A STUDY ON THE DESIGN OF RAW WATER CONSERVATION RESERVOIR IN THE EDUCATION AREA, NORTHERN BANDAR LAMPUNG

Ofik Taupik Purwadi[1*], Dyah Indriana Kusumastuti[1], Sugeng Triyono[2] and Slamet Budi Yuwono[2]
[1]Department of Civil Engineering, Faculty of Engineering, University of Lampung, Bandar Lampung, Indonesia
[2]Department of Agriculture Science, Faculty of Agriculture, University of Lampung, Bandar Lampung, Indonesia
*Email: ofik.taupik@eng.unila.ac.id

ABSTRACT *– The increasing demand for raw water in educational areas, especially in the north of Bandar Lampung, has led to potential water crises and environmental problems. One of the early symptoms is the emergence of inundation points to new floods every year due to natural infiltration ponds that have not been able to accommodate excess rainwater and overflow. This study aims to design a conservation reservoir from a natural pond located in the educational area of the University of Lampung. The data used are rainfall data from rain stations with a data range of 10 years, soil type data, cohesion value (c), internal shear angle (φ), specific gravity (Gs), water content (w), and soil density (γ). The reservoir design method includes hydrological and stability analysis. The ideal flood discharge analysis results according to the conditions at the research site are between 0.0847 - 0.7051 (m^3/s) for 24 hours for a repetition time of 10 years. The storage volume calculation with the current water level data is +110.00 m at 34777.5620 m^3. Analysis of flood discharge on the selected spillway building is the Ogee type with a repetition time of 10 years, and the outflow discharge occurs at the 8^{th} hour with a discharge of 0.2704 m^3/s. At the same time, the results of the analysis of the stability of the reservoir against landslides in three conditions obtained the values of Fs = 4.3087, 3.3797, and 1.7924 for the newly built condition, the condition of the reservoir in the event of a sudden decline, and the condition of the reservoir being fully filled, respectively. These results indicate that the reservoir design can be realized under stable conditions with Fs above the safe threshold (> 1.2).*

Keywords: Conservation, Northern Bandar Lampung, Reservoir, Water

I. INTRODUCTION

The need for groundwater in educational and urban areas in the north of Bandar Lampung is increasing every year [1]. This situation has triggered a significant increase in groundwater extraction in areas that do not yet have a water piping system, including the educational area in northern Bandar Lampung. With the increasing development of residential and industrial areas, groundwater extraction activities have also increased [2–4]. One of the impacts of this increase in the extraction process is a decrease in the groundwater table. If these impacts are not handled properly, there is the potential for more significant impacts such as subsidence that can trigger flood points, to clean water crises, including seawater intrusion in coastal areas [5, 6]. The groundwater crisis significantly impacts the community's economy [7, 8].

Massive groundwater extraction activities have shown some adverse impacts in some areas of Bandar Lampung, especially on the coast. Several densely populated sub-districts on the southern coast of Bandar Lampung have experienced a clean water crisis, especially those sourced from groundwater. This condition has been caused by seawater intrusion due to groundwater pumping for decades [1]. In addition, other impacts such as subsidence, have also been identified in this area [9]. There is a reasonably large groundwater basin in the northern part of the city [10]. However, there is also an increase in groundwater extraction in this area, including in the education area. Without good monitoring and handling activities, the groundwater crisis may continue in the northern part of Bandar Lampung City. Moreover, there is currently an increase in groundwater extraction from deep bore wells.

Control, utilization, and management of water quality must be carried out as early as possible and continuously. This action is needed to anticipate over-exploitation of water, including controlling water pollution and potential flooding. The current condition is that there are several ponds in the educational area of the University of Lampung. This pool holds water and stores water reserves in the dry season. However, during the rainy season, the pool has not been able to accommodate the excess rainwater and overflows. The area has the lowest elevation, and the water reservoir in the pond is small, so excess rainwater is not accommodated. In fact, rainwater is a source of water that can be used to sustain the availability of water in the area of the University of Lampung. So the effort to conserve raw water at the University of Lampung is to plan the construction of dams in these ponds. In addition, it is also to accommodate excess rainwater so that there is no overflow in the pool that can damage the surrounding buildings.

This study uses geohydrological data to analyze and design raw water conservation reservoirs in educational areas, especially the Lampung University. Hydrological analysis in the planning of the reservoir construction is carried out to determine the magnitude of the planned flood discharge that affects the volume of water reservoirs and the design of spillway buildings. The results of the data analysis are used as input data in planning the construction of the reservoir so that it is appropriate and follows the conditions in the field. This reservoir planning includes reservoir type planning, technical planning, reservoir foundation planning, and reservoir stability analysis.

II. MATERIAL AND METHODS

The research area is located in the northern part of the city of Bandar Lampung. It is an educational area growing rapidly accompanied by the development of residential and tourist areas [11, 12]. The study area for conservation pond design is focused on the educational area of the University of Lampung (Unila). Water demand in Unila increases over time. If water use increases, the water supply in water sources will decrease and even run out, so an effort is needed to keep water available.

Morphologically, the research area is dominated by plains with an altitude of about 90 – 150 meters above sea level. Only the area west of the study site is a hilly morphological area that is part of the foot of Mount Betung. Rainfall in this area reaches 2000 mm/year [13]. This high rainfall causes the water supply into the aquifer to be quite good, especially in unconfined aquifers. However, in the last 10 years, heavy rainfall has also resulted in runoff which causes inundation points to floods. This condition is in line with the increase in land-use change into built-up land and the increase in groundwater extraction.

In the educational area of the University of Lampung, there are several natural ponds. This pool holds water and stores water reserves in the dry season. However, during the rainy season, the pool has not been able to accommodate the excess rainwater and overflows. This is because the area has the lowest elevation and the water reservoir in the pond is small, so excess rainwater is not accommodated. Rainwater is a source of water that can be used to sustain the availability of water in the area of the University of Lampung. One of the most potential locations for constructing a conservation pond is shown in Figure 1.

Figure 1. Location of potential raw water conservation reservoir development in Unila.

In planning this conservation reservoir, the required data include topographic data, hydrological data in the form of rainfall data from rain stations with a data range of 10 years, soil data in the form of soil type data, cohesion value (c), internal shear angle (φ), specific gravity (Gs), moisture content (w) and soil density (γ). These data are secondary data because they are obtained from previous research and archival data from related agencies. Data analysis was carried out, including topographic, hydrological, soil, and water quality. The results of data analysis on topographic data are the capacity or storage volume. For hydrological data, the analysis results are in the form of planned flood discharge (maximum discharge).

Dam stability is a construction calculation to determine the reservoir's dimensions to withstand the loads and forces acting on the reservoir under any circumstances. Checking the stability of the reservoir body is carried out, including checking for overturning, shearing, and failure of the carrying capacity.

Hydrological analysis in the planning of the reservoir construction is carried out to determine the magnitude of the planned flood discharge that affects the volume of water reservoirs and the design of spillway buildings. The calculation of the planned flood discharge in this study was carried out using daily rainfall data due to the unavailability of discharge data. The daily rain data will be processed into planned rainfall, which is then analyzed into planned flood discharge. Rainfall data were obtained from the nearest rain station around the reservoir construction planning site. The rainfall data were obtained from the closest monitoring station to the research area, namely the Polinela Station, as shown in Table 1.

To determine the design of flood discharge used, the rational method. This rational method is oriented to calculating peak discharge which has the following equation form.

$$\mathrm{Q}_T = 0{,}278 \times C \times I \times A \tag{1}$$

where Q_T is the peak discharge (m^3/s) for repetition time T year, and I is the average rainfall intensity (mm/hour). Then A and C are the catchment area (m^2) and the runoff coefficient, respectively.

One of the functions of the reservoir is flood control at a river drainage location. If there is a flood, the water level in the reservoir will rise little by little until it reaches the threshold of the spillway. Some of the flood water will pass through the spillway, while the rest will cause the water level in the reservoir to rise. The floodwater surface elevation in the reservoir can be calculated using flood routing. The following is the basic formula for calculating load routing according to Soedibyo [14]:

$$\frac{I_1+I_2}{2}.t - \frac{O_1+O_2}{2}.t = S_2 - S_1 \tag{2}$$

where I is *inflow* (m^3/s) and O is *outflow* (m^3/s), while $S_2 - S_1$ is the additional water stored in the reservoir (m), while t is the period of time (s).

The results of the data analysis are used as input data in planning the construction of the reservoir so that it is appropriate and follows the conditions in the field. This reservoir planning includes reservoir type planning, technical planning, reservoir foundation planning, and reservoir stability analysis. The reservoir stability analysis includes an analysis of the forces acting on the

Table 1. Maximum daily rainfall data (mm) at Polinela Station

NO	YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AGU	SEP	OCT	NOV	DES	MAX
1	2009	75.2	20.5	41.7	52.7	37.2	63.4	29.7	26.6	8.5	35.2	30.9	31.3	75.2
2	2010	34.1	32.3	76.9	47	27.6	46.2	81.5	40.3	35.6	60.04	69.5	73.9	81.5
3	2011	52.1	17.8	23.6	68.9	11.1	28.1	25.2	11.3	2.9	58.8	50.3	41.5	68.9
4	2012	34.6	66.8	14.5	72.7	64	30.6	11.1	10.7	0	106.3	41.2	57	106.3
5	2013	54.7	107.9	70	93.8	40.8	25.4	93.5	15.5	16.7	55.6	26.2	59.3	107.9
6	2014	35.4	51.2	102.8	26.4	33.8	41.2	30.9	62.9	0	23.4	23.4	58.3	102.8
7	2015	54.7	75.7	79.8	91.6	36.6	19.1	64.8	26.6	44.2	59.8	23.5	60.1	91.6
8	2016	83.4	87.5	50.3	74.2	31.5	42.3	50.1	18.5	18.5	57.9	28.7	50.6	87.5
9	2017	39.2	159.6	52.5	50.1	10.1	36	12.9	17.1	40.5	18.2	74.4	52.3	159.6
10	2018	33.6	53.2	65.3	55.6	-	-	-	-	-	-	-	-	65.3

Table 2. Results of calculation of rainfall intensity and planned discharge

Hour	Recurrence Time (Years)						I (mm/hr)	Q (m^3/s)
	2 yr	5 yr	10 yr	25 yr	50 yr	100 yr		
1	30.6593	38.9764	53.8313	53.8313	60.8110	68.3634	53.8313	0.7051
2	19.3097	24.5479	33.9037	33.9037	38.2997	43.0563	33.9037	0.4441
3	14.7340	18.7310	25.8699	25.8699	29.2242	32.8536	25.8699	0.3388
4	12.1615	15.4606	21.3531	21.3531	24.1217	27.1175	21.3531	0.2797
5	10.4797	13.3226	18.4002	18.4002	20.7859	23.3674	18.4002	0.2410
6	9.2797	11.7971	16.2932	16.2932	18.4058	20.6917	16.2932	0.2134
7	8.3730	10.6444	14.7012	14.7012	16.6074	18.6699	14.7012	0.1926
8	7.6595	9.7373	13.4485	13.4485	15.1922	17.0790	13.4485	0.1761
9	7.0808	9.0016	12.4324	12.4324	14.0444	15.7886	12.4324	0.1628
10	6.6003	8.3908	11.5887	11.5887	13.0913	14.7171	11.5887	0.1518
11	6.1937	7.8739	10.8749	10.8749	12.2850	13.8107	10.8749	0.1424
12	5.8445	7.4300	10.2617	10.2617	11.5923	13.0320	10.2617	0.1344
13	5.5407	7.0437	9.7282	9.7282	10.9896	12.3544	9.7282	0.1274
14	5.2734	6.7040	9.2591	9.2591	10.4596	11.7586	9.2591	0.1213
15	5.0363	6.4025	8.8426	8.8426	9.9892	11.2298	8.8426	0.1158
16	4.8241	6.1327	8.4701	8.4701	9.5683	10.7566	8.4701	0.1109
17	4.6329	5.8897	8.1344	8.1344	9.1891	10.3303	8.1344	0.1065
18	4.4596	5.6694	7.8301	7.8301	8.8454	9.9439	7.8301	0.1026
19	4.3016	5.4686	7.5528	7.5528	8.5321	9.5917	7.5528	0.0989
20	4.1570	5.2846	7.2987	7.2987	8.2451	9.2691	7.2987	0.0956
21	4.0238	5.1154	7.0650	7.0650	7.9811	8.9723	7.0650	0.0925
22	3.9009	4.9591	6.8492	6.8492	7.7372	8.6982	6.8492	0.0897
23	3.7869	4.8143	6.6491	6.6491	7.5112	8.4441	6.6491	0.0871
24	3.6810	4.6795	6.4630	6.4630	7.3010	8.2077	6.4630	0.0847

reservoir horizontally and vertically and overturning and shear analysis. The output or result of the planning stage of this reservoir construction is in the form of a plan drawing.

III. RESULT AND DISCUSSIONS

3.1. The results of the calculation of rainfall and planned discharge

Maximum daily rainfall is the maximum daily rainfall data taken every year, which will be used to calculate the design flood discharge. The frequency analysis results using the Pearson Log method and rainfall intensity with the Mononobe equation obtained the results of calculations for a period of 1 to 24 hours for up to 100 years (Table 2). The calculation of the planned discharge is carried out using the Rational method because the watershed area is only 0.07291 km^2, less than 12 km^2 [15]. Determination of the repetition time following the Regulation of the Minister of Public Works No. 12 of 2014, which is if the watershed area is <10 ha, a 2-year repetition period is used, but to maintain the safety of the reservoir, there will be changes in land use in the watershed within a 2-year period, which will cause an increase in flood discharge. Therefore, a 10-year repetition period is used as time ideal repetition.

3.2. Reservoir Volume

The current reservoir volume of the reservoir can be calculated using topographic data (Figure 2). The topographic data used in this study has a height difference (contour) of 0.5 m. Each elevation has a different area. The volume can be calculated by constraining two successive contours.

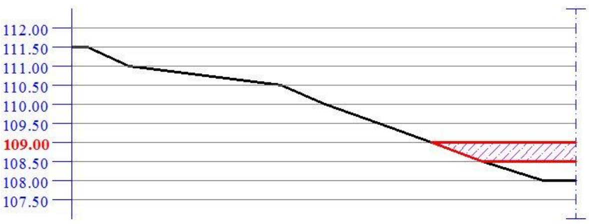

Figure 2. Topographic profile and existing water level

The design reservoir volume is calculated according to the height or depth, area of the reservoir, and the slope of the reservoir body. Basically, the planned reservoir volume calculation is not much different from the cur-

Table 3. Calculation of the planned reservoir storage volume

Elevation (m)	Z (m)	Total Area (m^2)	Storage Volume (m^3)
107.00	0.0	724.695	0.000
107.50	0.5	935.809	3158.142
108.00	0.5	1167.737	3475.879
108.50	0.5	1420.720	3806.111
109.00	0.5	1693.649	4147.046
109.50	0.5	1985.065	4496.923
110.00	0.5	2293.316	4857.300
Total			**34777.56**

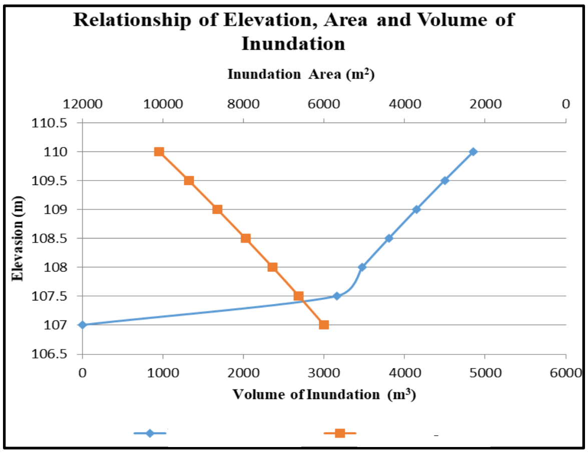

Figure 3. Graph of the relationship between elevation, area, and volume of inundation

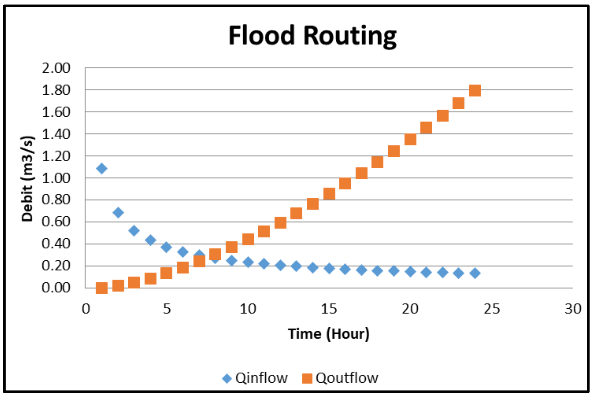

Figure 4. Flood Routing graph of 10 years of repetition.

-rent reservoir volume. The planned reservoir storage volume calculation can be seen in Table 3. Based on field measurements, the current water level is +110.00 m with a storage volume of 34777.5620 m^3. The relationship between elevation, area and volume of inundation is shown in Figure 3.

3.3. Flood Routing

In reservoir planning, the planned normal water level is +110.00 m with a reservoir volume of 34777.5620 m^3. The flood discharge that overflows on the spillway building is selected for the flood discharge with a repetition time of 10 years with the type of overflow building of the Ogee type. Based on the calculations that have been made, it is obtained that the outflow discharge occurs at the 8th hour with a discharge of 0.2704 m^3/s, as shown in Figure 4.

3.4 Reservoir Planning

The reservoir type in the conservation pond planning in the selected research area is a single pond type. The selection of this type is based on its purpose, namely to accommodate excess rainwater. In addition, based on its location in the flow of water, this reservoir is a reservoir in the flow of water because it does not stem the flow of water. Then based on the type of construction, this dam is planned as an embankment reservoir. It is a rain-fed and spring-loaded reservoir based on the water source. In planning the reservoir body, several parts need to be considered, namely the width of the peak, the slope of the embankment slope, the height of the guard, and the height of the reservoir body. Based on the Guidance for Making Reservoir [16], The width of the peak in the reservoir is determined based on the type of body of the reservoir and the height of the reservoir. For this reason, with an embankment reservoir body type and a height of fewer than 5 meters, the peak width of the reservoir is 2 meters. Meanwhile, the reservoir body is designed with a homogeneous fill and a height of fewer than 10 m. The slope is 1:3 with a guard height of 0.5 m, and the height of the reservoir body is 4 m.

The spillway is designed with the ogee type to drain the excess water when the flood discharge occurs. This spillway is designed from masonry/concrete. The following is the technical data needed in planning shown in Table 4.

Table 4. Technical data in the planning of the reservoir

Technical Data	Value
Overflow debit	0.5788 m^3/dt
Overflow width	1.5 m
Water level upstream of the threshold	0.4055 m
Planned flood elevation	+110.00 m
Elevation of the bottom of the reservoir	+107.00 m

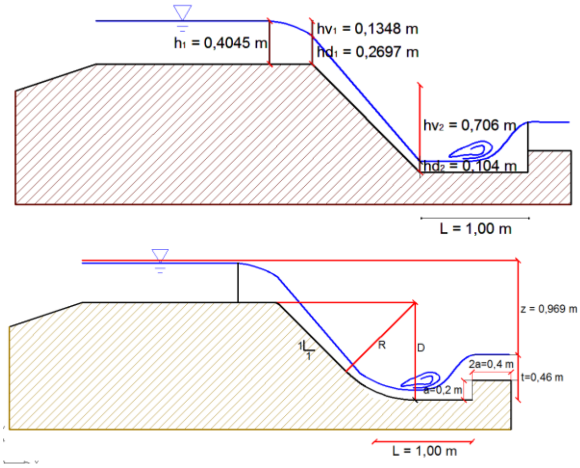

Figure 5. Design of spillway

The flow control channel regulates the flow of water so that the flow speed is low, but the water discharge is large. The flow control channel is designed by taking into account the overflow outflow so that the velocity of the water in the flow control channel can be obtained. In analyzing the launch channel, Bernoulli's equation is used, it is obtained that the depth of the water plus the difference in height between the energy line and the water surface is the same with the assumption of 0.969 m with a speed of 3.72 m/s so that the Froude number (Fr) is 3.69. The design of the spillway is shown in Figure 5.

3.5. Stability Analysis

In this study, the body of the pond that is reviewed is the body of the pond around the overflow building because the elevation behind the pond body is lower. To overcome the failure in the slope stability of the reservoir, analysis was carried out on three conditions: the condition of the newly completed construction, the condition of the reservoir in the event of a sudden decline, and the condition of the reservoir being fully developed filled. The scheme for calculating the stability of the embankment slope in a newly constructed condition is carried out using the circular sliding plane wedge method, as shown in Figure 6.

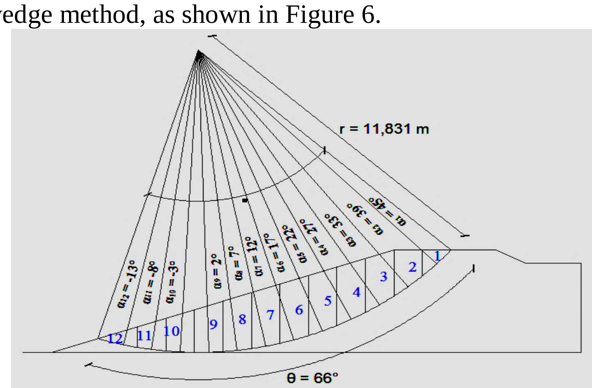

Figure 6. Stability analysis schematic using the newly constructed circular sliding plane wedge method.

Table 5. Stability of the reservoir against landslides when the reservoir has just been built

Slice	A (m^2)	W (t/m)	α (°)	sin α	cos α	T = W . Sin α (t/m)	N = W . cos α (t/m)	Ne = e . W . Sin α (t/m)	Te = e . W . Cos α (t/m)	l	C . l
1	0.5148	0.5251	45	0.7071	0.7071	0.3713	0.3713	0.0622	0.0622		
2	1.4132	1.4415	39	0.6293	0.7771	0.9071	1.1202	0.1521	0.1878		
3	1.9643	2.0036	33	0.5446	0.8387	1.0912	1.6803	0.1829	0.2817		
4	2.2044	2.2485	27	0.4540	0.8910	1.0208	2.0034	0.1711	0.3359		
5	2.3309	2.3775	22	0.3746	0.9272	0.8906	2.2044	0.1493	0.3695		
6	2.3377	2.3845	17	0.2924	0.9563	0.6971	2.2803	0.1169	0.3823		
7	2.2549	2.3000	12	0.2079	0.9781	0.4782	2.2497	0.0802	0.3771	13.6214	28.0601
8	2.0765	2.1180	7	0.1219	0.9925	0.2581	2.1022	0.0433	0.3524		
9	1.8225	1.8590	2	0.0349	0.9994	0.0649	1.8578	0.0109	0.3114		
10	1.4725	1.5020	-3	-0.0523	0.9986	-0.0786	1.4999	-0.0132	0.2514		
11	1.0428	1.0637	-8	-0.1392	0.9903	-0.1480	1.0533	-0.0248	0.1766		
12	0.5749	0.5864	-13	-0.2250	0.9744	-0.1319	0.5714	-0.0221	0.0958		
Total						**5.4209**	**18.9943**	**0.9088**	**3.1842**		**28.0601**

The parameter used for the value of γd (t/m^3) is 1.02, while depression high and $U = u.b/\cos\alpha$ is 0. The calculation results of reservoir stability in Figure 6 are shown in Table 5. Based on the values in Table 5, the safety factor (Fs) value is obtained as shown in the equation below.

$$F_s = \frac{\sum\{C\ .\ l + (N - U - N_e)\tan\phi\}}{\sum((T + T_e)} = \frac{28.0601 + (18.9943 - 0 - 0.9088)\tan 26.5^o}{5.4209 + 3.1842} = 4.3087 \quad (3)$$

With an Fs value of 4.3087, which has a value greater than 1.2, the slope stability of the design of the reservoir when it is just finished is considered very stable. The next step is to calculate the stability using the circular sliding plane wedge method if there is a sudden drop in water level, as shown in Figure 7.

According to the sudden decrease stability analysis scheme, the parameter used for the value of γd (t/m^3) increased from the 2nd to the 12th incision (Figure 7). while depression high and $U = u.b/\cos\alpha$ are not worth 0 at incisions 3 – 5 according to the red line in the area affected by the decline. The results of the calculation of the stability of the reservoir in Figure 7 are shown in Table 6.

Based on the values in Table 6, the safety factor (Fs) value is obtained as shown in the equation below.

$$F_s = \frac{\sum\{C\ .\ l + (N - U - N_e)\tan\phi\}}{\sum((T + T_e)} = \frac{28.0601 + (27.7304 - 4.4574 - 1.1594)\tan 26.5^o}{6.9161 + 4.6487} = 3.3797 \quad (4)$$

With an Fs value of 3.3797, which has a value greater than 1.2, the slope stability of the design of the reservoir when it is just finished is considered very stable. The next step is to calculate the stability using the circular sliding plane wedge method if the reservoir is full, as shown in Figure 8.

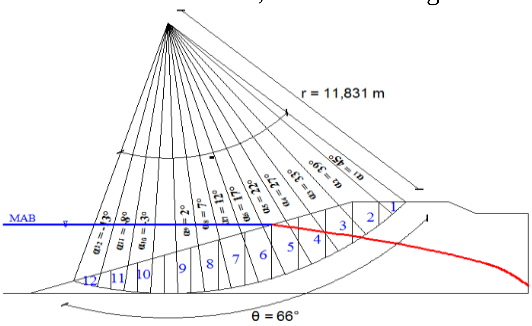

Figure 7. Schematic stability analysis using circular sliding plane wedge method in case of sudden settlement

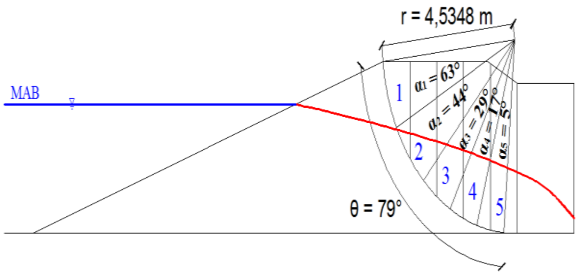

Figure 8. Schematic stability analysis using the circular slide section method in full reservoir conditions.

The parameter used for the value of γd (t/m^3) is only at slices 1 - 5, according to the stability analysis scheme in full reservoir conditions in Figure 8. According to the line, depression high and U=u.b/cos α are not 0 at incisions 2-5. red on areas affected by full pond conditions. The results of the calculation of the stability of the reservoir in Figure 8 are shown in Table 7. Based on the values in Table 7, the safety factor (Fs) value is obtained as shown in the equation below.

$$F_s = \frac{\sum\{C\ .\ l + (N - U - N_e)\tan\phi\}}{\sum((T + T_e)} = \frac{28.0601 + (14.1336 - 6.3064 - 1.1197)\tan 26.5^o}{6.6789 + 2.3694} = 1.7924 \quad (5)$$

With an Fs value of 1.7924, which has a value greater than 1.2, the slope stability of the design of the reservoir when it is just finished is considered to be relatively very stable. Based on the three results of stability analysis in each reservoir condition, it can be estimated that the conservation pond design is relatively stable and can be applied.

Table 6. Stability of the reservoir against landslides in case of sudden subsidence

Slice	A (m^2)	γ_d (t/m^3)	W (t/m)	α (^{O})	T = W . Sin α (t/m)	N = W . cos α (t/m)	Ne = e . W . Sin α (t/m)	Te = e . W . Cos α (t/m)	depression high (m)	U = u.b/cos α	l	C . l
1	0.514	1.02	0.525	45	0.371	0.371	0.0622	0.062	0	0	13.621	28.0601
2	1.360	1.02	1.387	39	0.873	1.078	0.1464	0.180	0	0		
	0.051	1.446	0.074	39	0.046	0.057	0.0078	0.009	0	0		
3	1.238	1.02	1.262	33	0.687	1.059	0.1153	0.177	0.77	0.844		
	0.726	1.446	1.050	33	0.572	0.881	0.0959	0.147	0	0		
4	0.731	1.02	0.745	27	0.338	0.664	0.0568	0.111	1.5	1.548		
	1.473	1.446	2.131	27	0.967	1.899	0.1622	0.318	0	0		
5	0.239	1.02	0.244	22	0.091	0.226	0.0154	0.038	2.08	2.063		
	2.091	1.446	3.025	22	1.133	2.804	0.1900	0.470	0	0		
6	2.337	1.641	3.836	17	1.121	3.668	0.1880	0.615	0	0		
7	2.254	1.641	3.700	12	0.769	3.619	0.1290	0.606	0	0		
8	2.076	1.641	3.407	7	0.415	3.382	0.0696	0.567	0	0		
9	1.822	1.641	2.991	2	0.104	2.989	0.0175	0.501	0	0		
10	1.472	1.641	2.416	-3	-0.126	2.413	-0.021	0.404	0	0		
11	1.0428	1.6412	1.7114	-8	-0.238	1.694	-0.039	0.284	0	0		
12	0.5749	1.6412	0.9435	-13	-0.212	0.919	-0.035	0.154	0	0		
Total					**6.9161**	**27.7304**	**1.1594**	**4.6487**		**4.4574**		**28.0601**

Table 7. Stability of the reservoir against landslides when the reservoir is full

Slice	A (m^2)	γ_d (t/m^3)	W (t/m)	α (^{O})	T = W . Sin α (t/m)	N = W . cos α (t/m)	Ne = e . W . Sin α (t/m)	Te = e . W . Cos α (t/m)	depression high (m)	U = u.b/cos α	l	C . l
1	1.153	1.02	1.176	63	1.048	0.534	0.175	0.089	0	0	13.621	28.0601
	0.127	1.446	0.184	63	0.164	0.083	0.027	0.014	0	0		
2	1.623	1.02	1.655	44	1.150	1.191	0.192	0.199	0	0		
	0.892	1.446	1.291	44	0.897	0.929	0.150	0.155	1.08	1.381		
3	1.806	1.02	1.843	29	0.893	1.612	0.149	0.270	0	0		
	1.339	1.446	1.937	29	0.939	1.694	0.157	0.284	1.58	1.662		
4	2.008	1.02	2.049	17	0.599	1.959	0.100	0.328	0	0		
	1.492	1.446	2.159	17	0.631	2.064	0.105	0.346	1.76	1.693		
5	1.996	1.02	2.036	5	0.177	2.029	0.029	0.340	0	0		
	1.412	1.446	2.043	5	0.178	2.035	0.029	0.341	1.7	1.570		
Total					**6.6789**	**14.1336**	**1.1197**	**2.3694**		**6.3064**		**28.0601**

IV. CONCLUSION

This study succeeded in calculating the ideal design flood discharge according to the conditions at the research location, which is between 0.0847 - 0.7051 (m^3/s) for 24 hours for a repetition time of 10 years. The storage volume calculation with the current water level data is +110.00 m. It is obtained at 34777.5620 m^3. The flood discharge that overflows on the spillway building is selected for the flood discharge with a repetition time of 10 years with the type of overflow building of the Ogee type. Based on the calculations that have been made, it is found that the outflow discharge occurs at the 8th hour with a discharge of 0.2704 m^3/s.

Based on the results of the analysis of the stability of the reservoir against landslides in three conditions, namely the condition of newly built, the condition of the reservoir in the event of a sudden decline, and the condition of the reservoir being filled, significant results were obtained all Fs values were above the safe threshold (> 1.2). These results confirm that the study on the design of reservoirs for raw water conservation in educational areas north of Bandar Lampung, particularly in the Unila area, can be realized in a stable condition. With the application of surface water conservation methods as part of rainwater harvesting techniques using ponds, it is hoped to overcome the increasing demand for raw water in the research area. In addition, the conservation pond area is also expected to be a location for flood retention and can recharge unconfined aquifers in the area.

V. REFERENCES

[1] Rustadi, Darmawan, I. G. B., Haerudin, N., Suharno, and Setiawan, A., "Geophysical approach for assessment of seawater intrusion in the coastal aquifer of Bandar Lampung, Indonesia," *IOP Conf. Series: Materials Science and Engineering*, **1173**: 1–8(2021).

[2] Foster, T., Brozović, N., and Speir, C., "The buffer value of groundwater when well yield is limited," *Journal of Hydrology*, **547**: (2017).

[3] Elshall, A. S., Castilla-Rho, J., El-Kadi, A. I., Holley, C., Mutongwizo, T., Sinclair, D., and Ye, M., "Sustainability of Groundwater," *Reference Module in Earth Systems and Environmental Sciences*, (2021).

[4] Akter, A. and Ahmed, S., "Modeling of groundwater level changes in an urban area," *Sustainable Water Resources Management*, **7**(1): (2021).

[5] Alfarrah, N. and Walraevens, K., "Groundwater overexploitation and seawater intrusion in coastal areas of arid and semi-arid regions," *Water*, **10**(2): 143(2018).

[6] Das, S., Maity, P. K., and Das, R., "Remedial Measures for Saline Water Ingression in Coastal Aquifers of South West Bengal in India," *MOJ Ecology & Environmental Sciences*, **3**(1): 16–24(2018).

[7] Lan, Y., Jin, M., Yan, C., and Zou, Y., "Schemes of groundwater exploitation for emergency water supply and their environmental impacts on Jiujiang City, China," *Environmental Earth Sciences*, **73**(5): (2015).

[8] Halder, S., Roy, M. B., and Roy, P. K., "Analysis of groundwater level trend and groundwater drought using Standard Groundwater Level Index: a case study of an eastern river basin of West Bengal, India," *SN Applied Sciences*, **2**(3): (2020).

[9] Zaenudin, A., Darmawan, I. G. B., Armijon, Minardi, S., and Haerudin, N., "Land subsidence analysis in Bandar Lampung City based on InSAR," *Journal of Physics: Conference Series*, **1080**(1): 1–7(2018).

[10] Zaenudin, A., Darmawan, I. G. B., and Laimeheriwa, G., “2D Gravity Qualitative Modeling to Identify Bedrock and Volcanic Rocks in South Lampung Region,” *Jurnal Ilmiah Pendidikan Fisika Al-Biruni*, **9**(1): 161–175(2020).

[11] Ali, S., Maharani, L., and Untari, D. T., "Development of religious tourism in Bandar Lampung, Indonesia," *African Journal of Hospitality, Tourism and Leisure*, **8**(5): (2019).

[12] May, L. F., Abdurrahman, A., Hariri, H., Sowiyah, S., and Rahman, B., “The Influence of Principal Managerial Competence on Teacher Performance at Schools in Bandar Lampung,” *Tadris: Jurnal Keguruan dan Ilmu Tarbiyah*, **5**(1): (2020).

[13] Kusumastuty, N. A. E., Manik, T. K., and Timotiwu, P. B., "Identification of temperature and rainfall pattern in Bandar Lampung and the 2020-2049 projection," *IOP Conference Series: Earth and Environmental Science*, **739**: (2021).

[14] Soedibyo, *Teknik Bendungan*. Banjarnegara: Pradnya Paramita, (2003).

[15] Triatmodjo, B., *Hidrologi Terapan*. Yogyakarta: Beta Offset, (2008).

[16] Sumadilaga, D. H., Irianto, E. W., Indrawan, D., Karma, A., Uzaer, M., Sofiyuddin, H. A., Kasiro, I., Adidarma, W. K., Rusli, B. S., Nugroho, C. L., and Sunarto, *Panduan Pembuatan Embung*. Jakarta : Kementerian Pekerjaan Umum dan Perumahan Rakyat, (2018).