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Hydrological Accurate Plan as the Main Supporting of Pejok Reservoir Design in Bojonegoro Regency, East Java Province of Indonesia

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Abstract: This study intends to carry out the hydrological analysis accurately for supporting the design of Pejok Reservoir. Pejok Reservoir is located in Bojonegoro Regency, East Java Province of Indonesia. Without the accurate plan of hydrology, it will cause some troubles and damages after a reservoir has been operated moreover Pejok is as an ungauged basin. The methodology consists of the design flood analysis and plan, flood routing through the reservoir, dependable discharge analysis and plan, and the irrigation water requirement analysis and plan. Result is hoped can support the accurate design of Pejok Reservoir, so there are not some troubles and damages after the reservoir has been operated for a long time.

Keywords: hydrology, flood, discharge, irrigation

I. INTRODUCTION

The lack of the data presentation of hydrology in the field has long become the drawback of the hydraulic structure planning [1][2]. Hydrological approaches in the watershed systems have granted great many contributions to the hydraulic structured planning. It is very difficult to understand the process of run-off thoroughly [3]. Such a conditional deficiency in particular, however has respectively placed one of the hydrological models such as the hydrograph model to be quite great utility. The hydrograph model can become the source of some important information that is necessary for the reliable of hydraulic structure [4]. A continuing case in hydrology is the peak discharges estimation due to the design purposes on the watersheds with only having the limited available data.

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One of the approaches to this case is to regionalize the flood frequency characteristics and analysis on a number of gauged watersheds, the other is to estimate the flood frequency distribution on the basis of more readily and available long rainfall records or intensity duration frequency rainfall statistics, and then to use a spesified design of storm or high flow [5].

For many years the hydrologists have been interested in the impacts of various uncertainties on the reliability and accuracy on the estimation of catchment hydrological variables such as the peak discharge and volume. The estimation of rainfall uncertainty will give an impact on the run-off simulation. The design flood values is very important for deciding the option especially when the lengths of recorded data are short and it may require the usage of various statistical distributions **[6].** Nowadays, the development of human civilization will not be free from the water function as one of the main supports. However, reminding to the water distribution on the earth which is not equally in the space as well as time [8][3], so the availability of water structures for the water control and usage is often becoming as a must. If a water source is decided to be developed for reaching a certain aim, there is needed a conception, design [8], plan, construction, and operation from any means for the control and usage [9][10]. Design which is always based on the expertise and accurate management is as a very important thing to reach the level of water usage usefulness that is nacessary on incoming period [11]. The hydrological analysis in this study is carried out for supporting the design of reservoir. The early survey for hydrological study has been carried out. Some hydrological parameters which support this study are as follow: the design flood, flood routing through the reservoir, dependable discharge, and irrigation water requirement. However, runoff as the main factor which has the great contributes in this analysis, is occurring on the uplands flows downstream in the various patterns. These patterns of flow are affected by many factors which are including spatial and temporal distribution of rainfall, hydraulics stream, watershed and channel storage, geology and soil characteristics, watershed surface and cover condition. The flood protection projects in Indonesia have been advanced as a first consideration of hydrology engineering for many years, though many fundamental problems are still remained unsolved.

The possible way to reach the clear formulation contributive to the further development of engineering works in such projects is to make basic and applied researches of the physics of stream behaviors, and especially, of the greatest importance, but most difficult, is to establish the essential features of runoff distribution by rainfall in the light of past experience and advanced knowledge in modern hydrology, while the practical approaches of the runoff distribution like the unit hydrograph method, distribution graph method, and so on, have been progressed. The graph of flow (rate versus time) at a stream section is called as a hydrograph, of which no two are exactly alike. The computation methods for analyzing the peak rates of flow are based upon the empirical relations, starting with the Rational Method for the peak discharge in the 19^{th} century, progressing to the unit hydrograph in the 1930s, and then to more recent use of the dimensionless or index hydrographs. The empirical relations are as the simple elements from which a hydrograph may be made as complex as needed. Difficulties with hydrograph development lie in the precise estimation of runoff from rainfall and the determination of flow paths [12][13]. As stream flow records are somewhat limited for most locations, it is necessary to relate the runoff to rainfall. Knowing the rainfall rates, a function to convert rainfall to runoff is required. Unit hydrograph (UH) is such a tool.

A variety of basic and applied research programs on runoff distribution by rainfall and stream behaviors by several approaches of scientific and engineering methods, unit hydrograph method, computation of runoff by the characteristic curves, and flood routing by electronic analog computer, has been promoted for recent several years. It is very difficult to understand the process of run-off thoroughly [3]. Ideally, every watershed has its own particular unit hydrograph. Realizing that the Synthetic Unit Hydrograph models have been researched and developed in the areas which the watersheds are far different than the ones applied, they therefore quite often come up with inaccurate results, which affects the design of the hydro structure. So that was needed to calibrate some parameters for the ideal cases [14]. In many parts of the world, the rainfall and runoff data are seldom adequate to analyse a unit hydrograph of a basin or watershed. This situation is common in Indonesia due to the lack of gauging stations along most of the rivers and streams. Generally, basic stream flow and rainfall data are not available for planning and designing the water management facilities and the other hydraulic structures in the undeveloped watershed [15][16]. However, many techniques have been evolved that allow the generation of the synthetic unit hydrograph that includes the Snyder method (researched in USA), the Nakayasu method (researched in Japan), and the Gama-I method which is found in Indonesia based on the research of 300 flood events in Java Island. However, this study intends to present the accurate hydrological analysis in ungauged watershed for supporting the design of reservoir.

II. MATERILAS AND METHODS

Design flood which is used for the need of design is as follow: a) Q_{PMF} as the reference of capacity design and the width of small dam weir; and b) Q_{100} as the reference of stilling basin structure. To obtain the value of design flood, there is used the analysis of synthetic unit hydrograph [17][18]. There are some methods for this analysis such as the Nakayasu (research in Japan), Snyder (research in USA), and Gama I (research in Indonesia) [19][20]. The four methods are available in the Indonesian National Standard.

The estimation of dependable discharge is carried out by using the rainfall data transformation to the discharge data. The transformation uses the methods of F.J. Mock and then the result is compared with the frequency analysis of observed data in the Kerjo Weir which is as the first weir in the downstream of the Pejok Reservoir Plan. As the design basic, it will be used the discharge with the certain dependable or probability and it will be remained based on the reaching evaluation result of the cropping area and water availability on the last period.

The irrigation water requirement in each location is designed by the agro-climate analysis method. The basic of cropping pattern determination is based on the reference which is ratified by the Regent of Bojonegoro as the team leader of regional irrigation. The planned cropping pattern is regarding to the general crop custom of the local society such as paddy-paddy-second crop. However, for the reaching of maximal crop intensity, it can modify the existing cropping pattern such as paddy-second crop-second crop regarding to .

pay attention to the balance between the discharge availability and the cropping water requirement.

A. General Illustration of Research Location

Based on the study map, there is obtained the parameters in the Pejok Reservoir watershed as follow:

Name of the river : Kali Brangkal/ Kali Kerjo/ Kali Semarmendem
Area of the watershed : 47.496 km^2
Length of the main river : 17.909 km
The river slope : 0.02

Area of the watershed is analyzed from the control point of the structure design where the determination of the structure design location point on the earth map uses the aids of GPS (Global Positioning System).

B. Data Collecting

The rainfall stations which are used in the hydrological analysis on the design of Pejok Reservoir are the Station of Panjang and Blubuk. Analysis of the regional average rainfall will use the method of Thiessen Polygon.

The recorded daily rainfall data in each of the rainfall station is 22 years. Before being used for the next analysis, the data have to be tested for the completion and consistency [21]. To analyze the dependable discharge is used the method of F.J. Mock at first, to transform the rainfall to discharge data, and then it is compared to the dependable discharge which is obtained from the recorded run-off discharge on the Kerjo Weir where is located in the downstream of reservoir plan. Figure 1 presents the catchment area of Pejok Reservoir

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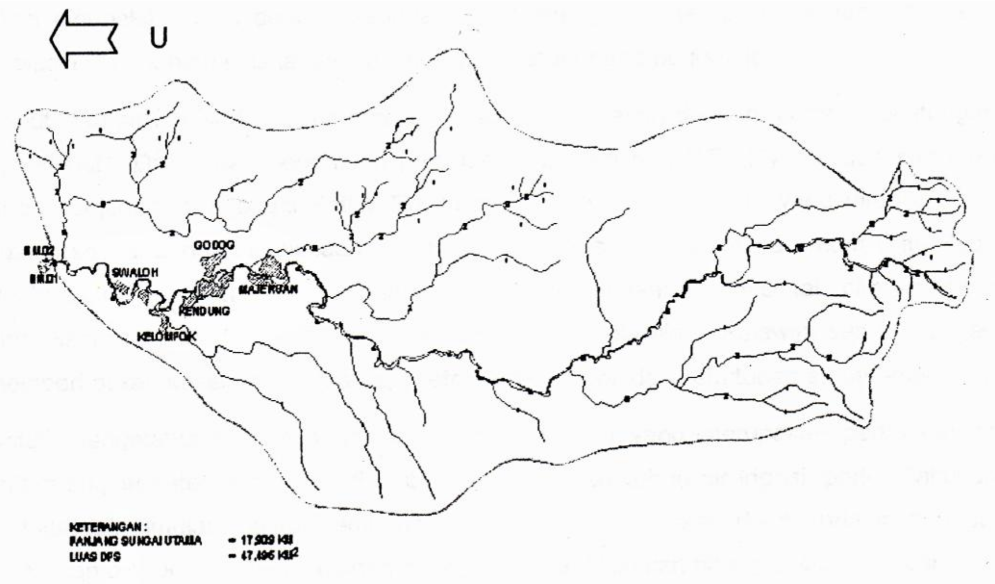

Fig.1 Catchment area of Pejok reservoir

C. Synthetic Unit Hydrograoh of Nakayasu

Flood peak discharge [20]:

$$Qp = \frac{c \cdot A \cdot Ro}{3{,}6(0{,}3Tp + T0{,}3)} \quad (1)$$

Where

Qp = Q max, as the flood peak discharge ($m^3.s^{-1}$}
c = coefficient of flow (= 1)
A = area of watershed, until outlet (km^2)
Ro = unit rainfall (mm)
Tp = time lag from the beginning of rain until flood peak (hour)
$T_{0,3}$ = time which is needed by the discharge recession from the peak discharge until 30% of peak discharge (hour)

Formula of the unit hydrograph as follow [20]:

Limb curve

$0 \leq t < Tp$

$$Qt = Q\ max \left(\frac{t}{Tp}\right)^{2,4} \quad (2)$$

Recession line

a. $Tp \leq t < (Tp + T_{0,3})$

$$Qt = Q\ max * 0{,}3^{\frac{t-Tp}{T0,3}} \quad (3)$$

b. $(Tp + T_{0,3}) \leq t < (Tp + T_{0,3} + 1{,}5\ T_{0,3})$

$$Qt = Q\ max * 0{,}3^{\frac{t-Tp+0,5T0,3}{1,5T0,3}} \quad (4)$$

c. $t \geq (Tp + T_{0,3} + 1{,}5\ T_{0,3})$

$$Qt = Q\ max * 0{,}3^{\frac{t-Tp+1,5T0,3}{2T0,3}} \quad (5)$$

D. Method of F.J. Mock

Dr. F.J. Mock (1973) has introduced a simple model of monthly water balance simulation for streamflow which consists of the rainfall data, evaporation, and the hydrological characteristic of the watershed. The potential evaporation and actual evapotranspiration are analysed by the formula as follow [20]:

$$Ea = ETo - \Delta E \rightarrow (Ea = Et) \quad (6)$$

$$\Delta E = ETo \times (m/20) \times (18 - n) \rightarrow (E = \Delta E) \quad (7)$$

Where:

Ea = actual evapotranspiration ($mm.day^{-1}$)
Et = limited evapotranspiration ($mm.day^{-1}$)
ETo = potential evaporation by Penman method ($mm.day^{-1}$)
M = percentage of the area which is not covered by the crop estimated from the land use map
m = 0 for area with the dense forest
m = 0 for area with the second forest on the end of rainy seasonand it is adding 10% every the next dry month.
m = 10 – 40 % for erosioned area
m = 30 – 50 % for prepared the agricultural area (like paddy field, field)
n = the number of rainy day in a month

The rainfall which reaches the soil surface can be formulated as follow [20]:

$$Ds = P - Et \quad (8)$$

Where:

Ds = rainfall which the reaches soil surface ($mm.day^{-1}$)
P = rainfall ($mm.day^{-1}$)
Et = limited evapotranspiration ($mm.day^{-1}$)

The formula for groundwater is as follow [20]:

$$Vn = k . V_{n-1} + ½ (1 + k) . In \quad (9)$$

$$DVn = Vn - V_{n-1} \quad (10)$$

Where:

Vn = volume of groundwater, n^{th} month
V_{n-1} = volume of groundwater, $(n-1)^{th}$ month

K = $qt.qo^{-1}$ = catchment area recession factor
qt = groundwater flow at the t^{th} period (t^{th} month)
qo = groundwater flow in the early month
In = infiltration on n^{th} month
DV_{n-1}= volume change of groundwater flow

III. RESULTS AND DISCUSSION

A. Analysis of Design Flood

Based on the data collecting, due to the ungagged watershed, it indicates that there is no more observed period of flooding that is ever happened in the study location. As the solution to the limited data, the estimation of design flood is analyzed based on the recorded observed rainfall [14] due to the some rainfall stations. Analysis of the synthetic unit hydrograph is suggested to be used the Nakayasu, Gama I, and Snyder. These three methods are presented in the Indonesian National Standard (SNI). The selection of the method is based on the consideration of historical flood which has ever happened due to the observation on the measurement points at the AWLR (Automatic Water Level Recorder) as well as the weirs along the river. Table- I present the recapitulation of design flood in the Pejok reservoir.

Table- I Recapitulation of design flood in the Pejok reservoir

Return period (year)	Design flood ($m^3.s^{-1}$)		
	Nakayasu	Gama I	Snyder
2	121.800	98.067	99.167
5	144.569	116.314	117.618
10	160.252	128.932	130.377
20	175.791	141.434	143.010
25	182.715	147.005	148.653
50	197.300	158.812	160.592
100	211.861	170.454	172.365
200	226.326	182.094	184.135
500	247.400	199.047	201.279
1000	258.654	208.102	210.435
PMF	620.425	499.167	504.764

Source: own study

B. The Selection of Design Flood Analysis

The synthetic unit hydrograph is intended to find out the nature of the watershed responses against the precipitation input data, whereby it can become as the supportive warning systems to the areas that are vulnerable to flooding. In addition it can resume up the hydrograph data availability that are previously vacant due to the operational problem to the Automatic Water Level Record (AWLR)

The synthetic unit hydrograph is intended to find out the nature of the watershed responses against the precipitation input data, whereby it can become as the supportive warning systems to the areas that are vulnerable to flooding. In addition it can resume up the hydrograph data availability that are previously vacant due to the operational problem to the Automatic Water Level Record watershed.

To attend the historical flood data, the analysis method of design flood which is closer with the field condition is the Nakayasu method by using the frequency analysis [5] of the Log Pearson III. Ideally every watershed has its own particular unit hydrograph. If the physical and hydrological condition in general is quite homogeneous, it would be quite possible to create a synthetic unit hydrograph model that resembles the ones made up by the previous researchers [1]. However, the historical flood which was happened on 8th February 2003 is equal with the Q_{10}. Attending the historical flooded data and the design flood of Pascal Reservoir, the analysis method of design flood which is closer to the field condition on the design of Pejok Reservoir (the watershed area is 47.496 km^2) is the Nakayasu method with the Q_{100} is 620.425 $m^3.s^{-1}$. Table- II presents the analysis on design flood of the Probable Maximum Precipitation by using the Nakayasu method and the hydrograph is presented as in the Fig 2.

Table- II Analysis of design flood hydrograph (Probable Maximum Flood-

T (hour)	U(t,1)	Q ($m^3.s^{-1}$) due to the netto rainfall (mm)						Base flow ($m^3.s^{-1}$)	Qflood ($m^3.s^{-1}$)
		144.581	37.580	26.361	20.986	17.722	15.491		
0.000	0.000	0.000	-	-	-	-	-	0.000	0.000
1.000	0.385	55.664	0.000	-	-	-	-	0.000	55.664
2.000	2.032	293.789	14.468	0.000	-	-	-	0.000	308.257
3.000	3.693	533.938	76.363	10.149	0.000	-	-	0.000	620.449

4.000	2.174	314.319	138.783	53.566	8.080	0.000	-	0.000	514.747
5.000	1.260	182.172	81.699	97.351	42.644	6.823	0.000	0.000	410.689
6.000	0.899	129.978	47.351	57.309	77.501	36.011	5.964	0.000	354.114
7.000	0.631	91.231	33.784	33.215	45.624	65.447	31.478	0.000	300.779
8.000	0.443	64.049	23.713	23.699	26.442	38.528	57.208	0.000	233.639
9.000	0.328	47.423	16.648	16.634	18.866	22.330	33.677	0.000	155.578
10.000	0.252	36.434	12.326	11.678	13.242	15.932	19.519	0.000	109.131
11.000	0.193	27.904	9.470	8.646	9.297	11.183	13.926	0.000	80.426
12.000	0.148	21.398	7.253	6.643	6.883	7.851	9.775	0.000	59.803
13.000	0.114	16.482	5.562	5.088	5.288	5.813	6.863	0.000	45.096
14.000	0.087	12.579	4.284	3.901	4.050	4.466	5.081	0.000	34.361
15.000	0.067	9.687	3.269	3.005	3.106	3.420	3.904	0.000	26.392
16.000	0.051	7.374	2.518	2.293	2.392	2.623	2.990	0.000	20.190
17.000	0.039	5.639	1.917	1.766	1.826	2.020	2.293	0.000	15.460
18.000	0.030	4.337	1.466	1.344	1.406	1.542	1.766	0.000	11.861
19.000	0.023	3.325	1.127	1.028	1.070	1.187	1.348	0.000	9.086
30.000	0.018	2.602	0.864	0.791	0.818	0.904	1.038	0.000	7.018
21.000	0.014	2.024	0.676	0.606	0.630	0.691	0.790	0.000	5.418
22.000	0.010	1.446	0.526	0.474	0.483	0.532	0.604	0.000	4.065
23.000	0.008	1.157	0.376	0.369	0.378	0.408	0.465	0.000	3.152
24.000	0.006	0.867	0.301	0.264	0.294	0.319	0.356	0.000	2.401

Source: own study

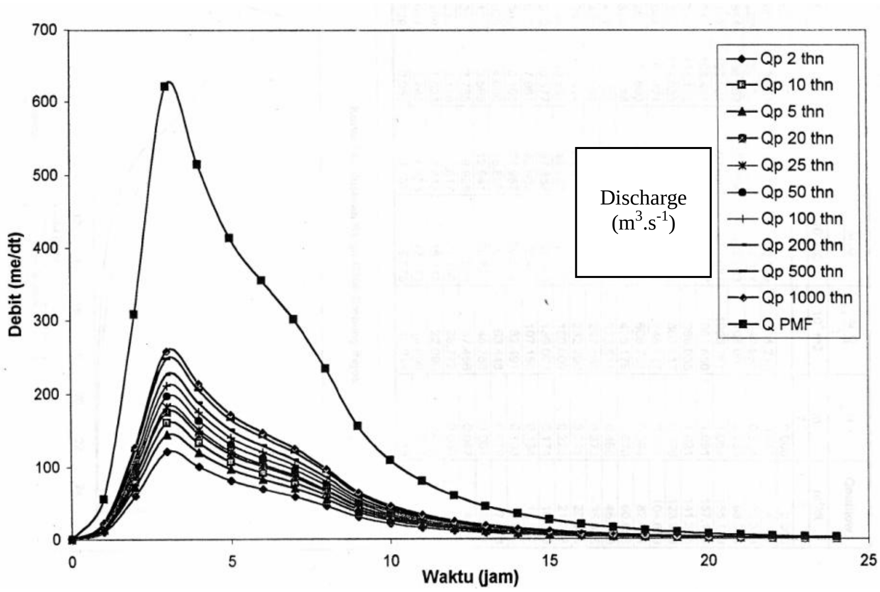

Fig. 2 Design flood hydrograph of PMF by using Nakayasu method

C. Flood Routing Through Reservoir

The data that are needed for analyzing the flood routing through reservoir are as follow: 1) the relation between the reservoir volume and elevation; 2) the relation between the outflow and water level elevation in the reservoir; the outflow and storage; 3) the inflow hydrograph, $I = I_{(t)}$; and 4) the initial value of storage (S), the inflow (I), and the outflow (Q) when t = 0. The initial value of storage is determined when there is the condition of normal water level. For the flood control reservoir, the storage value is regarded with the operation rule. For the dam safety, it is suggested that the intake gate is in the closed condition. To obtain the most economic value among the combination of the weir width and dam mercu height, the reservoir rooting is needed to be trialed for some weir widths. The analysis result of flood routing throgh the reservoir is presented as in the Table- III

Tabel- III. The analysis result of flood routing through the reservoir

Design alternative of weir height (m)	Parameter	Design flood ($m^3.s^{-1}$)		
		Q_{100}	Q_{1000}	Q_{PMF}
6	Width of weir (m)	65		
	Inflow ($m^3.s^{-1}$)	211.861	258.654	520.425
	Outflow ($m^3.s^{-1}$)	157.196	193.016	455.616
	Water head (H) over weir	1.091	1.247	2.259
	Reduction of flood (%)	25.80	25.38	25.09
	Deceleration of flood time (hour)	2	2	1
7	Width of weir (m)	45		
	Inflow ($m^3.s^{-1}$)	211.861	258.654	520.425
	Outflow ($m^3.s^{-1}$)	124.202	153.258	377.179
	Water head (H) over weir	1.188	1.371	2.516
	Reduction of flood (%)	41.38	40.76	39.21
	Deceleration of flood time (hour)	3	3	3
8	Width of weir (m)	40		
	Inflow ($m^3.s^{-1}$)	211.861	258.654	620.425
	Outflow ($m^3.s^{-1}$)	102.384	127.580	326.807
	Water head (H) over weir	1.131	1.310	2.472
	Reduction of flood (%)	53.17	51.84	47.4
	Deceleration of flood time (hour)	4	4	4
9	Width of weir (m)	35		
	Inflow ($m^3.s^{-1}$)	211.861	258.654	520.425
	Outflow ($m^3.s^{-1}$)	84.233	106.304	282.165
	Water head (H) over weir	1.087	1.267	2.449
	Reduction of flood (%)	60.24	58.90	54.52
	Deceleration of flood time (hour)	5	5	5
10	Width of weir (m)	30		
	Inflow ($m^3.s^{-1}$)	211.861	258.654	620.425
	Outflow ($m^3.s^{-1}$)	67.838	86.891	234.076
	Water head (H) over weir	1.046	1.227	2.455
	Reduction of flood (%)	67.58	86.41	62.11
	Deceleration of flood time (hour)	5	5	5

Source: own study

There are presented 5 alternatives of the weir width, so the selection of the width type is depended on the field condition and mainly for the suitable design flood. Certainly the design flood is selected due to the design of weir dimension regarding to the need and it has also to consider the flooding which has ever been happened.

D. Analysis of Dependable Discharge

To analyze the dependable discharge of the Pejok Reservoir, it is used the recorded run-off discharge on the weir where is located in the downstream of reservoir plan such as the Kerjo Weir. The record of the discharge data is during 7 years such as from 1997 until 2003. Therefore, the analysis of dependable discharge on the Pejok Reservoir is carried out by using the generated data from the recorded data. Table- IV presents the dependable discharge of Kerjo Weir where is located in the Pejok watershed.

Table- IV. Dependable discharge of Kerjo Weir due to the some dependable condition

Month	Dependable discharge ($m^3.s^{=1}$)				
	97.3%	80%	75.3%	50.7%	26.02%
	dry	dependable	sufficient	moderate	wet
January	0.000	11.810	14.052	24.932	38.519
February	0.000	9.209	12.444	28.148	47.479
March	0.000	16.133	19.898	38.165	60.967
April	0.000	12.153	16.636	38.396	65.571
May	0.000	4.058	5.179	10.821	17.416
June	0.000	3.602	5.886	16.973	30.819
July	0.000	1.222	2.551	9.001	17.055
August	0.000	0.000	0.001	0.008	0.017
September	0.000	0.000	0.000	0.000	0.000
October	0.000	0.134	9.436	54.577	110.950
November	0.000	2.977	4.039	9.195	15.635
December	1.371	12.550	14.504	23.985	32.825

Source: own study

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There are 4 methods which can be used for analyzing the dependable discharge depended on the data availability and the need of analysis. The method of minimum average discharge is used for the watershed with the fluctuation between the maximum and the minimum discharge is not too high from year to year [23] and the water need is relatively constant along the year. The method of flow characteristic is suitable for the watershed with the fluctuation between the maximum and minimum discharge is relatively high from year to year, the water need is not constant along the year, and the available data is long enough. The method of basic year is suitable only for supplying the irrigation. The method of basic month is almost similar with the method of flow characteristic which analyses the dependable discharge of every month. However, in this study there is used the method of flow characteristic. It is due to the consideration for planning the operation reservoir. Therefore, the dependable discharge is presented in the five discharge conditions such as the dry (97.3%), dependable (80%), sufficient (75.3&), moderate (50.7%), and wet (26.02%) discharge condition.

E.Cropping Water Requirement

One of the factors that influence the cropping water requirement is percolation which is in the range of 2-3 mm.day^{-1}. The percolation value, cropping coefficient of paddy, and the irrigation price of field cropping is presented in the Table- V, VI, and VII. However, the cropping water requirement is presented in the Table- VIII.

Table- V. Value of percolation

No	Type of soil	Percolation (mm.day^{-1})
1	Light	2.0
2	Medium	2.0-3.0
3	Medium-heavy	4.0
4	Heavy	5.0

Table- VI. Coefficient of paddy cropping

No	Month	Local varieties	Superior varieties
1	0.5	1.1	1.1
2	1.0	1.1	1.1
3	1.5	1.1	1.05
4	2.0	1.1	1.05
5	2.5	1.1	0.95
6	3.0	1.05	0
7	3.5	0.95	-
8	4.0	0	-

Source: own study

Table- VII. Irrigation value of field cropping

No	Item	Initial	The increasing which can be reached
1	Main irrigation network	0.75	0.8
2	Tertiary swath	0.65	0.75
3	Total	0.5	0.6

Source: own study

Analysis of the irrigation water requirement is presented for 3 types of the cropping patterns as follow: 1) paddy-paddy-second crop; 2) paddy-second crop-second crop; and 3) paddy-second crop and paddy-second crop. Analysis of the potential evaporation is presented in the Table- VIII and the cropping water requirement is presented in the Table- IX

.Table- VIII. Evaporation of Pejok reservoir

Month	Eo (mm.day^{-1})	Eo reservoir (mm.day^{-1})
January	3.21	2.47
February	3.11	2.39
March	3.09	2.38
April	3.37	2.6
May	3.67	2.83
June	4.04	3.11
July	4.13	3.18
August	5.21	4.01
September	5.85	4.5
October	5.71	4.4
November	4.35	3.35
December	3.76	2.9

Source: own study

Table- IX. Irrigation water requirement

Period		Water requirement ($l.s^{-1}.ha^{-1}$)		
		Paddy	Second crop	Intake
November	1	0.57	0.00	0.57
	2	0.54	0.00	0.54
	3	0.84	0.00	0.84
December	1	0.00	0.00	0.00
	2	1.51	0.00	1.51
	3	1.58	0.00	1.58
January	1	1.03	0.00	1.03
	2	0.79	0.00	0.79
	3	0.21	0.00	0.21
February	1	0.70	0.00	0.70
	2	0.13	0.00	0.13
	3	0.45	0.00	0.45
March	1	0.33	0.00	0.33
	2	0.67	0.00	0.67
	3	1.01	0.00	1.01
April	1	1.19	0.00	1.19
	2	1.24	0.00	1.24
	3	0.56	0.00	0.56
May	1	1.13	0.00	1.13
	2	0.67	0.00	0.67
	3	1.04	0.00	1.04
June	1	0.51	0.03	0.54
	2	0.12	0.00	0.12
	3	0.00	0.30	0.30
July	1	0.00	0.50	0.50
	2	0.00	0.62	0.62
	3	0.00	0.72	0.72
August	1	0.00	0.83	0.83
	2	0.00	0.83	0.83
	3	0.00	0.83	0.83
September	1	0.00	0.43	0.43
	2	0.00	0.10	0.10
	3	0.00	0.00	0.00
October	1	0.00	0.00	0.00
	2	0.00	0.00	0.00
	3	0.00	0.00	0.00

Source: own study

The cropping pattern is regarding to the society custom by the consideration of the available water supply. The cropping pattern consists of the type of crop and the schedule of crop. The Table 9 presents the water requirement for paddy and second crop on all of the crop schedulle.

IV. CONCLUSION

The hydrology analysis as above is carried out in the Pejok Reservoir as an ungauged watershed. Based on the problem formulation, the hydrological accurate plan as the main supporting for the design of Pejok reservoir produces the result as follow:1) Based on the results obtained so far for the ungauged watershed, it can be seen that the generation of unit hydrograph through synthetic methods has been found useful and effective. The evaluation of the storm hydrograph flows obtained in this study from the three methods employed have indicated that there are significant differences in the methods. Though all the three methods employed have been found useful in one way or the other, but the Nakayasu method has been considered distinct and more important since it utilizes most major unit hydrograph characteristics and watershed characteristics in the generation of unit hydrographs. This method is found simple, requiring only an easy determination of watershed and watershed characteristics. The result is very close with the field condition (historical data of flooding) such as the Q_{PMF} = 620.425 $m^3.s^{=1}$. For the other return periods (2, 5, 30, 35, 50, 100, 200, 500, and 1000 year) can be seen in Table- I above; 2) The flood routing is analyzed based on the result of design flood analysis with the Nakayasu method.

Analysis result of the flood rooting through reservoir with the height alternative of weir is 6 m, Q_{PMF}= 620.425 $m^3.s^{=1}$ and outflow= 465.616 $m^3.s^{-1}$. However, for the height alternative of weir is 6, 7, 8, 9, and 10 m, each for $Q_{100,}$ Q_{1000}, and Q_{PMF} can be seen as in Table- III above. The selected type of width is depended on the field condition and mainly for the suitable design flood; 3) The dependable discharge is analyzed by using the flow characteristic method. It is due to the need of analyzing the reservoir operation rule. Therefore, the dependable discharge is presented in the five discharge conditions such as the dry (97.3%), dependable (80%), sufficient (75.3&), moderate (50.7%), and wet (26.02%) discharge conditions. The maximum dependable discharge of 80% is on the March such as 16.133 $m^3.s^{-1}$. However for the other months in one year each for the condition of dry, dependable of 80%, sufficient, moderate, and wet discharge condition can be seen in the Table- IV above, and 4) The cropping water requirement is analyzed by considering the society custom. The water requirement is analysed for the paddy and second crop for every ten days schedule. The result can be seen in the Table- VIII above.

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