocities which were experienced with a residual storage depth of three feet near the dam wall can be adopted as the minimum permissible velocities for sand storage dams.

The catchment area of the sand storage dam at Aukeigas is 14 square miles, the maximum probable flood

$Q_1 = 3600 \times 14^{0.45} = 11,800$  cusecs (page 28) and the peak intensity of a series of equivalent floods  $Q \doteq 0.6 Q_1 \doteq 708$  cusecs, adopting a coefficient more or less in keeping with the observations plotted in Fig.21. (Definition of equivalent floods, page 79).

The velocities of flow occurring near the dam wall at various construction stages of the sand storage dam at Aukeigas will now be determined for the flood discharge  $Q = 708$  cusecs.

| Stage No. | $h$ (feet) | $H_1$ (feet) | $H_2 = \left(\frac{Q}{3.3A}\right)^{2/3}$ (feet) | $A_F = A(H_1 + H_2)$ (sq.ft) | $v_a = \frac{Q}{A_F}$ (ft./sec) |
|-----------|------------|--------------|--|------------------------------|---------------------------------|
| 1         | 73         | 4            | 2.05   | 441                          | 1.6                             |
| 2         | 77         | 1.5          | 1.98   | 268                          | 2.6                             |
| 3         | 87         | 3            | 1.83   | 420                          | 1.7                             |
| 4         | 90         | 1.5          | 1.78   | 296                          | 2.4                             |
| 5 in 1945 | 110        | 6            | 1.56   | 833                          | 0.8                             |
| 5 in 1949 | 110        | 3            | 1.56   | 502                          | 1.4                             |

$l$  = crest length of dam wall.

$H_1$  = height of weir crest above sand filling.

$H_2$  = depth of overflow over weir crest.

$A_F$  = flow cross section a short distance upstream of the dam wall.

$v_a$  = velocity of flow a short distance upstream of the dam wall.

Satisfactory sedimentation resulted in stages 1 to 4 with a value of  $v_a$  varying from 1.6 to 2.6 feet/second. In stage 5,  $v_a$  originally was only 0.8 ft./sec. which was insufficient to produce effective scouring. When  $v_a$  was increased to 1.4 ft./sec. as a result of shallower water depth, sedimentation was again satisfactory. It is suggested that  $v_a = 1.5$  ft./sec. be adopted as the limiting condition for the design of stages of sand storage dams.

The future stages of the sand storage dam at Aukeigas will now be designed on this basis.

$$Q = 708 \text{ cusecs as before} \quad \& \quad A_F = \frac{Q}{v_a} = \frac{Q}{1.5}$$

| Proposed Stage No. | $l$ | $H_2 = \left(\frac{Q}{3.3b}\right)^{2/3}$ | $A_F = \frac{Q}{1.5}$ | $H_1 = \frac{A_F}{l} - H_2$ | Adopted value of $H_1$ |
|--------------------|-----|---|-----------------------|-----------------------------|------------------------|
| 6                  | 119 | 1.5                                       | 472                   | 2.4                         | 2.0                    |
| 7                  | 130 | 1.4                                       | "                     | 2.3                         | 2.0                    |
| 8                  | 144 | 1.3                                       | "                     | 2.1                         | 2.0                    |
| 9                  | 155 | 1.3                                       | "                     | 1.9                         | 1.5                    |
| 10                 | 170 | 1.2                                       | "                     | 1.8                         | 1.5                    |

A full picture of existing and proposed stages of the sand storage dam at Aukeigas is as follows:-

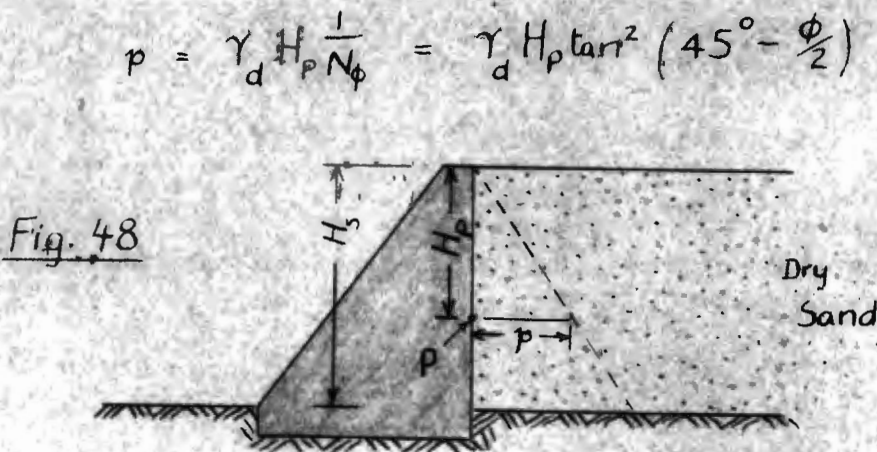


| Stage No.   | Total estimated sediment volume to end of stage | Sediment volume in each stage | Height of dam wall at end of stage (feet) |
|-------------|---|-------------------------------|---|
| 1(existing) | 116,700 cu.ft.                                  | 116,700 cu.ft.                | 7   |
| 2 "         | 242,800 "                                       | 126,100 "                     | 8.5                                       |
| 3 "         | 418,400 "                                       | 175,600 "                     | 11.5                                      |
| 4 "         | 548,400 "                                       | 130,000 "                     | 13  |
| 5 "         | 1,586,500 "                                     | 1,038,100 "                   | 19  |
| 6(proposed) | 1,880,000 "                                     | 293,500 "                     | 21  |
| 7 "         | 2,500,000 "                                     | 620,000 "                     | 23  |
| 8 "         | 2,340,000 "                                     | 840,000 "                     | 25  |
| 9 "         | 4,000,000 "                                     | 660,000 "                     | 26.5                                      |
| 10 "        | 4,820,000 "                                     | 820,000 "                     | 28  |

It has taken 13 years for the deposition of just over 1 million cubic feet of sediments and it is therefore clear that several decades will elapse before the dam will have reached the final stage envisaged.

In designing sand storage dams cognisance should be taken of the fact that the pressure on the dam wall due to the saturated sand is greater than that due to water alone.

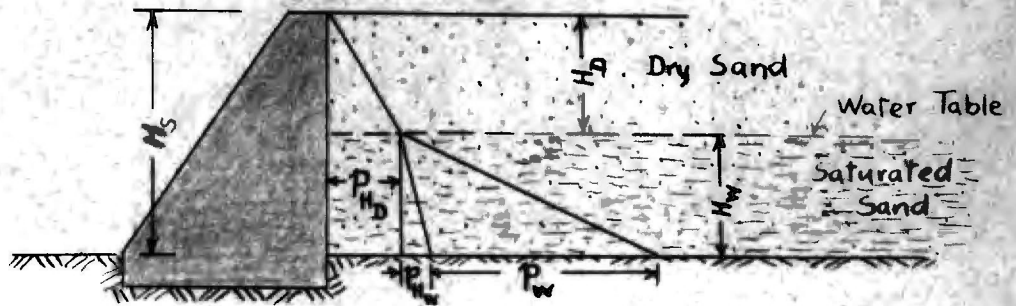
The pressure of dry sand on a vertical retaining wall is given in the following diagram:- (Rankine)



If the sand is saturated with water, full hydrostatic pressure will be exerted on the wall. The sand pressure will, however, be reduced, since only the submerged weight of the sand will now act<sup>12</sup>.

Partly saturated storage basin:-

Fig. 49



$$p_{H_d} = \text{pressure due to dry sand} \\ = \gamma_d H_d \tan^2(45^\circ - \phi/2)$$

$$p_{H_w} = \text{pressure due to saturated sand} \\ = \gamma' H_w \tan^2(45^\circ - \phi/2)$$

$$p_w = \text{water pressure} = \gamma_w H_w$$

$$\gamma' = \text{Submerged unit weight.}$$

The pressure on a vertical wall due to water alone and due to water plus submerged sand in a practical example, is compared in the following diagram:-

The basic data assumed are,

$$H_s = 10 \text{ feet}$$

$$\text{Depth of overflow over the wall} = 4 \text{ feet}$$

$$\gamma_d = 89 \text{ lb/cu.ft.}$$

$$\gamma' = 52 \text{ lb/cu.ft.}$$

$$\gamma_w = 62.5 \text{ lb/cu.ft.}$$

$$\phi = 30^\circ$$

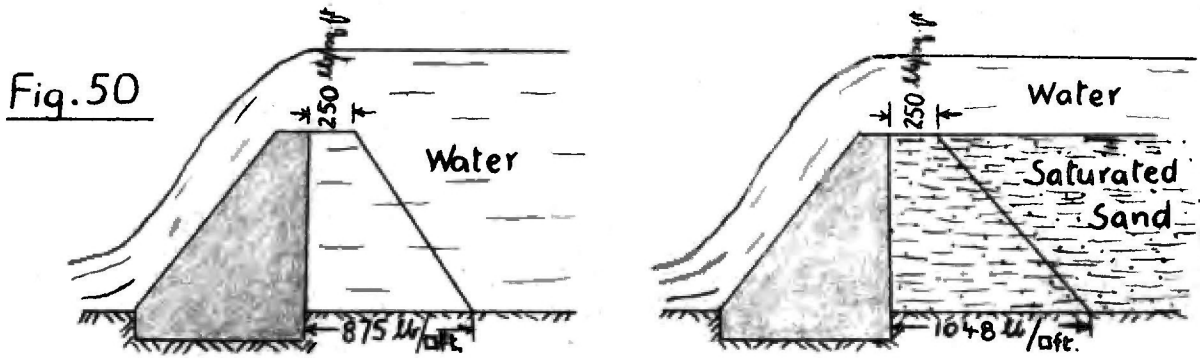
$$\text{Water pressure at crest} = 4 \gamma_w = 250 \text{ lb/sq.ft.}$$

$$\text{" " " base} = 14 \gamma_w = 875 \text{ lb/sq.ft.}$$

$$\text{Submerged sand pressure at base} = 52 \times 10 \times \tan^2(45 - 15)$$

$$= 520 \tan^2 30^\circ$$

$$= 173 \text{ lb/sq.ft.}$$



In practice weirs designed for retaining sand to store water will therefore be slightly more massive than weirs for retaining water only.

Special attention should also be given to the design of the well and drains. Fig. 51 shows details for a small sand storage dam.

According to Terzaghi and Peck<sup>12</sup> experiments have shown that the following relation should be observed between grading of material to be drained and material of which the filter may be constructed.

Let  $D_{15}$  denote the 15% size of the coarsest beds in contact with the filter and  $D_{85}$  the 85% size of the finest beds in contact. The 15% size of a suitable filter must then be between  $4D_{15}$  and  $4D_{85}$  (The notation  $D_{15}$ ,  $D_{85}$  etc. is explained on page 190 ).

The same authors on page 119 of their publication advocate drainage slots equal in size to  $D_{60}$  of the drained material.

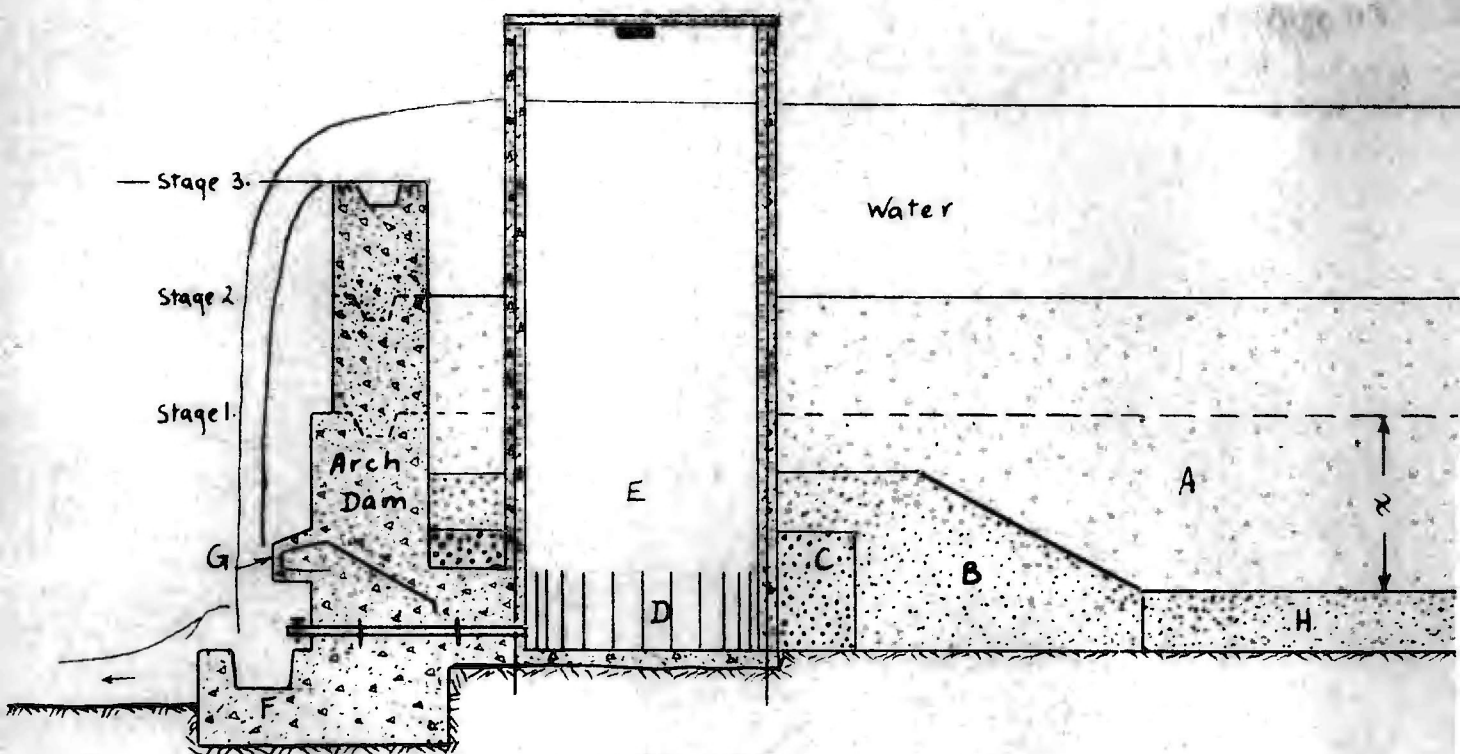


FIGURE 51  
DRAWOFF DETAILS FOR SAND STORAGE DAM.  
Scale 1" = 5'0"

- A. Sand deposited in basin by floods say  $D_{15} = .0045$  and  $D_{85} = .011$  inches. (see sieve analysis, page 181)
- B. Fine filter.  
 $D_{15}$  not coarser than four times  $D_{85}$  of A or say .044 inches.  
 $D_{85}$  say .088 inches.
- C. Coarse filter.  
 $D_{15}$  not coarser than four times  $D_{85}$  of B or say 0.3 inches.  
 $D_{60}$  say 0.5 inches.
- D Intake slots =  $D_{60}$  of C = 0.5 inches.
- E six feet diameter collecting well with 0.5 inch intake slots and  $1\frac{1}{2}$ " diameter outlet pipe.
- F Drinking trough for stock.
- G Reinforced concrete projection above outlet valve.
- H Fill similar in grading to B carted in to reduce the height of the first stage to the maximum permissible.
- Note:- 1). Details of filters and slots can be adapted in accordance with the materials available on the site.
- 2). If material similar to A is available it can be

filled in over the fine filter B in order to eliminate all possibility of clogging of filters by flood water.

The average permeability in a horizontal direction will determine the rate at which water can be extracted from a sand storage reservoir. In the sand storage dam at Aukeigas this average permeability was found to be 34 feet a day (page 96) and is sufficient to allow the water to percolate to the drainage system at the well at a rate commensurate with the consumption. In a large sand storage dam, designed for the delivery of a much more copious water supply, however, a longitudinal drain may have to be provided. The following example will illustrate the problem:-

A sand storage dam in a large river is to be drawn upon at the rate of 16,000 cubic feet a day. The slope of the sand is 1:300 and the cross sectional area of the basin near the dam wall 10,000<sup>sq.</sup> feet, when the dam is full; and 2500 sq.ft., when  $\frac{7}{8}$ th of the supply has been depleted. With the slope of the water table at 1:300 the seepage flow at the dam wall will be 1670 cu.ft/day when the dam is full and with a slope of the base of 1:150 a rate of extraction of 883 cu.ft/day will be possible when the dam is  $\frac{1}{8}$  full.\* The natural seepage is thus below requirements. A four inch diameter asbest-cement pipe will deliver the required quantity of 16,000 cubic feet a day at a hydraulic gradient of 1:300. The pipe should be slotted and surrounded with gravel and sand filters in accordance with the principles enumerated on page 150. The pipe diameter should not be so selected that it can deliver the desired quantity at a hydraulic gradient flatter than that of the surface of the sand; as this would result in too much surplus water being drained from the upper part of the sand reservoir to the lower and being discharged over the dam wall. It will not be necessary to extend the drain more than half-way up the length of the storage basin,

\*Assumed average permeability in a horizontal direction 50ft/day

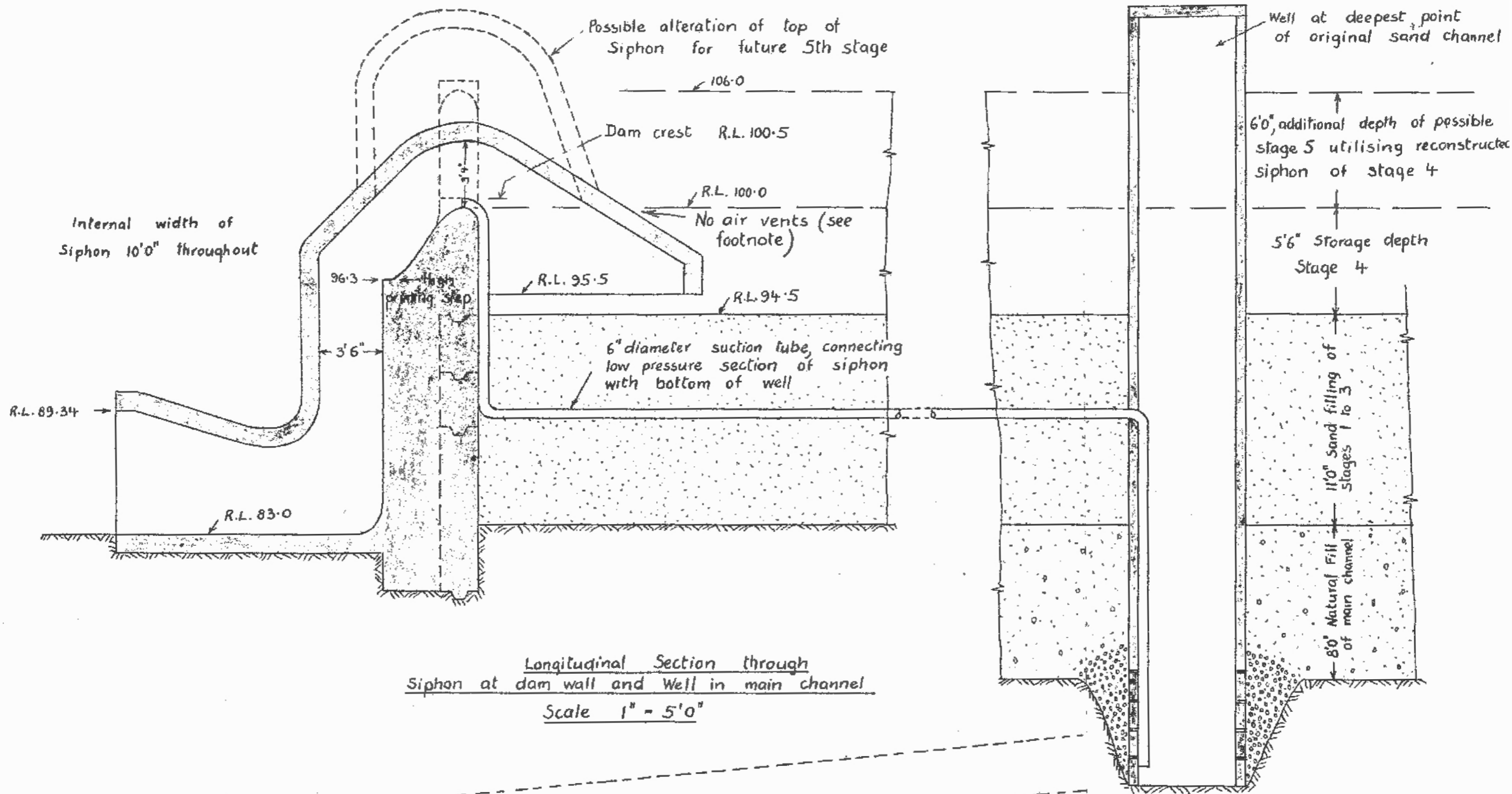
CHAPTER 5SAND STORAGE DAMS WITH SIPHONS.(1) Requirements to be met in the Design of the Siphon.

The siphon must prime during large floods and lower the water table so that scouring of the silt in the basin can take place. When the intensity of the inflowing flood has dropped to a certain value the siphon action must break off abruptly and the basin must again be filled before the end of the flood. The priming of the siphon must occur with sufficient frequency to bring about appreciable scouring of the basin. On the other hand it may not occur with floods which are too small, as this would leave the basin partly depleted after siphon action.

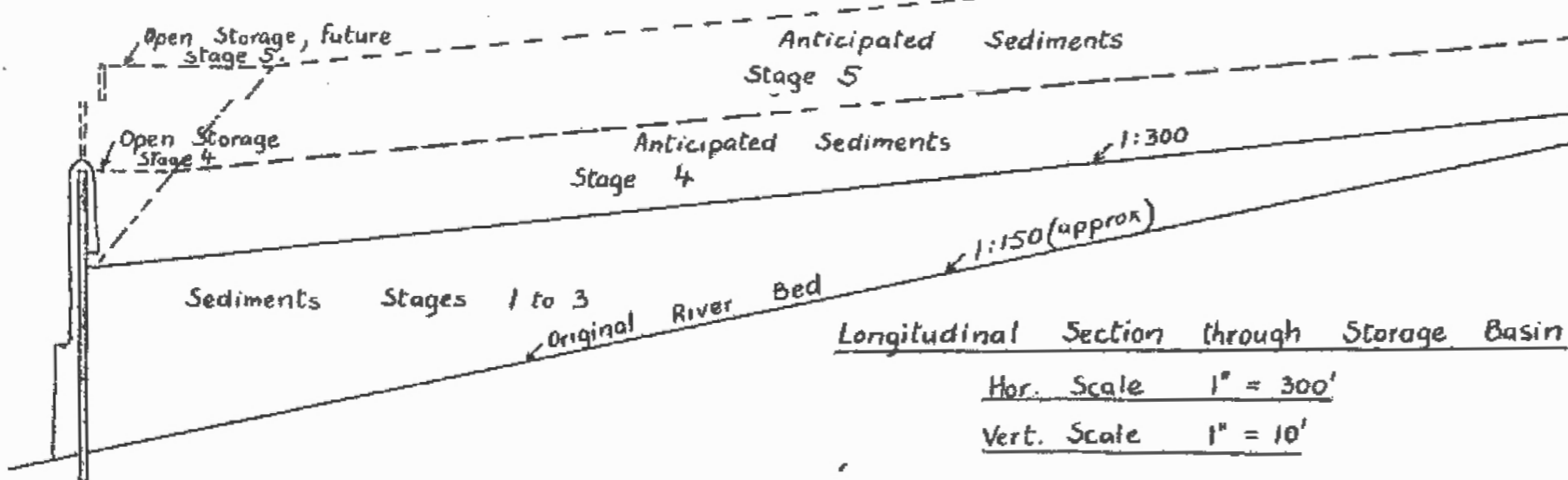
A siphon was designed for the fourth stage of Bulskop Dam ( a sand storage dam near Okahandja). Crest levels were so designed that the siphon will prime with a flood

$Q_p = 993$  cusecs. The size of the catchment area is 50 square miles and the maximum probable flood  $Q_1$ , therefore  $= 3600 \times 50^{0.45} = 21,000$  cusecs. i.e.  $\frac{Q_p}{Q_1} = 0.047$ . It will be seen from detailed statistics of catchment areas of the same order of magnitude (Fig.21) that approximately half the total runoff from the catchment area will probable be yielded by floods having a relative intensity equal to and greater than 0.47. It is, therefore, concluded that the priming of the siphon will occur with sufficient frequency. It is suggested as a general rule that  $Q_p$  should not be greater than the flood intensity corresponding to  $F_R = 0.5$  (adopting the notation defined in page 31 )

As can be seen from Fig.52 a priming step has been provided at a level slightly higher than the level of the siphon intake, with the object of bringing about the abrupt breaking off of the siphon action when the water level in the dam is lowered to siphon intake level.



Longitudinal Section through Siphon at dam wall and Well in main channel  
Scale 1" = 5'0"



Longitudinal Section through Storage Basin  
Hor. Scale 1" = 300'  
Vert. Scale 1" = 10'

Fig. 52 PRESENT STAGE OF DEVELOPMENT OF BULSKOP DAM AND POSSIBLE FUTURE FIFTH STAGE

Note: In ordinary siphon spillways air vents are provided above the intake so that siphon action breaks off gradually in keeping with the flood magnitude. In the sand dam siphon, however, lowering of the water level to the base of the open storage, followed by an abrupt breaking off of the siphon action when the flood is still large enough to fill the basin, is achieved by omitting air vents and placing the priming step high.

A further requirement to be met is that a sufficiently large open storage capacity must always be maintained at the dam wall to assist in the absorption of the small runoff of a poor rainy season.

It will be seen that a large variety of requirements have to be met involving the hydraulic properties of the siphon itself and the problem of sand transportation.

In order to be able to predict the behaviour of siphons and sediments in the basin with some degree of accuracy, the author constructed the apparatus shown in Fig.53 and 54; and conducted experiments which will be described in detail in the sections which follow.

## (2) Dimensions of a Model Siphon and Storage Basin.

The scale to which the model sand storage dam with siphon was constructed will now be discussed.

The model embraces flow in an open channel as well as in a closed conduit. For the sake of simplicity complete geometrical similarity of the model with the entire prototype is desirable.

Dynamic similarity must, however, also be attained if coefficients of discharge and the behaviour of the model generally are to be used to predict the result in the proposed structure.

The basic principles involved can be found in standard textbooks on hydraulics but are given in Appendix A for reference purposes.

Applying the second method discussed in the appendix, namely that in which a smooth geometrically similar model is adopted, to the model of the Bulskop Dam siphon, the following result is obtained:-

1) The prototype consists of

1) The Channel through the dam basin. The alignment is poor and rocky ridges project into the basin. The bottom consists of sand and gravel. The value of Kutter's  $N$  is



estimated at .035.

2) The siphon, consisting of concrete cast in fairly neat formwork. A value of  $N = .015$  is estimated.

A model scale of 1:20 was adopted. It was therefore necessary for the channel to possess a coefficient of roughness  $N = \left(\frac{1}{20}\right)^{\frac{1}{6}} \times .035 = .021$  and the siphon  $N = \left(\frac{1}{20}\right)^{\frac{1}{6}} \times .015 = .009$

In accordance with Kutter's coefficients as tabulated in reference 19, Page 824, it is considered that a model will comply with these conditions, if the channel is made straight, with concrete sides, and the siphon is made of smooth, tin coated metal.

The model was constructed in accordance with these requirements.

Flow in both the model and the prototype must furthermore be turbulent. A model with tranquil flow of a prototype with turbulent flow will be useless.

The Reynold's number,  $R = \frac{vm}{\sigma}$

Where  $\sigma$  = kinematic viscosity of water

$= 1.1 \times 10^{-5} \frac{\text{square feet}}{\text{seconds}}$  at usual temperatures.

$v$  = velocity in feet per second

$m$  = hydraulic radius in feet

i.e.  $R = 90,000 \ vm$

A check was made to see that in all experiments,  $R$  was well above the upper limit for tranquil flow namely 2,700. In other words in all experiments,

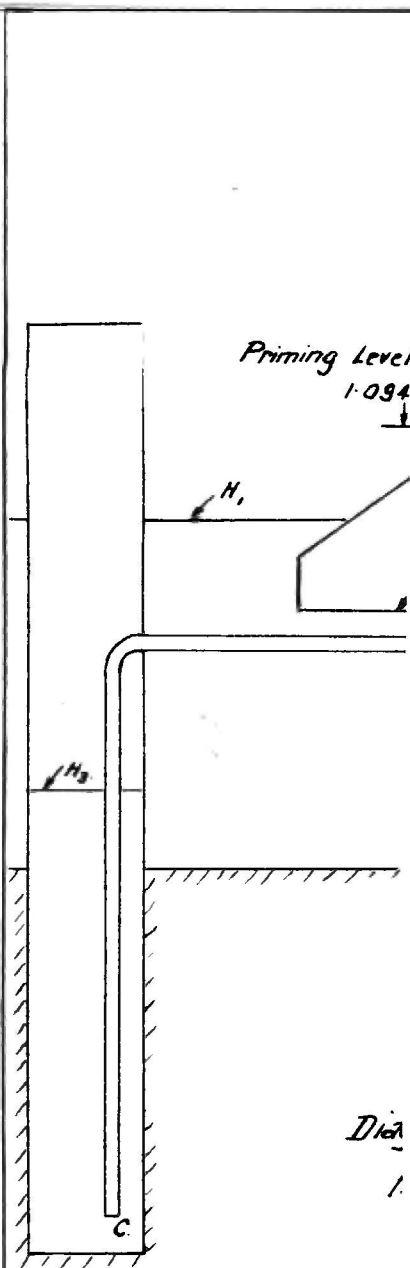
$vm > .03 \frac{\text{square feet}}{\text{seconds}} \dots\dots\dots(15)$

(3) Experiments to determine the Hydraulic Properties of the Siphon.

The model was mounted as shown in Fig.53 and the following results were observed:-

The siphon primes when the water level in the basin rises to R.L.1.094'. When the water level is lowered to R.L..775' air is drawn in and the siphon action breaks off abruptly, provided the efflux is not submerged. It is clear that the position of the priming step at a level higher than the intake of the siphon is responsible for the abrupt breaking off of the siphon action.

The reduced levels of the water tables  $H_A$ ,  $H_B$  &  $H_C$  and the discharge over the adjustable weir were noted for different adjustments of the tail water.



If the adjustable weir is lowered completely, efflux conditions are somewhat indeterminate. The water assumes an approximate surface level of 0.325 feet before leaving the siphon. When the weir is raised so as to submerge the efflux the total head  $H_A - H_B$  is more definite.

The equation of siphon discharge is as follows:-

$$Q_T = C (\text{sectional area at } X) \sqrt{2g(H_A - H_B)}$$

$$= 0.0875 C \sqrt{2g(H_A - H_B)}$$

The coefficient of discharge  $C$  will depend to some extent on the amount of velocity head recovered in the expanding portion of the siphon beyond  $XX$ .

The following experimental results were obtained.

( $Q_T$  was determined by allowing the discharge of the apparatus to fill a vessel of known capacity and noting the time taken):-

$$H_A - H_B = 0.7 \text{ feet} \quad Q_T = .445 \text{ cusecs} \quad \therefore C = 0.76$$

$$H_A - H_B = 0.5 \text{ feet} \quad Q_T = .366 \text{ cusecs} \quad \therefore C = 0.74$$

Submerged efflux:-

$$H_A - H_B = 0.5 \text{ feet} \quad Q_T = .418 \text{ cusecs} \quad \therefore C = 0.845$$

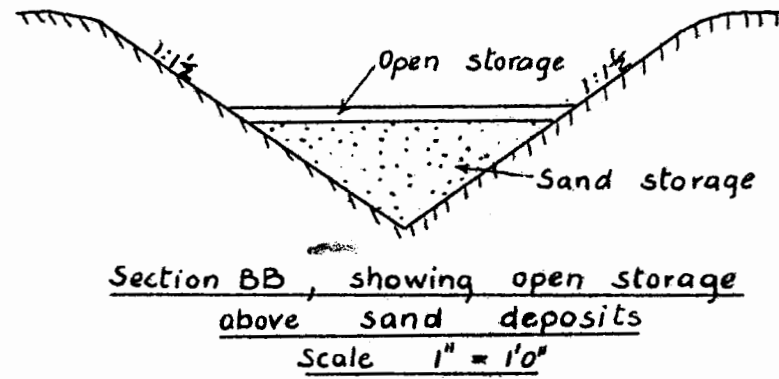
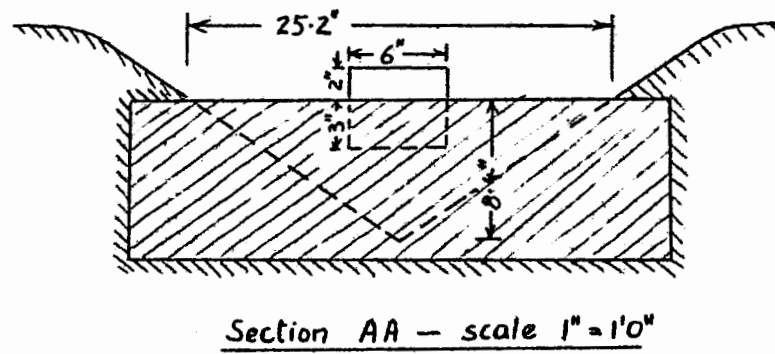
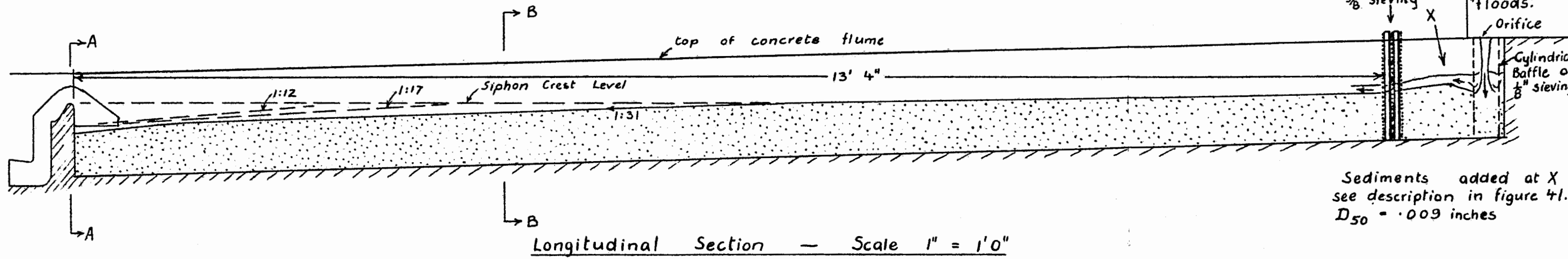
$$H_A - H_B = 0.3 \text{ feet} \quad Q_T = .307 \text{ cusecs} \quad \therefore C = 0.80$$

The suction head on the pipe  $ABC$  is equal to  $H_A - H_C$ . Observed values of  $H_A - H_C$  were found to agree very closely with the expression  $2.2 \frac{v_A^2}{2g}$ . Where  $v_A$  = velocity at the crest of the siphon i.e. at section  $AA$ . Bernoulli's theorem gives  $\frac{v_A^2}{2g}$  as the value of the suction head. The remaining  $1.2 \frac{v_A^2}{2g}$  is due to the suction tube facing downstream and the special pressure distribution at the bend in the siphon at  $AA$ .

#### (4) Experiments to determine the Effect on the Storage Basin.

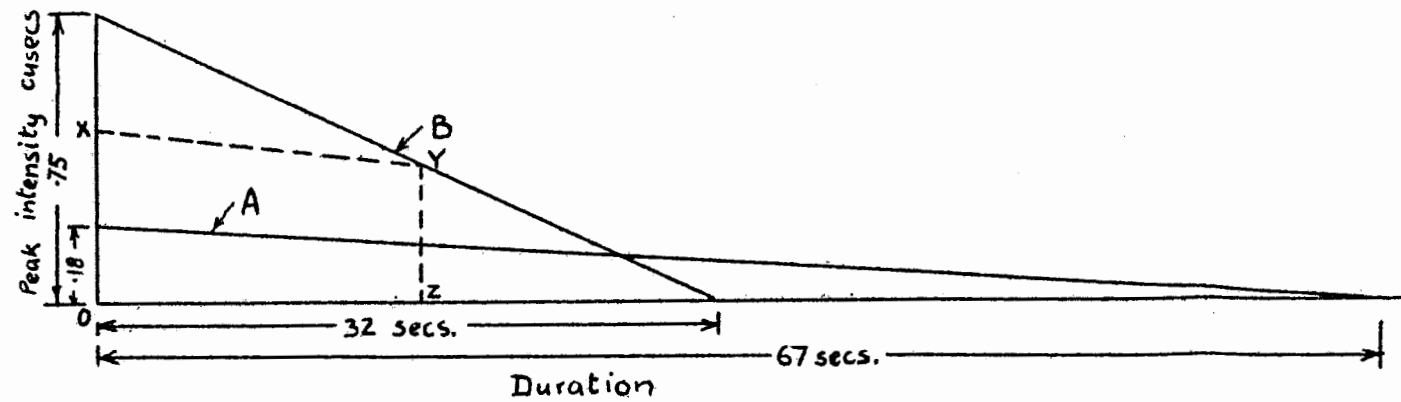
In Figs. 54 and 55 a further series of experiments is illustrated in which the siphon was mounted on the weir of the triangular flume described on page 67. The results show how the sand banks deposited by a series of five smaller floods

FIG. 54 DETAILS OF APPARATUS USED FOR INVESTIGATING SEDIMENTATION IN SAND STORAGE DAMS WITH SIPHONS



Result of Experiments

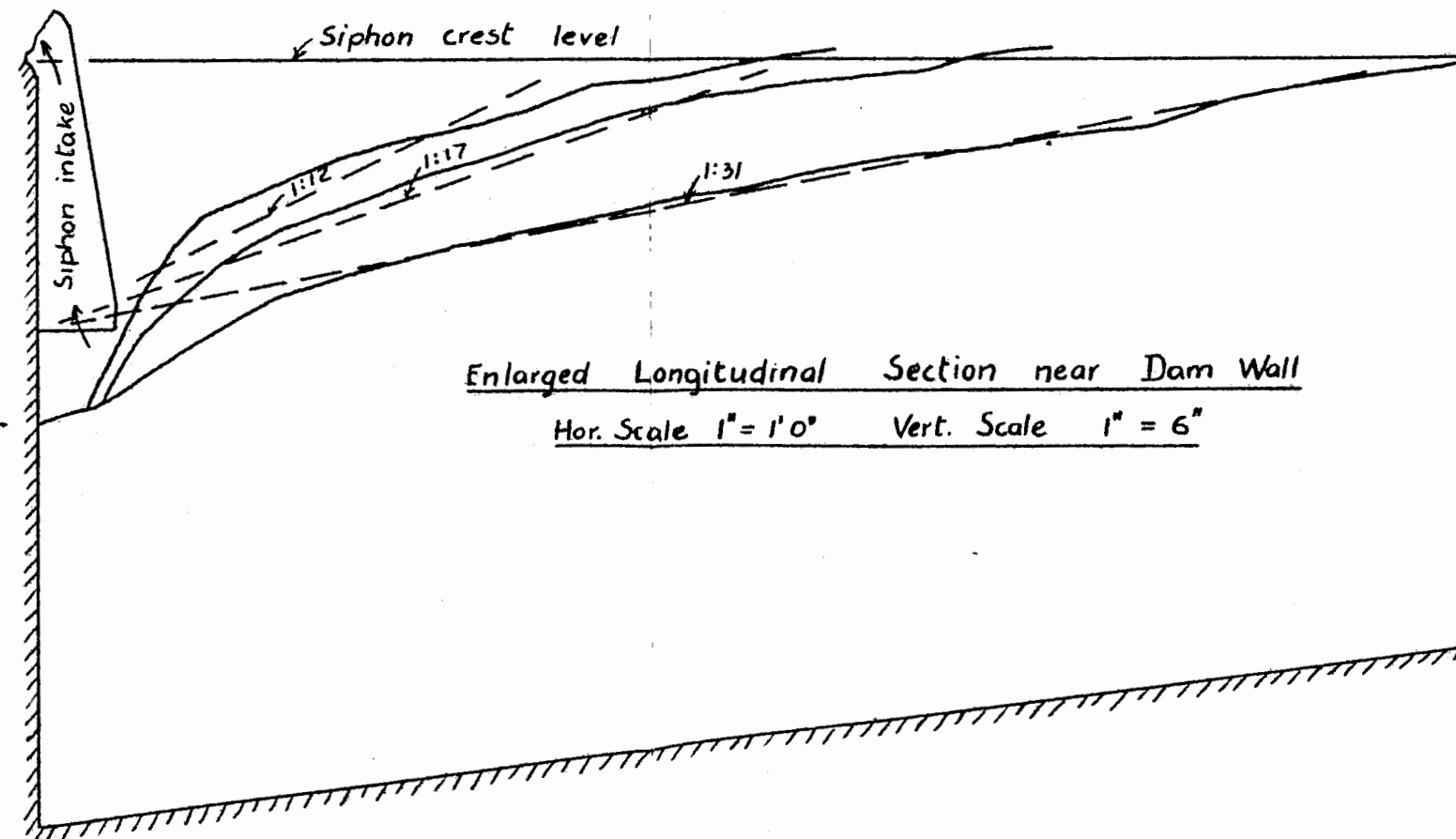
| P = percentage sediments added to the inflowing "floods" | i = slope of sediments in the basin, below siphon crest level. |
|--|--|
| 0.4%   | 1:31   |
| 0.8%   | 1:17   |
| 1.6%   | 1:12   |



"Flood" curves produced by Apparatus

Cycles consisting of 5 floods A followed by one flood B were repeated until sand levels in the flume assumed an equilibrium position.

XYZO = Siphon Discharge with small open storage capacity left = 6.5 cu. ft = 0.54 of total discharge of flood B.



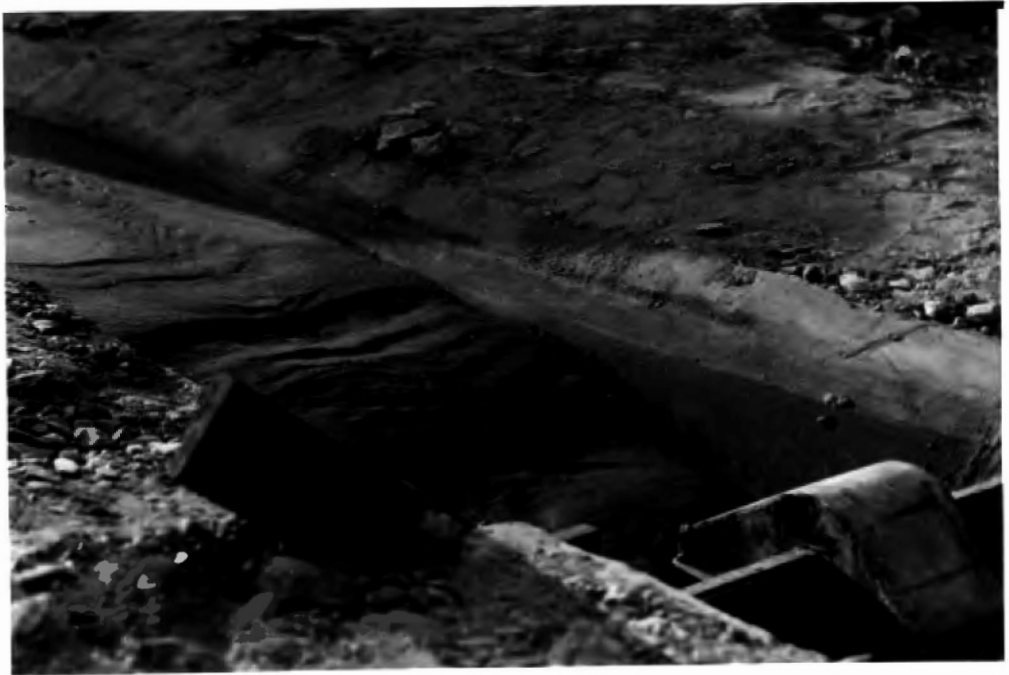


Fig.55.

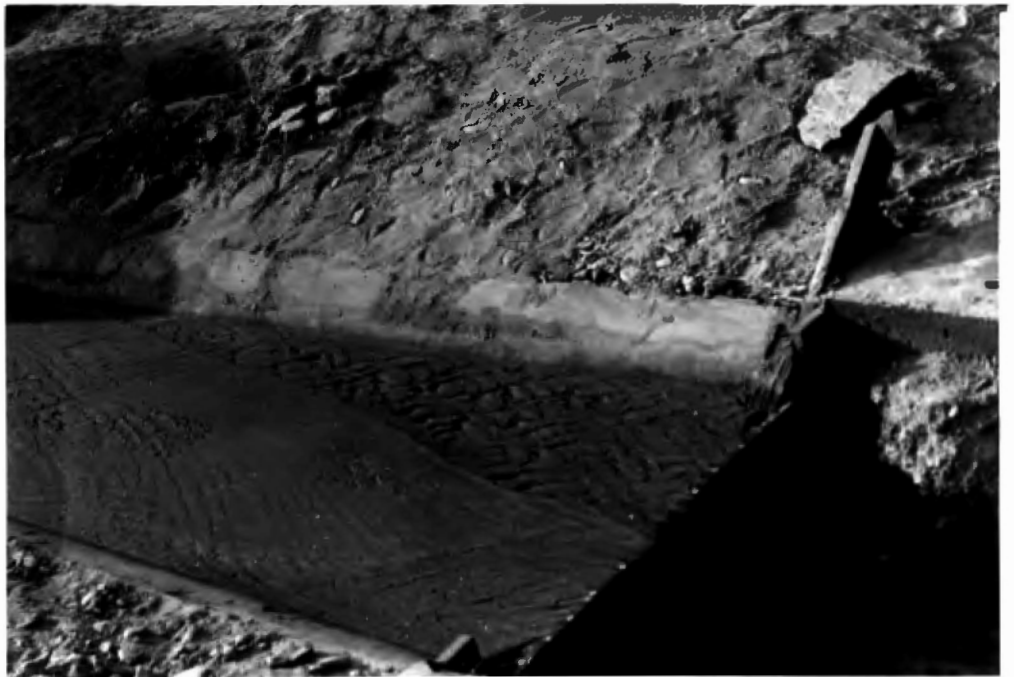


Fig.56.

Model Sand storage Dam. With and without siphon.

without siphon action were scoured by a sixth flood which was large enough to prime the siphon. With 0.4% of sediments added to the inflow the sediments in the basin reached equilibrium at a slope of approximately 1:31. With 1.6% of sediments a very steep equilibrium position was maintained for a long period Fig.54; but finally the intake with this percentage of sediments became too restricted for priming the siphon; and the basin was completely filled with sand.

The flood which caused priming had a discharge of 12 cubic feet compared with the total discharge of a cycle of 6 floods of 42 cubic feet.

Summary of results: Concrete flume  
with siphon (See also graph on Fig.54.)

| Percentage sediments in inflow | Quantity of sediments expressed as a percentage of the floods which cause priming. | Equilibrium slope in basin |
|--------------------------------|--|----------------------------|
| 0.4%                           | 1.4%   | 1:31=.0323                 |
| 0.8%                           | 2.8%   | 1:17=.0589                 |
| 1.6%                           | 5.6%   | 1:12=.0833                 |

The maximum siphon discharge (page ) was 0.445 cusecs or approximately .3 cusecs per foot width of channel.

Substituting  $q_f = 0.3$  and  $P = 1.4, 2.8$  and  $5.6$  in equation(9), (page ) the following slopes are obtained.

(Note  $D_{50} = .009$ " )

$$i_{1.4} = .00331 \times 1.4^{1.4} + .113 \times .009 \times \left(\frac{1}{0.3}\right)^{\frac{2}{3}}$$

$$= .0076$$

$$i_{2.8} = .00331 \times 2.8^{1.4} + .113 \times .009 \times \left(\frac{1}{0.3}\right)^{\frac{2}{3}}$$

$$= .0163$$

$$i_{5.6} = .00331 \times 5.6^{1.4} + .113 \times .009 \times \left(\frac{1}{0.3}\right)^{\frac{2}{3}}$$

$$= .0393$$

Equation (9) applies to free flow in an open channel. In the problem under consideration the flow is only freely scouring for a portion of the time during which siphon

action takes place. During the major portion of the time the velocity of flow and consequently also the scouring action is reduced by brackwater effect. This is the explanation for the discrepancy between the observed slopes tabulated on page 162 and the computed values above. The percentage sediment loading which actually corresponds to the observed slopes in accordance with equation (9) are 4.8%, 7.6% and 9.8% which suggests that the portions of the floods which caused priming which were effective in scouring were  $\frac{1.4}{4.8}$ ,  $\frac{2.8}{7.6}$  and  $\frac{5.6}{9.6}$  i.e. 0.29, 0.37 and 0.58 of the total priming flood of 12 cu.ft. In other words, when there was still a fairly large open storage capacity ( $i = .0323$ ) only 29% of the priming floods were effective in scouring, when the capacity was reduced ( $i = .0589$ ), 37% were effective and when it was further reduced ( $i = .0833$ ) 58% were effective. The siphon discharge when only small open storage capacity is left amounts to 54% of the flood which causes priming (Fig 54).

In designing sand storage dams with siphons the full siphon discharge will be assumed to be active in sediment transportation through the basin when storage capacities have been reduced to near their final value.

##### (5) Prediction of Results - Bulskop Dam.

After construction to a height of 11 feet in three stages, a fourth stage with a siphon was added to this dam (Fig.52)

The principal dimensions of the structure are as follows:-

- Crest level of siphon 100.0
- Crest level of arched weir 100.5
- Intake level of siphon 95.5
- Floor level at efflux 83.0
- Top of opening at efflux 89.34

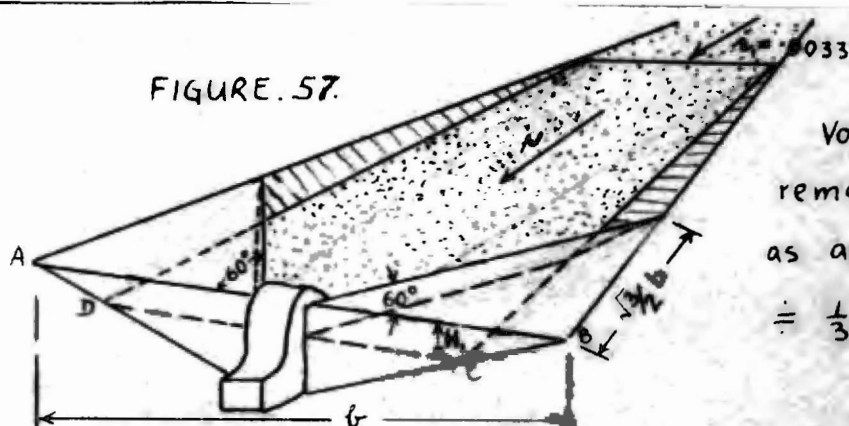
$2.2 \left( \frac{834}{33.4} \right)^2 \times \frac{1}{2g} = 21.4$  feet below water level in dam when this is at 101.8

$2.2 \left( \frac{790}{33.4} \right)^2 \times \frac{1}{2g} = 19.2$  feet below water level in dam when this is at 100.0

$2.2 \left( \frac{622}{33.4} \right)^2 \times \frac{1}{2g} = 11.9$  feet below water level in dam when this is at 95.5.

The initial open storage capacity of the stage up to siphon crest is 1.39 million cubic feet. The water body is roughly wedge shaped, with the top surface horizontal and the lower surface at a slope of .0033. If the sand deposits **assume** a new slope  $i$  as a result of deposition, the open storage capacity will be reduced to  $1.39 \times \frac{.0033}{i}$  million cubic feet very approximately.

Where the siphon width is small compared with the width of the basin it is desirable to make a further allowance for imperfect scouring towards the sides of the basin. The permanent open storage maintained by siphon scour is therefore assumed to take the following shape:-



Volume of silt not removed by scouring taken as approx.  $\frac{1}{3} \text{Area ABCD} \times \frac{\sqrt{3}}{2} b = \frac{1}{3} b^2 H \frac{\sqrt{3}}{2}$

This assumption results in a reduction of the open storage capacity by  $b \times 5.5 \times \frac{1}{3} \times \frac{\sqrt{3}}{2} b$

Which, substituting the dimensions of the Bulskop Dam is equal to  $170 \times 5.5 \times \frac{1}{3} \times \frac{\sqrt{3}}{2} \times 170 = 46,000$  cu.ft.



The residual open storage in Bulskop Dam will therefore be approximately equal to  $1.39 \times \frac{.0033}{1} = .046$  million cu.ft.

There are neither field nor experimental data to confirm the assumed  $60^\circ$  regions not affected by scour. There is reason to believe that such regions will be limited in volume. The sand dam with siphon is different from dams which are first allowed to silt up more or less completely over a number of years and in which scouring is then attempted during a good rainy season. In the former scouring occurs simultaneously with all floods which are large enough to bring appreciable volumes of silt; and surge waves occur at all levels on the sides of the basin during the emptying process. In the latter the scouring flow cuts into the deposits and initially at least leaves large deposits on the sides, which are then slowly undercut and eroded.

The following table gives the residual open storage for different values of  $i$

| $i$  | $V_R$ = residual open storage capacity |
|------|--|
| .005 | 0.90 million cu.ft.                    |
| .01  | 0.41                                   |
| .02  | 0.18                                   |
| .04  | 0.07                                   |
| .05  | 0.05                                   |

An estimate will now be made of the slope which the sediments will ultimately occupy within the storage basin of the Bulskop Dam. It has previously been estimated that approximately 50% of all runoff will constitute floods that will prime the siphon (page 154). The percentage of the flow of the priming floods which will be effective in scouring will depend on the residual open storage capacity,

the characteristics of the inflowing floods and the discharge curve of the siphon.

Point A on Fig.21 represent the assumed relative intensity of .047 for  $F_R = \frac{0.5}{A}$  for Bulskop Dam and point B the relative intensity of .094 for  $F_R = 0.25$ . The points are selected somewhat below those of the Gamams catchments as somewhat less favourable runoff conditions are expected. .047 is the relative intensity of the minimum flood which will prime the siphon and .094 can be accepted as the relative intensity of a typical flood which will cause priming.  $Q_1 = 21,000$  cusecs and the typical priming flood therefore has a peak intensity of  $.094 \times 21,000 = 1975$  cusecs. Fig.58 gives an estimate of the characteristics of a flood of this intensity, based on the detailed records given in Figs.22 and 23 and shows what happens when this flood passes through the storage basin of the Bulskop Dam. The conclusion is arrived at that 44% of the discharge of all floods which cause priming is effective in scouring i.e. 22% of all runoff priming and nonpriming.

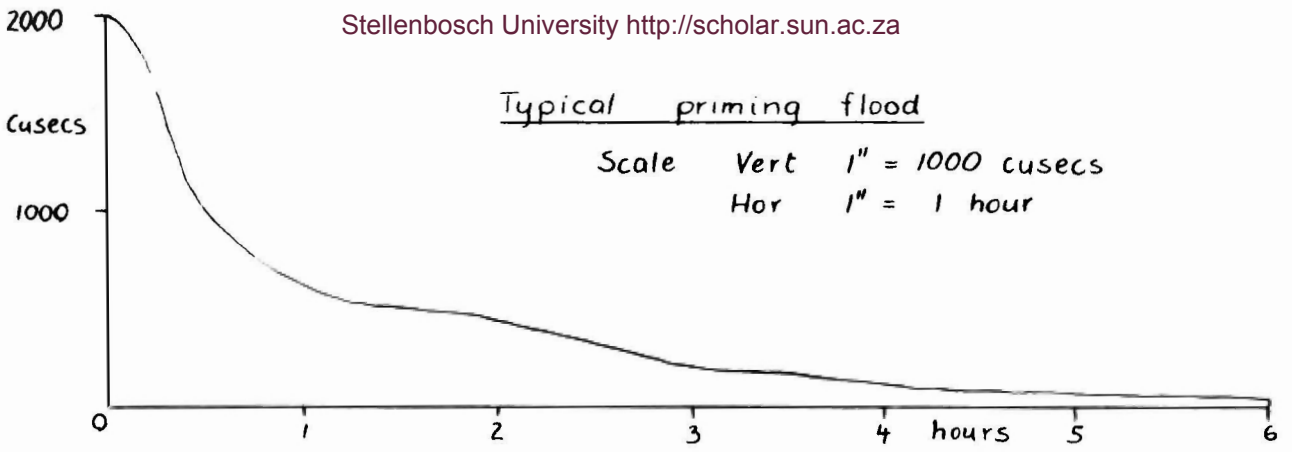
On page 92 the flood water arriving at the Bulskop Dam was estimated to contain 1.02% by weight of sand which means that the scouring portion will have to cope with  $\rho = \frac{1.2}{0.22} = 4.63\%$ . The peak intensity of the scouring portion is 840 cusecs (Fig.58) and the width of channel 170 feet i.e.  $q_b = \frac{840}{170} = 5.0$  cusecs per ft. width.  $D_{50} = 0.01$  inches.

Substituting in equation (9)

$$\begin{aligned} i &= .00331 \times 4.63^{1.4} + 0.113 \times 0.01 \times \left(\frac{1}{5.0}\right)^{2/3} \\ &= 0.284 + .0004 \\ &= .0288 \end{aligned}$$

i.e. Residual open storage capacity in accordance with the table on page 166 is given by

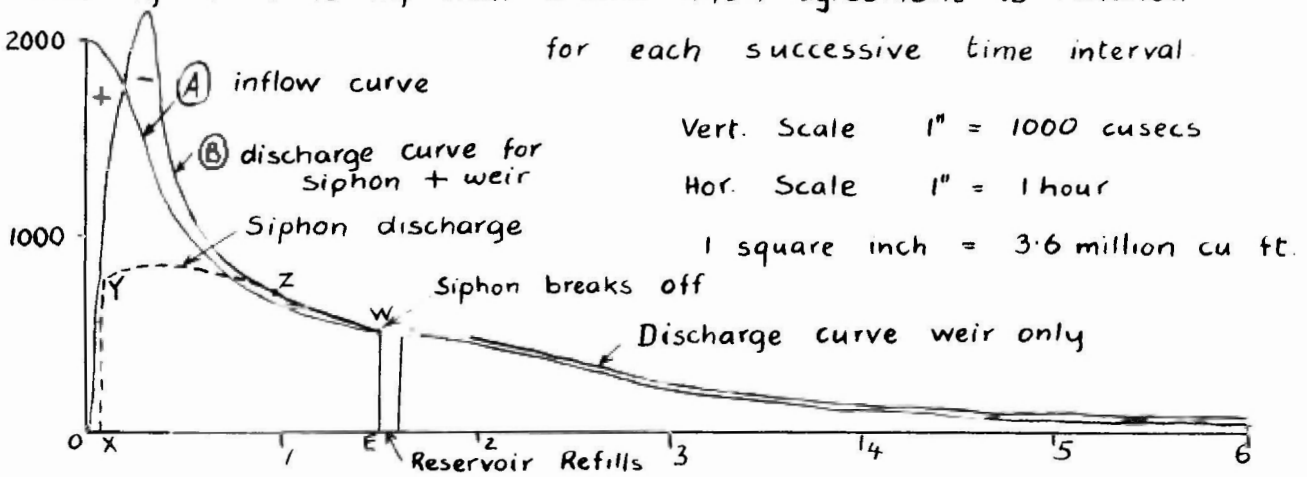
$$V_R = 0.113 \text{ million cu.ft.}$$



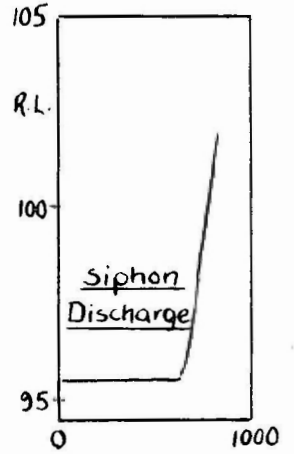
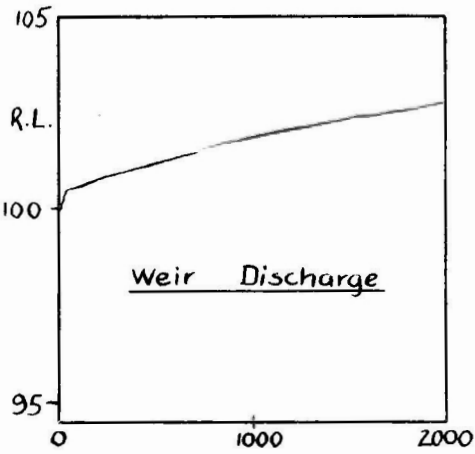
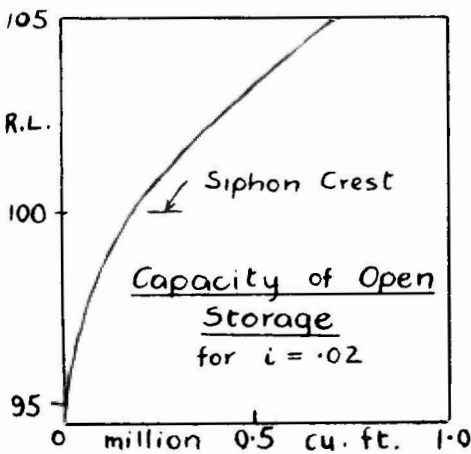
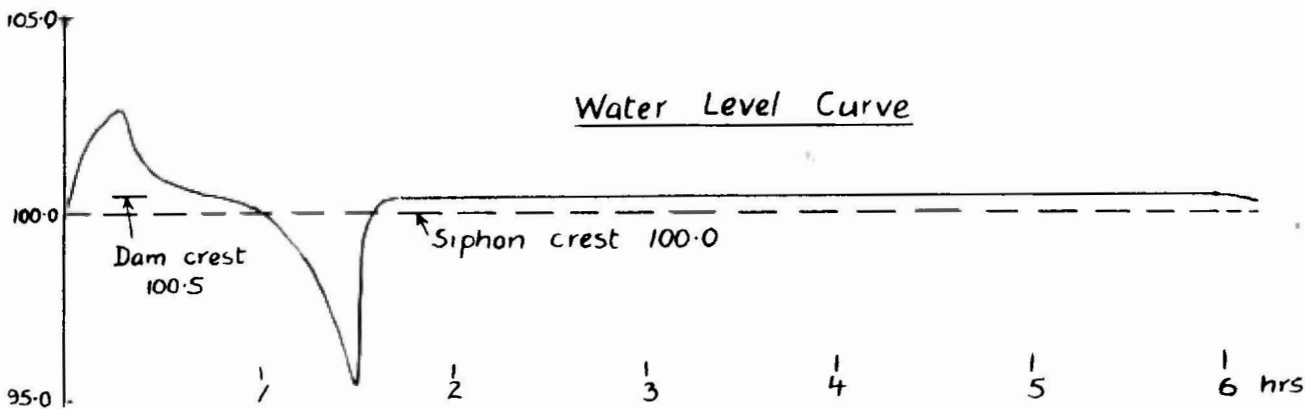
Construction of Efflux Curve B.

The curve is so constructed that the summation of areas between curves A & B, which represents the storage above crest level and the plotted discharge correspond to the same water level on the capacity & discharge curves, respectively.

Plotting is done by trial & error until agreement is reached for each successive time interval.



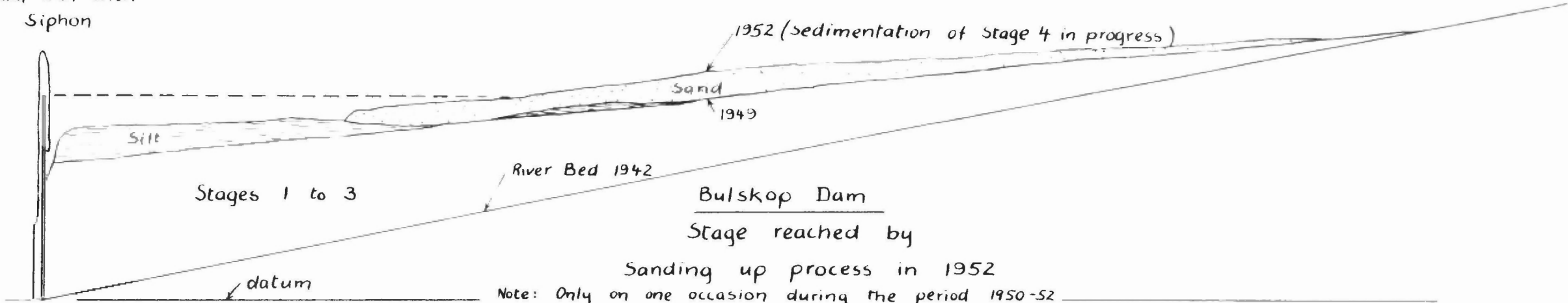
XYZWE = Siphon discharge = 3.64 million cubic feet.



Conclusion Total flood discharge 8.2 million cu. ft.

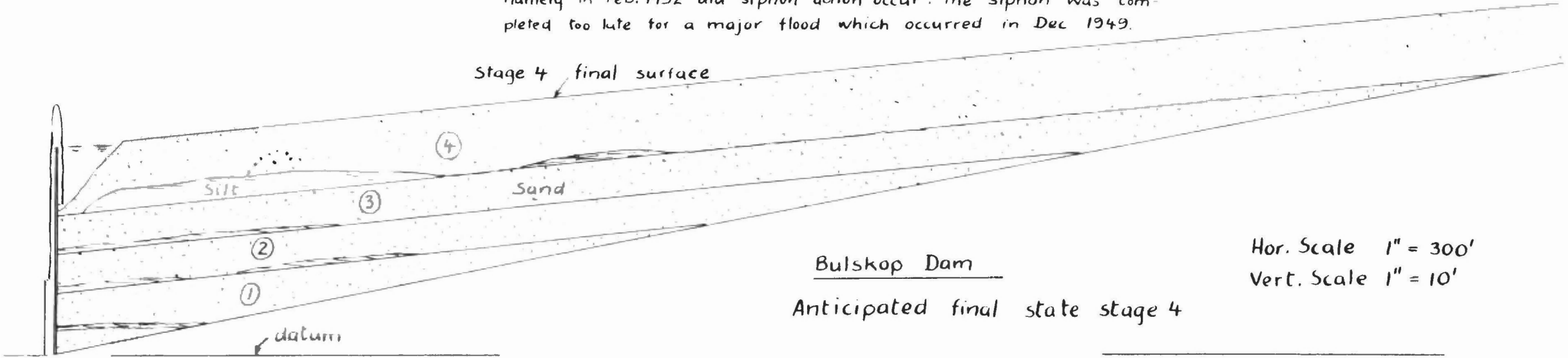
Portion of flood effective in scouring 3.6 million cu. ft.  
or 44% of the total.

Dam wall with Siphon



Note: Only on one occasion during the period 1950-52 namely in Feb. 1952 did siphon action occur. The siphon was completed too late for a major flood which occurred in Dec 1949.

Stage 4 final surface



Hor. Scale 1" = 300'  
Vert. Scale 1" = 10'

Fig. 59 Bulskop Dam

showing observed sedimentation up to 1952 and anticipated future sedimentation.

CHAPTER 6.ANALYSIS OF FLOW NETS AT WEIRS, EMBANKMENTS AND GROUND  
WATER DILUTION SYSTEMS.(1) Weir on Sheet Piling in River Bed with Uniform Permeability.

Seepage underneath sheet piling in a sandy river bed is analysed in the diagram which follows. The sheet piling is so constructed that it dams up the river three feet above bed level. Overflowing water is discharged on a heavy stone-in-wire-mat on a suitable gravel base. The groundwater flow net is constructed for the condition of surface flow occurring in the river and keeping the upstream sand body saturated.

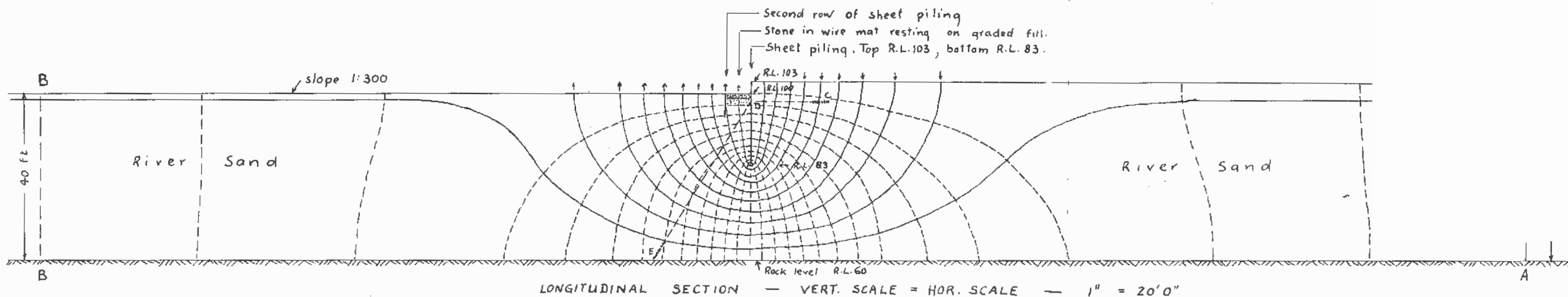
The sand volume available for water storage will be increased somewhat by deposition upstream of the weir at a surface slope parallel to the original river bed slope. The pump station which will benefit by the additional storage can be situated either in the immediate vicinity of the weir or some distance downstream.

A further effect of the weir is shown in Fig.60. The ground water flow underneath the sheet piling is several times the seepage flow of the river due to the additional head produced by the weir. In the example chosen the ratio was found to be 10:1. One part of the seepage underneath the sheet piling is thus derived from normal river seepage and nine parts from fresh water infiltration from floods. The velocities are comparatively high (near the base of the deposits approximately 10 to 15 times the normal seepage velocities.) The normal seepage flow is forced into a thin layer only a few feet thick over bed rock which is by no means as smooth in outline as in the theoretical case. All these factors will combine to bring about a mixing between the two types of water i.e. dilution of ground water flow during floods. A further dilution can be brought about by the tube CDE, slotted at C and E, as explained in

detail in Fig.60. The pump station to benefit by ground water dilution is best situated some distance downstream of the weir.

FIGURE 60

ANALYSIS OF FLOW WHERE RIVER BED IS RAISED 3 FEET BY SHEET PILE CUTOFF EXTENDING ABOUT HALF WAY DOWN TO ROCK



River bed material assumed to have a permeability  $k = 450$  feet a day

Condition of complete saturation  $\left\{ \begin{array}{l} h_1 = 3 \text{ feet} \\ \text{Number of flow channels } N_f = 11 \\ \text{" " potential drops } N_d = 23 \end{array} \right.$

Note

- ① For proposal to eject water from lower layers, thus causing replacement of underground flow by surface water, see chapter 3, Fig. 39.
- ② It is also possible to inject fresh water by pipe CDE, slotted at C and E. Difference of head between ends  $= \frac{17}{23} \times 3 = 2.20$  feet as long as condition of complete saturation persists upstream of the sheet piles.

Underground Flow at Sheet Piling  $= q_w = k h_1 \frac{N_f}{N_d}$   
 $= 450 \times 3 \times \frac{11}{23} = 645 \text{ cu.ft./day/unit channel width}$

Effective Velocity of flow at top of sheet piling  $= k \times \frac{5h}{8l} = 450 \times \frac{3}{23} \times \frac{1}{3} = 19.6 \text{ feet/day}$   
 " " " " " bottom " " " = " =  $450 \times \frac{3}{23} \times 1 = 58.8$  "  
 " " " " " sections AA & BB =  $k \cdot i = 450 \times 0.033 = 1.5$  "  
 " " " " " rock level under sheet piling  $= 450 \times \frac{3}{23} \times 3 = 19.6 \text{ feet/day.}$

Underground Flow at Section AA  $= q_{Au} = k \times i \times 43$   
 $= 450 \times \frac{1}{300} \times 43$   
 $= 64.5 \text{ cu.ft./day/unit channel width}$

Underground Flow at Section BB  $= q_{Bv} = 450 \times \frac{1}{300} \times 40$   
 $= 60 \text{ cu.ft./day/unit channel width}$

Surface flow at BB, as long as condition of complete saturation persists upstream of sheet piles,  $= 645 - 60 = 585 \text{ cu.ft./day/unit channel width.}$

The three foot hydrostatic head is divided into eleven parts in the diagram. Potential lines are closest at the sheet piling and the hydraulic gradient at efflux next to the piling amounts to a drop of  $\frac{3}{11}$  ft. in a distance of 3 ft. or 1:11. Heave can be expected to occur at the critical hydraulic gradient  $i_c = \frac{\gamma'}{\gamma_w}$ <sup>12</sup>. If  $\gamma' = 52$  lb per cu.ft. and  $\gamma_w = 62.5$  lb. per cu.ft. then  $i_c$  is approximately unity. There is therefore a considerable factor of safety against piping by heave. Empirical rules for permissible seepage gradient under hydraulic structures specify 1:5 to 1:8 for sand<sup>8 & 9</sup>. D.W.Taylor<sup>13</sup> in discussing such empirical rules points out that fine sand is generally the safest foundation where failure due to piping is concerned. This is probably due to the uniformity of fine sand and the resulting uniformity in seepage velocities. Since the computed gradient of efflux of 1:11 is well below the permissible gradient quoted by various authorities, there is no danger in the example analysed of failure due to undermining as a result of excessive seepage velocities.

The stone-in-wire-mat should be long enough to act as an effective apron against scour. Dixey<sup>10</sup> in his Handbook of Water Supply suggests a length of apron of eight feet for a weir height of three feet and for higher weirs a proportional increase in this length.

The stone-in-wire-apron should rest on gravel or crushed rock which is large enough not to be washed out by floods. A mixture of sizes from  $\frac{3}{4}$  inch to 3 inch is considered suitable for a nine inch layer immediately below the apron ( $D_{15}$  of this coarse gravel layer say 1 inch). In accordance with the rules given on page 150 the coarse gravel should be placed on a layer of finer material with  $D_{85} = \frac{1}{4}$  of  $D_{15}$  of the coarse material =  $\frac{1}{4}$  inch and a  $D_{15}$  size of say  $\frac{1}{10}$  th inch. This lower layer should be approximately 6 inches thick and

will seal off fine river sand ( $D_{85}$  equal to or greater than  $\frac{1}{40}$  th inch) against scouring by floods.

(2) Weir on Sheet Piling in River Bed with Variable Permeability.

If the weir were constructed in a river bed consisting of fine sand near the surface ( $k = 58$ ) followed by a layer of medium sand ( $k = 183$ ) and underlain by very coarse sand ( $k = 1580$ ) the flow net will assume the form shown in Fig.61. The ratio between permeabilities of the three layers is approximately  $\frac{1}{3} : 1 : 9$ .

When the river bed is completely saturated  $h_1 = 3$  feet.

It will be seen that the flow net has been drawn with square fields in the zone  $k = 183$  and rectangular fields in the other zones. In the fine sand zone the dimension of the rectangle in the direction of flow is  $\frac{1}{3}$  its width and in the zone of very coarse sand the length in the direction of flow is nine times the width. The reason for this construction is as follows:-

The flow between any <sup>two</sup>  $\wedge$  flow lines =  $\frac{q_w}{N_f} = k \frac{\delta h_1}{\delta l} \times \text{area of flow channel,}$  where  $\delta h_1 =$  loss of head in rectangle and  $\delta l =$  length of rectangle . For unit width of river bed,

$$\frac{q_w}{N_f} = k \frac{\delta h_1}{\delta l} \times \text{distance between flow lines.}$$

$$= k \frac{\text{width of rectangle}}{\text{length of rectangle}} \delta h_1,$$

$\frac{q_w}{N_f}$  is constant and  $\delta h_1$  is constant for equal pressure intervals

$$\therefore k \frac{\text{width of rectangle}}{\text{length of rectangle}} = \text{constant}$$

If square fields are, therefore, constructed in the central zone the fields in the other zones must have width to length ratios in inverse proportion to the value of  $k$ .

The following results are computed from the information given by the flow net:-



it will rise through the apron to be washed away by floods.

Injection tube  $X Y$  :- The difference in head between  $X$  and  $Y$  will be  $\frac{5}{N_d} h_1 = 1.5$  ft. Fresh water will enter the slotted section at  $X$  and will be injected into the lower ground water layers at  $Y$  causing dilution of the brack content of the latter.

(3) Sand Embankment on River Bed with Uniform Permeability.

Where a suitable spillway can be provided on the side of the river, it is possible to achieve the storage plus ground water diluting effect by means of a sand embankment properly designed against overtopping by floods and washing out by seepage.

Fig.62 illustrates such an embankment with adequate freeboard above high flood level. Seepage velocities will be greatest during high floods. In the example under consideration a concrete apron has been provided on the upstream side to lengthen the seepage path. The pressure gradient at efflux is  $\delta h_1 = \frac{h_1}{10} = 1$  foot in  $\delta l = 5$  ft. which is the steepest gradient permitted in empirical rules (page 170).

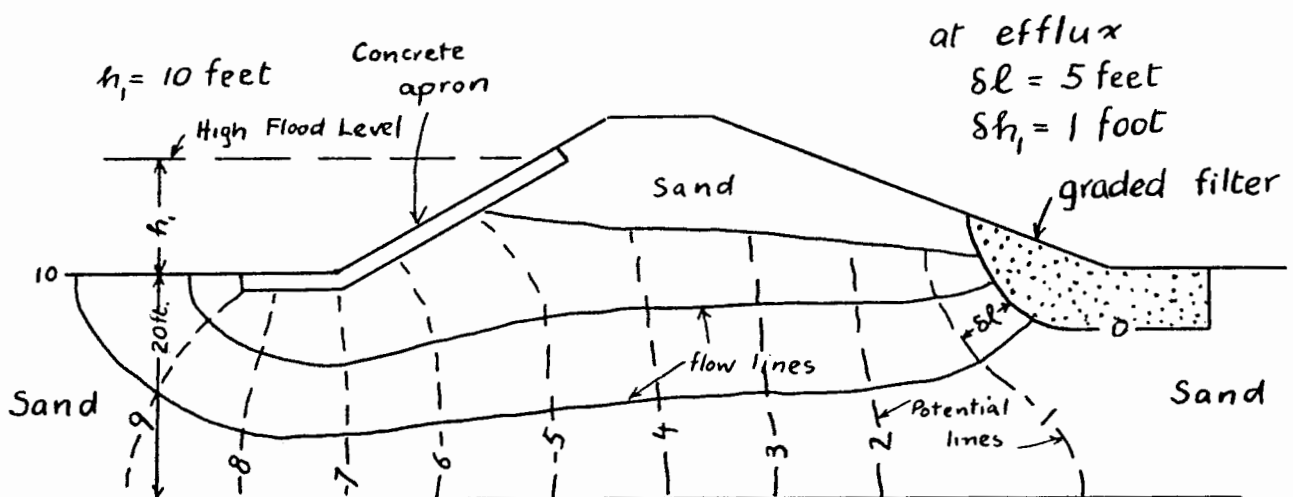


Fig. 62 Flow Net for Sand Embankment

Scale 1" = 5'0"

$$N_f = 14$$

$$N_d = 10$$

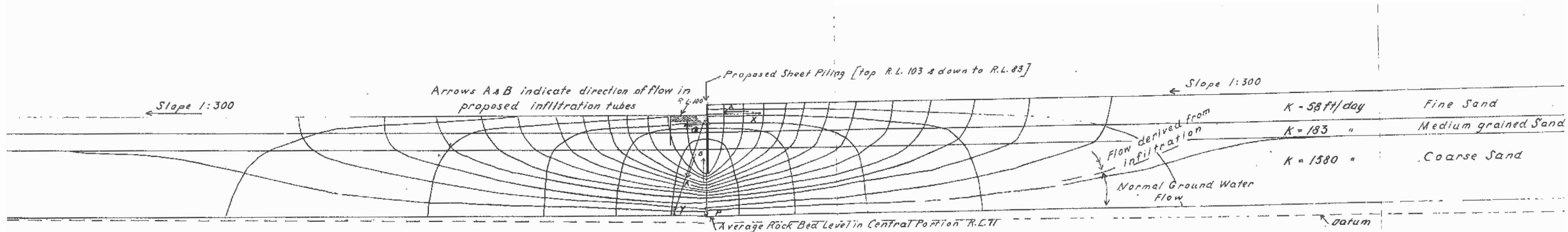
$$q_{gw} = \text{underground flow at sheet piling} = k h_1 \frac{N_f}{N_d}$$

where  $k$  is the permeability in the zone with square fields  
 $= 183 \times 3 \times \frac{14}{10} = 768 \text{ cu.ft./day/unit channel width.}$

Normal ground water flow in coarse sand  $= 1580 \times 19 \times \frac{1}{300}$   
 $= 100 \text{ cu.ft./day/unit width.}$

Ejection tube PQ :- The difference in head between P and Q will be  $\frac{2.5}{N_d} h_1 = \frac{2.5}{10} \times 3 = 0.75 \text{ ft.}$  Brack water will enter the slotted section at P and will be ejected at Q from where

Fig. 61



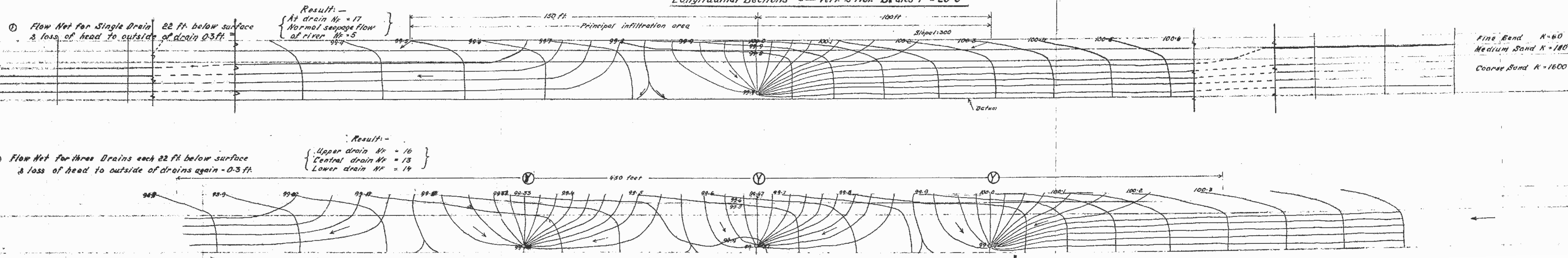
Proposed Weir for improving ground water supply — Flow Diagram during and shortly after floods.

Scale : 1" = 20'0"

The loss of head from the surface of the sand to the perforations for a total discharge of 201,000 cubic feet a day through the three drains is therefore 0.3 feet. If the area drained is divided into twelve parts as in the tube well system (Fig.39) and an amount  $q = \frac{201,000}{12} = 16,175$  cu.ft/day be taken from each part then the loss of head for any other discharge can be taken as approximately  $\frac{0.3}{16,175} q = .0000187 q$ . Where each part is drained by one tube well instead of a transverse horizontal drain the loss of head was found to be approximately .0000485  $q$  or almost three times as great (Fig.33.) This result is to be expected as the seepage flow is drawn through much smaller cross sectional areas of sand in the case of the tube well than in a horizontal drain extending over the full width of the sand.

Fig.63

Flow Studies, assuming transverse drains in Rectangular Channel  
 Longitudinal Sections - Vert. & Hor. Scales 1" = 20' 0"

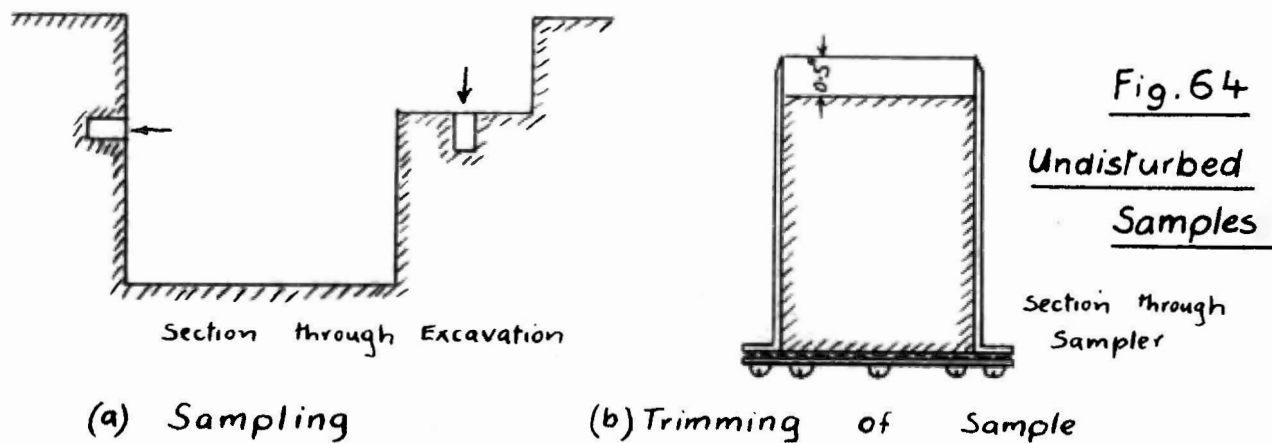


Although it is not practicable to lay horizontal drains 22 feet below river bed level, the analysis of this theoretical case gives a general picture of the type of flow which may be expected with rows of tube wells (e.g. the distance upstream and downstream from extraction points at which infiltration is induced, uniformity of infiltration rate, etc.)

CHAPTER 7.STANDARD METHODS OF DETERMINING THE HYDRAULIC PROPERTIES  
OF SAND.(1) Description of Apparatus and Methods.

Fig. 64 illustrates the apparatus used for taking undisturbed samples of sand and the procedure adopted in sampling.

An open excavation is made to the required depth and samplers pressed in horizontally and vertically Fig.64 a) The sampler used consisted of a cylinder of thin sheet metal with an inside diameter of 3.5 inches, a cutting edge formed by bevelling on the outside and a flanged end for screwing on a sieve cover on the other end. When testing fine sand, a fine sieve (U.S.No.40) backed by a coarse rigid sieve (approx U.S.No.4) was screwed onto the flanged end while the sampler was still in place, thus sealing off the trimmed end of the undisturbed sand sample. Finally, the sampler was dug out and trimmed on the other end to 0.5 inches below the cutting edge (Fig.64 b). Obviously only moist sand can be sampled in this way



With the apparatus used by the author the total height of the trimmed sample was 4.19 inches, giving a volume  $V_1$  of 40.3 cubic inches or 661 cubic centimeters. The next step is to determine the properties of the sample.  $W_1$  = weight of the sample as taken, is determined.

The sample plus sampler is next placed in a vessel and submerged in water and the total weight of apparatus plus contents determined. The following diagram shows how the submerged weight of the sample is deduced by subtracting the weight of apparatus filled with water only from the weight previously determined.

$W_2$  = submerged weight =



Fig. 65

The permeability is next determined with the sample still in its undisturbed state of compaction. The apparatus is illustrated in the following sketch:-

Inflow adjusted to  
keep water level constant.

Note: Water level A is kept constant by siphon feed from a large vessel.

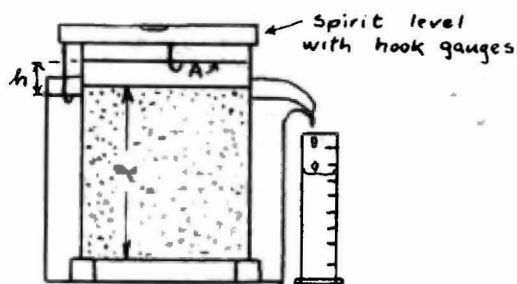


Fig. 66

Coefficient of permeability  $k = \frac{q}{a} \frac{L}{h}$   
where  $q$  is the rate of discharge  
and  $a$  the full cross section of the sample.

The sampler is now removed from the vessel and placed on the scale the moment the free water disappears from the surface. This gives a rough check on the saturated weight, which is, however, more accurately deduced from the submerged weight  $W_2$ .

The sample is now removed from the sampler and dried in an electric oven.  $W_3$  = weight of sample dry.

$V_2$  the volume of the dry material when poured loosely into a cylinder of known dimensions and  $V_3$  the volume when consolidated by ramming in thin layers, are next determined.

The angle of repose is determined by pouring the dry material from a low height onto a flat table and measuring the slopes with a clinometer.

Finally the material is subjected to a sieve analysis.

The densities of the samples in various conditions of compaction and moisture content are also determined. Formulae are as follows (mainly following the notation adopted in Reference No 12):-

Unit weight of dry material,  $\gamma_d = \frac{W_3}{V_1}$

Unit weight as sampled,  $\gamma_a = \frac{W_1}{V_1}$

Unit weight of saturated material  $\gamma = 1 +$  submerged unit weight  $= 1 + \frac{W_2}{V_1}$  <sup>12</sup>

Unit weight of dry material, loose  $\gamma_0 = \frac{W_3}{V_2}$

Unit weight of dry material, maximum compaction,  $\gamma_{max} = \frac{W_3}{V_3}$

Finally, a full list of "index properties" is compiled for each sample in accordance with the following system:-

1) Allen Hazen's effective size,  $D_{10}$

Allen Hazen's uniformity coefficient,  $\frac{D_{60}}{D_{10}}$

"Fifty percent size",  $D_{50}$

Porosity of undisturbed sample  $n = \frac{\gamma - \gamma_d}{\gamma_d}$

Void ratio of undisturbed sample  $e = \frac{n}{1-n}$

Void ratio of loose sample  $e_0 = \frac{\gamma_d}{\gamma_0} (1+e) - 1$

Void ratio of compacted sample  $e_{min} = \frac{\gamma_d}{\gamma_{max}} (1+e) - 1$

Relative density of undisturbed sample  $D_r = \frac{e_0 - e}{e_0 - e_{min}}$

2) Water content of sample immediately after sampling,

$$w = \frac{\gamma_a - \gamma_d}{\gamma_d}$$

The water content when saturated =  $\frac{\gamma - \gamma_d}{\gamma_d}$

The degree of saturation immediately after sampling

$$S_r (\%) = \frac{\gamma_a - \gamma_d}{\gamma - \gamma_d} 100$$

3) Permeability  $k$

4) Angle of repose  $\phi$

## (2) Some typical Samples from Sand Storage Dams.

Three representative samples analysed by the author will now be described, one from the Bulskop sand storage dam in the mica schist area and two from the Gamams sand storage dam with a catchment consisting largely of quartzite country. In the latter the deposits fall clearly into two distinct major groups, fine sand and coarse sand respectively, hence the tabulation of the results of two samples. In the mica schist catchment fine sands are by far the most important part of the deposits. Samples were taken for testing permeability in a horizontal direction.

### Sample 1.

Representative sample from the Bulskop Dam.

Rounded and angular quartz and hornblende grains with a large proportion of mica flakes.

| Grain Size | Percentage finer than this grain size. | Method of Determination |
|------------|--|-------------------------|
| .188"      | 100%                                   | Sieve U.S.A.No. 4       |
| .094       | 100                                    | 8                       |
| .047       | 100                                    | 16                      |
| .037       | 100                                    | 20                      |
| .023       | 99.5                                   | 30                      |
| .018       | 99.5                                   | 40                      |
| .012       | 98.5                                   | 50                      |
| .009       | 49.4                                   | 80                      |
| .006       | 34.0                                   | 100                     |
| .003       | 4.0                                    | 200                     |



| Grain Size. | Percentage finer than this grain size. | Method of Determination |
|-------------|--|-------------------------|
| .188"       | 88.4                                   | Sieve U.S.A.No 4        |
| .094        | 79.9                                   | 8                       |
| .047        | 74.5                                   | 16                      |
| .037        | 68.8                                   | 20                      |
| .023        | 25.2                                   | 30                      |
| .018        | 21.9                                   | 40                      |
| .012        | 14.2                                   | 50                      |
| .009        | 2.5                                    | 80                      |
| .006        | 2.0                                    | 100                     |
| .003        | 1.0                                    | 200                     |

Allen Hazen's effective size  $D_{10} = .011$

" " uniformity coefficient  $\frac{D_{60}}{D_{10}} = \frac{.036}{.011} = 3.3$

"Fifty percent size"  $D_{50} = .032$ .

Further properties of the samples.

|  | Sample No 1 | Sample No 2 | Sample No 3. |
|--|-------------|-------------|--------------|
| Weight $W_1 =$                               | 910         | 1190        | 1255 gram    |
| $W_2 =$                                      | 456         | 547         | 649 "        |
| $W_3 =$                                      | 827         | 945         | 1097 "       |
| Volume $V_1 =$                               | 661         | 661         | 661 cc.      |
| $V_2 =$                                      | 665         | 680         | 680 "        |
| $V_3 =$                                      | 568         | 612         | 592 "        |
| $\gamma_a = \frac{W_1}{V_1} =$               | 1.38        | 1.80        | 1.90         |
| $\gamma_d = \frac{W_3}{V_1} =$               | 1.25        | 1.43        | 1.66         |
| $\gamma = 1 + \frac{W_2}{V_1} =$             | 1.69        | 1.83        | 1.98         |
| $\gamma_o = \frac{W_3}{V_2} =$               | 1.24        | 1.39        | 1.62         |
| $\gamma_{max} = \frac{W_3}{V_3} =$           | 1.46        | 1.54        | 1.85         |
| $n = \gamma - \gamma_d =$                    | 0.44        | 0.40        | 0.32         |
| $e = \frac{n}{1-n} =$                        | 0.79        | 0.67        | 0.47         |
| $e_o = \frac{\gamma_d}{\gamma_o}(1+e) - 1 =$ | 0.80        | 0.72        | 0.51         |

|  |          |           |           |
|--|----------|-----------|-----------|
| $e_{min} = \frac{\gamma_d}{\gamma_{max}}(1+e) - 1 =$ | 0.53     | 0.55      | 0.32      |
| $D_r = \frac{e_0 - e}{e_0 - e_{min}} =$              | 0.04     | 0.29      | 0.22      |
| $w = \frac{\gamma_a - \gamma_d}{\gamma_d} =$         | 0.10     | 0.26      | 0.14      |
| $w_{sat} = \frac{\gamma - \gamma_d}{\gamma_d} =$     | 0.36     | 0.28      | 0.19      |
| $S_r(\%) = 100 \frac{w}{w_{sat}} =$                  | 28%      | 93%       | 74%       |
| $k =$  | 11ft/day | 10 ft/day | 97 ft/day |
| $\phi =$   | 31°      | 33°       | 35°       |

Notes on the significance of  $D_r$  and  $\gamma_a$  :-

Relative density  $D_r = 0$  means material completely uncompacted.

$D_r = \infty$  means material with compaction equal to the maximum attainable in the laboratory.

$D_r = 1$  compaction such that void ratio is midway between that of loose and completely compacted material.

The samples taken are, therefore, all three in a relatively uncompacted state. They were all taken within two feet of the surface.

$\gamma_a$  and units derived therefrom such as  $w$ ,  $S_r$ , etc. show the moistness of the sample when taken. They are given here merely for the sake of completeness. Moisture content determinations are of interest where a number of samples are taken to explore the capillary zone, etc. The moisture which the isolated samples from the three dams happened to contain when taken is of no immediate significance.

CHAPTER 8  
CONCLUSIONS.

(1) Natural Sand Reservoirs.

Where a more or less V-shaped channel, eroded in rock and subsequently filled with sand, constitutes the reservoir, the slope of the sand and the bed rock will be consistently in one direction and frictional resistance to underground flow will be the cause of ground water being stored from one flood season to the next.

The most favourable drawoff point is the deepest point in the cross section (Fig.31). In estimating the safe rate of extraction which can be maintained without jeopardising the permanence of the supply, the computed seepage at lowest known water table can be accepted as a guide (page 112 ).

Pump stations, spread along the length of the river bed, must be sufficiently far apart. There must be a sufficient volume of saturated sand between pump stations to maintain the supply from one flood season to the next. Ground water cutoffs at the pump stations will store surplus seepage of the early part of the dry season to be available at the drawoff points late in the dry season (Fig. 42). Further development of the supply which can be considered can be in the form of embankments or weirs to increase the volume of sand deposits. Open storage dams in tributaries of the main river can also be considered, to supply water during the early part of the dry season so that the more efficient storage in the sand can be reserved for the later part.

When the ground water in river beds contains appreciable percentages of dissolved solids, evaporation in sections with high water table will cause a serious increase in the concentration (page 121 , , sample b 1 and page 121 samples h 1 to 5). Even very copious floods

developed in seven years (Fig.18). The rate of development is limited by the size of the catchment area, 14 sq. miles in this instance. If a supply of 100,000 gallons a day is to be developed in this region, within a similar period, by means of stage construction, a catchment area approximately 600 square miles in size must be selected.

In small sand storage dams a well at the dam wall, surrounded by suitable filters, will provide adequate drainage. In larger dams, However, a drainage system extending some distance <sup>up</sup> the length of the basin has to be designed as the permeability of the material in sand storage dams is low.

By providing a siphon at the dam wall the transporting power of the floods passing through the basin can be increased. Larger stages can therefore be adopted. A permanent open storage capacity will remain at the dam wall which will assist in the absorption of flood water during poor rainy seasons.

### (3) General.

Detailed information of hydrological factors is required for the proper design of sand storage dams. The following list gives these factors together with the values or rules which were found to apply to the central mica schist area in S.W.A.:-

a) Catchment yields in a semi-arid region are extremely erratic. Yields from 0 to 8 inches per annum are experienced, the latter only in exceptional flood years. The average annual runoff for some good runoff producing catchments during a series of years which did not include exceptional flood seasons, varied from 0.21 to 0.67 inches (page 27 ). Years with no runoff occur occasionally and in the design of open storage dams it

is desirable to allow for at least two years in succession with negligible runoff compared with the capacity of such dams. Years with runoff which is negligible compared with the relatively small capacity of sand, storage dams occur only occasionally and a number of such years in succession are unlikely to occur.

b) Maximum Probable Floods.

For catchment areas in S.W.A., the formulae given by G.B.Williams for the Rocky Mountains have been found to apply irrespective of the average annual rainfall of the area concerned. The formulae are as follows:-

$$Q_1 = 1900 A^{0.75} \text{ for catchment areas up to 10 sq.miles}$$

$$Q_1 = 3600 A^{0.45} \text{ for catchment areas over 10 sq.miles.}$$

A reduction factor should, however, be applied where catchments are extremely absorbant (sand-veld, Dolomite, etc.).

c) Variation in peak intensity of floods.

In the Swakop River catchment near Okahandja fifty percent of all runoff was found to occur with floods which have a peak intensity of at least .025 times the maximum probable flood and twentyfive percent with peak intensities of at least .040 times the maximum probable flood. In the smaller catchments near Gamams the corresponding values average .070 and .150, respectively, (Fig.21).

d) Particulars of flood curves.

Flood curves almost invariably show a sharp rise to the maximum value followed by a ~~much~~ gentle drop to zero.

e) Slope of river beds.

Experiments with a small scale model suggest the formula

$$i = .00331 P^{1.4} + .113 D_{50} \left( \frac{1}{q_b} \right)^{\frac{2}{3}}$$

The formula can also be applied to the slope of sediments in a sand storage dam with siphon if  $q_b$  and  $P$  are based on the siphon discharge instead of the total runoff of the catchment (pages 91 and 163 )

(4) The Future of Sand Storage in South West Africa. .

The exploitation of natural sand storage reservoirs can still be carried very much further. More comprehensive data of the basins and their replenishment must, however, be accumulated so that extraction can be properly regulated in the interests of all concerned.

The results of experiments in ground water dilution still have to be awaited, but it is expected that good results can be achieved with the methods described.

From the point of view of regional planning of water supplies, sand storage dams are worth encouraging where numerous small but dependable water supplies are required. The lower river course will be deprived of a very much smaller quantity of water than where open storage is adopted (page 14 ). Due to the reduction in evaporation losses the water usefully consumed may, nevertheless, be greater in the case of the sand storage dams than with open storage structures. Large sand storage dams, or weirs on porous foundation in natural sand reservoirs, will moreover have a regulating effect on runoff, increasing the period during which open water flows in river beds after rainy seasons.

NOTATION.

|                                  |  |
|----------------------------------|--|
| $A$ (square miles)               | = catchment area.  |
| $A_F$ (square feet)              | = cross section of flowing water a short distance upstream of a sand storage weir.                             |
| $a$ (sq.cm.)                     | = cross section of sample in permeameter.  |
| $C_2$ (constant)                 | = constant in Mueller's formula for bed load transportation.   |
| $A_s$ (sq.ft)                    | = surface area of water at storage depth $h_s$   |
| $L$ (feet)                       | = crest length of weir.  |
| $C$ (constant)                   | = constant of integration, coefficient of discharge.   |
| $C_1$ (constant)                 | = constant in the expression for the surface area of a reservoir.  |
| $I$ (inches)                     | = grain size of uniform material.  |
| $D$ (feet)                       | = dimension in the prototype, depth of ground water.   |
| $D_{10}, D_{50}$ etc. (inches)   | = 10%, 15% etc. of the material is finer than the grain size $D_{10}, D_{50}$ etc.                             |
| $D_r$ (gr.per cc.)               | = relative density of undisturbed sample.  |
| $d$ (feet)                       | = dimension in the model corresponding to $I$ in the prototype, depth of water at a tube well.                 |
| $E$ (feet per annum)             | = rate of evaporation from a free water surface.   |
| $e$ (dimensionless)              | = Void ratio of undisturbed sample.  |
| $e_0$ "                          | = void ratio of sample, loose.   |
| $e_{min}$ "                      | = void ratio of sample in state of maximum compaction.   |
| $F$ (dimensionless)              | = efficiency of a stage reservoir.   |
| $F_K$ "                          | = fraction of all runoff occurring at a peak intensity equal to or greater than any given peak intensity $Q$ . |
| $g$ (feet per sec <sup>2</sup> ) | = acceleration due to gravity  |
| $H$ (feet)                       | = loss of head in prototype.   |
| $H_1$ (feet)                     | = height of weir above sand in sand storage dam.   |
| $H_2$ (feet)                     | = depth of overflow over weir.   |
| $H_A, H_B, \& H_C$ (feet)        | = reduced levels of water observed at various points in the model.   |
| $H_D$ (feet)                     | = depth of sand above water table in a sand reservoir.   |
| $H_P$ (feet)                     | = depth of sand above point F  |

|                         |   |   |
|-------------------------|---|---|
| $H_s$ (feet)            | = | full storage depth at weir.   |
| $H_w$ (feet)            | = | depth of sand below water table.  |
| $h$ (cm,feet)           | = | hydraulic head (cm in laboratory test with permeameter, otherwise feet), head in model corresponding to $H$ in prototype. |
| $h_s$ (feet)            | = | water depth in reservoir.   |
| $h_1$ (feet)            | = | total hydraulic head.   |
| $i$ (radians)           | = | hydraulic gradient, gradient of bed of open channel.  |
| $i_{2.7}$ etc.(radians) | = | gradient of open channel with $P=2.7$ etc.  |
| $i_c$ (dimensions)      | = | critical hydraulic gradient in seepage flow.  |
| $l$ (cm, feet)          | = | length of seepage path (cm in laboratory test with permeameter, otherwise feet)   |
| $m$ (feet)              | = | hydraulic mean depth.   |
| $N$ (dimensionless)     | = | Kutter's coefficient of roughness.  |
| $N_d$ "                 | = | number of potential drops in flow net.  |
| $N_f$ "                 | = | number of flow channels in flow net.  |
| $N_m$ "                 | = | Kutter's coefficient of roughness in model.   |
| $N_p$ "                 | = | Kutter's coefficient of roughness in prototype.   |
| $n$ "                   | = | porosity = water content by volume of saturated sand.   |
| $n_1$ "                 | = | average water content by volume of a body of sand not necessarily saturated.  |
| $n_2$ "                 | = | specific yield corresponding to $n_1$   |
| $P$ (percentage)        | = | Sediment load expressed as a percentage by weight of the flow.  |
| $p$ (lbs/sq.ft.)        | = | pressure intensity on vertical retaining wall at any point $P$ .  |
| $p_d$ (lbs/sq.ft.)      | = | pressure intensity due to dry sand at depth $H_d$   |
| $p_s$ (lbs/sq.ft.)      | = | pressure intensity due to saturated sand at depth $H_s$   |
| $p_w$ (lbs/sq.ft)       | = | pressure intensity on retaining wall due to water   |
| $Q$ (cusecs)            | = | peak intensity of a flood.  |
| $Q_1$ (cusecs)          | = | maximum probable flood in accordance with G.B.Williams' formulae.   |



- $Q_A$  (cu.ft./annum) = rate of useful drawoff from a reservoir.
- $Q_P$  (cusecs) = priming flood (siphon).
- $Q_T$  (cusecs) = siphon discharge.
- $q_f$  (cc/day, cu.ft./day) = rate of percolation (cc./day in permeameter tests, cu.ft./day in all other cases.
- $q_{f_1}, q_{f_2} \& q_{f_3}$  (cu.ft./day) = rate of percolation or extraction at various points in a stage basin.
- $q_{fa}$  (cusecs) = rate of flow per unit width of channel.
- $q_{fb}$  (cusecs) = peak intensity of river bed forming flood per unit width of channel.
- $q_{fu}$  (cu.ft./day) = seepage flow in river bed upstream of a sheet pile weir.
- $q_{fv}$  (cu.ft./day) = seepage flow in river bed downstream of a sheet pile weir.
- $q_{fw}$  (cu.ft./day) = seepage flow under sheet piles.
- $R$  (dimensionless) = Reynolds number.
- $R$  (feet) = radius of circle of influence.
- $R_E$  (dimensionless) =  $\frac{\text{Volume of upper three feet in sand reservoir}}{\text{Total volume of sand in reservoir}}$ .
- $r$  (feet) = radius of well.
- $S_r$  (percentage) = degree of saturation.
- $T_D$  (years) = time taken to deplete a reservoir from full storage depth  $H_s$  to empty.
- $t_D$  (years) = time taken to deplete a reservoir from depth  $h_s$  to empty.
- $T_E$  (years) = time taken for the depletion of the top three feet of a sand reservoir.
- $t$  (days, years) = duration of seepage flow from a sand reservoir (days), time required to deplete a residual storage depth (years)
- $V$  (ft./sec) = velocity of flow in prototype
- $V_1$  (cc) = volume of undisturbed sample.
- $V_2$  (cc) = volume of sample, loose.
- $V_3$  (cc) = volume of sample subjected to maximum compaction.
- $V_R$  (cu.ft.) = residual open storage capacity in sand storage dam with siphon.
- $V_S$  (cu.ft.) = Volume of reservoir corresponding to a storage depth  $H_s$ .

|   |  |
|---|--|
| $v$ (ft/sec)  | = velocity of flow in model corresponding to $V$ in prototype.             |
| $v_A$ (ft/sec)  | = velocity at crest of siphon.   |
| $v_a$ (ft/sec)  | = average velocity of flow.  |
| $v_s$ (cu.ft)   | = volume of reservoir corresponding to a storage depth $h_s$               |
| $v_t$ (ft/sec)  | = velocity of flow in tube well.   |
| $W$ (lbs/sec)   | = weight of sediments transported per unit time per foot width of channel. |
| $W_1$ (grams)   | = weight of sample as taken.   |
| $W_2$ (grams)   | = submerged weight of sample.  |
| $W_3$ (grams)   | = weight of sample, dry.   |
| $w$ (gr/cc)   | = water content of sand.   |
| $w_{sat}$ (gr/cc)   | = water content of saturated sand.   |
| $X$   | = an unknown in a mathematical analysis of a problem.                      |
| $\gamma$ (gr/cc)  | = unit weight of saturated material.                                       |
| $\gamma_a$ "  | = unit weight of material as sampled.                                      |
| $\gamma_d$ "  | = " " " dry material, undisturbed.   |
| $\gamma_{max}$ "  | = " " " " " , max.com-paction.   |
| $\gamma_o$ "  | = " " " " " , loose.   |
| $\gamma'$ "   | = " " " submerged sample.  |
| $\gamma_{water}$ (lbs/cu ft)  | = density of water.  |
| $\phi$ (degrees)  | = angle of repose.   |
| $\sigma$ $\left\{ \frac{\text{square feet}}{\text{seconds}} \right\}$ | = kinematic viscosity.   |

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APPENDIX A  
DYNAMIC SIMILARITY.

In closed conduits one method of attaining dynamic similarity in geometrically similar structures is to subject the model to a much greater head than that which occurs in the full size structure, thus increasing the velocity through the model and making the Reynolds numbers equal.

$$\text{i.e.} \quad \frac{VD}{\sigma} = \frac{vd}{\sigma}$$

$\sigma$  is the kinematic viscosity.

$V$  and  $v$  are velocities and  $D$  and  $d$  dimensions in the prototype and model respectively.

Or if the same liquid is used in each case,

$$VD = vd$$

$$\text{i.e.} \quad \frac{V}{v} = \frac{d}{D} \quad \dots\dots\dots(10)$$

If both the model and the prototype have perfectly smooth walls, equal Reynolds numbers will mean dynamic similarity <sup>7</sup>.

With dynamic similarity,  $\frac{\text{Total energy loss}}{\text{Kinetic energy produced}}$  is

the same in each case

$$\text{i.e.} \quad \frac{h}{v^2} = \frac{H}{V^2} \quad \dots\dots\dots(11)$$

Combining equations (10) and (11)

$$\frac{h}{H} = \left(\frac{D}{d}\right)^2 \quad \dots\dots\dots(12)$$

If a model is half the size of the prototype it must therefore be subjected to four times the head which occurs in the prototype; in order that dynamic similarity will be achieved by this method.

The method cannot be applied to open channels where the head of necessity obeys the scale ratio. In any case equation (12) will only be strictly correct for the theoretical case of perfectly smooth walls.

A method of attaining <sup>dynamic</sup> similarity which is applicable to either conduits or open channels, is to make the model very much smoother than the prototype. In this method heads can be allowed to obey the model scale ratio. Manning's formula<sup>7</sup> will show that for any two sets of conditions of roughness there will be one scale ratio which will result in dynamic similarity.

Manning's formula for flow in open and closed conduits is:

$$v = \frac{1.486}{N} m^{2/3} i^{1/2} \dots\dots\dots(13)$$

where  $v$  = velocity of flow in feet per second

$m$  = hydraulic mean depth

$i$  = hydraulic gradient, or head lost divided by distance travelled.

$N$  = Kutter's coefficient of roughness

As heads obey the same scale ratio as the model dimensions,  $i$  will be the same in the model as in the prototype and

$$\frac{v}{V} = \left(\frac{m_d}{m_D}\right)^{2/3} \times \frac{N_p}{N_m} = \left(\frac{d}{D}\right)^{2/3} \times \frac{N_p}{N_m}$$

Where suffix  $p$  refers to the prototype and suffix  $m$  to the model.

But for dynamic similarity equation (12) must also be satisfied.

$$\text{i.e. } \frac{h}{H} = \left(\frac{v}{V}\right)^2$$

$$\therefore \left(\frac{h}{H}\right)^{1/2} = \left(\frac{d}{D}\right)^{2/3} \frac{N_p}{N_m}$$

But  $\frac{h}{H} = \frac{d}{D}$  as heads are made to follow the scale ratio.

$$\therefore \left(\frac{D}{d}\right)^{1/6} = \frac{N_p}{N_m} \dots\dots\dots(14)$$

APPENDIX B

ANALYSIS OF SOME EXAMPLES OF FLOW IN SAND CHANNELS AND RESERVOIRS.

(1) Flow in a sand channel of uniform cross section with the water table parallel to the surface of the sand.

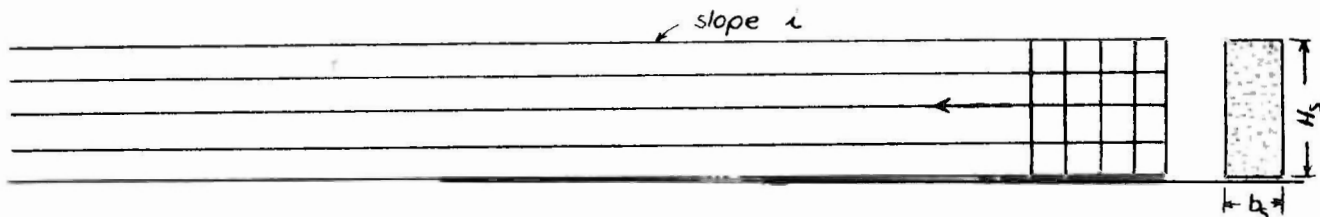


Fig. 67

The cross sectional area of saturated sand =  $H_s b_s$

The rate of flow  $q_f = k H_s b_s i$

(2) Effect of a ground water cutoff on the flow in example(1)

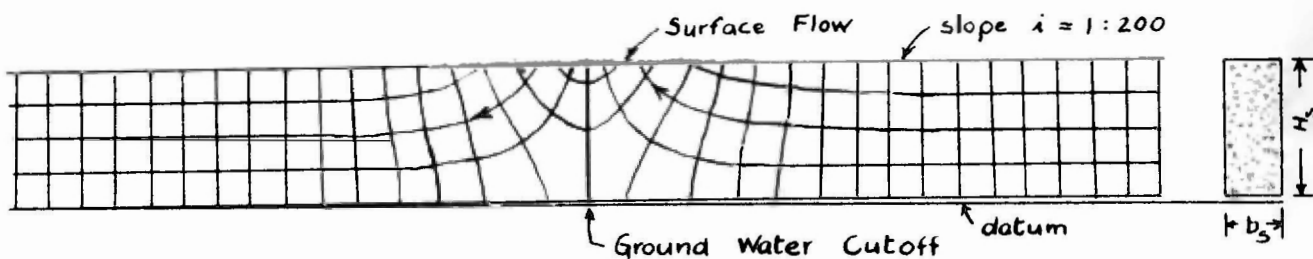
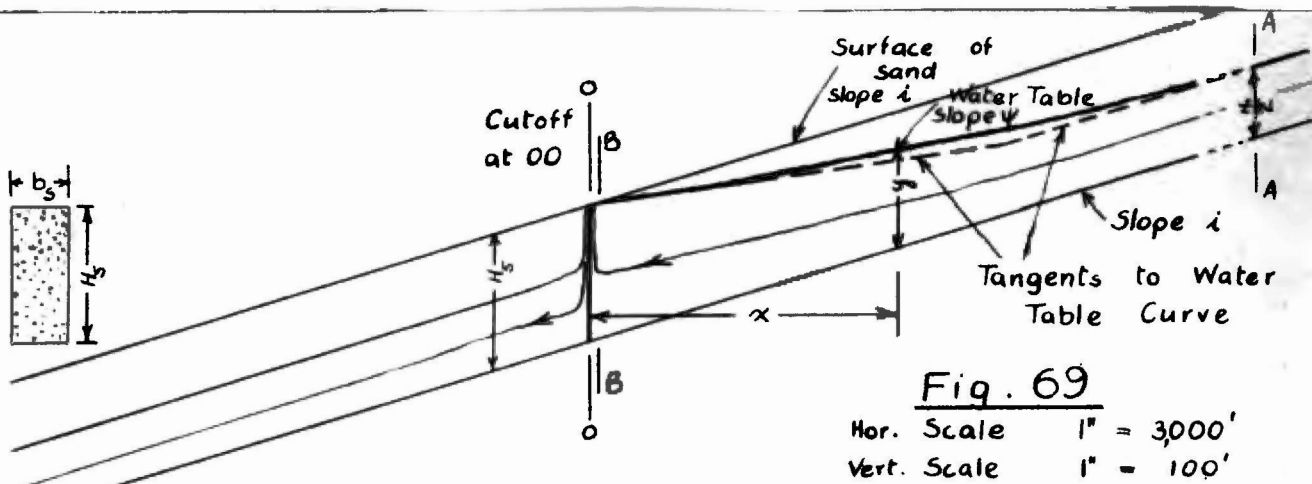


Fig. 68

The total flow  $q_f = k H_s b_s i$  is brought to the surface at the cutoff.

Fig.68 represents a sand channel 70 feet deep to a scale of 1" = 100'. The flow net shows that 10feet from the cutoff  $\frac{5}{4}$  of the flow is at the surface and  $\frac{1}{4}$  in the sand. At 25 feet the proportions are  $\frac{1}{2}$  and  $\frac{1}{2}$  and at 55 feet  $\frac{1}{4}$  and  $\frac{3}{4}$  for surface and underground flow, respectively. At 100 feet from the cutoff the flow is almost entirely underground.

(3) Effect of a ground water cutoff when the sand channel yields only half the flow of examples (1) and (2).



At section AA at an infinite <sup>distance</sup> upstream of the cutoff

$$q_f = \frac{1}{2} k H_s v_s i$$

At any other section, distance  $x$  from OO, the flow =  $k y v_s \psi$

Since the flow is constant throughout,

$$k y v_s \psi = \frac{1}{2} k H_s v_s i$$

i.e.  $y \psi = \frac{1}{2} H_s i$

At section BB, 150 feet upstream of OO, the flow is for all practical purposes completely underground (see example 2) and  $y$  is practically equal to  $H_s$

i.e.  $\psi_B = \frac{1}{2} i$

In comparison with the extent of the back water curve sections BB and OO are very close together.

Analysis of the back water curve will therefore be based on the following data:-

Anywhere between OO and AA,  $y \psi = \frac{1}{2} H_s i$

$$y_0 = H_s$$

$$\therefore \psi_0 = \frac{1}{2} i$$

$$y_A = \frac{1}{2} H_s$$

It is clear from Fig.69 that  $\psi = i + \frac{dy}{dx}$

(4) Analysis of the general case of example (3) where the saturated depth at section AA has any value  $y_A$ .

$$\begin{aligned}\text{Flow per unit width} &= k y \psi \\ &= k y \left( i + \frac{dy}{dx} \right)\end{aligned}$$

At section AA the slope  $\psi = i$  and the depth  $y = y_A$

$$\therefore k y \left( i + \frac{dy}{dx} \right) = k y_A i$$

$$i + \frac{dy}{dx} = \frac{y_A i}{y}$$

$$\frac{dy}{dx} = -i \left( 1 - \frac{y_A}{y} \right)$$

$$\frac{dx}{dy} = - \frac{y}{i(y - y_A)}$$

$$\begin{aligned}x &= - \frac{1}{i} \int_{H_s}^y \frac{y dy}{y - y_A} \\ &= \frac{1}{i} \int_y^{H_s} \frac{y dy}{y - y_A} \\ &= \frac{1}{i} \int_y^{H_s} \left( 1 + \frac{y_A}{y - y_A} \right) dy \\ &= \frac{1}{i} \left[ y + y_A \log(y - y_A) \right]_y^{H_s} \\ &= \frac{1}{i} \left\{ (H_s - y) + y_A \log \frac{H_s - y_A}{y - y_A} \right\}\end{aligned}$$

This expression gives the distance upstream of the cutoff at which any given depth of water table will be maintained.

For instance, if the depth, of saturated sand at the cutoff,  $H_s = 70$  feet and the depth at an infinite distance from the cutoff,  $y_A = 35$  feet then a depth of  $\frac{3}{4} H_s$

or 52.5 feet will be maintained at

$$\begin{aligned}x &= 200 \left\{ 17.5 + 35 \log_e \frac{35.0}{17.5} \right\} \\ &= 8400 \text{ feet}\end{aligned}$$



The volume of saturated sand per unit width of channel for a distance  $x$  upstream of the cutoff  $= \int_{H_s}^y y dx$  and can be derived from the expression  $\frac{dx}{dy} = -\frac{y}{i(y - y_A)}$  determined above.

Multiplying by  $y dy$  and integrating the expression becomes

$$\begin{aligned} \int y dx &= - \int \frac{y^2 dy}{i(y - y_A)} \\ \text{i.e. } \int_{H_s}^y y dx &= \frac{1}{i} \int_{y}^{H_s} \frac{y^2 dy}{y - y_A} \\ &= \frac{1}{i} \int_{y}^{H_s} y dy + \frac{y_A}{i} \int_{y}^{H_s} \frac{y dy}{y - y_A} \\ &= \frac{1}{2} \frac{1}{i} (H_s^2 - y^2) + y_A x \end{aligned}$$

It will be seen that the first term,  $\frac{1}{2} \frac{1}{i} (H_s^2 - y^2)$  gives the volume of saturated sand above the asymptote

$y = y_A$  and the second term  $y_A x$  the volume below the asymptote.

If  $i = \frac{1}{200}$ ,  $H_s = 70$ ,  $y = 52.5$ , and  $y_A = 35$  as before then the volume of saturated sand in section  $x$  is

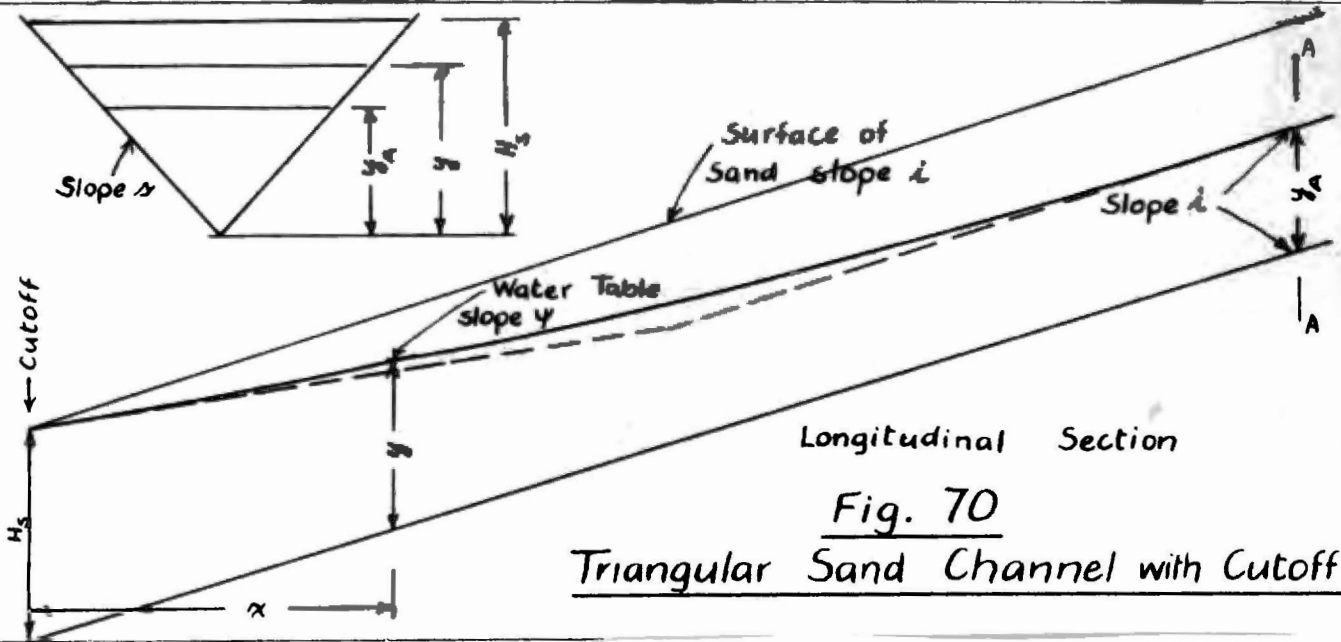
$$\begin{aligned} \frac{1}{2} \frac{1}{i} (H_s^2 - y^2) + y_A x &= 100 (4900 - 2760) + 8400 \times 35 \\ &= 214,000 + 294,000 \text{ cu.ft.} \end{aligned}$$

i.e. The additional volume of saturated sand due to the cutoff in the lower 8400 feet of the basin

= 214,000 cu.ft. per unit width of basin. and normal volume of saturated sand in 8400 feet of basin = 294,000 cu.ft.

The total additional storage due to the cutoff up to the limit  $\alpha \rightarrow \infty$  is given by  $\frac{1}{2} \frac{1}{i} (H_s^2 - y_A^2) = 100(70^2 - 35^2)$   
 $= 100 \times 4900 \times \frac{3}{4}$   
 $= 368,000 \text{ cu.ft.}$

(5) Ground Water Cutoff in sand channel of triangular cross section.



At distance  $\alpha$  upstream of the cutoff

depth of saturated zone =  $y$

and saturated area =  $\frac{1}{2} y^2$  (see cross section)

$$\begin{aligned} \text{Flow in channel} &= k \frac{1}{2} y^2 \psi \\ &= k \frac{1}{2} y^2 \left( i + \frac{dy}{dx} \right) \end{aligned}$$

At section AA where  $\alpha \rightarrow \infty$  the slope  $\psi = i$

and the depth  $y = y_A$

$$\therefore k \frac{1}{2} y^2 \left( i + \frac{dy}{dx} \right) = k \frac{1}{2} y_A^2 i$$

$$i + \frac{dy}{dx} = \frac{y_A^2 i}{y^2}$$

$$\frac{dy}{dx} = -i \left( 1 - \frac{y_A^2}{y^2} \right)$$

$$\frac{dx}{dy} = - \frac{y^2}{i (y^2 - y_A^2)}$$

$$\begin{aligned}
 x &= -\frac{1}{i} \int_{H_s}^y \frac{y^2 dy}{y^2 - y_A^2} \\
 &= \frac{1}{i} \int_y^{H_s} \frac{y^2 dy}{y^2 - y_A^2} \\
 &= \frac{1}{i} \int_y^{H_s} \left( 1 + \frac{y_A^2}{y^2 - y_A^2} \right) dy \\
 &= \frac{1}{i} \left[ y + \frac{1}{2} y_A \log \frac{y - y_A}{y + y_A} \right]_y^{H_s} \\
 &= \frac{1}{i} \left\{ (H_s - y) + \frac{1}{2} y_A \log \left( \frac{H_s - y_A}{H_s + y_A} \frac{y + y_A}{y - y_A} \right) \right\}
 \end{aligned}$$

Substituting the numerical values  $H_s = 70$ ,  $y = 52.5$ ,

$$y_A = 35, \text{ and } i = \frac{1}{200}$$

$$x = 200 \times 70 \left\{ 0.25 + 0.5 \times 0.5 \log \left( \frac{1}{5} \times 5 \right) \right\}$$

$$= 200 \times 70 (0.25 + .125)$$

$$= 200 \times 70 \times .375$$

$$= 4200 \text{ feet.}$$

The volume of saturated sand for a distance  $x$  upstream

of the cutoff =  $\int_{H_s}^y \frac{1}{s} y^2 dx$  and can be derived from the expression  $\frac{dx}{dy} = -\frac{y^2}{i(y^2 - y_A^2)}$  determined above.

Multiplying by  $\frac{1}{s} y^2 dy$  and integrating, the expression becomes

$$\int \frac{1}{s} y^2 dx = -\int \frac{y^4 dy}{i s (y^2 - y_A^2)}$$

$$\text{i.e. } \int_{H_s}^y \frac{1}{s} y^2 dx = \frac{1}{i s} \int_y^{H_s} \frac{y^4 dy}{y^2 - y_A^2}$$

$$= \frac{1}{i s} \int_y^{H_s} y^2 dy + \frac{y_A^2}{i s} \int_y^{H_s} \frac{y^2 dy}{y^2 - y_A^2}$$

$$= \frac{1}{3} \frac{1}{i s} (H_s^3 - y^3) + \frac{1}{s} y_A^2 x$$

It will be seen that the first term  $\frac{1}{3} \frac{1}{i \alpha} (H_s^2 - y^2)$  gives the volume of saturated sand above the asymptote

$y = y_A$  and the second term  $\frac{1}{\alpha} y_A^2 x$  the volume below the asymptote.

If  $i = \frac{1}{200}$ ,  $H_s = 70$ ,  $y = 52.5$ ,  $y_A = 35$  and  $\alpha = \frac{1}{10}$  then the volume of saturated sand in section  $x$  is

$$\begin{aligned} \frac{1}{3} \frac{1}{i \alpha} (H_s^3 - y^3) + \frac{1}{\alpha} y_A^2 x &= \frac{1}{3} \times 10 \times 200 (70^3 - 52.5^3) + 10 \times 35^2 \times 4200 \\ &= 667(343000 - 145000) + 10 \times 1225 \times 4200 \\ &= 667 \times 198,000 + 51,450,000 \\ &= 132,000,000 + 51,450,000 \end{aligned}$$

i.e. The additional volume of saturated sand due to the cutoff in the lower 4200 feet of the basin is 132,000,000 cubic feet and is far in excess of the volume of saturated sand in an unobstructed 4200 feet section of the channel, which amounts to only 51,400,000 cubic feet.

The total additional storage due to the cutoff up to the limit  $x \rightarrow \infty$  is given by

$$\begin{aligned} &= 667(343,000 - \frac{1}{4} \times 343,000) \\ &= 171,000,000 \text{ cu.ft.} \end{aligned}$$

(6) Sand Storage Basins in which the inflow breaks off abruptly at the end of the rainy season.

All examples considered up to now have been for a uniform rate of discharge throughout the length of the channel and the water table constant at any section.

Where a sand storage reservoir receives no inflow from upstream, drainage will cause a lowering of the water table.

In the examples which follow this condition of no inflow during depletion will apply.

(7) Sand Storage Basin of triangular cross section with depth increasing in direction of flow and without cutoffs or other barriers.

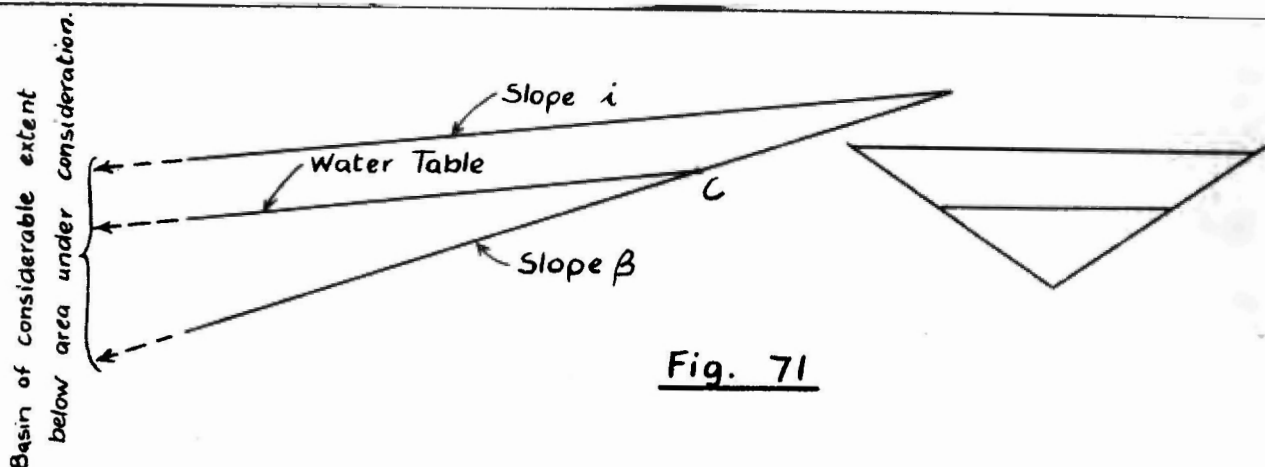


Fig. 71

Under the conditions stated the water will behave like a wedge sliding down an inclined plane.

The slope  $\beta$  and not the surface slope will determine the velocity of flow.

The velocity of flow will be equal to  $k\beta$  irrespective of the slope of the water table. Even if the water table were level the saturated zone will "slide" down the incline  $\beta$  at a velocity  $= k\beta$ .

The point C will recede in the direction of flow at a velocity  $\frac{k\beta}{n_2}$  where  $n_2$  = the specific yield of the material

(8) Sand Storage Basin as in example (7) but with a water tight cutoff at the lower end of the area under consideration.

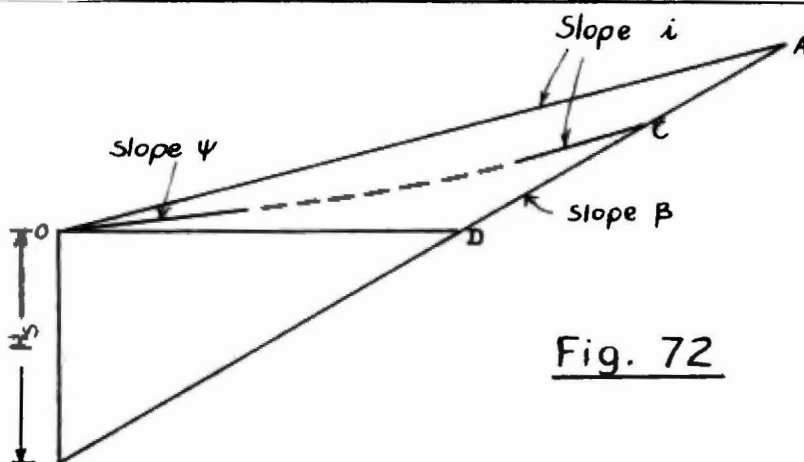


Fig. 72

At the cutoff the water table will remain at a constant level as in examples 1) to 5) and the velocity of flow will be proportional to the surface slope  $\psi$ . Near the upper end of the basin the velocity will be equal to  $k\beta$  as in example 7). Eventually the water table will come to rest on line OD.

As an approximation the water table will be assumed to have a slope  $\psi$  throughout. The error made will be due to volumes in the upper portion of the basin, which are small compared with volumes near the cutoff.

As a basis of analysis the condition  $\frac{d\gamma}{dt} = q_f$  will be laid down where  $\gamma$  is the volume of water drawn off up to a certain instant and  $q_f$  is the rate of drawoff at that instant.

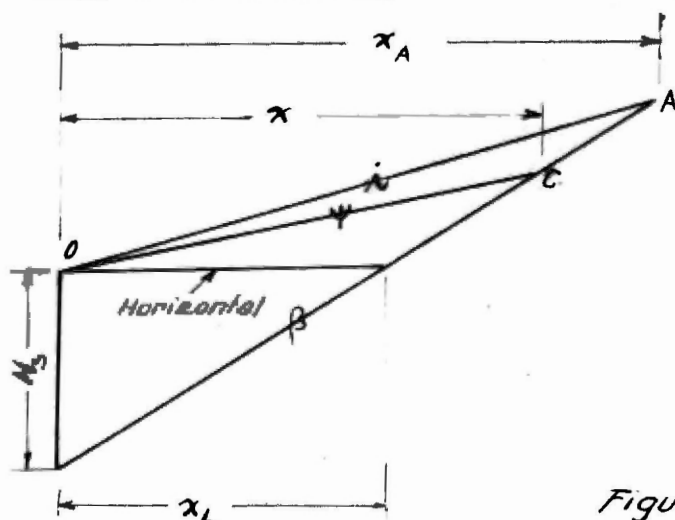


Figure 73.

It is desired to determine the time  $\tau$  in which the water table recedes from OA to OC.

$$x_A = \frac{H_s}{\beta - i} \quad x = \frac{H_s}{\beta - \psi} \quad x_L = \frac{H_s}{\beta}$$

At any position of the water table  $OC$ .

$$z = z_A - \frac{1}{3} n_2 \frac{1}{s} H_s^2 x$$

$$\begin{aligned} q &= k \frac{1}{s} H_s^2 \psi \\ &= k \frac{1}{s} H_s^2 \left( \beta - \frac{H_s}{x} \right) \end{aligned}$$

but  $\frac{dz}{dt} = q$

i.e.  $-\frac{1}{3} n_2 H_s^2 \frac{1}{s} \frac{dx}{dt} = k \frac{1}{s} H_s^2 \left( \beta - \frac{H_s}{x} \right)$

$$\begin{aligned} \therefore t &= -\frac{n_2}{3k} \int \frac{dx}{\beta - \frac{H_s}{x}} + C = -\frac{n_2}{3k\beta} \int \frac{x dx}{x - \frac{H_s}{\beta}} + C \\ &= -\frac{n_2}{3k\beta} \int \frac{x dx}{x - x_L} + C = -\frac{n_2}{3k\beta} \int \frac{x - x_L + x_L}{x - x_L} dx + C \\ &= -\frac{n_2}{3k\beta} \left\{ \int dx + x_L \int \frac{dx}{x - x_L} \right\} + C \\ &= -\frac{n_2}{3k\beta} \left\{ x + x_L \log(x - x_L) \right\} + C \end{aligned}$$

but  $t = 0$  if  $x = x_A$

$$\therefore t = \frac{n_2}{3k\beta} \left\{ (x_A - x) + x_L \log \frac{x_A - x_L}{x - x_L} \right\}$$

(9) Sand Storage Basin as in example (7) but with a weir of height  $H_1$  constructed on a pervious foundation.

Due to sand deposition upstream of the weir the capacity of the basin will increase from  $O_1BL$  to  $O_2OA$

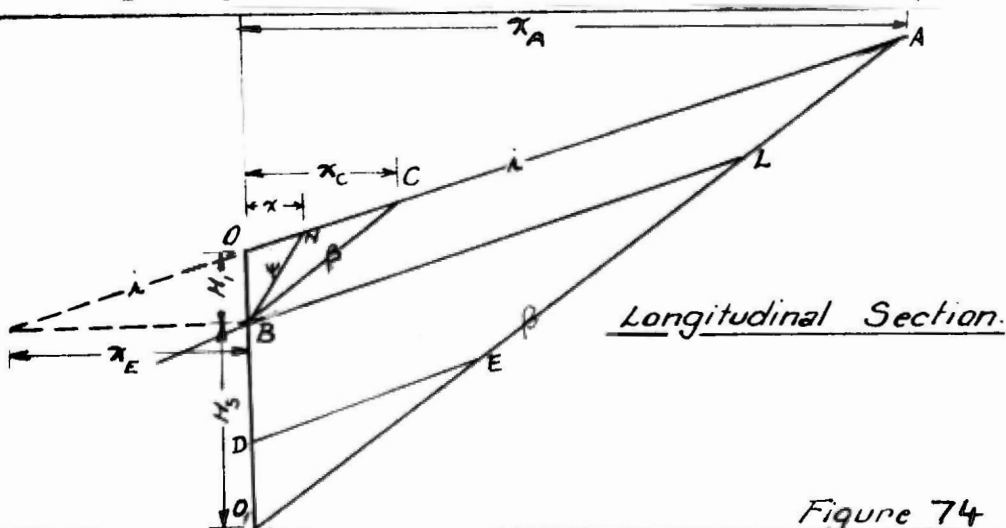
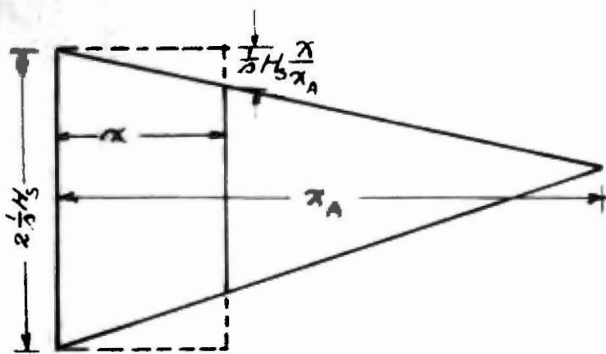


Figure 74

$OBN$  represents the volume drained in time  $t$ .

$$x_E = \frac{H_1}{i} \quad x = \frac{H_1}{\psi - i} \quad x_A = \frac{H_1 + H_s}{\beta - i}$$

As an approximation the sides of the valley above line  $AB$  are assumed vertical.

Plan

As an approximation the sides of the valley above line A-B are assumed vertical.

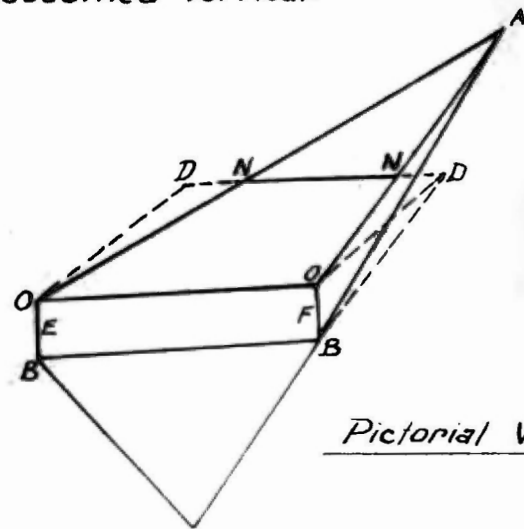
Pictorial View

Figure 75

$$z = n_2 \left( \text{volume DDFE} - 2 \times \text{volume NDBO} \right)$$

$$\begin{aligned} \text{i.e. } z &= 2 \frac{1}{3} H_s H_1 \frac{x}{2} n_2 - \frac{1}{3} 2 \frac{1}{3} H_s \frac{x}{x_A} H_1 \frac{x}{2} n_2 \\ &= n_2 \frac{1}{3} H_s H_1 x - \frac{1}{3} n_2 \frac{1}{3} H_s H_1 \frac{x^2}{x_A} \\ &= n_2 \frac{1}{3} H_s H_1 \left( x - \frac{x^2}{3x_A} \right) \end{aligned}$$

$$\begin{aligned} q &= k \frac{1}{3} H_s^2 \psi \\ &= k \frac{1}{3} H_s^2 \left( i + \frac{H_1}{x} \right) \\ &= k \frac{1}{3} H_s^2 H_1 \left( \frac{1}{x_E} + \frac{1}{x} \right) \end{aligned}$$

$$\text{but } q = \frac{dz}{dt} \quad \therefore k \frac{1}{3} H_s^2 H_1 \left( \frac{1}{x_E} + \frac{1}{x} \right) = n_2 \frac{1}{3} H_s H_1 \left( 1 - \frac{2}{3} \frac{x}{x_A} \right) \frac{dx}{dt}$$

$$dt = \frac{n_2}{k H_s} \left( \frac{1 - \frac{2}{3} \frac{x}{x_A}}{\frac{1}{x_E} + \frac{1}{x}} \right) dx$$



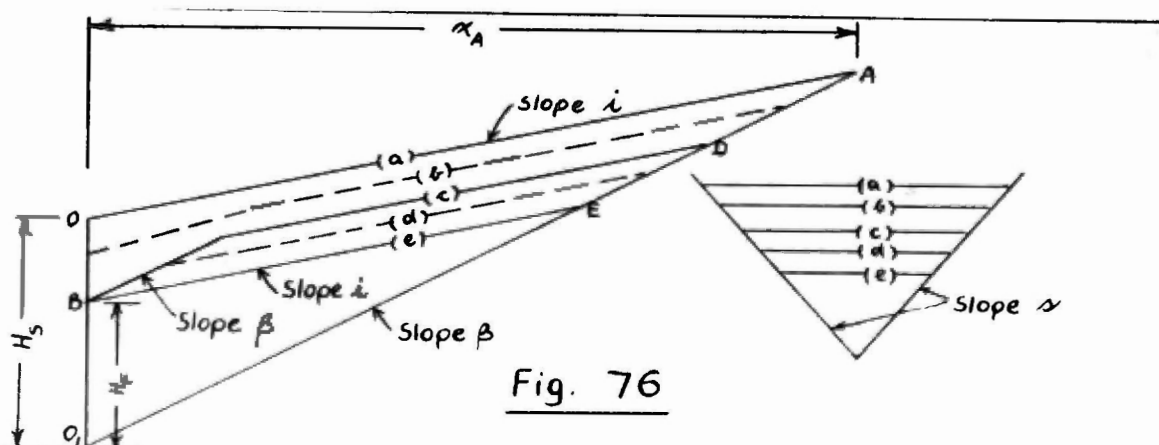
Integrating between the limits  $x=0$  and  $x=x_c$

$$\begin{aligned}
 \tau &= \frac{n_2}{k H_s} \int_0^{x_c} \frac{x - \frac{2}{3} \frac{x^2}{x_A}}{\frac{x}{x_E} + 1} dx \\
 &= \left\{ \frac{2}{3} \frac{n_2 x_E}{k H_s x_A} \right\} \int_0^{x_c} \frac{-x^2 + \frac{3}{2} x_A x}{x + x_E} dx \\
 &= \left\{ \begin{array}{c} \text{"} \\ \text{"} \end{array} \right\} \int_0^{x_c} \frac{-x^2 - x x_E + x x_E + \frac{3}{2} x_A x}{x + x_E} dx \\
 &= \left\{ \begin{array}{c} \text{"} \\ \text{"} \end{array} \right\} \left\{ \int_0^{x_c} -x dx + (x_E + \frac{3}{2} x_A) \int_0^{x_c} \frac{x}{x + x_E} dx \right\} \\
 &= \left\{ \begin{array}{c} \text{"} \\ \text{"} \end{array} \right\} \left[ -\frac{1}{2} x^2 + (x_E + \frac{3}{2} x_A) (x - x_E \log(x + x_E)) \right]_0^{x_c} \\
 &= \frac{2}{3} \frac{n_2 x_E}{k H_s x_A} \left\{ -\frac{1}{2} x_c^2 + (x_E + \frac{3}{2} x_A) (x_c - x_E \log \frac{x_c + x_E}{x_E}) \right\}
 \end{aligned}$$

This is an approximate expression for the recession of the water table from OA to BCA.

Further recession will be governed by the condition that the ground water body O, BCA "slides" down the incline  $\beta$  at a velocity  $k\beta$ .

(10) Sand Storage Basin as in example 8) but with a constant rate of extraction, equal to the initial rate of efflux, being applied.



The lines (a) (b) (c) (d) and (e) indicate the position of the water table with successive stages of depletion.

Initially the water table is at the surface of the sand and the rate of extraction is made equal to the initial rate of efflux which would have taken place over the cutoff,  $q_c = \frac{1}{S} H_s^2 k i$

To maintain this rate of extraction with smaller cross sections of flow, as depletion proceeds, it is necessary for the slope of the water to become steeper at the cutoff (Line (b) ). In the upper region of the basin the water table will retain the slope  $i$  and the ground water will "slide" down at a velocity  $k\beta$  as in example (7).

When the steep portion near the cutoff has attained the slope  $\beta$  (line (c) ) the depth at the cutoff will be given by  $\frac{1}{S} H_F^2 k \beta = q_c$

$$= \frac{1}{S} H_s^2 k i$$

$$\text{i.e. } H_F = \sqrt{\frac{i}{\beta}} H_s$$

After this <sup>the</sup> ground water body  $O_1BCD$  will "slide" down the slope  $\beta$ , giving a constant discharge  $q_c$  until  $C$  reaches the position  $B$  i.e. up to the position of the water table (e).

The residual water storage below (e) will be denoted by  $\gamma_E$  and will be available at a rate of extraction less than  $q_c$ .

$$\begin{aligned} \gamma_E &= \left( \frac{H_F}{H_s} \right)^3 \gamma_A \\ &= \left( \frac{i}{\beta} \right)^{3/2} \gamma_A \end{aligned}$$

$t_N$  = period during which constant drawoff  $q_c$  is maintained

$$\begin{aligned} t_N &= \left( \frac{\gamma_A - \gamma_E}{q_c} \right) \\ &= \left\{ 1 - \left( \frac{i}{\beta} \right)^{3/2} \right\} \frac{\gamma_A}{q_c} \end{aligned}$$

(11) Application of the analytical results obtained to saturated and partly saturated storage basins.

Examples 1 to 10 have been analysed on the basis of completely saturated material. This requirement in most instances is only satisfied in very favourable rainy seasons. The record of the total water yield of the sand storage dam at Aukeigas near Windhoek shows, that a body of fine sand yields up to 25% of its volume after very favourable rainy seasons; but that the yield in most seasons is of the order of 10%. (See chapter 3) It has been observed that in wells drawing their supply from sandy river beds the possible rate of extraction is decreased as a result of incomplete saturation after poor runoff years and that the decline of the water table is more rapid even with the lower rate of extraction.

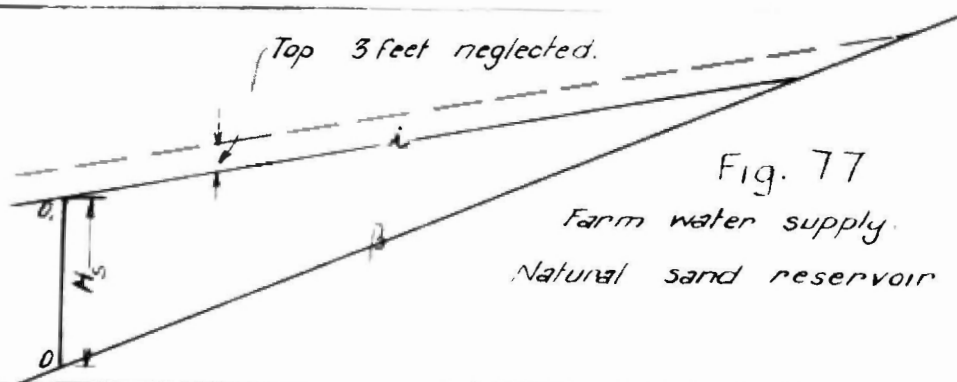
The ratio between the total yield of a sand body in any particular year and the total yield with complete saturation will be referred to as the yield ratio and will be denoted by the symbol,  $Y$ .

$$q_f t = Y q_s t_s$$

where  $q_f$  and  $t$  are the rate of percolation and its duration with a yield ratio  $Y$  and  $q_s$  and  $t_s$  the corresponding amounts when the sand body is completely saturated.

Conditions of flow with incomplete saturation must be very complex and difficult of solution. It is suggested that estimates of yield and duration be based in the assumption that  $q_f/q_s = t/t_s = \sqrt{Y}$  which satisfies the condition  $q_f t = Y q_s t_s$ .

As a practical example, to illustrate the method of analysis, the following farm water supply will be considered:-



$$H_s = 20 \text{ feet}$$

$$i = .0033$$

$$\beta = .0066$$

$$\Delta = 0.10$$

To allow for evaporation, the top three feet have been neglected.

(A) Complete saturation will first be considered.

$$n_2 = .25$$

$$k = 400 \text{ feet a day.}$$

(a) Depletion with undisturbed flow as in example (7)

The initial rate of discharge at  $OO_1$  will be

$$\begin{aligned} q_0 &= k \frac{1}{\Delta} H_s^2 \beta \\ &= 400 \times 10 \times 20^2 \times .0066 \text{ cubic feet a day.} \\ &= 10540 \text{ cubic feet a day.} \end{aligned}$$

The discharge will be reduced to 2000 cubic feet a day when

$$\begin{aligned} k \frac{1}{\Delta} h_s^2 \beta &= 2000 \\ h_s &= \sqrt{\frac{2000}{400 \times 10 \times .0066}} \\ &= 8.7 \text{ feet.} \end{aligned}$$

The time during which the rate of discharge is 2000 cubic feet a day or more can be deduced from the distance which the point  $C$  recedes as the water depth at  $OO_1$  drops from 20 to 8.7 feet.

$$\text{Distance} = (20 - 8.7) \frac{1}{.0066 - .0033} = 3390 \text{ feet.}$$

$$\begin{aligned} \text{Velocity of recession as deduced in example (7)} &= \frac{k\beta}{n_2} \\ &= \frac{400 \times .0066}{.25} = 10.6 \text{ ft/day.} \end{aligned}$$

$$\therefore t = \frac{3390}{10.6} = 320 \text{ days.}$$

(b) Depletion with groundwater cutoff.

The original rate of discharge =

$$K \frac{1}{S} H_s^2 i = 400 \times 10 \times 20^2 \times .0033 = 5270 \text{ cubic feet a day (example (8) ).}$$

(i) The flow will be reduced to 2000 cubic feet a day when a groundwater slope  $\psi$  is reached after time  $t$ .

$$\begin{aligned} \psi &= \frac{2000}{K \frac{1}{S} H_s^2} \\ &= \frac{2000}{400 \times 10 \times 20^2} \\ &= .00125 \end{aligned}$$

Determining the terms in Fig.73,

$$\begin{aligned} \alpha &= \frac{H_s}{\beta - \psi} \\ &= \frac{20}{.0066 - .00125} \\ &= 3740 \text{ feet} \end{aligned}$$

$$\begin{aligned} \alpha_A &= \frac{20}{.0033} \\ &= 6050 \text{ feet} \end{aligned}$$

$$\begin{aligned} \alpha_L &= \frac{H_s}{\beta} = \frac{20}{.0066} \\ &= 3025 \text{ feet} \end{aligned}$$

Applying the final equation of example (8),

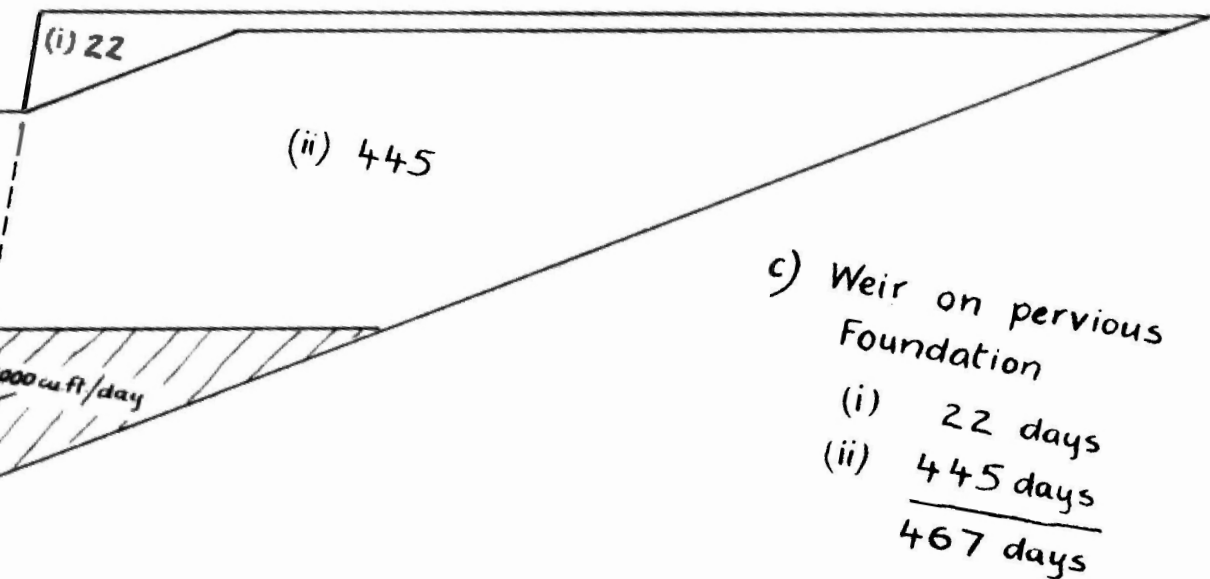
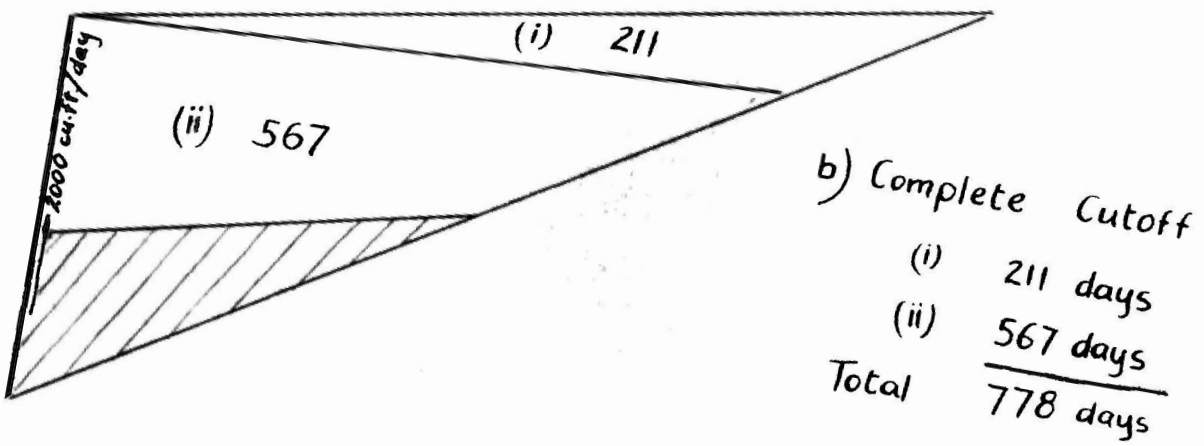
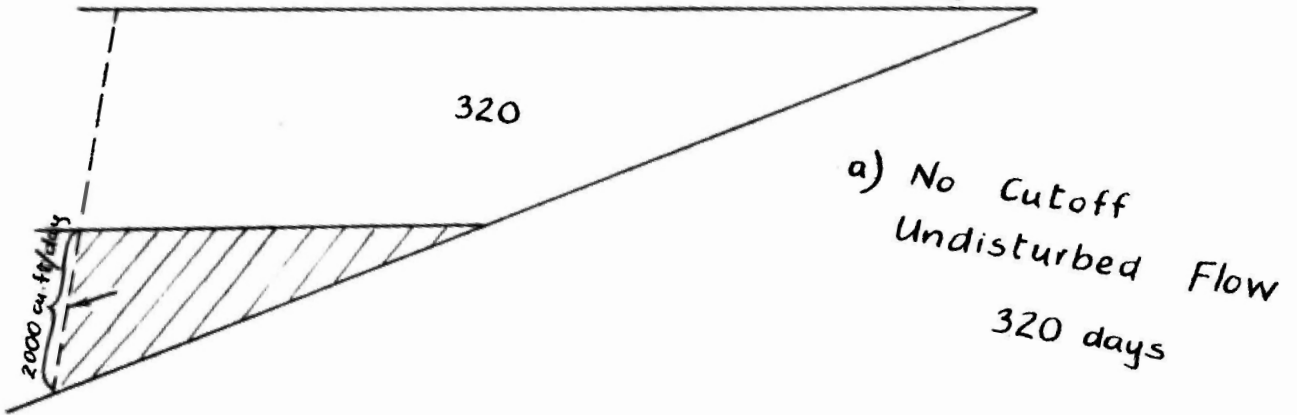
$$\begin{aligned} t &= \frac{.25}{3 \times 400 \times .0066} \left\{ 2310 + 3025 \log \frac{3025}{715} \right\} \\ &= .0316 (2310 + 3025 \times 1.442) \\ &= .0316 \times 6670 \\ &= 211 \text{ days} \end{aligned}$$

(ii) Extraction can now be continued at the rate of 2000 cubic feet a day. In accordance with the final equation determined in example (10), the time during which that rate of extraction can be maintained will be given by

$$t_N = \left\{ 1 - \left( \frac{.00125}{.0066} \right)^{\frac{3}{2}} \right\} \frac{\gamma_A}{q_c}$$

$\gamma_A$  is the available water storage at the beginning of this period of extraction when the slope of

Fig. 78  
Summary of Comparative  
Depletion Analyses



(B) Allowance will now be made for partial saturation.

Assume a yield ratio  $Y = 0.4$ .

| Practical example | Time during which a yield not less than $\sqrt{.4} \times 2000 = 1260$ cu.ft. per day is maintained. |
|-------------------|--|
| a                 | $\sqrt{.4} \times 320 = 202$ days = $6\frac{1}{2}$ months.   |
| b                 | $\sqrt{.4} \times 778 = 492$ days = 16 months.   |
| c                 | $\sqrt{.4} \times 467 = 296$ days = $9\frac{1}{2}$ months  |

The watertight cutoff will thus make it possible to use a small quantity of water for a long period and will constitute a considerable improvement on natural conditions. The weir on pervious foundation will only extend the availability of the supply by three months but it should not be lost sight of that this method has doubled the sand storage capacity, surplus water being allowed to flow to the lower reaches of the river. This means river regulation and an approach to conditions of perennial flow.

Frequently the sand bed of the river can be dammed by means of a wide sand embankment and the excess flood water allowed to escape over a rocky shelf on the side of the valley. With this variation the cost may be less than where a watertight cutoff is adopted. The decision as to which ground water augmentation system to adopt will depend on the circumstances of each case.

NOTATION USED IN APPENDICES.

|                          |  |
|--------------------------|--|
| $b_s$ (feet)             | = width of sand channel.   |
| $H_s$ (feet)             | = depth of sand channel.   |
| $q$ (cu.ft/day)          | = rate of extraction from a sand storage dam or rate of flow past a cross section.                   |
| $q_c$ (cu.ft/day)        | = constant rate of extraction from a sand storage dam.   |
| $t$ (days)               | = period of drawoff.   |
| $t_N$ (days)             | = period during which a constant drawoff $q_c$ is maintained.  |
| $x$ (feet)               | = distance upstream of a cutoff  |
| $y$ (feet)               | = depth of saturated sand at distance $x$ upstream of a cutoff.                                      |
| $y_A$ (feet)             | = depth of saturated sand where $x \rightarrow \infty$   |
| $\gamma$ (dimensionless) | = yield ratio.   |
| $\mathcal{Z}$ (cu.ft.)   | = volume of water drawn off from a sand storage basin in time $t$ .                                  |
| $\mathcal{Z}_A$ (cu.ft.) | = total volume of water which can be drawn from a sand storage basin                                 |
| $\mathcal{Z}_E$ (cu.ft.) | = residual volume of water in a sand storage dam available at a rate of extraction less than $q_c$ . |

Other symbols used in the appendices can be found in the notation at the end of Chapter 8 and in the diagrams illustrating the analyses.