HYDROLOGICAL DAM DESIGN

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Hydrological dam design

(Honors class design project)

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ABSTRACT

Civilization has always been developed along rivers, whose presence guaranteed success to and from the seacoast, irrigation for crops, water supplies for urban communities and lately power development and an industrial water supply. However, the many advantages have always been counter balanced by the dangers of floods. In the past levees or flood banks were built along many major rivers to prevent inundation in the flood season. In more recent times storage reservoirs have been built as the principle of dam construction became better understood. Other measures like relief channels, storage basins and channel improvements are continuously under construction in many parts of the world. It is important for such works that estimates can be made of how the measures proposed will affect the behavior of flood waves in rivers so that economic solutions may be found in particular cases.

Reserving the water and making available to the drought season is a means to solve water shortage problems. However, safety must be insured in the design to protect dangers that may arise from flooding and over toping of the dam body. Therefore, considering the degree of lagging time and attenuation of a wave traversing along a reservoir or a reach is important in the design of dam spillways. Based on past information, the low flow characteristics of the river is the factor that affects the storage required and the normal full supply level of the reservoir. High flow records and flood forecasting techniques provide the basis for the design of the spillway and the flood storage required above the normal full supply level.

The objective of this study report is to estimate the water requirement of an irrigation plot at Cedara and the water reserve that may be needed to serve the required irrigation requirement. Both the SCS-SA program and hydrological flood routing techniques were used to estimate the peak design flood and the hydrographs respectively. The spillway width was determined using hydraulic formula.

Finally, from the results obtained, both the reservoir volume and irrigation water requirement for the irrigation area were estimated and cost-benefit analyses was done accordingly.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

1. INTRODUCTION

The proposed site, gauging weir U2H016, is located at the geographical coordinate system of 29^0,32’E latitude and 30^0,17’E longitude, near Cedara Agricultural College, around 25 km north of Pietermaritzburg town in the province of KwaZulu Natal, South Africa. The average elevation of the watershed area is 1067m above mean sea level (amsl), covering a total area of 5.17km^2.

The Cedara drainage basin is composed of different land cover types and has five distinctive hydrological response units. 30 percent of the area is covered by veld and pasture in good condition, the rest 70 percent of the catchment is covered by dense forests. The plantation type includes, Eucalyptus Grandis, Acacia Mearnsii, Pinus Patula, Indigenous Forest, Populus Canescens, Gum regrowth, Wattle regrowth, Pine, Felled, Saplings, Mixed Peach Orchard, Veld Kikuyu Pasture, Veld in good condition, Veld in fair condition, Scrub and Maize are the main. 87percent of the soil texture of the area is clay and 13 percent sandy clay loam. The Cedara area is commonly a combined variety of sandy clay loam and clay, which is suitable for maize crop .The soil classification including the catchment land use and different soil types are given in the Appendix.

The irrigation area with its front edging inclined to the east and its back edging towards the dam is located in the alluvial plain region. The plain terrain is closed and slopes gently to wards the U2H016 dam site with an average slope of 16.4 percent. In general, the topography of the down stream site of U2H016 area appears favorable for surface, drip and sprinkler irrigation. According to the statistic of meteorological data obtained from the Cedar station, the daily annual maximum series varies from 27mm to 313mm. There is one rainy season per year. This season mainly appears to be from September to December.

The reservoir formed by the construction of the proposed dam would contain a total capacity of 15000 m^3 of water at the full supply level. It has a dead storage of 10 percent of the full supply level. The live storage (90 percent) of the capacity of the dam is available for utilization. The dead storage could be used for drinking of domestic animals during the prolonged summer drought. The dead storage volume will also serves for accumulating silt for a certain period of time. The reservoir shall contribute to the development of ground water table at the downstream side of the dam. The water-spread area of the reservoir is estimated to be 0.67ha. Topographic suitability affects the position of the spillway. So the position of spillway should be placed to a suitable position in which retrogressive erosion of the dam toe protection should be assured and also the economical feasibility has to be foreseen. An earth excavated spillway with a trapezoidal section and longitudinal slope of 2 percent to be covered by a Kikuyu grass is preferred for the erosion protection of the spillway surface. Kikuyu grass is preferred as it can grow under the climatic and soil condition of Cedara. The width of the spillway is 20.05m with a maximum flow depth of 0.50m. Freeboard for the spillway is considered 30 percent of the depth .Hence, total depth of the spillway will be 0.65m. Maximum flow estimated to spill through this section is 16.64 $m3.s^{-1}$.

Salient features of the dam

Catchment

Area of the catchment	5.17 km^2.
Végétation cover	Forest 70 %, Veld 30%
Average slope of thé catchment	16.4%

Reservoir

Water spread area	0.67ha
Gross Storage	15000 $m.^3$
Dead storage	1500m^3
Live storage	13500m^3
Depth	6.7m

Fetch length (back water length)	402m
Dam width	33.5m

Spill Way

Type	earth excavated & covered with grass Ours wrote
Design Discharge	15.77 $m^3.s^{-1}$
Width.	20.05m
Depth of flow	0.50m
Free board	0.15m
Total depth	0.65m

Irrigation Planning

The command area is specified as per the water holding capacity of the reservoir to irrigate a certain land in the Cedara area. This project shall enable the farmers of the Cedara area to grow crop like maize in the months of November to February. Pietermaritzburg being near to this village, market for the maize crop is predicted (envisaged). Irrigation farming shall bring a remarkable change in the economic standard of the villagers resulting in social and cultural changes.

Environmental impact assessment

The reservoir supposed to help in cooling down the atmosphere with humid conditions. The growth of vegetation and trees in the catchment and hills shall be possible with the availability of water and other activities in the area. The cattle shall have better fodder than at present. For that a design consideration of environmental requirement once in every 5 years is considered.

2. DESIGN FLOOD ESTIMATION

In design flood estimation the hydrologist are faced with conflicting requirement in terms of safety and economy.

As noted by Andrew (1997), there are two methods used these days to estimate a design flood:

- The statistical analysis of past floods with extrapolation to estimate the magnitude and probability of occurrence of future floods, and
- The estimation of probable maximum precipitation of a particular catchment under the worst meteorological conditions likely to occur over the catchment, followed by an estimation of the run-off that would result from such a storm.

In this study design report, 85 years annual maximum series data of rain fall from Cedara meteorological station is processed for 2, 5, 10, 20, 50 and 100 year of return periods. Detail of extreme value distributions investigated to determine the one–day design rainfall and the result obtained contained at the Appendix. The probability distribution curve like, Gumble (EV1) vs. Return periods and Log Pearson III vs Return period and Normal distribution vs Return period are plotted and compared with the annual maximum series (AMS) vs. return period curve. The Log Pearson III curve is fit with the AMS vs. Return period curve. Hence for the 20 years return period, a 122.64mm annual maximum series value is obtained from the Log Pearson III distribution for the one-day design rainfall.

The result is compared with the known events for the coordinate latitude 29^0 32'E and longitude 30^0 17'E. The map depicting one-day rainfall depth over Southern Africa, for return period of 20 years shows a 119mm for the Cedara area. Hence, 119mm is adopted to be a one-day design rainfall to calculate the peak flow in SCS-SA model.

Results of hydrological calculation obtained from Log Pearson III distribution curve and one-day design rainfall of long year event record from SCS-SA manual are contained in Table 2.1.

Table 2.1 Results of hydrological calculations

Return Periods in Years	One day design rain fall From Log Pearson III distribution curve	One day design rainfall for Cedara catchment from SCS –SA manual.
2	55	60
5	74.13	85
10	95.56	100
20	122.62	119

Calculation Steps

The daily rainfall is ranked from the largest to the smallest. For each ranked value, the return period T is calculated using the Weibull Plotting position formula: -

$$T=\frac{N+1}{m} \tag{2.1}$$

Where: -

T	=	return period or recurrence interval,
N	=	total number of events in the data series, and
m	=	rank of each events in the data series.

Sample calculation

$$T=\frac{87+1}{6}=14.667\text{yrs}$$

Probability (col. 6, Table-A1 annex**)**

$$p = \frac{m}{n+1} \tag{2.2}$$

where

m = rank of each events, (col.5, Table-A1), and

n = total number of events.

Example (for row 1, col 6 Table-A1)

$$0.011 = \frac{1}{86 + 1}$$

Total sum of each event

$$= \sum_{i=1}^{n} x_i$$

where *x* is an event

=5446

Mean is average of total events

$$\overline{\chi} = \frac{1}{n} \sum_{i=1}^{n} x_i$$

where

$\overline{\chi}$ = mean of events,

n = total number of events, and

X_i = each event from the data series.

=63.33

Variability is given by

$$\sigma^2 = \frac{1}{n} \sum_{i=1}^{n} (x_i - \overline{x})^2 \qquad (2.3)$$

Standard deviation $= \sqrt{\sigma^2}$ or σ

σ^2 = 1280.92, and

σ = 35.79.

Skew ness (the third moment, α) is given by:

$$\alpha = \frac{n}{\sigma^3 (n-1)(n-2)} \sum_{i=1}^{n} (x_i - \bar{x})^3 \tag{2.4}$$

α = 4.52

Where

n	=	total number of events,
σ	=	standard deviation, and
$\bar{x}$	=	mean of total events.

Sample Calculation

Table1-1 (Annex) of column, 2 and 3 are frequency factors taken from the probability table for Normal and Gumble (EV1) distribution.

Since the Skew ness of events in the rainfall data series is greater than 1.0, that is equal to 4.52, the frequency factor for Log Pearson Type III distribution could not be taken directly from the Pearson type III distribution table. Hence the frequency factors for Log Pearson III, are calculated from Eq 2.5 using Table 2.2 of log normal distribution (W_{TLN}) and log transformed value of mean as variables and the results are contained in Column 4, Table A1-1(Appendix).

$$K_T = W_{TLN} + (W_{TLN}^2 - 1)(\mathrm{Log}\frac{\bar{X}}{6}) + \frac{1}{3}(W_{TLN}^3 - 6W_{TLN})(\mathrm{Log}\frac{\bar{X}}{6})^2 - (W_{TLN}^2 - 1)(\mathrm{Log}\frac{\bar{X}}{6})^3 + W_{TLN}(\mathrm{Log}\frac{\bar{X}}{6})^4 + \frac{1}{3}(\mathrm{Log}\frac{\bar{X}}{6})^5 \tag{2.5}$$

Where

K_T	=	frequency factor for Log Pearson III,
$\bar{X}$	=	mean of total events, and
W_{TLN}	=	log normal distribution.

K_T (col4, row1 of Table,1-1)=0.00+(0.0^2-1)(1.76/6) + 1/3(0.0^3-6*0.0)(1.76/6)^2-(0.0^2-1)(1.76/6)^3+ 0.00(1.76/6)^4 +1/3(1.76/6)^5=-0.27

The magnitude of an extreme event for different return periods is calculated from the formula:

$$X_T = \overline{X} + K_T \sigma \tag{2.6}$$

where

$\overline{X}$	=	mean of total events,
K_T	=	frequency factor, and
σ	=	standard deviation.

Sample calculation for Normal distribution (column 5, row 1, of TableA1-1)

= 63.33 + 0 *35.79

=63.33

Sample calculation for Gumble EV1 distribution (Column 6, Row 1, Table A1-1)

=63.33 + (-0.16)*35.79

=57.6

Log Pearson III distribution (Col.7, Row 1, Table A1-1)

=1.76 + (-0.27*0.167)

=1.71

Table 2.2 Values of the standardized variate for the Gumble (W_{TG}) and Log-Normal (W_{TLN}) distributions (Schulze et al., 1995)

Return Period T (years)	Exceedence Probability P_e	W_{TG} Gumble Distribution	W_{TLN} Log-Normal Distribution
2	0.5	0.37	0
5	0.2	1.5	0.04
10	0.1	2.25	1.28
20	0.05	2.97	1.64
50	0.02	3.9	2.05
100	0.01	4.6	2.33

In order to determine which distribution best fit with the annual maximum series data, curves are plotted from the data series of (Col 5,6,8 of Table A1-1, Appendix) and event data Series of AMS (Table-A1, Col 4, Appendix) and return period (column 7, Table A1, Appendix) verses EVD value distribution curve.

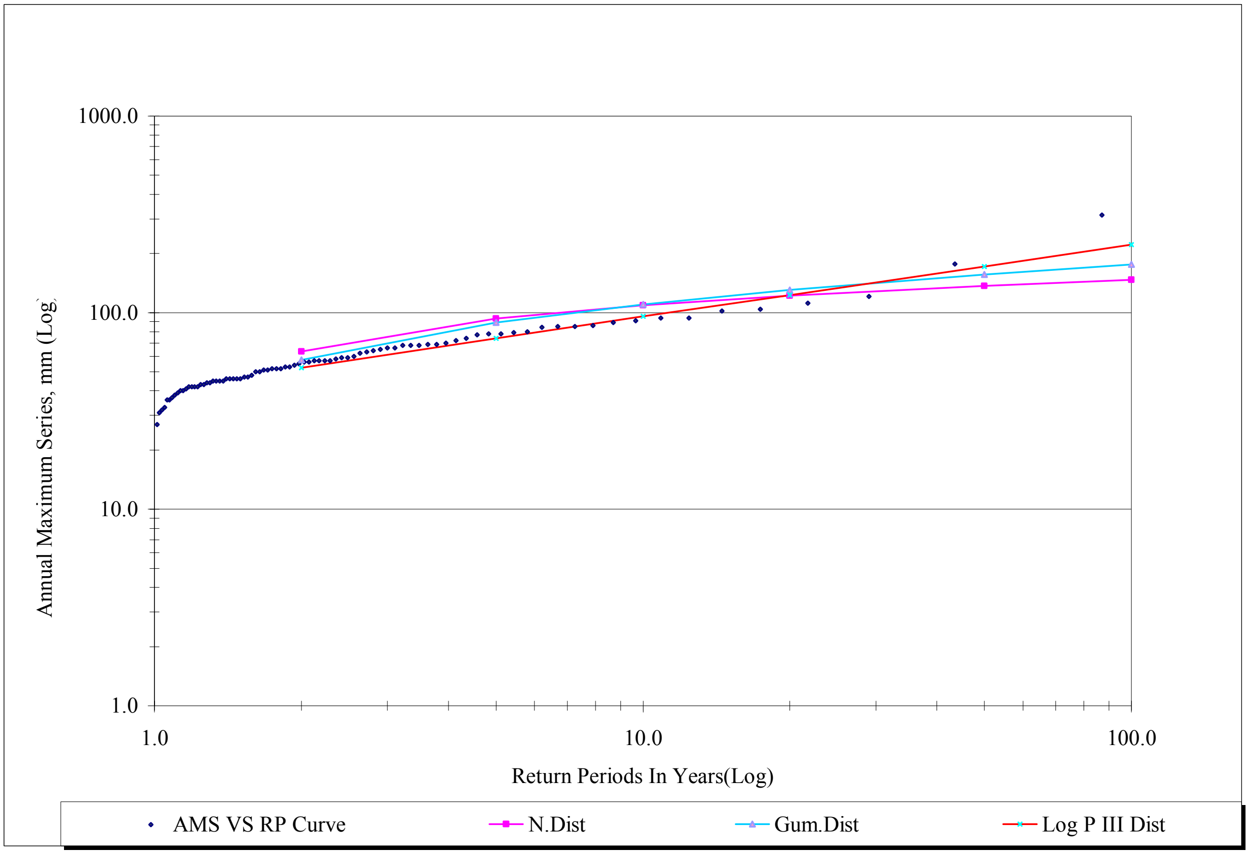

Figure 2.1 Extreme value distribution curve

3. SCS CURVE NUMBER DETERMINATION

As noted by Hawkins (1980; cited by Schulze et al., 1992), Storm flow volume and peak discharge rates are commonly required for selected design return periods. These values often need to be estimated with the use of simulation models. One such model that has became accepted and established for use on small catchments is the SCS method.

3.1 Running SCS-SA Model

The reasons to use SCS-SA method of calculating peak flow are: -

1. the equations are simple,
2. Inputs are related to physical properties of the catchment (e.g. soils; land cover properties and its " wet ness" just prior to design events),
3. The method provide uniform answers, and
4. It uses as readily available and graphical solutions renders user friendly.

3.2 SCS Storm Flow Equation

As noted by Schulze et al. (1992), storm flow is defined as the direct runoff response to a given rainfall event and it consists of both surface runoff and subsurface flows, but excludes base flow (i.e. the delayed subsurface response).

$$Q = \frac{(P - I_a)^2}{(P - I_a + S)} \text{ for..} P > I_a \tag{3.1}$$

Where: -

Q = storm flow depth,

P = daily rainfall depth (mm), usually input as a one day design rainfall for a given return period,

S = potential maximum soil water retention (mm) (Index of the wetness of the catchment soil prior to a rainfall event), and

I_a = Initial losses (abstraction) prior to the commencement of storm flow, Comprising of depression storage, interception and initial infiltration (mm)=0.1*S*.

3.3 Adjustment of Curve Number

As noted by Schulze et al. (1992), the curve number assigned to a particular soil and land cover condition is an index of stormflow response prior to consideration of catchment soil moisture conditions. At the preliminary catchment investigation initial curve numbers and the actual catchment condition in relation to land cover and humus depth are determined. Soil forms, code, texture, depth class SCS-soil grouping, land cover, cover class, land use and treatment storm potential is investigated as shown in Table 3.1 as per the SCS-SA manual specification and based on the data obtained from the preliminary site investigations.

Table 3.1 Land use and soil information of the catchment

Sub–Area		**Soil**					**Land**				**CN**
No	**%**	**Form**	**Code**	**Texture**	**Depth Class (Approximate Depth)**	**SCS Group**	**Cover**	**Cover Class**	**Practice /Treatment**	**Storm Flow Potential**	
1	8	Mispah	MS10	Sacllm	Deep (Class 3)	C	Forest	Dense (Class3)	Humus Depth 100mm/Compactness, Loose	Low	61
2	25	Hutton	HU28	Clay	Intermediate (Class 2)	A/B	Veld And Pasture	Intermediate (Class 2)	Veld & Past In Good Cond	Low	51
3	5	Mispah	MS10	Sacllm	Intermediate (Class 2)	C	Veld & Pasture	Intermediate (Class 2)	Veld &Pasture In Good Cond	Low	74
4	45	Hutton	HU28	Clay	Deep (Class 3)	A/B	Forest	Dense (Class 3)	Humus Depth 50mm/Compactness, Moderate	High	54
5	17	Clovelly	CV28	Clay	Deep (Class 3)	B	Forest	Dense (Class 3)	Humus Depth 25mm/Compactness, Moderate	High	68

Under forest condition a high storm flow potential exists when undergrowth is sparse and there is a compact shallow organic and litter layer (humus < 50mm deep), while a low storm flow potential exists when undergrowth is dense and there is loose, deep organic and litter layer (humus > 100mm deep). Since in veld or pasture condition, a low stormflow potential is associated with light grazing or plant cover > 75% of the area, the storm potential at the Cedara area for veld and pasture condition is considered to be low.

While running the model, typical soil moisture related adjustment to the curve number made in joint association method. For a median antecedent soil moisture condition based on large number of storms does not account for the effect that the event by event variation of soil moisture status can have when estimating design runoff response. The joint association method between rainfall amount and catchment moisture status may result in the second, third or even the fourth largest rainfall event of a year producing the largest flood as a result of specific moist soil conditions prevailing just prior to rainfall event (Schulze et al., 1992). Storm flow series thus derived using the daily rainfall records of the Cedara station by joint association method.

In determining the catchment response time, the Schmidt-Schulz lag equation is used since all the known variables are available to use the equation. The constraints to use the other equation were the variables such as time of concentration, summation of travel times along the flow path hydraulic length of catchment along the main channel were unknown. Further the SCS lag equation is preferably used in semi-arid and arid areas of limited vegetation cover and shallow soils (Schulze et al., 1992). Also as per the SCS-SA manual recommendation Schmidt-Schulze equation is best suited when storm flow response is comprised not only of surface runoff but also of a subsurface component, which occurs frequently in areas of high mean annual precipitation or in natural catchments with good surface cover.

The known values to run the SCS-SA model in this study project are:

- Average catchment slope=16.4%,
- Area of catchment=5.17 km^2,
- MAP=875mm,
- Average Elevation of the area =1067m, (above mean sea level),
- Latitude =29^0 32’, and
- Longitude =30^0 17’.

The model was run using catchment derived input information at first and then using known input values at the second run. The inputs are the design one day rainfall depth from the Log Pearson III type distribution curve for recurrence interval 2, 5, 10, 20 years .The SCS-SA run output is contained in the Appendix.

The output obtained from the SCS-SA model run is:

- Catchment lag time = 1.03hr,
- Adjusted curve number for 20 years = 55.6 (dimension less),
- Peak discharge for 20 yrs return period = 15.77 $m^3.s^{-1}$, and
- The inflow obtained for each distinct change of time (Δt) values are used in determining the outflow in each distinct Δt value at the spillway of the dam (11 minutes).

4. FLOOD ROUTING

As defined by Fread (1981) and Linsley *et al*. (1982), flood routing is a mathematical method for predicting the changing magnitude and celerity of a flood wave as it propagates downriver or through reservoirs. Generally, routing may be divided as channel routing and reservoir routing. The detail of the reservoir routing and channel routing is described in the following section.

4.1 Reservoir Routing

Reservoir routing is a design procedure to estimate the outflow hydrograph in the reservoir for the design of spillway and other outlet structures. Reservoir routing can be conducted using either the storage indication methods or the Muskingum method.

4.1.1 Storage indication methods

As noted by Viessman et al. (1989), for a reservoir or river reach where a natural or artificial structure forms an approximately level pool, it can be assumed that the temporary storage (S_n) is uniquely related to the head (H) of water over the crest of the control section as water stored in a reservoir is released as pipe flow through turbines or outlet works and during flood events over an emergency spillway. The discharge can be estimated by Eq 4.1 as follows:

$$Q_{pillway} = C_d \, W_s \, H^{(3/2)} \qquad \text{-for gate uncontrolled Ogee spillway.} \qquad (4.1)$$

Where

$Q_{spillway}$ = flow rate ($m^3.s^{-1}$) over an uncontrolled spillway,
C_d = coefficient of discharge, input by the user (Theoretically =3.0),
W_s = width (m) of spillway, input by the user, and
H = depth (m) of storage above spillway crest.

The storage indication method is a widely used method for both reservoir and channel routings (Wendell, 1972). Storage values for various pool elevations may be computed from topographic maps. Since storage and outflow are both depend only on water elevation above full supply level, the resulting storage–elevation curve and outflow-elevation curve can be combined to from storage–outflow graph (Smithers and Caldecott, 1995).

From the available data the storage discharge curve is developed and shown as in Figure 4.1

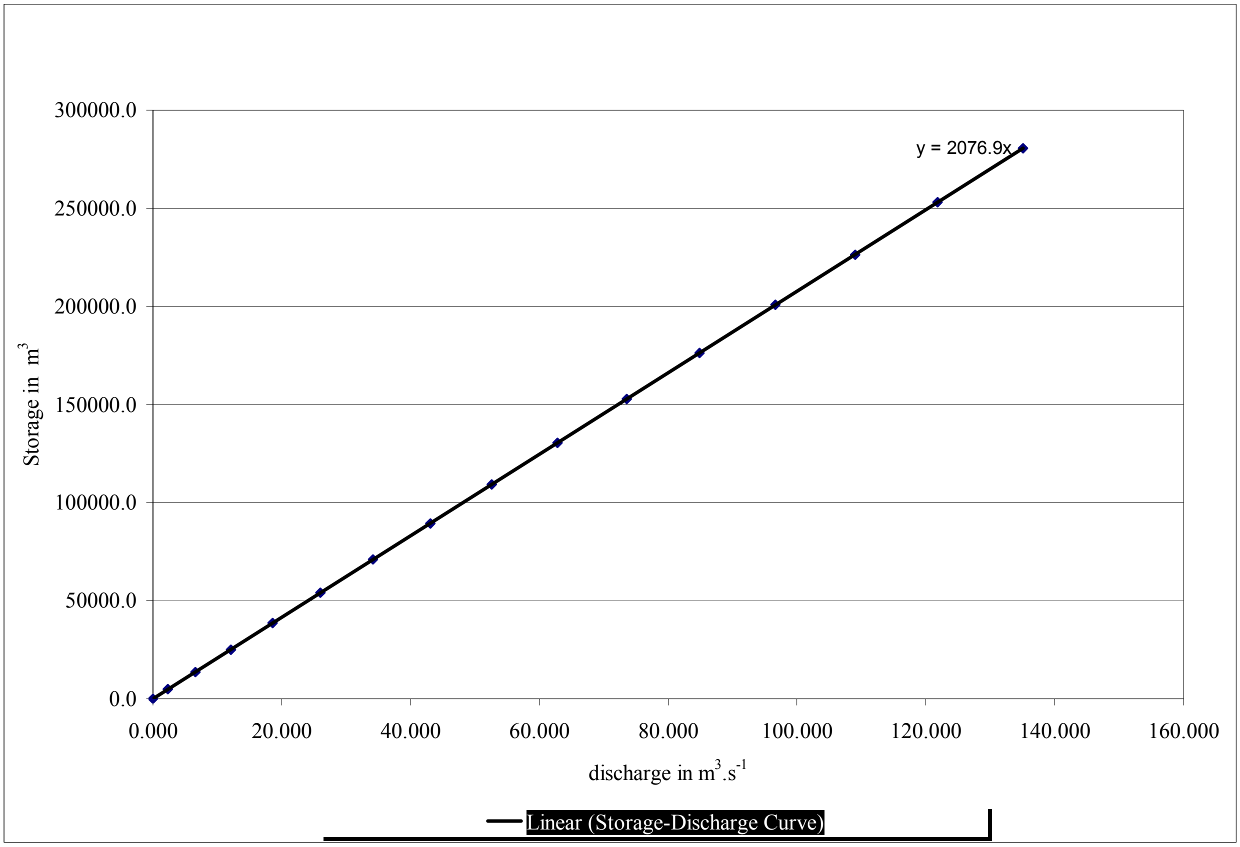

Figure 4.1 Storage discharge curve

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From Figure 4.1, the reservoir is a linear reservoir. When, the storage out flow relation ship is linear, the slope of the storage outflow curve would be K. Then the storage can be expressed as:

$$S=K*Q \tag{4.2}$$

Flow routing through a linear reservoir is accomplished by first dividing time into a series number of increments and then substituting Eq 4.2 in Eq 4.3.

$S_{(n+1)} = KQ_{(n+1)}$ in to equation

$$S_{n+1}-S_n=(I_n+I_{n+1})*\frac{dt}{2}-(Q_n+Q_{n+1})*\frac{dt}{2} \tag{4.3}$$

Equation 4.4 can be derived from Eq 4.3 as follows:

$$KQ_{(n+1)}-S_n=(I_n+I_{n+1})*\frac{dt}{2}-(Q_n+Q_{n+1})*\frac{dt}{2} \tag{4.4}$$

Then $Q_{(n+1)}$ is the only unknown variable in Eq 4.4

where

S	=	storage (m3),
I	=	inflow ($m^3.s^{-1}$),
Q	=	discharge ($m^3.s^{-1}$),
dt	=	change in time (s), and
n	=	number of series.

Since the peak time of the inflow hydrograph is 1.8 hr from the commencement of the rain Δt taken 1/6 of the peak time (0.30 hr). In order to determine the correct spillway size the spillway discharge depth curve and the depth storage curve are plotted and contained in the Appendix based on the flowing equations:

$$S=5.4*10000*D^{1.5}m^3 \tag{4.5}$$

$$Q=1.3*W*D^{1.5}m^3.s^{-1} \tag{4.6}$$

Where

S	=	storage (m^3),
Q	=	discharge ($m^3.s^{-1}$),
D	=	depth above full supply level (m), and
W	=	spillway width (m).

From the above equation D can be substituted and the equation becomes

$$S = 5.4 * 10000 * \frac{Q}{1.3W}$$

When the width is 20m a linear reservoir can be related by the formula:

$$S = 2076.9Q$$

Then the slope (K) is = 2076.9.

Calculation Steps to estimate the storage depth and discharge relationship of the reservoir

From equation, $S = 5.4 * 10000 * D^{1.5} m^3$

Sample calculation (the results are contained in Row 2, Column 4 of Table A2)

$$S = 5.4 * 10000 * 0.2^{1.5} m^3$$

$$= 4829.9 m^3$$

Considering change in depth to be 0.2m with a fixed width of 20m discharge is calculated**.**

Sample calculation (the results are contained in Row 2, Column 3 of Table A2).

$$Q = 1.3 * W * D^{1.5}$$

$$Q = 1.3 * 20 * 0.2^{1.5}$$

$$Q = 2.326 m^3.s^{-1}$$

From Table 2 (Appendix), storage discharge curve, depth discharge curve, discharge storage curve and the storage indication curve are plotted. From this the discharge storage is proved to be linearly related. Hence Eq 4.2 can be used to route the outflow and inflow hydrograph..

$$S = K Q$$

where-

S	=	storage (m^3),
Q	=	discharge ($m^3.s^{-1}$), and
K	=	slope.

The columns in Table A3 (Appendix) are described as follows:

- Col 1, specified time for each distinct inflow from SCS-SA out put,
- Col 2 inflow hydrograph out put from running the SCS-SA model,
- Col 3 change in time considered 11 minute(660 second),
- Col 4 sum of two adjoining inflows in column 2, and
- Col 5 out flow.

Outflow is calculated from Eq 4.7

$$\frac{0.5(I_n+I_{n+1})\Delta t-0.5\Delta tO_n+S_n}{(K_1+0.5\Delta t)}=O_{(n+1)} \quad (4.7)$$

where:-

I_n	=	subsequent or adjoining inflows ($m^3.s^{-1}$),
Δt	=	change in time (s),
S_n	=	storage prior to discharge Q $_{n+1,}$ (m^3) and
K_1	=	slope of the linear curve of discharge and storage.

Sample calculation

The first assumption is storage and discharge (Q_1) is zero when Δt is zero. The first row is computed from the above assumptions. Storage is used to calculate Q_2 in the next step using the above formula.

Sample calculation for (Row 14, Col 5 of Table- A3). The maximum outflow estimated as:

$$\frac{0.5(30.7)660-0.5*660*15.34+31853}{(2076.9+0.5*660)}=O_{(n+1)}$$

$$O_{(n+1)}=15.34\,m^3.s^{-1}$$

4.1.2 Muskingum method

For the Muskingum method, since the reservoir is linear reservoir the Muskingum constants are taken to be X=0, K= slope (2076.9 Sec) of the discharge vs. storage curve. Choosing the change in time to be (Δt) =660 seconds. The storage relation in the Muskingum can be described by the Eq 4.5

Then total storage is expressed as

$$S=K[XI+(1-X)Q] \tag{4.8}$$

Where-

S	=	storage (m^3),
K	=	the storage time constant for the reach and approximately the wave travel time through the reach (second),
I	=	inflow ($m^3.s^{-1}$), and
Q	=	outflow ($m^3.s^{-1}$).

Since X=0 for linear reservoirs the storage is only a function of outflow, then Eq 4.5 becomes

$$S=KQ$$

The routing can be estimated using Eq 4.9

$$Q_2= C_0I_2+C_1I_1+C_2Q_1 \tag{4.9}$$

The coefficients in Eq 4.9 can be estimated using the variables X, K, and Δt as follows:

$$C_0=\frac{-KX+0.5\Delta t}{K-KX+0.5\Delta t} \tag{4.10}$$

$$C_1=\frac{KX+0.5\Delta t}{K-KX+0.5\Delta t} \tag{4.11}$$

$$C_2 = \frac{\text{K-KX-0.5}\Delta\text{t}}{\text{K-KX+0.5}\Delta\text{t}} \tag{4.12}$$

K and Δt have the same unit (seconds) and the sum of $C_1+C_2+C_3=1$

The theoretical stability accomplished if Eq 4.13 satisfied

$$2KX \leq \Delta t < 2K(1\text{-}X) \tag{4.13}$$

$$2 * 2076.9 * 0 \leq 660 < 2 * 2076.9 * (1-0)$$

$$0 \leq 660 < 4153.8$$

Sample calculation (results are contained in Table A4 the appendixes)

(row 14 column 4)= (row 14, col. 3)*C_0

=16.20*0.137

=2.22

(row 14, column 5)= (row 13,column 3)*C_1

=17.64*0.137

=2.42

(row 14, col 6)=(row13, column 7)*C_2

=15.337*0.726

=11.13

(row14, col 7)=(row 14,col. 4)+(row 14, col 5)+(row14, col.6)

=2.22+2.42+11.13

=15.77m^3/s

Outflow (Q) from the storage indication method and Muskingum method equal to be 15.771 $m^3.s^{-1}$.

The depth of flow the specified discharge can be calculated from the relation

$$Q = 1.3 * W * D^{1.5}$$ or can be obtained from the depth-discharge relation curve.

$$16.91 = 1.3 X 20 X D^{1.5}$$

$$D = \left(\frac{16.91}{1.3 * 20}\right)^{2/3}$$

$$D = 0.75m$$

The depth of water on the spillway for the first assumption width of 20m.is estimated to be 0.75m.

5. HYDRAULIC DESIGN OF THE SPILLWAY

While designing an earth-excavated spillway the important thing to consider is the erosive ability of the spilling water and the retardance effect of the grass growing on the spillway surface. Considering erosive ability of the flowing water and flow resisting effect of the grass, the maximum permissible velocity and maximum length of the growing grass is considered in designing.

A trapezoidal section is adopted for the cannel so that side bank stability could be assured.

5.1 Stability Analyses

<u>When the grass is at the stage of establishment</u>

Failure of slopes on the sides of embankment is frequent when the water level is nearly empty (Andrew, 1997). From the site visit and the information obtained, the canal surface is erosion resistance and the soil is made of Sandy clay loam and clay. As noted by Koegelenberg et al. (1997), recommendation for sandy loam soil surface, z =1:1.5 side slope is safe. For the design of the spillway, different maximum permissible velocity and different longitudinal slopes and a fixed side slope of z = 1:5 assumed, from which the width of spillway is calculated. The one, which is best fitted with the first width assumption has been taken for the design (Table A5, Appendix).

From the calculated results, when maximum permissible velocity is $2.1 m.s^{-1}$ and slope 2 percent, the design base width becomes 20.05m, which has no significance difference with the assumed width. Hence the final design width is taken to be 20.05m.

Pennisetum Clandestinum (Kikuyu) grass is selected for the cannel surface cover as it can survive and grow well under the Cedara climate condition and able to resist the maximum permissible velocity of $2.1 m.s^{-1}$.

Sample calculation

$$A = \left(\frac{Q}{v}\right) = \frac{15.77}{2.1} = 7.51\ m^2 \tag{5.1}$$

From Figure 7.10 (Koegelenberg et al., 1997), for a trapezoidal channel with z =1.5,

W/y=69 -------------------------- $W = 69 * y$

From Table 7.2 (Koegelenberg et al., 1997),

$$W = b + 2Zy \tag{5.2}$$

$$P = b + 2y\sqrt{1 + z^2} \tag{5.3}$$

Substituting in equation 5.2

$69y = b + 3y, then, b = 66y$

$$R = \frac{A}{p}\ then, P = \frac{A}{R} = \frac{7.51}{0.355} = 21.15 \tag{5.4}$$

From Eq 5.3, $P = b + 2y\sqrt{1 + z^2}$

$$21.15 = 66y + 2y\sqrt{1 + 1.5^2}$$

21.15=69.61y

y=0.304m

W=20.05m

W=20.05m then this value is taken as the design discharge for the spillway

Taking S=2%, b=20m, depth 0.304m, design discharge =15.77m^3.s^{-1}

5.2 Design for Maximum Capacity

<u>Grass already established.</u>

Velocity is taken from Figure 7.6, (Koegelenberg et al.1997),

Table 5.1 Maximum capacity of the spill way

V , from fig 7.6

S (%)	z	b	y(m)	A(m^2)	P(m)	R(m)	V($m.s^{-1}$)	Q($m^3.s^{-1}$)
2	1.5	20.05	0.30	6.2	21.15	0.295	0.6	3.74
2	1.5	20.05	0.40	8.3	21.49	0.384	1.05	8.67
2	1.5	20.05	0.45	9.3	21.67	0.430	1.4	13.06
2	1.5	20.05	0.50	10.4	21.85	0.476	1.6	16.64

To design the maximum capacity of a spillway, the slope and z and b are taken from the part of *design for Stability*. The area, perimeter and hydraulic radius are calculated as follows:

$$A = b y + z y^2$$

$$P = b + 2 y \sqrt{1 + z^2}$$

$$R = \frac{A}{P}$$

where

A = area (m^2),

B = base width (m),

Y = depth (m),

Z = horizontal move of side slope for one move of vertical (dimensionless),

P = perimeter (m), and

R = hydraulic radius (m),

Velocity is taken from the fig 7.6 for the respective hydraulic radius when the slope is 2 percent. Then discharge could be calculated from the continuity equation Q=VA

Sample calculation

For (Row 4, Col 5) - Table A3 (Appendix)

$$A = 20.05 * 0.50 + 1.5 * 0.5^2$$
$$=10.4$$

$$P = 20.05 + 2 * 0.5\sqrt{1 + 1.5^2}$$

P = 21.85m

$$R = \frac{10.4}{21.85} = 0.476\ m$$

V= from Figure 7.6 (Koegelenberg et al., 1997) =1.6m.s^{-1}

Q=VA=1.6 *10.4=16.4m^3/s which is above the discharge required to pass. Hence the maximum depth could be taken 0.5m plus free board will be the total depth of spillway.

Hydraulic mean depth for a channel width 20.05 m should be

From Table 7.2 (Koegelenberg et al., 1997) for a trapezoidal section

$$D_m = \frac{(b + zy)y}{b + 2zy} \tag{5.6}$$

$$D_m = \frac{(20.05+1.5*0.5)0.5}{20.05+2*1.5*0.5} = \frac{10.4}{21.55} = 0.48\,m$$

So taking the maximum capacity depth 0.5m is safe for the design of the spillway.

5.3 Free Board

As explained by Koegelenberg et al. (1997), free board can be specified as 5-30 percent of the depth of flow. Hence, adding for unforeseen conditions, the depth of spillway considered to be 0.65m.

6. RESERVOIR DESIGN

6.1 Reservoir Storage Analysis

In designing a reservoir the storage analyses and the irrigation strategy must be known before fixing the dam width and height. The methodologies in estimating reservoir size and the area to be irrigated using the reserve water are described in the next subsections.

6.1.1 Estimating the required reservoir capacity

Design Criteria's

- The capacity of the dam is designed considering the irrigation system should not fail in more than once in every five years,
- Normal flow is considered only in the months in which irrigation takes place. That is in the months of November to February,
- Environmental requirements are considered at least once in every 5 years
- Overflow from the reservoir during January has to be equal to the one in two year stream flow during January prior to the construction of the reservoir, and
- For the first run the dam has assumed to have a storage capacity of 50000 m^3, surface water spread area 1.5ha, wall length 50m, and the irrigable area to be 20 ha of land.

In the first run, the catchment is considered with out dam, no irrigated crop or irrigation system included in the analysis. Normal flow and overflow from the reservoir is fixed by running ACRU model assuming no reservoir on the river.

From the formula:

$$P_e = \frac{1}{T} \tag{6.1}$$

where

P_e = probability of an event being exceeded every year

T = return period (years)

Probability of exceedence= $P_e = \frac{1}{5} * 100 = 20\%$

The non-exceedence probability = 100-20 = 80 %

The data available in the ACRU model for the Cedara catchment is only from 1978 –1991, which is 14 years data, so 0.8 * 14 years = 11.2 years of supply is required from the dam.

Normal flow

The lowest flow in the months of November - February is obtained 0.21 mm/day which is specifically related to the month of February (Annexed). This is obtained running ACRU model without reservoir for the statistics of 30 percentile that can be obtained interpolating between 33 and 20 percentile statistical analyses. Then, multiplying by catchment area $5.17 * 10^6 m^2$ and dividing by 1000 to change in to meter, the average flow is calculated to be 1085.7 $m^3.day^{-1}$. The result obtained is used in the second run as a variable to QNORM considering a reservoir in the stream.

Environmental requirement

The environmental requirements are considered once in five years of water release that should meet the two years amount of January flow prior to the construction of the dam. Hence the 80 percentile value of non exceedence probability of January has to be checked every time when the model runs and then compare with the value which is obtained from once in two years of January flow when the dam was with out reservoir.

The 50-percentile value for January when ACRU model run's with out a reservoir is 0.46mm.day^{-1} or 2378.2$m^3.day^{-1}$ for the whole of the catchment (obtained by multiplying with the catchment area). If the over flow value is greater than 0.46mm.day^{-1}, then the dam capacity fulfills the environmental requirements.

Variables to be determined by running ACRU, when the stream is with out reservoir or with no detention dams is CELRUN (total stream flow sub-catchment including upstream contribution). The stream flow which is obtained with out the reservoir is used to determine the normal flow of the stream, the South African water law has no fixed definition as to what constitutes normal flow is. However, a commonly used rule is that normal flow equals that amount of stream flow that would likely have been exceeded on 70 percent of occasions in the month of typically lowest flows. In order to obtain such a value, a simulation without the reservoir, but with a frequency analysis of the total catchment's stream flow is first undertaken to determine a value of the 70th percentile of flow exceedence in the critical month, and this value can then constitute normal flow in a second simulation with the reservoir. In the ACRU model, when the total stream flow into a reservoir on a given day is less than normal flow releases, these releases are reduced to equate those of the total inflows.

Variables to be determined by running ACRU, when the stream is with reservoir and with different irrigation strategies are PIERR, OVERFL.

PIERR is an actual irrigation water applied to the field; expressed as a percentage of net irrigation required.

OVERFL is over flow from the dam spillway

- The ACRU models run for different scenarios to determine the required dam storage capacity to irrigate 20 ha of land.
- PEIRR and OVERFL values are checked for different volume of storage and contained in Table 6.1, the statistical data contained in table and graphs are shown in the Appendix.

Table 6.1 OVERFL and PIERR data for different storage volume

Volume	Width	Length	Depth	Area	Exceedence probability	Once in 5 yr probability overflow (mm/day)
m^3	m	m	m	ha		
50000	50	600	10	1.50	100%	0.87
30000	42.2	506	8.4	1.07	99%	0.87
28000	41.2	495	8.3	1.02	98%	0.87
20000	36.9	442	7.4	0.81	95%	0.87
15000	33.5	402	6.7	0.67	90%	0.87
10000	29.3	351	5.9	0.51	81%	0.88
5000	23.2	279	4.7	0.32	65%	0.9

From Table 6.1, the minimum storage volume to irrigate 20ha of land satisfying the design criteria is 10,000m^3, butut for the unforeseen conditions and contingency 15000 m^3 is considered as a reservoir volume.

Formulas used to calculate the values of width, length and depth in Table 6.1

Assuming a triangular shape reservoir the methods to calculate the area and width and depth are used from ACRU theory.

$$S_v = \frac{W * L * D}{6} \qquad (6.2)$$

Where

S_v = total dam storage, i.e. at full supply capacity (m^3),

W = dam wall width at full supply capacity (m),

L = dam length at full supply capacity (m), and

D = dam wall depth at full supply capacity (m).

The surface area

$$A = \frac{W * L}{2} \tag{6.3}$$

Where

A = reservoir surface area in m^2

Whenever the dam width changes, the new width, length and depth could be calculated from the following relations.

$$w_w = \left(\frac{s}{S_v}\right)^{\frac{1}{3}} * W \tag{6.4}$$

$$l_w = \left(\frac{s}{S_v}\right)^{\frac{1}{3}} * L \tag{6.5}$$

$$d_w = \left(\frac{s}{S_v}\right)^{\frac{1}{3}} * D \tag{6.6}$$

Where

w_w = new width of the dam (m),

s = new dam capacity (m^3),

l_W = new length of the dam (m), and

d_W = new depth of the dam (m).

6.1.2 Irrigation strategy

The model runs several times to determine the irrigation area, applying water in DUL and varied cycle but fixed application, provided that the reservoir capacity should fulfill the above design criteria's. Changing the irrigation strategy in the model, the maximum area of maize that can be irrigated by the proposed dam capacity is investigated and commented for the maximum area of maize to be irrigated, maize yield and the profit or loss to be made from each strategy.

Maximum area of maize to be irrigated

By running the model several times with different irrigation land area, the results obtained are contained in Tables 6.2 and 6.3.

Table 6.2 The non–exceedance probability of PIERR and OVERFL variables when the irrigation strategy is changed to DUL and PAW 0.5

Run No	Area of Land (ha)	PEIRR	OVERFL/day January 80 percentile value of non exceedence in the month of January
Required	-	>Or = 80%	>Or =0.46mm
1	20	36%	0.84mm
2	15	59%	0.87mm
3	10	84%	0.90mm
4	12	73%	0.89mm
5	11	79 %	0.89mm

From Table 6.2, 10ha of land could be irrigated using the reservoir water when the irrigation strategy is changed to DUL and PAW 0.5, and fulfilling the environmental requirement and the non-exceedence probability of 80 percent.

Table 6.3 The non–exceedance probability of PIERR and OVERFL variables when the irrigation strategy is changed to varied amount but fixed cycle

Run No	Area of Land (ha)	PEIRR	OVERFL/day January 80 percentile value of non exceedence in the month of January
Required	-	>Or = 80%	>Or =0.46mm
1	10	100%	0.89mm
2	15	94%	0.87mm
3	20	88%	0.83mm
4	25	81%	0.79mm

From Table 6.3, 25 ha of land could be irrigated using the reservoir water when the irrigation strategy is changed to varied application in a fixed cycle fulfilling the environmental requirement and the non-exceedence probability.

By changing the irrigation strategy the profit is compared and contained in Table 6.4.

Table 6.4 Maize yield

No	Area (ha)	Irri. Strategy	Price/tone(Rand)	Median profit (50%) R/ha	Yield (t/ha) Mean	min	Max	σ(SD)	(Macvicar et al.)	G. profit in Ra.
1	20	Fixed cycle	540	661	7.01	6.18	8.36	0.68 t	9.73%	75,708
2	10	Refill up to DUL	540	1438	8.23	7.79	8.47	0.22t	2.62%	44,442
3	25	Varied amount in a fixed cycle	540	1005	7.51	6.71	8.61	0.72t	9.53%	101385

The profit or loss to be made from each Strategy.

From Table 6.4, the profit per hectare of median value from irrigation area using UDL strategy is higher than the other two strategies. The mean yield (t/ha) is also higher than the DUL application strategy.

The coefficient of variation measures the amount of risk relative to the expected profit, thus the higher the value of CV the greater the risk. The same is also true for the standard deviation, where the higher the standard deviation value, the higher the risk involved. For 10ha, the value of the standard deviation and the coefficient of variation are relatively low so the risk involved in this form of irrigation is in an acceptable level. Finally the gross profit for the crop is found by multiplying the mean yield to the irrigated area and then multiplying by the expected selling price per tonne of maize, which was taken as R540.00.

Recommendation

Thus with the above discussion in mind, the client is recommended to use a method of irrigation scheduling that refills the soil profile to the drained upper limit as this will give the highest yield per hectare and is also relatively low in risk.

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