

Hydrological Study on the Rehabilitation of Aghor Pond for Refugee Livestock in the Commune of El Meghve, Mauritania

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Abstract

This study presents a hydrological analysis for the rehabilitation of Aghor Pond in Mauritania to support refugee livestock. Using topographic, soil, and hydrological data, the authors designed hydraulic structures to increase the pond's storage capacity to 315,178 m³ and extend its water retention. The study determines key parameters such as project flood discharge and sediment deposition to ensure the dam's stability and operational effectiveness. The findings support a plan to mitigate water scarcity for local and refugee communities.

Keywords

Livestock, Pond, Flood Discharge, Refugee, Mitigation

1. Introduction

Located at the crossroads of North Africa and the Sahel-Saharan zone, Mauritania is a vast semi-arid country facing significant vulnerability in terms of natural resources and access to basic social services—particularly water supply and healthcare. These challenges are further compounded by the impacts of migration flows and trade disruptions caused by conflicts in neighboring countries. In this unstable regional context, Mauritania stands out as an island of relative stability.

The Mauritanian economy largely depends on livestock farming, a sector that accounts for approximately 10.1% of GDP, nearly 70% of the added value in the rural economy, and employs around 10% of the active population [1]. However, this sector remains highly susceptible to climatic hazards, particularly drought and high evaporation rates. These conditions, combined with the high concentration of livestock, place increasing pressure on existing water points, which

are no longer sufficient to meet the growing needs of communities and their animals. As a result, the effectiveness of herders' efforts is significantly reduced.

In response to this situation, various alternatives are being implemented to improve water access, particularly through hydraulic infrastructure designed to enhance water resource mobilization. This study falls within that framework, aiming to sustainably improve the living conditions of refugees through the rehabilitation of the Aghor Pond.

2. Materials and Methods

2.1. Topographic Surveys

Topographic surveys in planimetry were conducted in the project area. The collected data were exported to Covadis software, where longitudinal and cross-sectional profiles were generated. Analysis of the point data sets allowed for the determination of the normal water level and the pond basin elevation.

2.2. Soil Studies

A soil reconnaissance survey was carried out using a method that combines the ring method with complementary manual techniques. The collected samples allowed for the identification of soil structure through visual inspection and laboratory testing.

2.3. Hydrological Studies

Hydrological studies were a critical component of this project and were conducted through the following steps:

2.3.1. Rainfall Data Analysis

Rainfall data were obtained from the meteorological station in Néma—the closest station to the Aghor area. The dataset includes thirty-four (34) years of records, covering the period from 1990 to 2023.

Using Excel and Hyfran Plus software, we performed analysis, verification, and adjustment of annual average rainfall and maximum daily rainfall data. These were modeled using both the Gaussian and Gumbel distribution laws.

2.3.2. Watershed Description

Based on a Digital Elevation Model (DEM) of the intervention area, derived from Mauritania's geographic data and processed using ArcGIS, the hydrographic network and watershed boundaries were delineated.

1) Geometric Characteristics: Perimeter and Area of the Watershed

These parameters were automatically calculated using ArcGIS. According to the classification by Rodier in FAO Bulletin No. 54 [2], watersheds are grouped into four (4) size categories based on surface area.

2) Equivalent Rectangle

The equivalent rectangle provides a simplified representation for comparing

watersheds in terms of how their geometry affects runoff behavior [3]-[5]. Its dimensions are calculated using the following formula:

$$L_{eq} = \frac{P + \sqrt{P^2 - 16 \times S}}{4} \quad (1)$$

$$l_{eq} = \frac{P - \sqrt{P^2 - 16 \times S}}{4} \quad (2)$$

where:

L_{eq} : Equivalent length (km); l_{eq} : Equivalent width (km); P : Watershed perimeter (km); S : Watershed area (km²).

3) Characteristic Elevations

Characteristic elevations refer to the highest (maximum altitude) and lowest (minimum altitude) points within the watershed. These elevations, typically located at the watershed outlet, are directly extracted using the ArcGIS tool.

4) Average Watershed Slope

The average slope of the watershed is derived from the characteristic elevations and provides insight into the topographical gradient of the catchment area. It is calculated using the following formula:

$$I_{moy} = \frac{H_{max} - H_{min}}{\sqrt{S}} \quad (3)$$

5) Longitudinal Slope of the Watershed

The longitudinal slope is estimated based on the size of the watershed using the simplified GRESILLON formula:

$$I_{long} = \frac{0.026}{\sqrt{S}} \quad (4)$$

where:

I_{long} : Longitudinal slope (m/km); S : Watershed area (km²).

Based on the value of the longitudinal slope, watersheds can be classified into six (6) slope classes (FAO, 1996), as defined by ORSTOM (now IRD).

6) Transverse Slope of the Watershed

The transverse slope is determined by analyzing the slope across 4 to 6 cross-sections of the watershed using Google Earth. The average of these individual slopes gives the watershed's mean transverse slope.

7) Hypsometric Curve

The hypsometric curve is derived from the Digital Elevation Model (DEM) data. It represents the distribution of watershed area as a function of elevation [3]-[5]. This curve allows us to determine the elevations corresponding to 5% and 95% of the cumulative area.

8) Global Slope Index

The global slope index reflects the impact of topography on peak runoff by influencing flow velocity [6]. It is expressed in meters per kilometer (m/km) and characterizes the overall relief of the watershed. It is defined by the following formula:

$$I_g = \frac{\Delta H}{L_{eq}} \quad (5)$$

where: I_g : Global slope index (m/km); ΔH : Elevation difference between the 5% and 95% cumulative area thresholds (m); L_{eq} : Equivalent length of the watershed (km).

9) Corrected Slope Index

This index is calculated only when the transverse slope exceeds the longitudinal slope by more than 20%. It is computed using the following formula:

$$I_{gcorr} = \frac{(n-1) * I_g + I_t}{n} \quad (6)$$

where: I_{gcorr} : Corrected global slope index (m/km); I_g : Global slope index (m/km); I_t : Transverse slope (m/km); n : Coefficient depending on the equivalent length of the watershed.

10) Drainage Density

Drainage density is calculated as the ratio of the total length of all stream channels within the watershed feeding the pond to the total area of the watershed:

$$D_d = \frac{\sum L_t}{S} \quad (7)$$

$L_{streams}$: Total length of watercourses (km); S : Watershed area (km²).

11) Specific Relief (D_n)

The specific gradient is used to characterize various landform types. It is measured in meters and can be calculated using the following formula:

$$D_s = I_g * \sqrt{S} \quad (8)$$

The resulting D_n value allows for the classification of three (03) types of relief: Low relief: when D_n is less than 50 m, Moderate relief: when D_n is between 50 m and 100 m, and High relief: when the specific relief exceeds 100 m [7].

A. Watershed Morphometric Characteristics: Form Coefficient

Also known as the Gravelus Compactness Index, it is expressed as:

$$I_{comp} = 0.282 \times \frac{P}{\sqrt{S}} \quad (9)$$

where:

I_{comp} = Gravelus compactness index; P = watershed perimeter in km; S = watershed area in km². Once the compactness index is calculated, the shape of the watershed can be determined.

B. Determining the Project Flood Discharge

The flood discharge for the project will be determined using both the ORSTOM method by Auvrey and Rodier and the CIEH method described in the FAO manual on flood and runoff estimation [2].

B.1. ORSTOM Method by Auvrey and Rodier

Calculated over a 10-year period, the decadal runoff discharge (Q_{r10}) results

from intense rainfall causing significant surface runoff into watercourses. The formula is as follows

$$Q_{r10} = A * P_{10} * K_{r10} * \alpha_{10} * \frac{S}{T_{b10}} \quad (10)$$

A = reduction coefficient, P_{10} = 10-year return period daily rainfall (in mm), K_{r10} = 10-year runoff coefficient, α_{10} = 10-year peak flow coefficient S = watershed area (in km²), T_{b10} = base time (in hours).

a. Calculation of Decadal Runoff Discharge Parameters (Q_{10}^d): Reduction Coefficient (A):

It adjusts the predicted discharge based on watershed characteristics such as vegetation, topography, urban development, etc. It is expressed as:

$$A = \left(1 - \frac{161 - 0.042 * P_{am}}{1000} \right) \log_{10} S \quad (11)$$

where:

P_{am} = mean annual rainfall (in mm).

b. Decadal Runoff Coefficient (K_{r10}):

This coefficient is determined by an exceptional rainfall event occurring once every 10 years, typically involving precipitation between 70 mm and 100 mm.

If the 10-year rainfall amount (P_{10}) does not match exactly 70 mm or 100 mm, a method should be applied for estimation.

This method is based on the trend observed in the chart dedicated to the determination of the 10-year runoff coefficient. Specifically, the extrapolation involves linear interpolation (or extrapolation, where applicable) between the known data points to estimate a suitable runoff coefficient for intermediate rainfall values. This approach ensures continuity and improves the accuracy of hydrological estimates when direct values are not available in the chart [7].

c. The base time (T_{b10}) of the hydrograph:

It is obtained by interpolating between the values of I_g that frame the catchment's index on either side, using the chart for determining the base time.

d. Estimation of the decadal peak discharge:

$$Q_{10} = m * Q_{r10} \quad (12)$$

where:

m = amplification coefficient, which depends on the infiltration capacity class of the catchment and the climatic zone.

B.2. The CIEH Method

This method relies on empirical relationships established from hydrological and meteorological data. It was developed as a statistical approach by Puech and Chabi-Gonni in 1983.

$$Q_{10} = a * S^S * P_{am}^P * I_g^i * K_r^K * D_d^d \quad (13)$$

In our study, we will use four (04) regression formulas that are the most representative and likely to approximate the 10-year flood event. Based on the parameters S , I_g , and K_r , the following equations will be considered: Equations 11 and 12, which account for the delineation of the study area where annual rainfall (P_{am}) is less than or equal to 1000 mm; Equation 42, which is a function of the country group; Equation 40, which is specific to the study area. Given the uncertainty associated with these different equations, the maximum discharge value obtained will be adopted as the 10-year flood discharge.

$$Q_{11} = 0.41 \times S^{0.524} \times K_{r10}^{1.038}; Q_{12} = 0.095 \times S^{0.643} \times K_{r10}^{1.038} \times I_g^{0.406} \quad (14)$$

$$Q_{42} = 0.0912 \times S^{0.643} \times K_{r10}^{1.019} \times I_g^{0.399}; Q_{40} = 0.254 \times S^{0.462} \times K_{r10}^{0.976} \times I_g^{0.101} \quad (15)$$

B.3. The GRADEX Method

The project's 100-year flood discharge will be derived from the 10-year flood discharges using an amplification coefficient C , also referred to as the multiplication coefficient. This method aims to ensure maximum safety by leading to the highest flood estimates (100-year flood) through the following relationship:

$$Q_{100} = C \times Q_{10} \quad (16)$$

In the Sahelian zone, the amplification coefficient is calculated as follows:

$$C = 1 + \frac{P_{100} - P_{10}}{P_{10}} * \frac{T_b^{0.12}}{K_{r10}} \quad (17)$$

C. Determination of the Flood Hydrograph

The flood hydrograph is a chart that illustrates how discharge varies over time during a flood event. It depends on both the base time and the time required for the rising limb of the flood. The peak discharge, combined with the recession flow, forms the recession curve of the hydrograph. Its determination is carried out using the following formula:

$$\frac{Q}{Q_{\max}} = \frac{2}{\alpha_{10}} * \frac{T_{b10} - \alpha_{10} * T_{m10}}{T_{b10} - 2 * T_{m10}} \quad (18)$$

D. The Design Flood of the Dam

This involves selecting the return period for the project's design flood. To ensure the safety of the structure, Degoutte and Fry [8] recommend choosing the flood return period based on the following relationship:

$H^2 \sqrt{V}$ where the result represents the return period of the project flood (in years).

Table 1 presents the obtained return period.

Table 1. Table for selecting the return period.

$H^2 \sqrt{V} < 5$	$5 < H^2 \sqrt{V} < 30$	$30 < H^2 \sqrt{V} < 100$	$100 < H^2 \sqrt{V} < 700$
0 (Centennial)	500 (Quinquennial)	1000 (Millennial)	5000 (Five-millennial)

E. The Dam Break Flood

This refers to the flood that the dam must be able to withstand without sustaining damage that could affect its operation under exceptional conditions.

The formula proposed by the International Commission on Large Dams (ICOLD) for its estimation is as follows:

$$Q_r = 2 \cdot I \cdot (a + 0.2)^{1.5} + 0.15 \cdot L$$

I : Length of the spillway

L : Length of the dam without spillway

a : Total freeboard.

F. Inflows at the Dam Site

F.1. Estimation of Water Inflows

To ensure the reservoir is filled, it is necessary to estimate the volume of water—*i.e.*, the inflows from the catchment area. To determine the annual inflows of the basin, we will use the estimation methods of Coutagne and Rodier.

The volume of inflows is obtained using the following parameters, depending on the method applied.

$$V = K_e \cdot S \cdot P \quad (19)$$

V = Volume of water at the outlet

K_e = Runoff coefficient

P = Average annual rainfall

F.1.a. The COUTAGNE Method

The parameters used to calculate annual inflows according to the Coutagne method are as follows:

$$\begin{aligned} L_e (\text{mm}) &= P (\text{mm}) - D (\text{mm}) \\ D (\text{m}) &= P (\text{m}) - \lambda P^2 (\text{m}) \\ \lambda &= \frac{1}{0.8 + 0.14 \cdot T} \end{aligned} \quad (20)$$

T = Average annual temperature

P = Average annual rainfall

D = Annual runoff deficit

L_e = Annual runoff depth (in mm or equivalent water layer).

The following correlations, used by ONBAH [3]-[5], make it possible to estimate the annual inflows during dry years (5-year and 10-year droughts), using:

$$K_e = \frac{L_e (\text{m})}{P (\text{m})} \quad (21)$$

The inflows during a 5-year dry period and a 10-year dry period are obtained respectively as follows. **Table 2** shows calculation of runoff coefficients for 5-year and 10-year dry years.

F.1.b. The RODIER Method

This method is based on identifying the reference or type basin corresponding

Table 2. Calculation of runoff coefficients for 5-year and 10-year dry years.

Runoff coefficient for a 5-year dry year	Runoff coefficient for a 10-year dry year
$K_{e5} = 0.7 \sqrt{2217K_e}$ (22)	$K_{e10} = 0.5 \sqrt{2217K_e}$ (23)

to our catchment area, along with its area of influence. The distribution curve of runoff depth as a function of cumulative non-exceedance frequencies makes it possible to determine the runoff coefficient and runoff volume for 5-year and 10-year return periods, based on the adjustment of rainfall data using the normal distribution law. Considering the characteristics of the type basin, our catchment area can be classified as part of the Oued Djajibine type basin, which belongs to the Oued Ghorfa system in Mauritania. This basin has a drainage density of 2.02 km/km and a specific elevation drop of 25 meters.

F.2. Losses at the Dam Site

F.2.1. Estimation of Evaporation Losses

Rainfall data collected from the NEMA station were used for the studies and to evaluate evaporation losses.

F.2.2. Estimation of Infiltration Losses

Mauritania, located in a semi-desert region with sandy terrain, has an average daily infiltration ranging from 2 to 4 mm. However, since the project area is dominated by clay soils, a value of 3 mm was selected for infiltration calculations.

F.2.3. Estimation of Spatial Losses Due to Sediment Deposition

Water carried by the various streams reaches the dam with solid debris. These materials occupy space and therefore reduce the storage capacity of the reservoir.

To estimate the specific degradation, we use the empirical formula of KARAM-BIRI, which, in addition to the geometric parameters of the watershed, also considers anthropic and morphological factors.

$$D_1 = 137 \times \left(\frac{P}{100} \right)^{-2.2} \times S^{-0.05} \times (0.25 + 1.13 \times (h + r))^{1.15} \quad (24)$$

P = average annual rainfall;

S = watershed area;

h = anthropogenic parameter, equal to 0.3 for a watershed containing small towns, medium-sized villages, or located near such settlements (FAO, 1996);

r = morphological parameter, equal to 0.2 for slightly developed relief (FAO, 1996);

D = specific annual degradation ($\text{m}^3/\text{km}^2/\text{year}$).

Thus, the annual volume of sediment input is obtained using the following formula:

$$V = D \times S \quad (25)$$

F.2.4. Reservoir Capacity

The topographic data collected in the project area allowed us to determine the reservoir's capacity. This data was subsequently processed using AutoCAD software. The estimation of losses made above, as well as the needs for watering the

livestock of the refugees, will allow us to define the dimensions of the dam control structures.

Various registration centers and local offices in the villages surrounding the pond were consulted in order to estimate the livestock population within the intervention area.

3. Results and Discussion

3.1. Topographic Studies

The analysis of the surrounding terrain facilitated the generation of various elevation levels ranging from 264 m to 328 m, as well as the determination of the surface area occupied by each elevation. These data are essential for adjusting the leveling of the basin in order to ensure proper sizing of the spillway and the dam. The results of the topographic surveys are shown in **Table 3**.

Table 3. Results of the topographic surveys.

Elevation (m)	cumulate area		
	Km ²	Km ²	%
328	0	0	0%
320	0.5460	0.5460	1.22%
310	2.6420	3.1880	7.14%
296.5	4.8272	8.0152	17.95%
290	3.9946	12.0098	26.90%
280	5.9658	17.9756	40.26%
270	14.6021	32.5777	72.96%
264	12.0747	44.6524	100%

The distribution of the water volume according to the elevations of the basin is shown in the height-volume curve below. Each elevation corresponds to a specific volume of water it contains.

The quantity of water held in these different pockets constitutes the total volume of water contained in the pond. **Figure 1** presents the height-volume curve of this basin.

The processing of the topographic base made it possible to obtain the height-surface curve of the dam basin between the elevations of 296.5 m and 299 m. **Figure 2** shows the height-surface curve of our reservoir.

3.1.1. Soil Studies

The soil studies show that the basin of the pond, from the left bank to the right bank, is composed of a clay layer throughout the surface area. Soil sampling through one-meter (1 m) deep test pits also confirms that the foundation soil of the dam is made of clay.

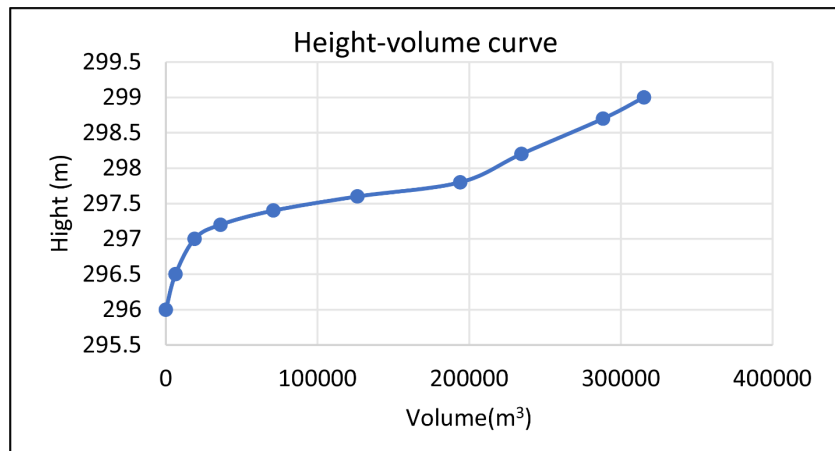


Figure 1. Height-volume curve.

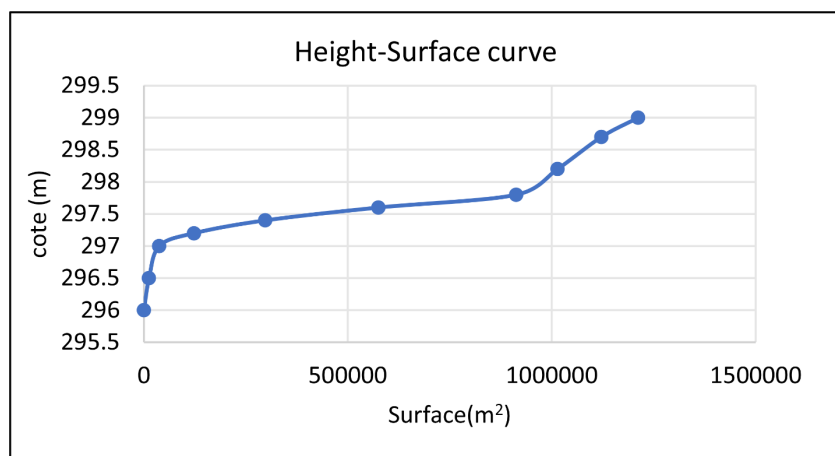


Figure 2. Height-surface curve.

3.1.2. Hydrological Studies

• Analysis of Rainfall Data

The analysis of rainfall data provides insight into precipitation variations in the study area, offering crucial information for water resource management. As part of this study, 34 data points were obtained from the NEMA station for rainfall analysis. These data were analyzed and verified in Excel using the method of moments, and then fitted using the HYFRAN software with the GAUSS normal distribution and the GUMBEL distribution. The results for rainfall data from the NEMA station are presented in **Table 4**.

Table 4. Rainfall data results from the NEMA station

Parameter	Annual Average Rainfall (mm)	Maximum Daily Rainfall (mm)
Number of data points	Thirty-four (34)	Thirty-four (34)
Maximum value	354.1	204.6
Minimum value	67.5	36

Continued

Average value	204.04	82.18
Coefficient of variation	0.358	0.487
Standard deviation	72.8	39.8
Confidence level (%)	95%	95%

Statistical analyses using the Gauss and Gumbel methods on both data series facilitated the identification of quantiles associated with various return periods.

- **Determination of Wet Quantiles (5-year; 10-year; 100-year Return Periods)**

The following **Table 5** lists the determination of wet quantiles.

Table 5. Determination of wet quantiles.

SUMMARY OF WET QUANTILES					
Return Period T (years)	5	10	20	50	100
Non-exceedance Probability P	0.8	0.9	0.95	0.98	0.99
Gumbel Reduced Variable	1.4999	2.2503	2.9701	3.9019	4.6001
Wet Quantiles (mm)	301.860	332.097	361.1013	398.643	426.7767

- **Determination of Dry Quantiles (5-year, 10-year, 100-year Return Periods)**

Table 6 displays the results for the determination of dry quantiles.

Table 6. Determination of dry quantiles.

SUMMARY OF DRY QUANTILES					
Return Period T (years)	5	10	20	50	100
Non-exceedance Probability P	0.8	0.9	0.95	0.98	0.99
Gumbel Reduced Variable	-1.4999	-2.2503	-2.9701	-3.90	-4.6000
Dry Quantiles (mm)	90.0064	68.279	47.439	20.4	0.2486

3.1.3. Description of the Watershed Characteristics Related to Geometry

The watershed feeding the dam has an area of approximately forty-five square kilometers (45 km²) with a perimeter of 42.2 km. This places it in the category of medium-sized watersheds, according to the classification defined by RODIER in FAO Bulletin 54, which considers watersheds in the range of $40 < S < 1000$ km² as medium-sized.

The results of the watershed characteristics related to geometry are presented in **Table 7**.

The value of the longitudinal slope allows us to classify our watershed as an R2 class basin (*i.e.*, lowland basins), according to the classification defined by ORSTOM (FAO, 1996).

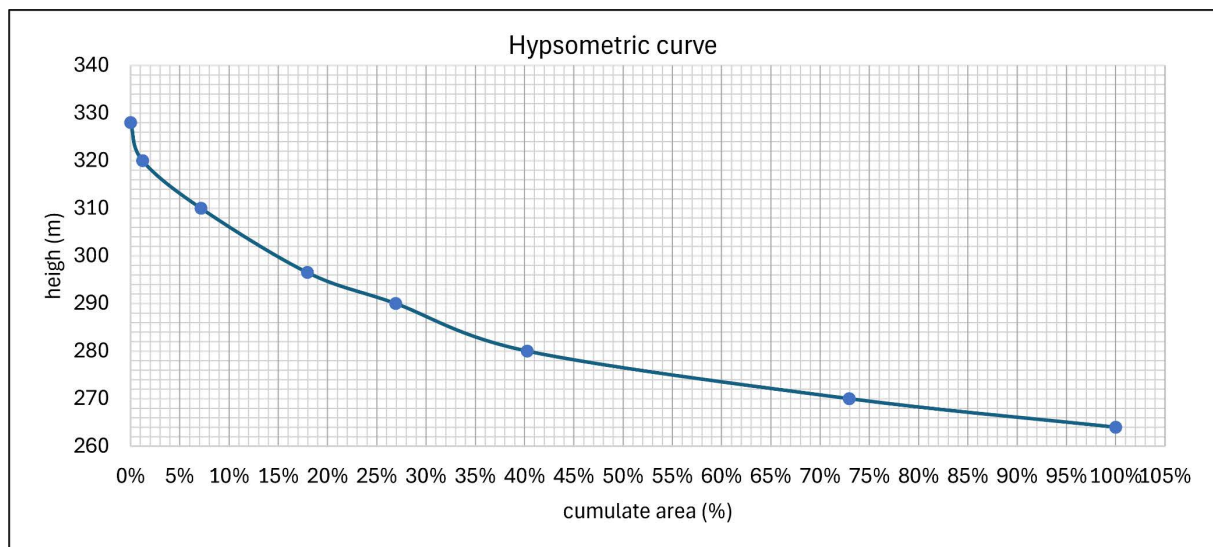
Table 7. Results of the watershed (WS) characteristics related to geometry.

Watershed geometric characteristic	Results
Length of the equivalent rectangle (L_{eq})	18.70 km
Width of the equivalent rectangle (l_{eq})	2.39 km
Average slope of the watershed (I_{moy})	9.56 m/km
Average transverse slope of the watershed (I_t)	12.52 m/km
Longitudinal slope of the watershed (I_l)	3.88 m/km
Overall slope index (I_g)	2.42 m/km

The transverse slope of 12.52 m/km was obtained by averaging four (4) transverse slopes using AutoCAD and Google Earth tools.

3.1.4. Hypsometric Curve Plotting

The following plot provides a graphical representation of the elevation distribution in the study intervention area. It is often used to illustrate the altitudinal variation of the surface area within the watershed of a given region. **Figure 3** shows the hypsometric curve plot of the watershed.

**Figure 3.** Hypsometric curve plot of the watershed.

Based on this hypsometric curve, the characteristic elevations of the watershed were determined graphically. The following values were obtained: A maximum value of 310.2 m, corresponding to the elevation at 5% of the watershed surface area; a minimum value of 265 m, corresponding to the elevation at 95% of the watershed surface area; 264 m and 328 m as the extreme elevations (minimum and maximum) of the watershed surface area. The transverse slope exceeds the longitudinal slope by 43%, highlighting the need to calculate the corrected overall slope index. In the numerical application for calculating this corrected slope index, the value of n , which corresponds to a coefficient based on the length of the

equivalent rectangle, is set to 3 ($n = 3$), as it falls within the interval $5 \text{ km} \leq L \leq 25 \text{ km}$ defined in FAO Bulletin 54.

3.1.5. Calculation of Watershed Geometric Characteristics Determining the Type of Relief Drainage Density (D_d) and Specific Elevation Difference (D_s)

The drainage density is obtained from the total length of the various watercourses feeding the outlet, using the ArcGIS tool. Regarding the specific elevation difference, the corrected slope index was used for its calculation. The results of geometric characteristics determining the watershed relief are presented in **Table 8**.

Table 8. Results of geometric characteristics determining the watershed relief

Geometric Characteristics Determining Relief	Results
Total length of watercourses	94.36 km
Drainage density (D_d)	2.1 Km/km ²
Corrected slope index (I_{gcorr})	5.91m/km
Specific elevation drop (D_s)	39.55 m

The value of the specific elevation drop indicates that the relief is low because $D_s < 50 \text{ m}$.

3.1.6. Watershed Morphometric Characteristics

The Gravelus compactness index shows that the shape of the watershed is elongated, since the form coefficient is $1.78 > 1.3$. Determination of the Project Flood Discharge ORSTOM Method by Auvrey and Raudier. This method begins with the determination of the ten-year runoff discharge of the ten-year flood.

Watershed Infiltration Capacity

With an area of 44.8 km^2 , projection on the determination chart for K_{r70} and K_{r100} gives the following values: For an infiltration category RI and a slope index $I_g = 3 \text{ m/km}$: $K_{r70} = 19$ and $K_{r100} = 23$ For RI and $I_g = 7 \text{ m/km}$: $K_{r70} = 16$ and $K_{r100} = 26$ For an infiltration category RI and $I_g = 5.91 \text{ m/km}$, the ten-year runoff coefficient is determined by interpolation between the surrounding slope values.

The calculations by interpolation yield a ten-year runoff coefficient of 21.54%. The base time T_{b10} is obtained by linear interpolation between the slope index values surrounding the studied watershed.

The results are as follows: Based on our area of 44.8 km^2 and the global slope index: For $I_g \leq 3 \rightarrow T_{b10} = 1010$ minutes and for $I_g = 7 \rightarrow T_{b10} = 600$ minutes. The following **Table 9** summarizes the results of the parameters used in the discharge calculation of surface runoff flood.

The CIEH Method

Based on the parameters S , I_g , and K_r , four (04) regression formulas will be retained in our study, as they are the most representative and likely to approximate the ten-year flood. **Table 10** shows the results of the calculation of the ten-year flow using the CIEH method.

Table 9. Results of the parameters used in the discharge calculation of surface runoff flood.

Parameter	Results
Reduction coefficient (A)	0.75
Ten-year runoff coefficient (K_{r10})	21.54%
Base time	898.27 mn
Ten-year flood peak coefficient (m)	2.6
Ten-year daily rainfall	132 mm
Watershed area (S)	44.8 km ²
Ten-year runoff discharge $Q_{r10} = A * P_{10} * K_{r10} * \alpha_{10} * \frac{S}{T_{b10}}$	44.93 m ³ /s
Ten-year flood peak discharge $Q_{10} = m * Q_{r10}$	
Q_{10} calculated using the ORSTOM method	47.18 m ³ /s

Table 10. Results of the calculation of the ten-year flow using the CIEH method.

Regression Formula	Results (m ³ /s)
$Q_{11} = 0.41 \times S^{0.524} \times K_{r10}^{0.982}$	61.27
$Q_{12} = 0.095 \times S^{0.643} \times K_{r10}^{1.038} \times I_g^{0.406}$	54.53
$Q_{40} = 0.254 \times S^{0.462} \times K_{r10}^{0.976} \times I_g^{0.101}$	35.23
$Q_{42} = 0.0912 \times S^{0.643} \times K_{r10}^{1.019} \times I_g^{0.399}$	48.77
Moyenne	50
Q_{10} using the CIEH method	61.27

Based on the ten-year flow rates obtained using the ORSTOM and CIEH methods, and the amplification coefficient determined using the GRADEX method (taking into account the study area), the hundred-year flow rates were calculated.

Table 11 presents a summary of the project's flood flow rates (ten-year and hundred-year flows).

Table 11. Ten-year flood flow (CIEH and ORSTOM methods).

Methods	Q_{10} (m ³ /s)	Q_{100} (m ³ /s)
ORSTOM	47.18	74.54
CIEH	61.27	100

After calculating the hundred-year and ten-year flood discharges using these two methods, we will adopt the CIEH method. This method yields a higher discharge, placing us in the most unfavorable scenario, which helps ensure that the flow rates are not underestimated.

Flood Hydrograph

The variation of discharge over time is represented by the flood hydrograph. This plot depends on the base time and the rise time. It will show the peak hundred-year discharge selected for the project, as well as the break-point discharge (Q_{rp}). The results below allow us to plot the flood hydrograph (Table 12).

Table 12. Results for plotting the flood hydrograph.

Discharges Q (m^3/s)	0	$Q_p = 100$	$Q_{rp} = 32$	$Q_f = 0$
Time (min)	0	$T_m = 296.43$	$T_r = 434.85$	$T_{b10} = 898.27$

Plotting of the Flood Hydrograph

The flood hydrograph displays the peak discharge and the break-point discharge, along with the times at which they occur. Figure 4 presents the flood hydrograph plot.

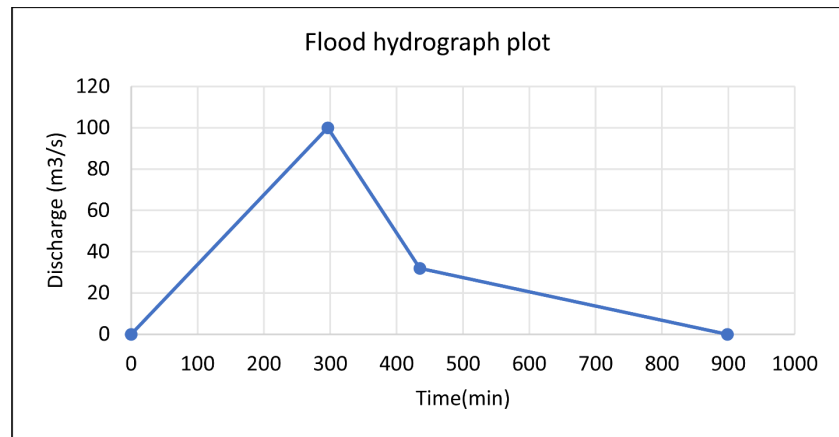


Figure 4. Flood hydrograph Plot.

The flood hydrograph shows the different phases of the flood period.

The first phase represents the rising limb, starting from the initial state ($Q = 0$; $T = 0$) up to the project flood peak ($Q_p = 100$; $T_p = 296.43$). The second phase corresponds to the recession limb, going from the project discharge to the recession break-point discharge ($Q_r = 32$; $T_r = 434.85$). The final phase is the drying or baseflow return curve, bringing the discharge back to its initial state.

Inflow to the Dam

Estimation of inflows using the COUTAGNE method

The advantage of using this method lies in its ability to provide a relatively accurate estimate of liquid inflows, which is essential for the operation and efficient management of the dam. The estimation using this method yielded the following results, shown in Table 13.

Estimation of Liquid Inflows Using the RODIER Method

The RODIER method, like the ORSTOM method, uses empirical formulas to calculate liquid inflows. However, the RODIER method is more prone to uncertainties because it relies on graphs. The liquid inflows obtained using this method

are presented in **Table 14**.

Table 13. Parameters for estimating liquid inflows.

Parameters for Estimating Water Inflows: COUTAGNE Method	Results
Average annual rainfall (m)	0.204
Annual runoff deficit (m)	0.196
Annual runoff depth (mm)	8.04
Runoff coefficient (%)	3.94
Runoff coefficient in a dry 5-year period (%)	2.76
Runoff coefficient in a dry 10-year period (%)	1.97
Annual water inflow (m ³)	360084.48
5-year dry period volume (m ³)	252059.136
100-year dry period volume (m ³)	180042.24

Table 14. Parameters for estimating liquid inflows.

Parameters for Estimating Water Inflows: RODIER Method	Results
Average annual rainfall (m)	204.04
Annual runoff deficit (m)	9.8
Runoff coefficient in a dry 5-year period (%)	6
Runoff coefficient in a dry 10-year period (%)	4.8
Annual water inflow (m ³)	895,817
5-year dry period volume (m ³)	548,459
100-year dry period volume (m ³)	447,908

The two methods used allowed for the estimation of liquid inflows at the basin level. Both methods provide values of annual liquid inflows sufficient to fill the normal water level of the basin. However, for convenience and reliability, we will use the average of the two methods, which is 627950.74 m³, for the subsequent calculations.

Losses at the dam

Evaporation losses

The table below presents the losses due to evaporation. These data were obtained from the Department of Agriculture in the municipality of El Meghve. The evaporation data obtained are in **Table 15**.

Table 15. Evaporation data.

Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	TA
177.1	180.3	212.7	204.1	197.6	171	145.8	134.9	146.9	172.8	166.7	163.1	2073

Source: Department of Agriculture of the Municipality of Meghve.

The maximum evaporation is recorded in March, with a total annual value of 2073 mm.

Losses due to infiltration

The project area is predominantly composed of clay, so we have selected a minimum infiltration rate of 3 mm per day for the infiltration calculation.

Spatial losses due to solid deposits

Using the empirical formula of KARAMBIRI, we were able to estimate the specific degradation.

Table 16 presents the results for the annual volume of sediment inflows.

Table 17 presents the results of estimated need and losses.

Table 16. Results of the annual volume of sediment inflows.

Specific Degradation (SD) (D_s)	Annual Inflow Volume (V)
112.33 m ³ /km ² /an	5032 m ³ /an

Table 17. Estimated needs and losses results.

Month	October	November	December	January	February	March
water need	30,679	30,679	30,679	30,679	30,679	30,679
lost (mm) (evap + inf)	175.8	169.7	166.7	180.1	183.3	212.7

4. Conclusions

The project to rehabilitate the Aghor Pond for the benefit of refugee livestock aims to improve the living conditions of Malian refugees in the commune of El Meghve, particularly those around the village of Aghor. Action Against Hunger, in line with its humanitarian mission to ensure the availability of essential resources and basic social services, is planning the rehabilitation of the Aghor Pond. This rehabilitation project is an initiative that benefits the local population, whose main activity is livestock farming.

Aware of their situation and the pressure exerted by livestock on water points intended for household use, the refugees welcome this proposal and request that the construction work be carried out as soon as possible, as it directly impacts their economy.

We have conducted technical studies to increase the pond's water retention capacity and to strengthen the availability of water resources during lean periods. This will be achieved by transforming the pond into a small dam equipped with control structures (dyke, spillway, dissipation basin, etc.) and a water intake system to supply watering troughs.

While this study provides a solid hydrological basis for the rehabilitation of Aghor Pond, it is important to acknowledge certain limitations. One of the main challenges lies in the reliability and completeness of the available hydrological and meteorological data, which may affect the precision of runoff estimations and flood forecasting. Additionally, sedimentation rates are estimated based on re-

gional assumptions, which may not fully reflect local variability. These uncertainties could influence the long-term performance of the designed system and should be addressed through continuous monitoring and adaptive management.

The next practical step in the project will involve the detailed engineering design of the control structures, including the dyke, spillway, dissipation basin, and water intake system. This phase will require comprehensive geotechnical surveys, structural analysis, and cost assessments to ensure both the technical feasibility and sustainability of the proposed interventions. Engaging local communities during the design and implementation phases will also be essential to ensure the project's success and long-term ownership.

Conflicts of Interest

The authors declare no conflicts of interest regarding the publication of this paper.

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