Hydrological and hydraulic study on the Wayai stream in Spa

Assessment of the retention volume required at Lake Warfaaz for flood control.

Interreg
Euregio Meuse-Rhine
EMR Flood Resilience
EUROPEAN UNION
European Regional Development Fund
TER CONSULT
Ingénieurs au service de l'arbre, la forêt, l'environnement
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Study framework: Project INTERREG – EMR228 - EMfloodResilience

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Numéro compte bancaire : BE45 5230 8079 2789

Review	Date	Version	Writing	Validation
Rev 0	26/01/23	Creation document	CBU/LR	CBU
Rev 1	10/02/23	V1	CBU/LR	CBU

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The EMfloodResilience project is being carried out within the context of Interreg V-A Euregio MeuseRhine and is 90% funded from the European Regional Development Fund

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1 BACKGROUND AND OBJECTIVES

The City of Spa is considering the use of the Warfaaz lake for flood control of the Wayai stream in order to reduce the risk of flooding in Spa.

In this context, the present study aims to estimate the retention required at Lake Warfaaz in order to delay flooding.

The study considers several rainfall event severities, with a return period of 5 years, 10 years, 25 years, 50 years, 100 years and a 100-year rainfall increased by 30%.

The absence of hydrological data (measuring station) on the Wayai in the study area implies that a hydrological study must be carried out in order to know the quantities of water involved. More specifically, the hydrological study allows the definition of the flood hydrographs (the flow of water over time) of the Wayai, upstream of the Lake and its tributaries located downstream of the Lake.

In addition, the Wayai stream is in a "pertuis" (vaulted passage) at its entrance to the city of Spa. Flooding in Spa is most often the result of saturation of these channels, leading to overflow. Reducing the risk of downstream flooding requires a hydraulic approach to determine the maximum flow through the channel.

2 SITUATION OF THE STUDY RIVER AND THE STUDY AREA

The study area is defined by the catchment areas of the Wayai and its tributaries from its source to its exit from the town of Spa (Figure 1).

The Wayai stream is a 3rd category river, originating upstream of Spa and flowing into the Hoëgne at Theux. Its catchment area is 93.78 km² (hydrometrie.wallonie.be).

The tributaries of the Wayai considered during the study are the following: Le passage, Le Soyereux, La Sauvenière, La Picherotte Le Barisart and finally Le Creppe. As the latter flows into the Wayai downstream of the town, it will not be considered further.

The catchment areas included in the study area and their surface area are presented in Figure 2.

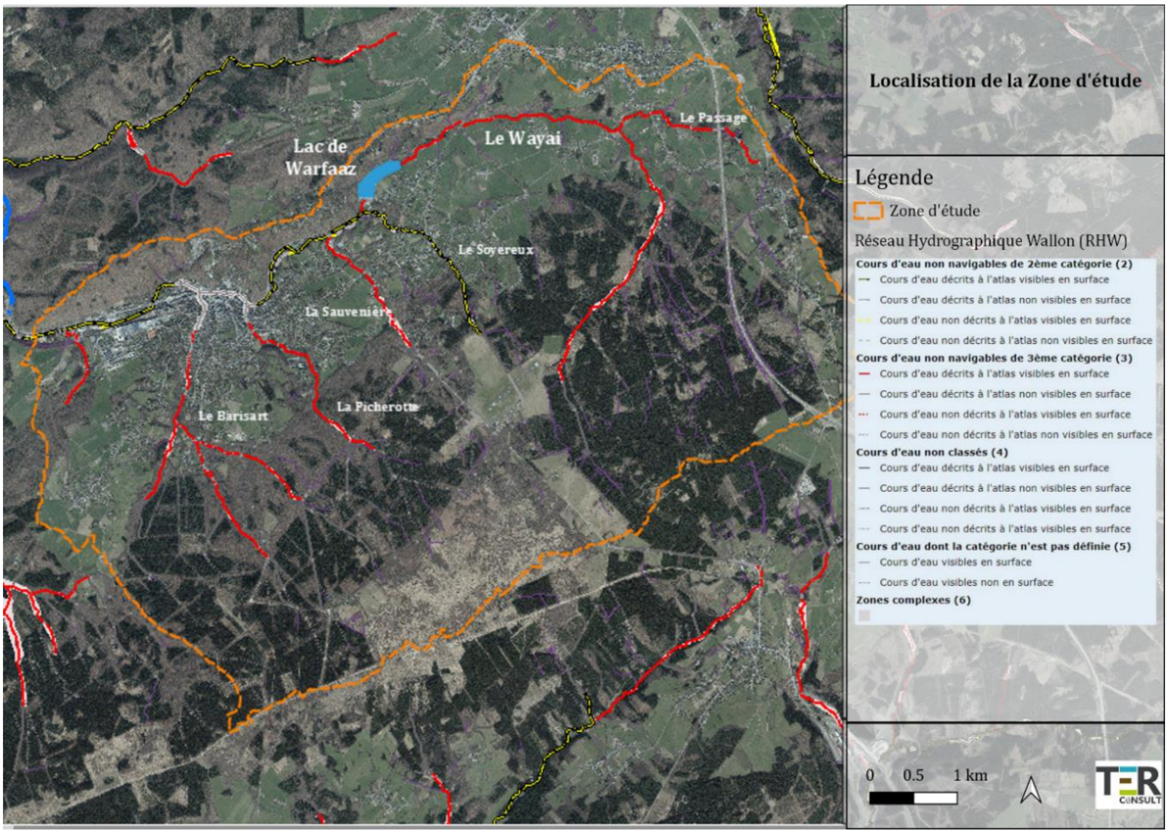


Figure 1. Location of the study area. The Wayai is a 3rd category stream, with a disturbance on the portion of its route through Spa (greyed out part of the stream) (geoportail.wallonie.be).

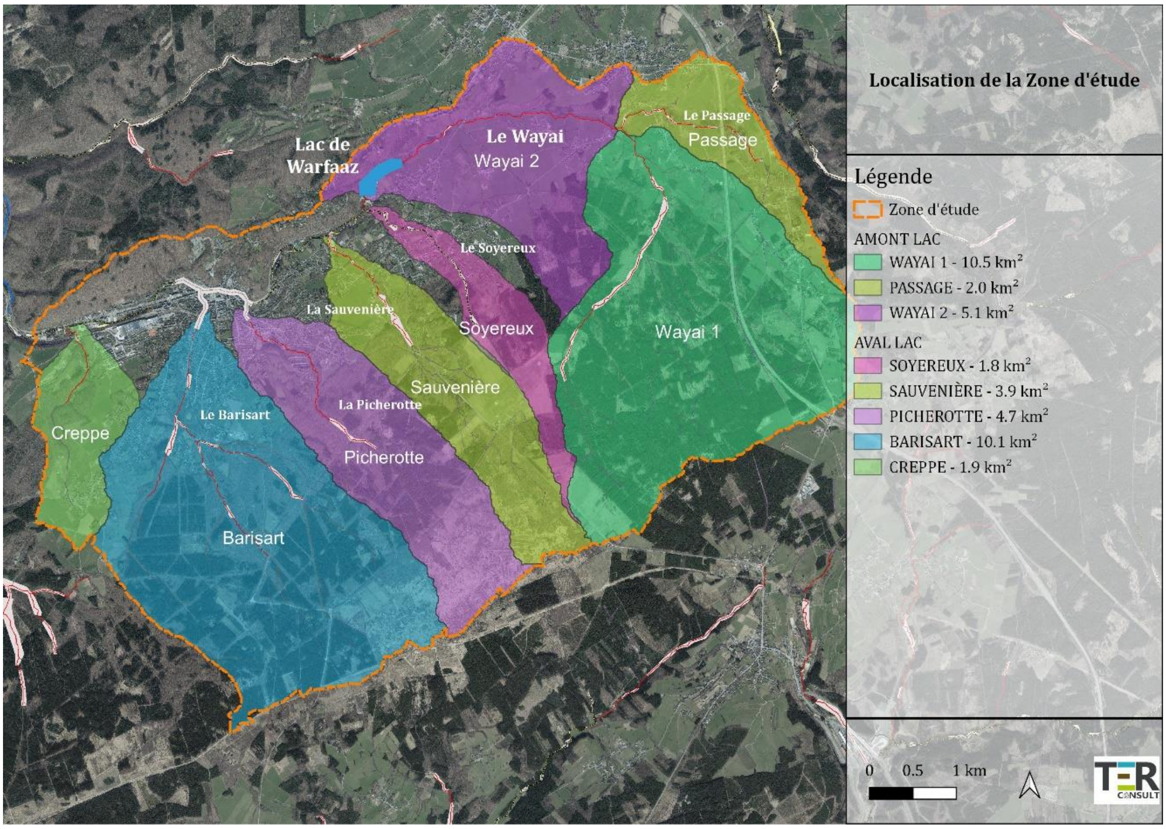


Figure 2. Catchment areas included in the study area and their size in km².

3 OVERALL APPROACH

Figure 3 shows schematically the methodology followed to calculate the retention volume of Lake Warfaaz. The methodology is applied according to return periods of 5, 10, 25, 50, 100 and 100 increased by 30%. The return period is abbreviated in the graphs and tables as TR (for return period).

The first step (1. - QCRIT) consists of calculating the maximum hydraulic capacity before overflow (full flow) of the openings and critical areas identified on the Wayai based on the existing profiles. The objective is to determine the most constraining critical section for the flow of the Wayai. Coupled with the flood hydrographs of the tributaries (3b - QAFF), it will determine the permissible leakage rate of the Lake of Warfaaz (4. - QOUT).

The hydrological modelling (3a. and 3b.) makes it possible to determine the flood hydrographs of the catchment areas upstream of the lake (3a. - QIN) and of the catchment areas of the tributaries downstream of the lake (3b. - QAFF) according to the selected return periods.

The calculation of the flow over time is based on the project rainfall for each catchment. The hyetograms of rainfall for increasing return periods are constructed using the double triangle method (2.).

The evolving volume of the lake is calculated at each time step, when the leakage flow is lower than the inlet flow (5.). Their difference (QIN - QOUT) is multiplied by the time step and gives the accumulated volume of the lake during the chosen time step.

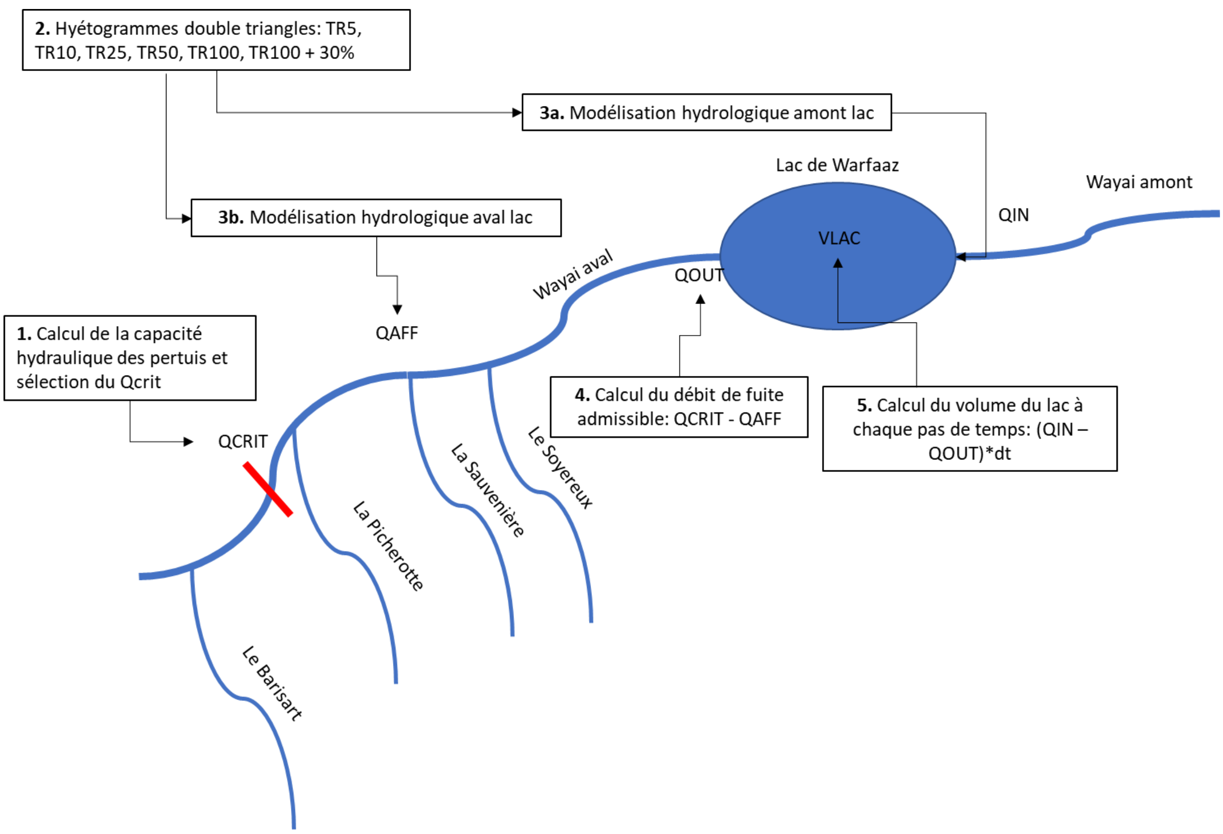


Figure 3. Schematic representation of the methodology followed for the calculation of the lake retention volume.

4 Available data and descriptions of the topographic surveys used

4.1.1 Geometry of the Wayai, Picherotte and Barisart channels

The hydraulic capacity of the sections studied in the Wayai and its tributaries (Picherotte and Barisart, which would present certain overflows in the event of a flood) is reconstituted from the longitudinal and cross-sectional profiles of the watercourse and its vaults, the longitudinal slope and the roughness coefficient (Manning).

The roughness coefficients are taken from the Tables of Chow, 1959 and ASCE, 1982.

The geometry of the openings is taken from the cross-sections provided by the City of Spa and the STP Liège. The sections come from different sources, more or less recent:

- Atlas of non-navigable waterways (1967)
- Surveys (2001)
- Repairs under the Parc des 7h (2019) (Wayai)
- Repairs under the Barisart road (2022) (Barisart)

The dimensions used to approximate the geometry of the openings are taken from the most recent sections provided to us (assumed to be up to date). As their typology is not always consistent according to the source used, a new denomination was established within the framework of this study. The sections selected to assess the hydraulic capacities of the three watercourses and their denomination are listed in Appendix 1.

The slopes of the streams in the channels are calculated from the elevations referenced in the cross-sections presented above, while the slopes between the engineering structures (bridges) are approximated from the SPW Digital Terrain Model (DTM). The DTM data was acquired by a Lidar flight in 2013-2014. This DTM is characterised by a planimetric resolution of 1 m and an altimetric accuracy of 0.12 m (source: geoportail.wallonie.be). This DTM represents the altitude of the ground, without considering the elements located on its surface (buildings, vegetation, etc.).

4.1.2 Geometry of Lake Warfaaz

The geometry of the Lake of Warfaaz is determined on the basis of the cross-sections transmitted by the City of Spa, carried out in the context of the lake's rehabilitation in 1978, as well as the reconstructed cross-section of the dam on the sluice gate side in 2022. The lake bottom elevations used are the project elevations, reached after the sludge has been removed. The relevance of this hypothesis will have to be verified once the bathymetric surveys in progress have been carried out.

The maximum water level before the lake is completely filled is at DNG (Second General Level) 100.00 m, determined on the basis of the longitudinal profile dating from 1978, corroborated by the plans for the dam on the gate side, dating from 2022. This is the maximum elevation considered for the lake reservoir.

The lake bottom elevations and the bottom and full widths are determined by sections C5 to C25 (Figure 6).

4.1.3 Data needed for the hydrological study.

The topography used for the hydrological study is taken from the SPW ERRUISSOL Digital Terrain Model (DTM), a DTM adapted to the study of runoff and soil erosion, which allows the delimitation of Hydrographic Sub-Catchments (HSC). It dates from 2005 and has a planimetric resolution of 10 m.

The ERRUISSOL DTM was obtained by interpolating altimetric data from the Digital Surface Model (DSM) of navigable and non-navigable rivers of the first category (1 point per m²), from the Computerised Continuous Mapping Project (PICC, 1/1,000) and from the Digital Terrain Model (DTM) 1/10,000 of the Belgian National Geographic Institute (IGN) using the Topo To Raster extension of Spatial Analyst 9.1 (source: geoportail.wallonie.be).

The data required for the hydrological modelling Soil Conservation Service (SCS) - Curve Number (CN) are as follows:

- Quantity-Duration-Frequency (QDF) curves for the municipality of Spa, from the Royal Meteorological Institute of Belgium (IRM) (https://www.meteo.be/fr/climat/climat-de-la-belgique/climat-dans-votre-commune).
- Land cover: based on the most recent land use map (WALOCS 2019, source: Geoportail.wallonie.be).
- The hydrological group: which defines the water infiltration potential in soils from the article by Demarcin et al. (2011).
- Tables of CN from the SCS describing the value of CN as a function of the hydrological group, the soil cover and the hydrological conditions (Annex 2).
- Tables describing the roughness coefficient according to land use (USACE, 1998, Appendix 2).
- The SCS synthetic unit hydrograph describing the response over time of the theoretical catchment flow caused by one unit of rainfall runoff (Appendix 3).

Finally, there is a flow measurement station on the Wayai stream in Spixhe (REF L6790 - catchment area 93.78 km²), downstream of Spa. The station has been in operation since 29/03/2002. The hourly flow data were downloaded from 29/03/2002 to 14/07/2021 (station shutdown during the floods of July 2021).

5 METHODOLOGY

5.1 DETERMINATION OF HYDRAULIC CAPACITY

Critical sections are those parts of the watercourse (or structures such as sluices) that overflow first during floods and are therefore likely to determine the permissible flow of the Wayai watercourse. In other words, this is the full flow.

A field visit carried out on 23/11/22 in the presence of representatives of the City of Spa and the STP Liège made it possible to identify certain problematic sections, located in the crossing of Spa by the Wayai and in line with certain watercourse crossing structures.

Given the large number of sections mentioned during the visit and their underground characteristics for a good number of them, the hydraulic capacity of several sections was calculated on the Wayai and its tributaries. Initially, 18, 3 and 7 profiles were selected for the Wayai, the Picherotte and the Barisart respectively (Appendix 1).

The full flow calculated corresponds to the maximum flow admissible by the watercourse before overflowing. For each of the critical sections, the full flow is calculated by the following Manning-Strickler formula:

$$Q = K * RH^{\frac{2}{3}} * i^{\frac{1}{2}} * S$$

With; Q: the flow rate [m^3/s], K: the Strickler roughness coefficient [m1/3/s], RH: the hydraulic radius of the flow [m], i: the hydraulic slope [m/m], S: the wetted area of the section [m^2].

The roughness coefficient (K) reflects the resistance of the opening to the passage of water. The K retained in this study for vaulted openings is 50 (value recommended by SETRA (2011) for old masonry and very degraded concrete).

The formula of the wetted surface of the section is dependent on the type of section considered to approximate the real geometry of the opening: rectangle, half-ellipse or semi-circle. The details of the type of section considered for each arch are presented in Annex 4.

The critical sections are designated by the minimum hydraulic capacity value for the Picherotte and the Barisart. For the Wayai, the hydraulic capacity value retained is that which implies the most constraining flow at the lake outlet, i.e. the minimum value after consideration of the tributary inflows. In addition, following exchanges with the watercourse manager, the presence of significant sedimentary obstructions in the left arm of the separate double sluiceway, observed during the inspection of the sluiceways after the floods of July 2021, was mentioned. Two modelling scenarios are therefore envisaged.

(i) **dpert_W_Rel_05**: A section upstream of the Barisart, considering one arm of the separate double inlet as blocked.
(ii) **spert_W_Rep_10**: A section upstream of the Barisart, in single channel

The critical sections of the Wayai are presented in Figure 4. We will see in section 4.1 that we will ultimately retain only one critical flow value for the hydrological modelling.

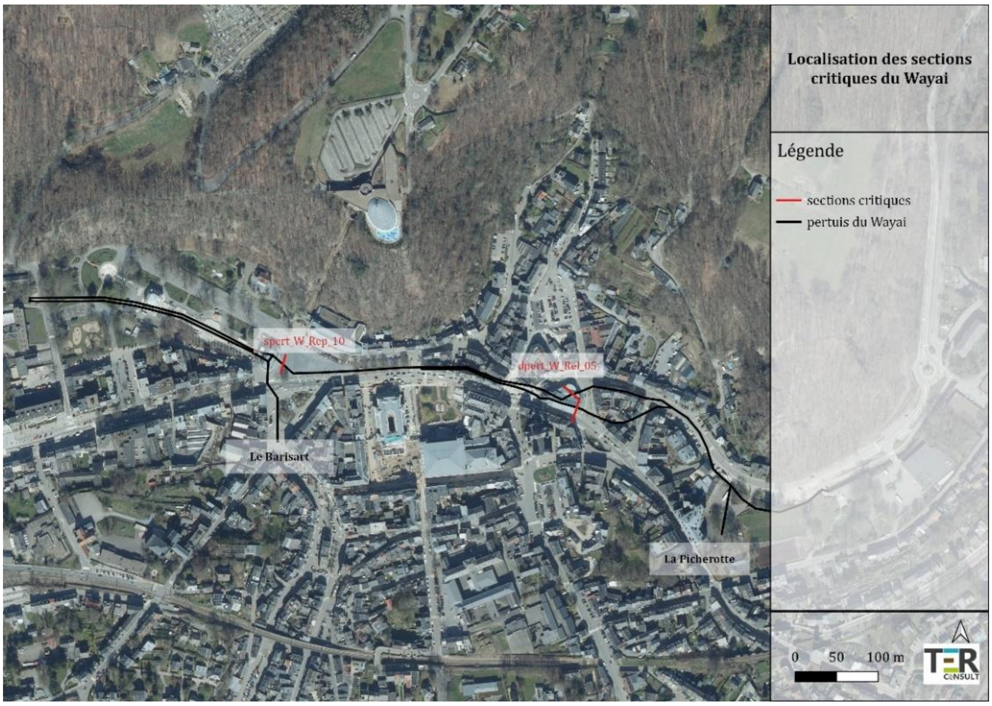


Figure 4. Location of critical sections of the Wayai.

5.1.1 Estimation of uncertainty

The uncertainty is estimated by setting the roughness coefficient K which can vary between 40 (old masonry/degraded concrete) and 65 (smooth concrete).

5.2 Hydrological modelling

Each catchment area upstream of the lake and downstream of the lake is the subject of a hydrological model. The simulation takes as input a project rainfall according to the return period and applies it to each catchment area, the characteristics of which will condition the dynamics of water runoff and therefore the flow of the watercourse. The time step of the simulation is 5 minutes.

5.2.1 Construction of hyetograms associated with each project rainfall

A project rainfall is characterised by an associated hyetogram. From the QDF curves of the IRM, it is possible to reconstruct synthetic hyetograms, i.e. a hyetogram associated with a theoretical project rainfall, with a given return period and duration (example for the rainfall with a return period of 25 years and 120 hours in Figure 5).

For each return period, the duration of the rainfall that maximises the volume of water in the lake is selected for further sizing.

The choice of the method was made in favour of the double triangle representation, adapted for catchments with a response time higher than 10 minutes, which is the case of the catchments under study (source: http://wikhydro.developpement-durable.gouv.fr).

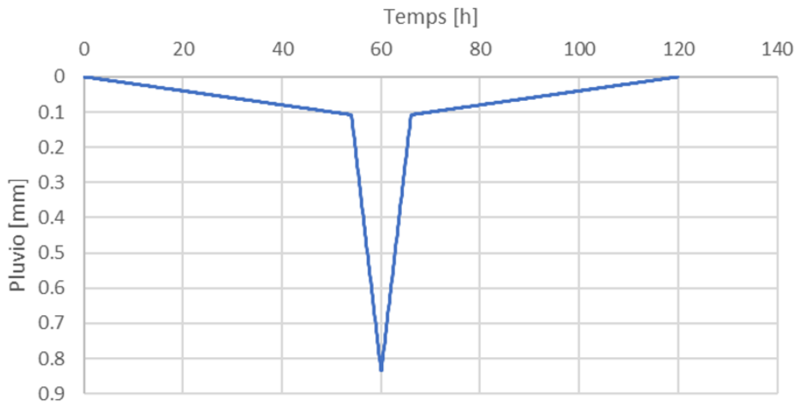


Figure 5. Example of a hyetogram: Return period of 25 years and duration of 120H.

5.2.2 Calculation of runoff

The first step of the SCS CN hydrological modelling is to calculate the runoff [mm] under the action of a design rainfall. The proportion of the rainfall that will run off the catchment depends solely on the CN.

The CN is calculated per sub-catchment, under average moisture conditions and is parameterised by

- Land cover
- The hydrological group
- Hydrological conditions

The CN values corresponding to each land cover and hydrological group are presented in Annex 2.

It is then adjusted according to the slope of the basin, using the formula of Sharpley and Williams (1990):

$$CNpente = \frac{CNIII - CNII}{3} * (1 - 2 * e^{-13.83*pente}) + CNII$$

The runoff is then calculated by the following formulas:

$$R = \frac{(P - I_a)^2}{(P - I_a) + S}$$

$with\ S = 25{,}4\left(\frac{1000}{CN} - 10\right)\ and\ Ia = coeff\_Ia * S$

- R: runoff height [mm] P: precipitation height [mm
- P: precipitation [mm]
- Ia: initial interception [mm].
- Coeff_Ia: initial interception coefficient, usually equal to 0.2. However, the value of 0.05 is suggested by more recent literature (Jiang R. (2001), Hawkins et al. (2002), Woodward et al. (2003)) and is therefore used for this study.
- S: Potential soil water retention [mm]

5.2.3 Calculation of flood hydrographs

The transfer function is then used to translate the rainfall into flow and is carried out by the SCS unit hydrograph model, taking into account the characteristics of the terrain. The method consists in reconstructing, at a defined time interval, the synthetic triangular unit hydrograph parameterised by:

- The runoff height (calculated in the step presented above).
- The time of concentration.

The time of concentration is determined by the SWRRB (Simulator for Water Resources in Rural Basins) method, which is the sum of the slope time of concentration (tcs - the time it takes for the most upstream drop in the catchment to reach the drainage channel) and the channel time of concentration (tcc - the time it takes for the drop to travel to the outlet from the drainage channel).

Finally, the global hydrograph is obtained by superimposing the unit hydrographs (convolution) for each studied return period.

5.2.4 Estimation of uncertainty

The Spixhe station, located downstream, is used to validate the order of magnitude of the flows obtained by the modelling. The time series of flows recorded at Spixhe allow the probability of occurrence of a given flow to be estimated.

The frequency analysis is carried out on the maximum annual flow by the Gumbel law frequency model. The points are then fitted by the method of moments.

The flow values at Spixhe are extrapolated to the lake inlet by a watershed ratio using the following formula:

$$Q_{BV-LAC} = Q_{BV-SPIXHE} * (\frac{A_{BV-LAC}}{A_{BV-SPIXHE}})^{\alpha}$$

With Q: peak flow [m³/s], A: area in [km²], α: Myer coefficient: 0.8 [-].

The flow values adapted by a catchment transfer coefficient can be compared with those simulated for each return period.

Table 1. Frequency analysis: flow value of the Wayai at Spixhe associated with each return period. Time series: 03-2002 TO 07-2021 - annual maximums

TR	Flow rate [m³/s] - Gumbel moments
2	27
5	46
10	58
25	74
50	86
100	97

5.3 ESTIMATED RETENTION VOLUME

The retention volume (RV) corresponds to the total volume of water runoff on the catchment area upstream of the lake, minus the volume of water leaving the lake, over the time interval of the rainfall. It is calculated for each project rainfall at the following return times (RT): TR5, TR10, TR25, TR50, TR100, TR100 flow + 30%.

It requires the calculation of the allowable leakage rate from the Lake preventing flooding by the Wayai in Spa. The allowable leakage flow is the one calculated for the critical section minus the inflow from the tributaries, for each return period. For each section, if we go upstream from downstream, the critical flow is reduced by the inflows of the different tributaries. The equation below summarises the approach:

$$Q_{OUT}(t) = Q_{CRIT} - \sum Q_{AFF}(t)$$

With QOUT [m³]: the lake leakage flow: variable over time. QCRIT [m³]: the full flow of the critical section: constant, QAFF [m³]: the inflows of the tributaries: variable over time.

Then the lake flood hydrograph can be reconstructed by the following equation:

$$Q_{LAC}(t) = Q_{IN}(t) - Q_{OUT}(t)$$

Finally, the retention volume (RV, [m³]) is calculated by the area under the curve of the lake flood hydrograph according to :

$$\text{VR} = \int_{tstart}^{tend} Qlac(t)\ dt.$$

With; tstart [min]: the start time of the rain and tend [min]: the end time of the rain.

The volume of water in the lake at each time step is then related to the water level in the lake.

5.3.1 Estimation of uncertainty

The value of CN is the determining parameter of the SCS method, however there is an uncertainty on the CN, resulting from several parameters: the assignment to the land use classes (dating from 2019), the hydrological group (dating from 2014 and with a planimetric resolution of 20 m), and the choice of the hydrology class. In order to take this uncertainty into account, a sensitivity analysis is performed by varying the CN by [-2 + CN, CN +2].

5.4 HEIGHT OF THE LAKE

The height of the lake is established at each time step according to the retention volume. For this purpose, it is necessary to approximate the volumetry of the lake.

It is important to note that the lake is considered to be empty at the beginning of the rainfall.

The height of the water is calculated in terms of the DNG altitude, which lies between the minimum height of the lake (92.36 m) and the maximum height (100.00 m).

The volume of the lake is decomposed into three trapezoidal parallelepipeds (trapezoid considered isosceles) (Figure 6). Their dimensions are presented in Table 2. The maximum volume of the lake before overflow in this context is estimated at 278,000 m³ by this method.

The maximum lake volume is calculated by the following formula: V = (b+B)*H*l

With b [m]: width of the lake bottom, H [m]: maximum water height, B [m]: width of the water line when H is reached, l [m]: length of the parallelepiped, V [m³]: maximum volume of water.

When the volume is variable, the height of the water is variable but also the width of the water line which corresponds to the large base of the trapezoid (two unknowns). The solution therefore depends on two equations: the volume equation of a trapezium and the equation from Thales' theorem applied to the triangles resulting from the decomposition of the trapezium (into two triangles and a rectangle). The combination of these two first degree equations results in a second-degree equation with one unknown (h), shown below:

$$\frac{B-b}{2H} * l * h^2 + l * b * h - V = 0 \; avec \; 92.36 \, m \; < h < 100.00 \, m$$

With b [m]: width of the lake bottom, H [m]: maximum water height, B [m]: width of the water line when H is reached, l [m]: length of the parallelepiped, V [m³]: volume of water in the lake at time t.

Table 2. Geometric characteristics of the three sub-volumes of the lake, in [m].

	Sections	**DNG background**	**b** (width of the lake bottom)	**H** (maximum water height before overflow)	**B** (full width of the water line when H)	**L** (length of the parallelepiped)
V1	C25 to C18	92.36	70	7.7	140.0	217
V2	C18 to C11	94.64	70	6.1	112.0	175
V3	C11 to C5	96.51	75	3.6	96.5	100

Figure 6. Division of the lake volume into 3 sub-volumes.

6 RESULTS

The results presented are summarised in this document but all the data can be provided to the project owner on request.

6.1 EXPLANATION OF MODEL DEVELOPMENT, CALIBRATION AND VALIDATION

Several models were developed for this study:

- The hydrological model was calibrated by comparison with conventional NC values in this type of study. The results were also compared and validated by a sensitivity analysis of this parameter.
- The hydraulic model was calibrated by comparison with friction coefficient values taken from the scientific literature and by a sensitivity analysis of these for the type of hydromorphological characteristics encountered.
- The volumetric model and the evolution of the water level in the lake were elaborated on the basis of a simplified geometrical configuration of the lake while confronting the results obtained with the old available plans and the maximum capacity of the lake.

6.2 CATCHMENT CHARACTERISTICS

The characteristics of the catchment areas are implemented in the hydrological model. They condition the dynamics of rainfall runoff. The catchment areas are in Figure 2 and their characteristics are summarised in Table 3.

Table 3. Characteristics of the catchment areas of interest: surface area, average altitude, Curve Number (CN), Initial Interception (Ia) and time of concentration (Tc)

	Area [km²]	**Average altitude [m]**	**CN [-]**	**Ia [mm]**	**Tc [min]**
Upstream of the lake					
Wayai 1	10.5	475	68	6	278
Le Passage	2.0	396	64	7	160
Wayai 2	5.1	356	71	5	210
Downstream of the lake					
Le Soyereux	1.8	428	69	6	186
La Sauvenière	3.9	445	68	6	385
La Picherotte	4.7	448	68	6	412
Le Barisart	10.1	436	69	6	426

6.3 HYDRAULIC CAPACITY – ANALYSIS OF THE EXISTING

The hydraulic capacity of the identified critical profiles are presented in Table 4. The detailed values of the calculated cross-sections are presented in Appendix 4. The Picherotte and the Barisart respectively accept a maximum flow before overflow of approximately 19 and 18 m³/s.

The hydraulic capacities of the critical sections identified for the Wayai are:

(i) **dpert_W_Rel_05** = 28 m³/s (left arm of the opening blocked)
(ii) **spert_W_Rep_10** = 35.5 m³/s

They are both located upstream of the confluence with the Barisart.

The sensitivity analysis of the parameter K on the value of the hydraulic capacity shows an uncertainty of -20% to +30%. The uncertainty is absorbed by the application of a safety coefficient of 20% in relation to the flow of 35.5 m³/s, which also ensures safety in the event that the left arm of the separate opening is blocked. The hydraulic capacity of the Wayai retained for the calculations is therefore **28 m³/s**.

Table 4. Hydraulic capacity of the Picherotte, Wayai and Barisart channels.

New name	Brook	Hydraulic capacity (m³/s)		
		Left	Right	Total
spert_P_Rel_03	La Picherotte	/	/	**18.7**
spert_B_Rep_04	Le Barisart	/	/	**18.3**
dpert_W_Rel_05	Le Wayai	29.9	**28.0**	57.9
spert_W_Rep_10		/	/	**35.5**

Table 5. Results of the sensitivity analysis of the parameter K on the hydraulic capacity of the Wayai channel on the ***spert_W_Rep_10*** *section. K = 50 is the reference value used for the modelling. The percentages indicate the difference with respect to the reference.*

K = 40	**K = 50**	K = 65
28.4 m³/s	**35.5 m³/s**	46.1 m³/s
- 20%		+ 30%

6.4 HYDRAULIC MODELLING – ANALYSIS OF THE EXISTING SITUATION

The hydrological modelling made it possible to define the maximum modelled volume for each return period and duration (Annex 5).

6.4.1 Results

Considering an empty lake at the beginning of the simulation, the rain duration with the highest volume is selected for each return period (Table 6).

As an indication, the 5-year return period does not lead to any accumulation of water in the lake, as the estimated leakage flow at each time step is greater than the lake inflow. Under these conditions, a rainfall of low severity (5 years) would not cause the level of the park to rise.

Under this management, the lake would overflow between a 50- and 100-year return period flood, with an estimated maximum volume of 359,000 m³ for the 100-year flood, i.e. greater than the lake's capacity of 278,000 m³. The critical return period is therefore situated between the 50-year and 100-year return periods. The current volumetry of the lake does not therefore make it possible to temporise floods with a return period of more than 50 years.

Table 7 summarises the results obtained from the hydrographs associated with each return period. The runoff coefficient may appear to be high for such a catchment area with little urbanisation and woodland. These values can however be explained, on the one hand, by the hydrological group of the soil "C" (low infiltration), on the other hand, by the important average slope which leads to a high NC and, finally, by the taking into account of an initial infiltration coefficient of 0.02 (results of recent studies whereas usually posed at 0.5). It should also be noted that the duration of the rainfall considered in the method is the one that is the most constraining for the considered basin, and therefore leads to the highest runoff coefficient.

Figure 7 to Figure 11 show the evolution over time of the inflow to the lake, the tributary flow, and the allowable leakage flow under the action of a project rainfall of given return period. The lake fills up when the inflow is greater than the outflow, i.e. when the inflow curve QIN (orange curve) exceeds the permissible outflow curve QOUT (yellow curve).

Tableau 6. Durée de la pluie retenue, volume de rétention associé pour chaque période de retour.

Return period	Duration [days]	Retention volume [m³ - rounded to the thousands]
10	2	6000
25	2	95000
50	5	206000
100	5	359000
100 + 30%	5	717000

Table 7. Summary of the results obtained for each return period. Ptot: sum of precipitation over the duration of the rainfall event, Cr: average runoff coefficient ?, QmaxIN: maximum flow at the lake inlet, QmaxAFF: maximum flow of the tributaries, QmaxOUT: maximum admissible leakage flow at the lake outlet.

	10	**25**	**50**	**100**	**100 +30%**
Ptot [mm]	88	102	152	166	216
Cr [-]	38%	42%	53%	55%	55%
QmaxIN [m³/s]	21	28	28	32	42
QmaxAFF [m³/s]	10	13	15	18	23
QmaxOUT [m³/s]	18	15	13	10	5

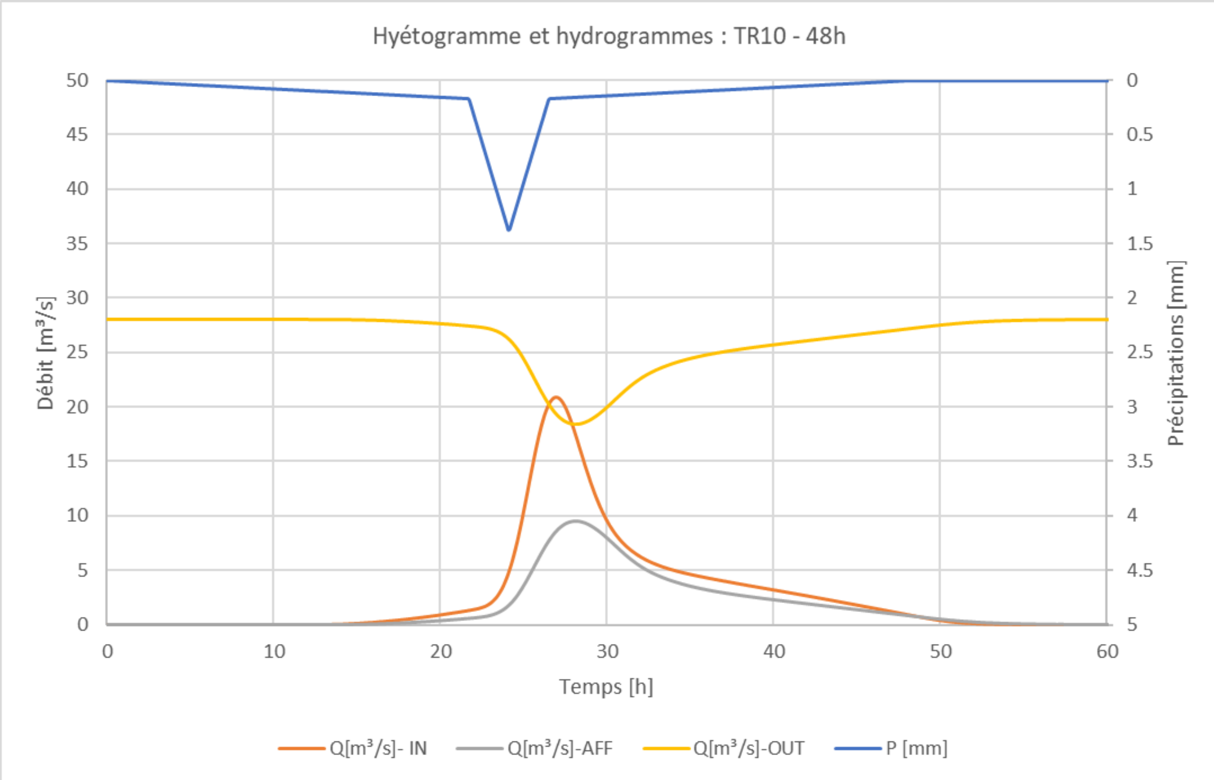


Figure 7. Hyetogram of rainfall and associated flood hydrographs with a 10-year return period and a duration of 48 hours. P: precipitation, QIN: inflow into the lake, QAFF: inflow from tributaries downstream of the lake, QOUT: permissible outflow from the lake.

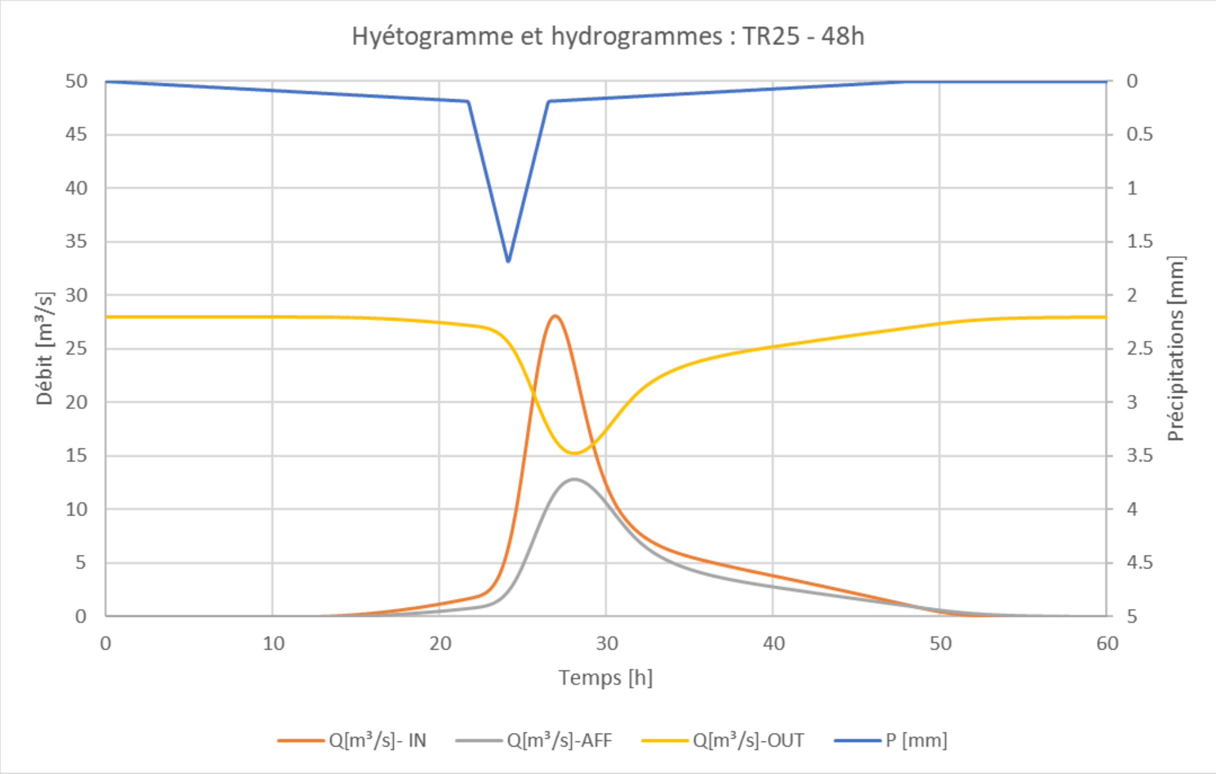


Figure 8. Hyetogram of rainfall and associated flood hydrographs with a return period of 25 years and a duration of 48 hours. P: precipitation, QIN: inflow into the lake, QAFF: inflow from tributaries downstream of the lake, QOUT: permissible outflow from the lake.

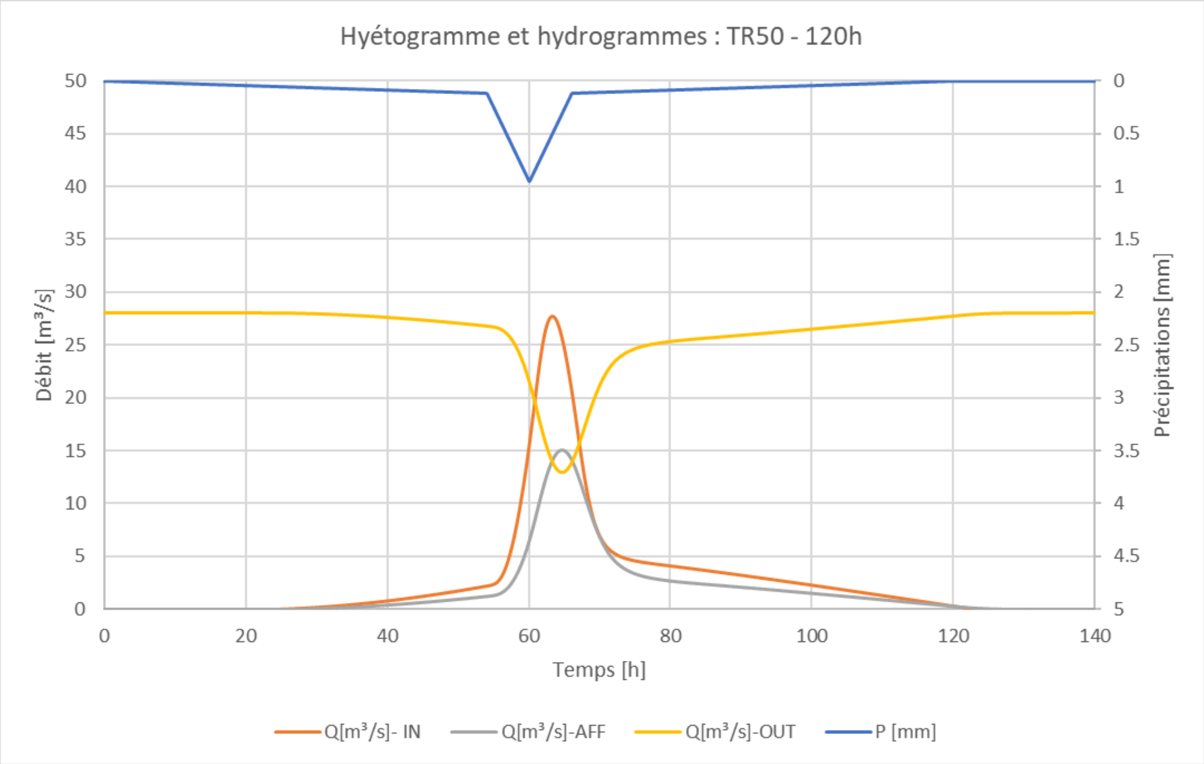


Figure 9. Hyetogram of rainfall and associated flood hydrographs with a 50-year return period and a duration of 120 hours. P: rainfall, QIN: inflow to the lake, QAFF: inflow from tributaries downstream of the lake, QOUT: permissible outflow from the lake.

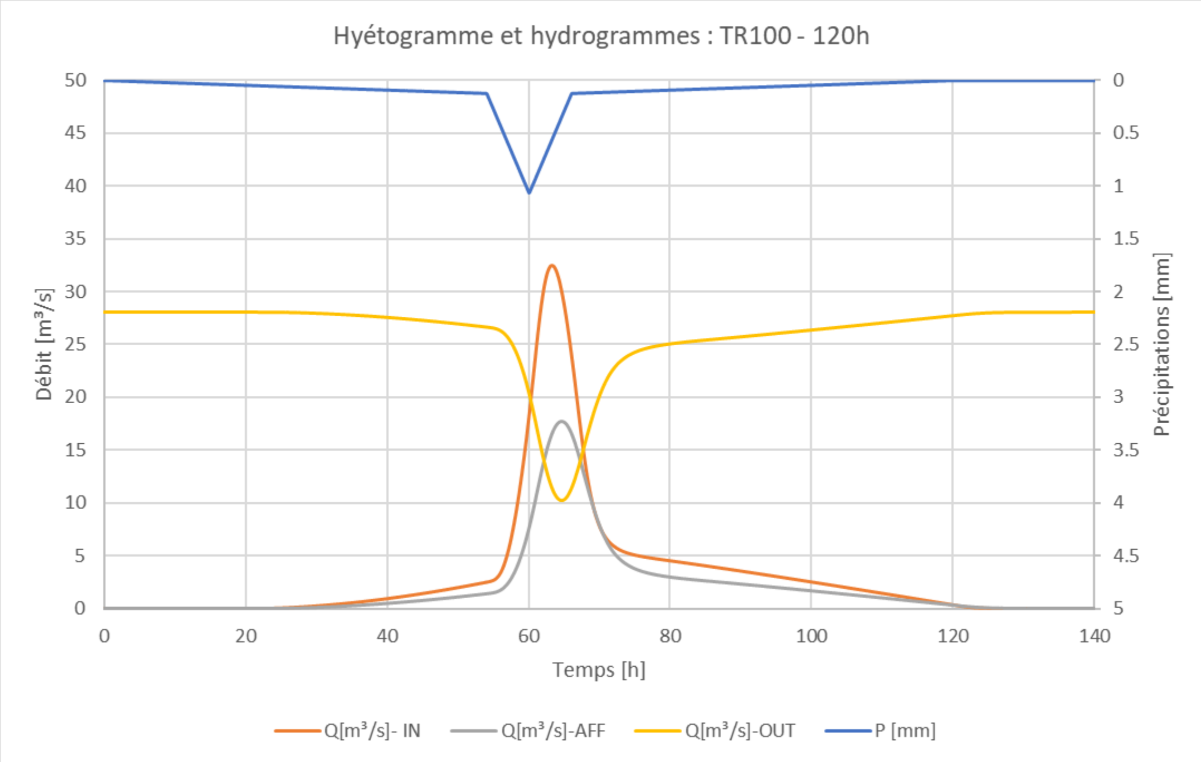


Figure 10. Hyetogram of rainfall and associated flood hydrographs with a return period of 100 years and a duration of 120 hours. P: precipitation, QIN: inflow into the lake, QAFF: inflow from tributaries downstream of the lake, QOUT: permissible outflow from the lake.

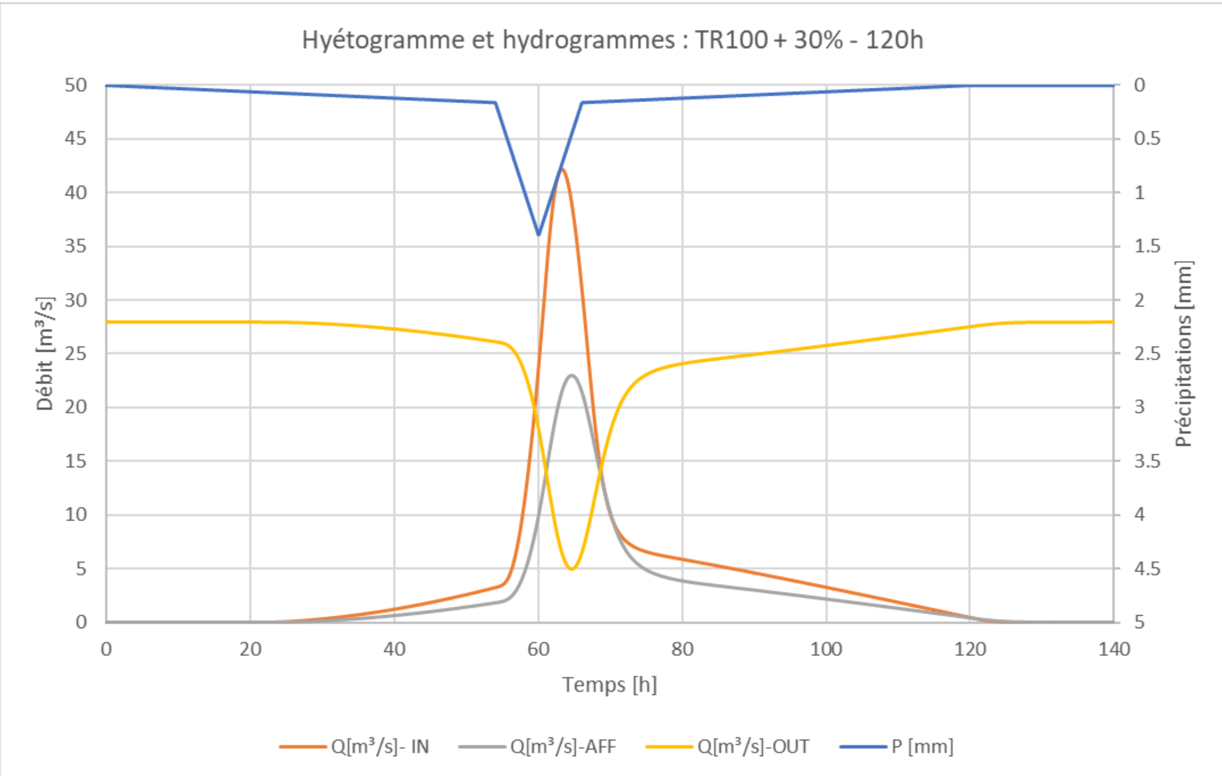


Figure 11. Hyetogram of rainfall and associated flood hydrographs with a return period of 100 years increased by 30% and a duration of 120 hours. P: rainfall, QIN: inflow to the lake, QAFF: inflow from tributaries downstream of the lake, QOUT: allowable outflow from the lake.

6.4.2 Validation and sensitivity analysis

Table 8 shows the comparison between the reconstructed flow at the Spixhe station, extrapolated to the lake inlet by the watershed ratio, and the simulated flow at the lake inlet. This reconstitution of the flows is an approximation, but nevertheless gives an idea of the order of magnitude of the flows expected at the lake inlet. The flows are much larger in the case of the hydrological simulation we have carried out. However, the model is probably more appropriate as it takes into account the actual characteristics of the basin of interest and avoids the assumption of similar catchments required by the use of a catchment ratio.

The sensitivity analysis of the NC presented in Table 7 shows the influence of the NC on the obtained volume values. The sensitivity of the model to the NC remains significant. Nevertheless, it decreases drastically for large return periods, reaching a deviation of about -20% to +10% for the maximum flood considered.

Table 8. Comparison between the flood flows calculated on the basis of the monitoring of the Spixhe station and extrapolated to the lake inlet and the flow simulated by the present study at the lake inlet.

TR	Reconstituted flow rate [m³/s] –lake outlet	Simulated flow rate [m³/s] – lake outlet	Difference
5	11.9	16.0	35%
10	15.1	20.9	38%
25	19.2	28.1	46%
50	22.3	34.2	54%
100	25.3	41.0	62%

Table 9. Results of the sensitivity analysis of CN on lake volume.

	CN – 2 (%)	CN – 2 (m3)	**CN** (m3)	CN + 2 (m3)	CN + 2 (%)
5	0%	0	**0**	0	0%
10	-100%	0	**6000**	18000	+200%
25	-30%	67000	**95000**	125000	+30%
50	-20%	168000	**206000**	247000	+20%
100	-20%	296000	**359000**	410000	+15%
100 + 30%	-17%	596000	**717000**	792000	+10%

6.5 VOLUME - HEIGHT EVOLUTION - ANALYSIS OF THE EXISTING SITUATION

The evolution of the volume and height of water in the lake for each return period are represented in Figure 12 to Figure 17, noting that the scales of the Y axes vary from one figure to another to ensure a representation adapted to the volume and height values.

It should also be remembered that the water height is referenced from altitude DNG 92.36 m (reference level considered for the lake bottom) whereas the lake is at maximum capacity above altitude DNG 100.00 m (maximum filling level considered on the basis of the plans transmitted). The height evolution graphs are capped at this height considered as maximum before overflow. It is possible that an additional margin exists beyond this plan reference.

Based on the figures below, the lake overflows and exceeds the 100.00 m altitude between the 50- and 100-year return periods (Figure 15).

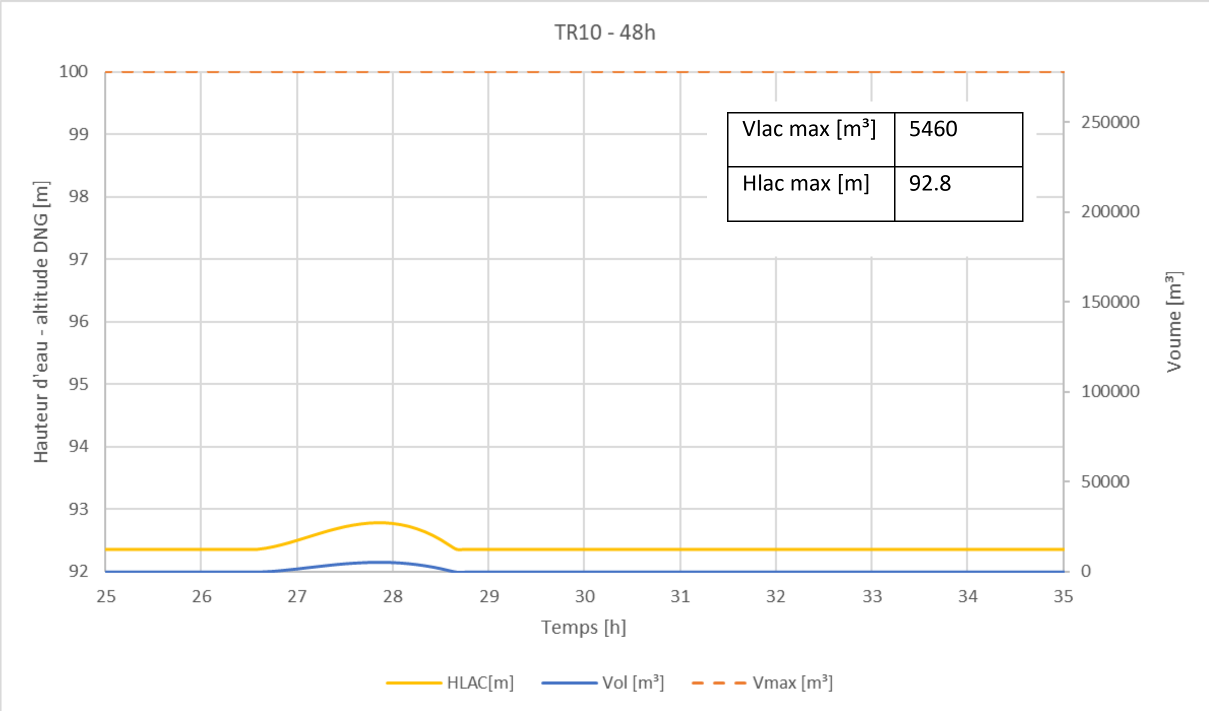


Figure 12: Evolution of the volume and height of the lake for the 10-year return period and a duration of 48 hours. Vmax represents the maximum capacity of the lake.

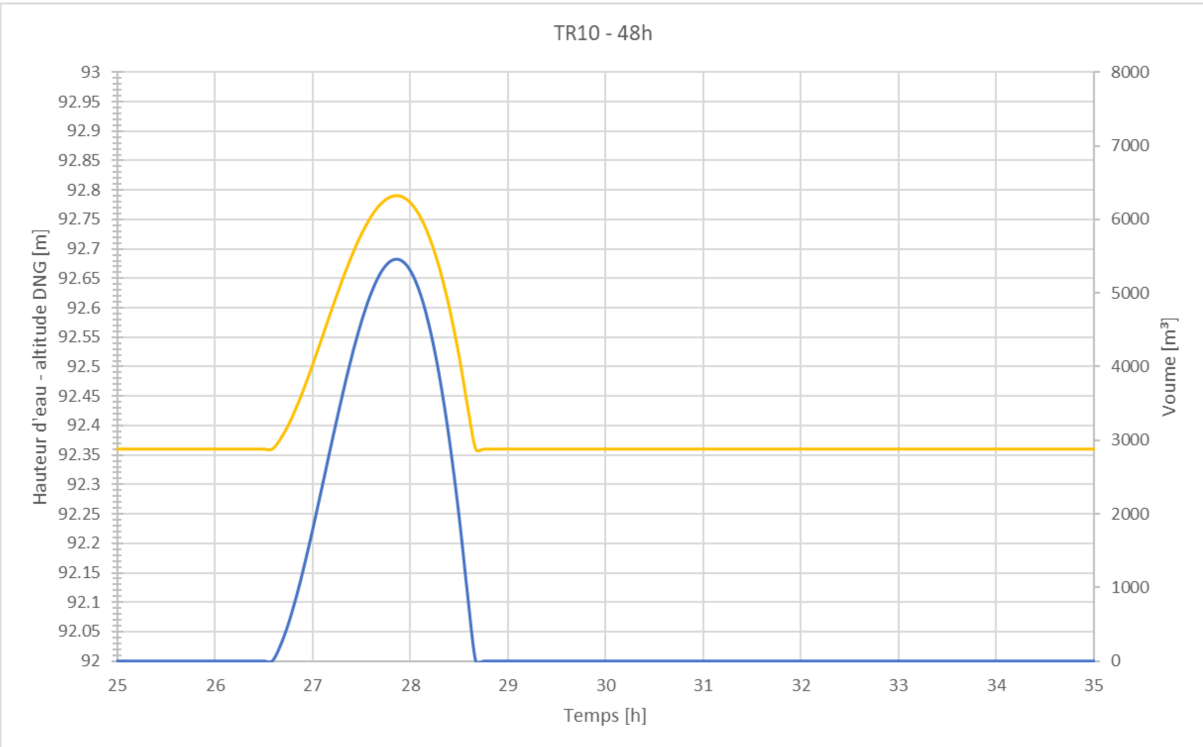


Figure 13. Zoom on the evolution of the volume and the water height of the lake for the return period of 10 years and a duration of 48 hours. Vmax represents the maximum capacity of the lake.

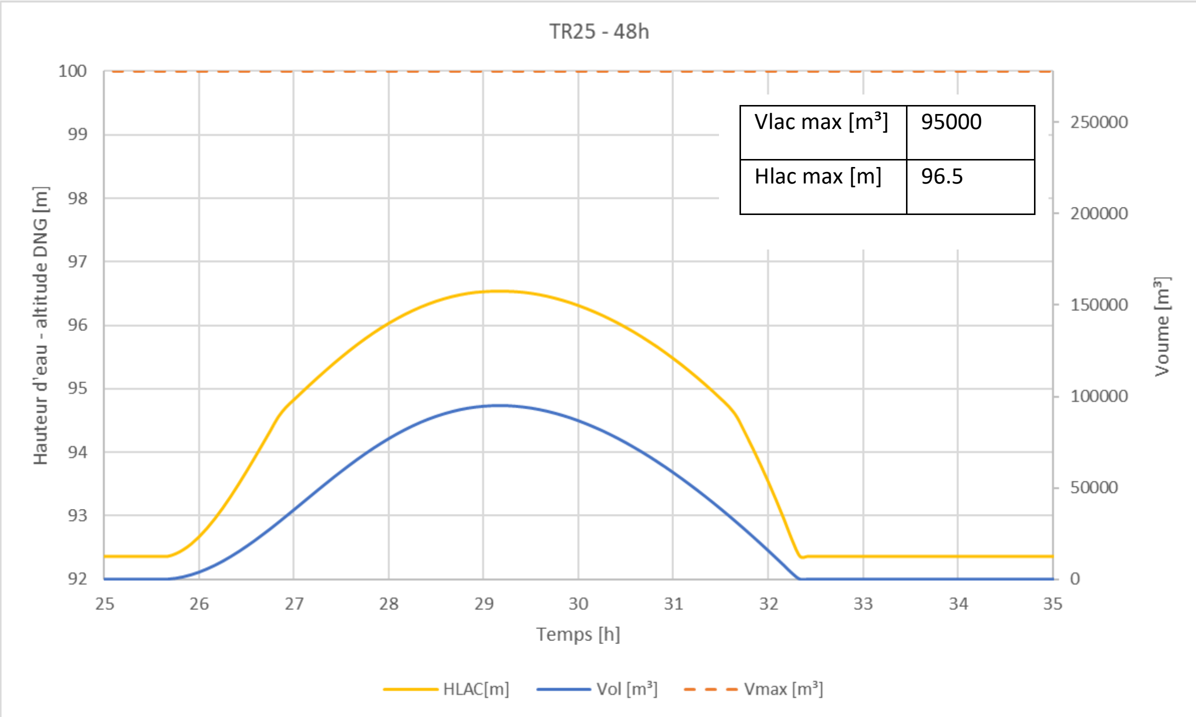


Figure 14. Evolution of the volume and water height of the lake for the return period of 25 years and a duration of 48 hours. Vmax represents the maximum capacity of the lake.

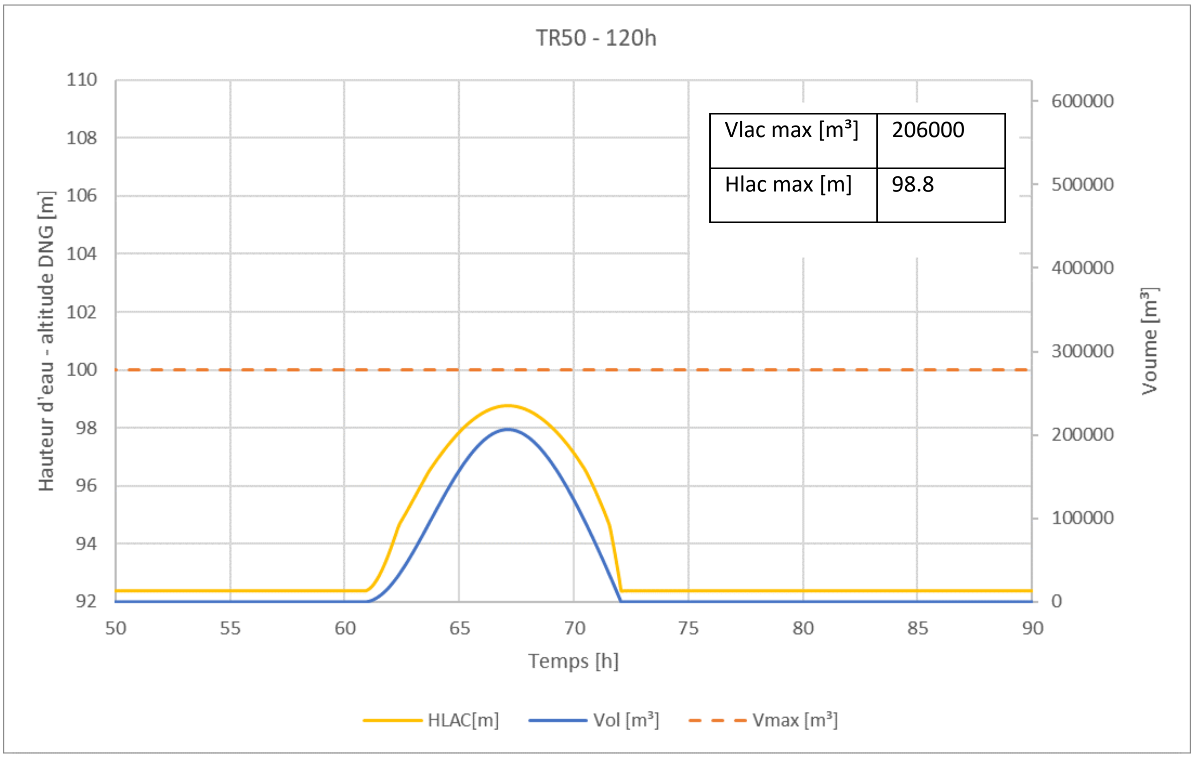


Figure 15. Evolution of the volume and water height of the lake for the return period of 50 years and a duration of 120 hours. Vmax represents the maximum capacity of the lake.

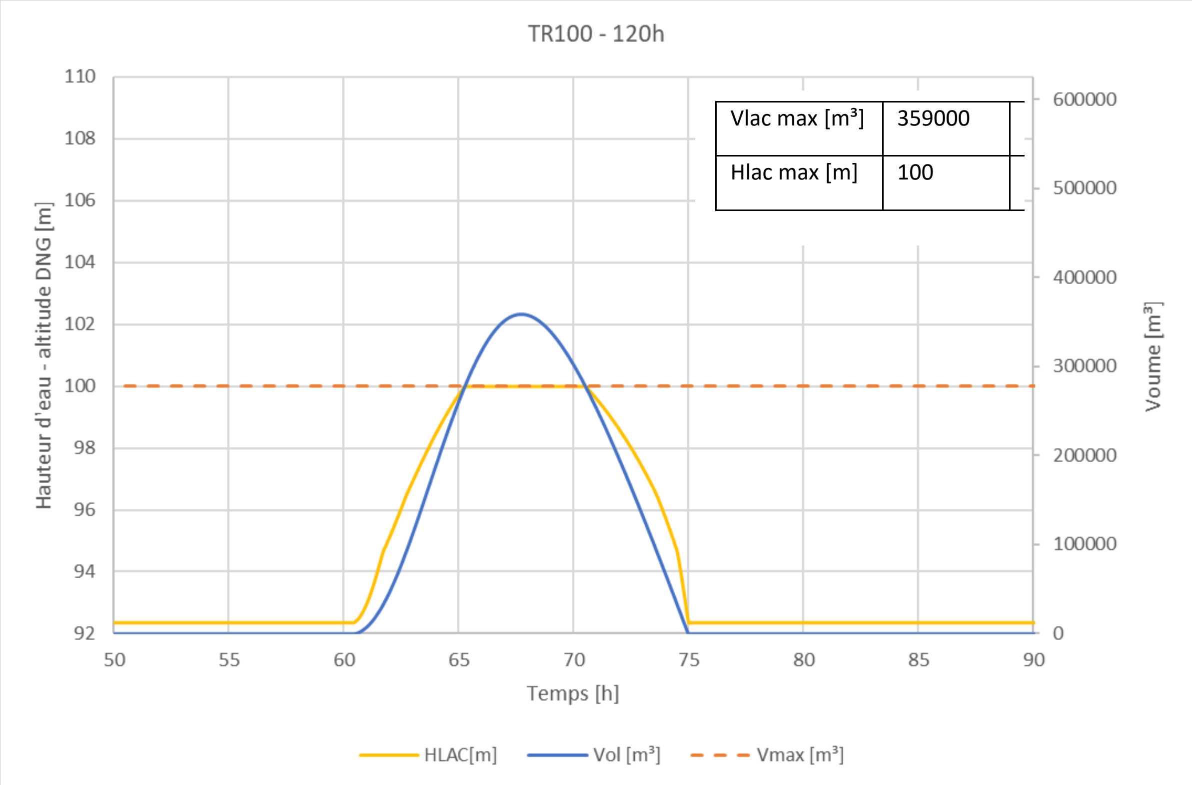


Figure 16. Evolution of the volume and water height of the lake for the return period of 100 years and a duration of 120 hours. Vmax represents the maximum capacity of the lake.

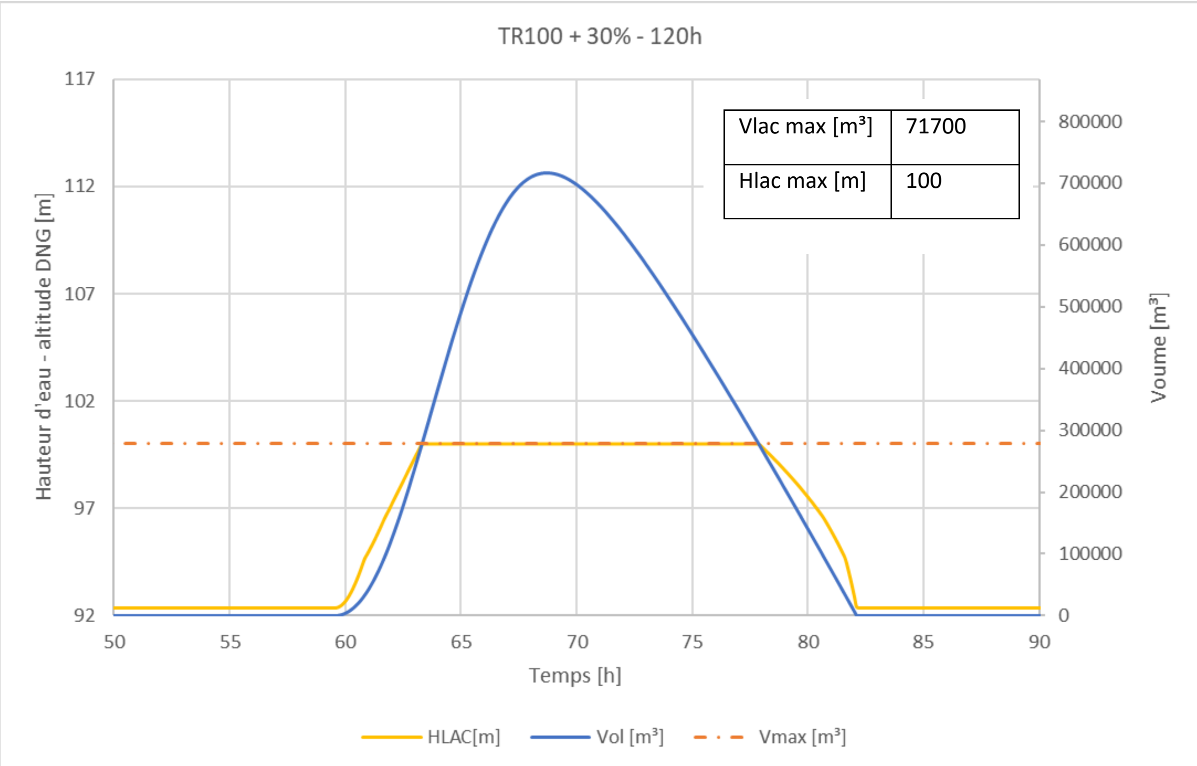


Figure 17. Evolution of the volume and water height of the lake for the return period of 100 years increased by 30% and a duration of 120h. Vmax represents the maximum capacity of the lake.

7 LIMITATIONS OF THE STUDY AND POINTS OF ATTENTION

7.1 HYDRAULIC CAPACITY OF THE OPENINGS

For safety reasons, the hydraulic capacity of the openings has been reduced by 20% in the case where no openings are obstructed. However, floods could generate logjams or significant sedimentation which reduce the hydraulic capacity of the openings (such as the left arm partially blocked in the spa under the tourist office during the inspection after the flood of July 2021). This security corresponds approximately to the capacity of one of the two arms of the above-mentioned double sluice. It is therefore important to limit the arrival of logjams in the underground areas as much as possible and to carry out regular maintenance of the network.

It should also be remembered that this initial estimate is based on the simplified Manning Strickler flow equation and cannot replace a complete hydraulic model.

Finally, the calculation of the hydraulic capacity of the openings is based mainly on sections dating from 2001 and 1967, and certain modifications during the various rainfall events as well as their obsolescence may have modified their geometry.

7.2 HYDROLOGICAL MODELLING

Hydrological models are based on a simplified representation of reality, leading to numerous assumptions on runoff dynamics, which cannot always be verified (calibration). Some hydrological phenomena are sometimes overestimated, underestimated or neglected (such as in this study evapotranspiration, rainfall spatialization, interception by vegetation etc.). For example, rainfall was considered to be uniform over each sub-catchment in the hydrological model used in this study. The analysis of the actual rainfall behaviour shows, however, a heterogeneous distribution of precipitation on the sub-catchments which influences the discharge at the outlet (depending on the distance between the core of the precipitation and the outlet). From the point of view of a first approach, the assumption of uniformity of rainfall is acceptable in view of the surface area of the sub-catchments studied.

Finally, the sensitivity analysis of the NC showed a significant impact on the calculated lake volumes. The calculation of the NC value is based on the most recent data on land use and hydrological groups, but is still subject to interpretation.

7.3 VOLUME AND HEIGHT OF THE LAKE

Some assumptions were made for the assessment of the volume and height of the lake, such as

- The lake is considered empty at the arrival of the rain: if the lake is not empty at the arrival of the rain, it is necessary to revise the proposed estimates based on the defined tidal range.
- The lake is considered to be cleaned to the project level specified on the 1978 lake drainage plans.
- The maximum leakage rate from the lake is optimised and variable over time. From a management point of view, the adaptation of the leakage flow in real time can be complicated. In the event that the leakage rate cannot be monitored in real time, it is important to redefine

the flood hydrographs and associated volumes and to define how the lake leakage rate should be managed.

- The height of the lake depends on the approximate volumetry of the lake, which is based on sections dating from 1978. The volumetry can be refined by a complete volumetric modelling of the lake.

The present study is therefore an indicative study and TER-Consult cannot be held responsible for any discrepancy with reality. The proposed results are verified on the basis of the elements at our disposal and their interpretation is intended to guide the project owner in the choice of his developments.

8 CONCLUSIONS AND IDEAS FOR RELEVANT SOLUTIONS

The aim of the study was to predict the retention capacity required for Lake Warfaaz in order to control flooding and the evolution of the volume and level of water in the lake, taking into account the severity of several rainfall events: a return period of 5 years, 10 years, 25 years, 50 years and 100 years, as well as a rainfall of 100 years increased by 30%.

The study showed an overflow of the lake from a rainfall event with a return period between 50 and 100 years. The Warfaaz lake could therefore constitute a useful protection structure for the temporisation of 5, 10, 25 and 50 year floods, with a leakage flow monitored in real time and an empty lake when the rain arrives. Under these conditions, the following objectives are achieved: (i) the lake does not overflow, (ii) the risk of flooding is limited downstream.

The ecological component of the lake must nevertheless be taken into account by the manager and a less significant tidal range (a few metres) could be taken into consideration to temporise the floods. The estimates will therefore have to be revised according to the desired tidal range and the ecological constraints of the lake (aquatic environment).

Considering that a safety margin has already been taken on the calculations of the critical flow before overflow for an unobstructed network (point 5.2) and that the configuration of the volume of the lake has been simplified to a parallelepiped with a trapezoidal base, the most important uncertainty remains in the hydrological model. In view of the sensitivity of the hydrological modelling results to variations in NC, we propose to apply a final safety factor of 20% to the final lake volume. In this framework, the volume of 206,000 m3 to be stored in TR50 becomes 247,000 m3 which remains acceptable in view of its total estimated storage capacity (278,000m3).

It is now important to define the ambition of the project, i.e. for what severity of rainfall event the lake will serve as a retention volume, which will condition the level of water in the lake outside of flood times. In addition, the method of managing the lake's leakage flow must also be defined.

Finally, if the severity of the rainfall is beyond the chosen return period, this implies either an overflow of the lake or an overflow of the tributaries. In this case, it is important to define risk reduction measures.

9 ANNEXES

Annex 1. List of sections used and their names used in this study for the Wayai, Picherotte and Barisart channels

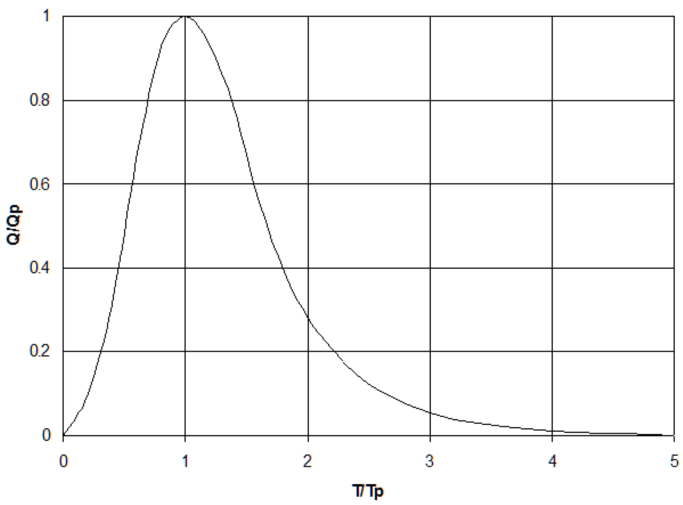
Section type	Watercourse	Source	Description	Source name	New name
single opening	picherotte	Surveys 2001	entry into opening	P9	**spert_P_Rel_01**
single opening	picherotte	Surveys 2001	into opening	P7	**spert_P_Rel_02**
single opening	picherotte	Surveys 2001	near confluence with Wayai	P2	**spert_P_Rel_03**
single opening	Wayai	Surveys 2001	single opening downstream of the Picherotte confluence	W49	**spert_W_Rel_01**
double opening	Wayai	Surveys 2001	double opening that splits into two arms	W44	**dpert_W_Rel_02**
double opening	Wayai	Surveys 2001	double opening that splits into two arms	W42-43	**dpert_W_Rel_03**
double opening	Wayai	Surveys 2001	double opening that splits into two arms	W40-41	**dpert_W_Rel_04**
double opening	Wayai	Surveys 2001	double opening that splits into two arms	W39	**dpert_W_Rel_05**
double opening	Wayai	Surveys 2001	start of adjacent double openings	W37	**dpert_W_Rel_06**
double opening	Wayai	Surveys 2001	adjacent double openings	W36/36a	**dpert_W_Rel_07**
double opening	Wayai	Surveys 2001	end of adjacent double openings	W32	**dpert_W_Rel_08**
single opening	Wayai	Repairs 2019	single opening with guniting	Stretch II Section 24-24	**spert_W_Rep_09**
single opening	Wayai	Repairs 2019	single opening repaired	StretchI Section C-C'	**spert_W_Rep_10**
single opening	Wayai	Repairs 2019	single opening repaired	Stretch I Section A-A'	**spert_W_Rep_11**
double opening	Wayai	Surveys 2021	double opening downstream of the Barisart confluence	W15	**dpert_W_Rel_12**
double opening	Wayai	Surveys 2021	double pertuis aval confluence Barisart	W14	**dpert_W_Rel_13**
double opening	Wayai	Repairs 2019	double pertuis aval confluence Barisart	Section A- A	**dpert_W_Rep_14**
double opening	Wayai	Surveys 2021	double opening park exit, under the road	W4-5	**dpert_W_Rel_15**
double opening	Wayai	Surveys 2021	double opening park exit, end gate	W1	**dpert_W_Rel_16**
bridge	Wayai	Atlas 1967	bridge under Av. Reine Astrid	NA	**pon_W_Atl_17**
Bridge	Wayai	Atlas 1967	bridge under Chem. De la Fagne Raquet	NA	**pon_W_Atl_18**

single opening	Barisart	Repairs 2022	upstream elbow	BB	**spert_B_Rep_01**
single opening	Barisart	Repairs 2022	downstream elbow	AA	**spert_B_Rep_02**
single opening	Barisart	Repairs 2022	downstream elbow	BB	**spert_B_Rep_03**
single opening	Barisart	Repairs 2022	downstream 2nd bend	Cross-section 11	**spert_B_Rep_04**
single opening	Barisart	Repairs 2022	downstream 2nd bend	Cross-section 12	**spert_B_Rep_05**
single opening	Barisart	Surveys2001	near confluence with Wayai	VS6	**spert_B_Rel_06**
single opening	Barisart	Surveys 2001	just before Wayai confluence	VS1	**spert_B_Rel_07**

Annex 1. CN values used according to land use and hydrological groups and Manning's coefficient values according to land use.

Land use (WALOUS 2019)	TABLE CN SCS	Known name	A	B	C	D	Roughness coefficient
No data		85	85	85	85	85	0.03
Artificial floor covering		98	98	98	98	98	0.013
Artificial structures above ground		98	98	98	98	98	0.013
Railway network		90	90	90	90	90	0.033
Bare floors	fallow (bare soil)	87	77	86	91	94	0.03
Surface water	water	100	100	100	100	100	0.04
Herbaceous cover in rotation during the year (e.g. annual crop)	favourable temporary grassland	74	58	72	81	85	0.035
Year-round grass cover	permanent grassland	59	30	58	71	78	0.04
Softwoods (>3m)	favourable forest	57	25	55	70	77	0.1
Softwoods (<3m)	unfavourable forest	68	45	66	77	83	0.05
Hardwoods (>3m)	favourable forest	57	25	55	70	77	0.1
Hardwoods (< 3m)	unfavourable forest	68	45	66	77	83	0.05

Annex 2. Illustration of the SCS unit hydrograph. Q (flow), Qp (peak flow), T (time), Tp (time of peak)



Annex 3. Results of the hydraulic capacity calculations for the sections considered. In red the hydraulic capacities retained for each watercourse (Wayai, Picherotte and Barisart). Left = left section if double; Right = right section if double; total = single section or sum of both.

New name	Simplified geometry considered	Hydraulic capacity (m^3/s)		
		Left	Right	Total
spert_P_Rel_01	half ellipse	/	/	20.7
spert_P_Rel_02	half ellipse	/	/	19.1
spert_P_Rel_03	half ellipse	/	/	18.7
spert_W_Rel_01	half ellipse	/	/	88.8
dpert_W_Rel_02	half ellipse	43.6	33.0	76.5
dpert_W_Rel_03	half ellipse	41.5	45.5	87.0
dpert_W_Rel_04	half ellipse	32.3	40.7	72.9
dpert_W_Rel_05	half ellipse	29.9	**28.0**	57.9
dpert_W_Rel_06	half ellipse	21.0	25.5	46.4
dpert_W_Rel_07	half ellipse	34.9	28.4	63.3
dpert_W_Rel_08	half ellipse	21.2	21.2	42.4
spert_W_Rep_09	rectangle	/	/	45.5
spert_W_Rep_10	rectangle	/	/	35.5
spert_W_Rep_11	rectangle	/	/	51.0
dpert_W_Rel_12	half ellipse	40.8	40.0	80.8
dpert_W_Rel_13	half ellipse	40.3	41.8	82.1
dpert_W_Rep_14	rectangle	41.1	23.8	64.9
dpert_W_Rel_15	half ellipse (g) and half ellipse + rectangle (d)	31.5	30.5	62.0
dpert_W_Rel_16	half ellipse	35.8	49.6	85.4
pon_W_Atl_17	semi-circle	47.2	47.2	94.5
pon_W_Atl_18	rectangle	34.8	34.8	69.5
spert_B_Rep_01	rectangle	/	/	26.8
spert_B_Rep_02	rectangle	/	/	42.4
spert_B_Rep_03	rectangle	/	/	34.9
spert_B_Rep_04	half ellipse	/	/	18.3
spert_B_Rep_05	rectangle	/	/	28.6
spert_B_Rel_06	half ellipse	/	/	45.5
spert_B_Rel_07	half ellipse	/	/	44.0

Annex 4. Retention volume [m³ - rounded to thousands] required for each return period and duration (h for hours, d for days). The maximum volume retained for each return period is highlighted in yellow.

	3h	**6h**	**12h**	**24h**	**2d**	**3d**	**4d**	**5d**	**7d**	**10d**	**15d**	**20d**	**25d**	**30d**
5	0	0	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	6000	2000	0	0	0	0	0	0	0	0
25	0	0	1000	29000	95000	93000	84000	84000	38000	0	0	0	0	0
50	4000	7000	52000	105000	204000	205000	200000	206000	148000	70000	0	0	0	0
100	58000	61000	146000	215000	343000	346000	344000	359000	295000	200000	29000	0	0	0
100 + 30%	163000	176000	327000	435000	632000	648000	666000	717000	676000	605000	349000	148000	10000	0