Hydrological and hydraulic study of the "Haveignée" stream in Trooz to improve the crossing of the village of Fraipont.

Interreg
Euregio Meuse-Rhine
EMFlood Resilience

EUROPEAN UNION
European Regional Development Fund

Contracting authority: Province de Liège

Place Saint Lambert, 18A. 4000 Liège

Contact : bruno.khuatduy@provincedeliege.be

Company number: BE0207 725 104

Framework of the study: Project INTERREG – EMR228 -

Realisation: TER-Consult SPRL
Rue de Serbie, 78/2. 4000 Liège
Contact: burton@ter-consult.be
Company number: BE0648 631 872
Bank account number: BE45 5230 8079 2789

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

Review	Date	Version	Writing
Rev 0	20/04/23	Creation document	LP/CBU
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1 CONTEXT AND OBJECTIVES

The Province of Liège has commissioned TER-Consult SPRL to carry out a hydrological and hydraulic study aimed at improving the crossing of the Haveignée stream in Fraipont by adapting (i) the dimensions of the channel under the Route du Village and Place Emile Vandervelde and (ii) the course of the Haveignée and opening up the part of the channel under the houses (**Figure 4**).

This request follows the deterioration of the channels caused by the exceptional rainfall event that took place from 13 to 15 July 2021, as well as the regular overflowing of the course of the stream. As for the open-cut section, it was envisaged that the residents would no longer have the stream channelled under their homes, restore the hydromorphological quality of this sector and improve the hydraulic conditions of the watercourse.

The aim of the hydraulic study we are proposing is to analyse the existing situation and to characterise, for several return periods, the projected situation by sizing the opening under the Route du Village and Place Emile Vandervelde and the open-cut section.

The purpose of the hydrological study is to reconstruct the flow rates of the Haveignée stream and the Vesdre in the vicinity of the project area.

The absence of flow measurements on the Haveignée stream meant that hydrological modelling had to be carried out to determine the amount of water coming from this tributary, using the Soil Conservation Service (SCS) method. The hydrological data for the Vesdre used in this study is based on the Chaudfontaine measuring station (ref 6228).

A number of preliminary discussions with the project owner guided the definition of the study (methodology, scope, etc.) and the return periods to be considered, the peak flows of which were recalculated on the basis of available hourly data. These discussions also helped to gather the basic data used in this study.

The return periods considered are 2-5-10-25-50-100 years.

2 LOCATION OF THE STUDY AREA, THE PROJECT AREA AND THE PERIMETER OF THE HYDRAULIC STUDY

The project area concerns the section of the watercourse where the works are planned, i.e. the Haveignée stream, from the railway bridge to the confluence of the stream and the river Vesdre (**Figure 3** and **Figure 4**).

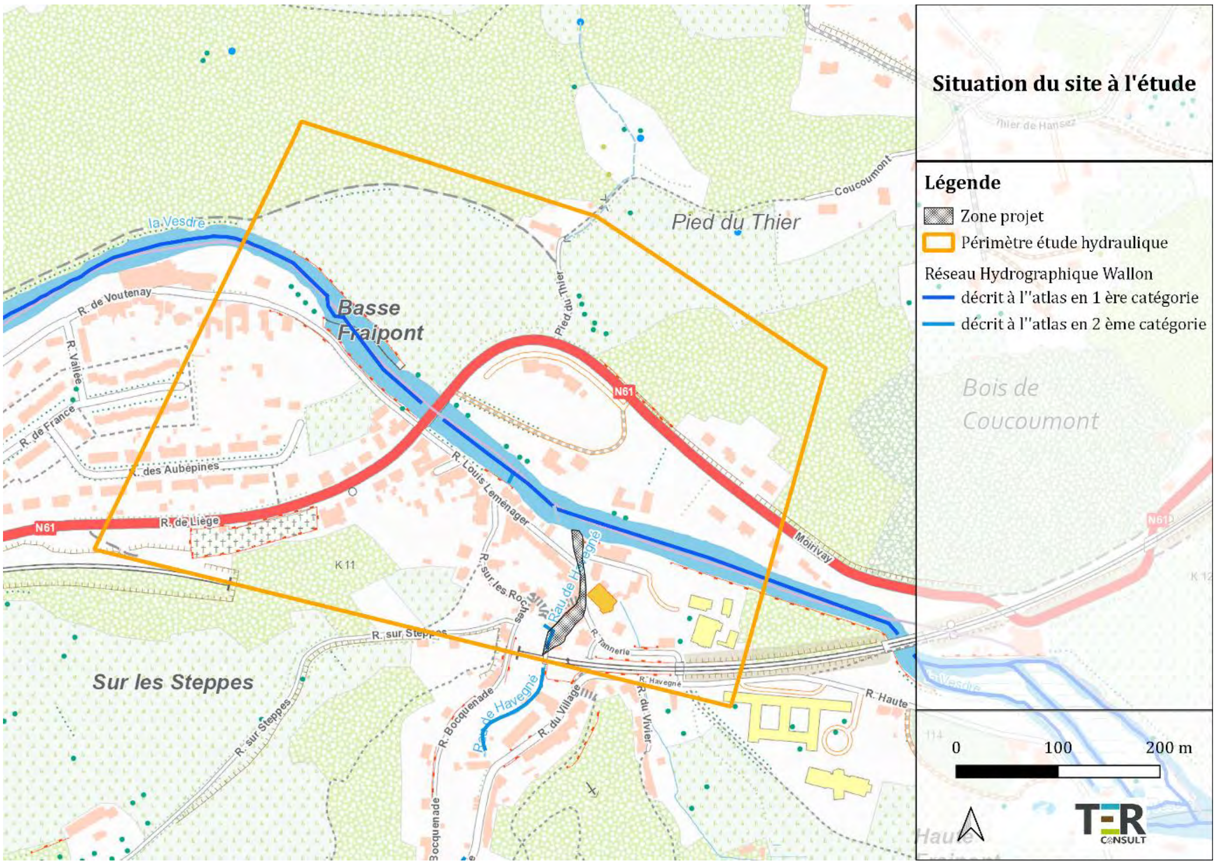
However, in order to take a broader view of the problem, we can define the perimeter of the hydraulic study as follows (Figure 1). From the meander of the Vesdre downstream of the weir at rue de Voutenay to the railway bridge upstream on the Haveignée stream. Part of the river Vesdre has been taken into account in the hydraulic model, as it has a significant impact on the flow within the project area (water rises during floods through the opening in the stream). The downstream limit is therefore located more than 450 m downstream of the project area in order to be as free as possible from the downstream constraints of the model.

Transversal to the watercourse, the boundaries of the study area are determined on the basis of the terrain (DTM) and flood zone maps provided by the SPW.

Within the project area, the Haveignée stream is open-cut from the railway bridge to plot 130H. Thereafter, it is channelled into the river Vesdre with various channel profiles.

Figure 2 shows the extent of the perimeter considered for the hydrological study of the Haveignée stream. In other words, the catchment area whose outlet is the inlet to the hydraulic model.

Figure 1 : Location of the project area, the hydraulic study perimeter and the Walloon hydrographic network (source: geoportail.wallonie.be).



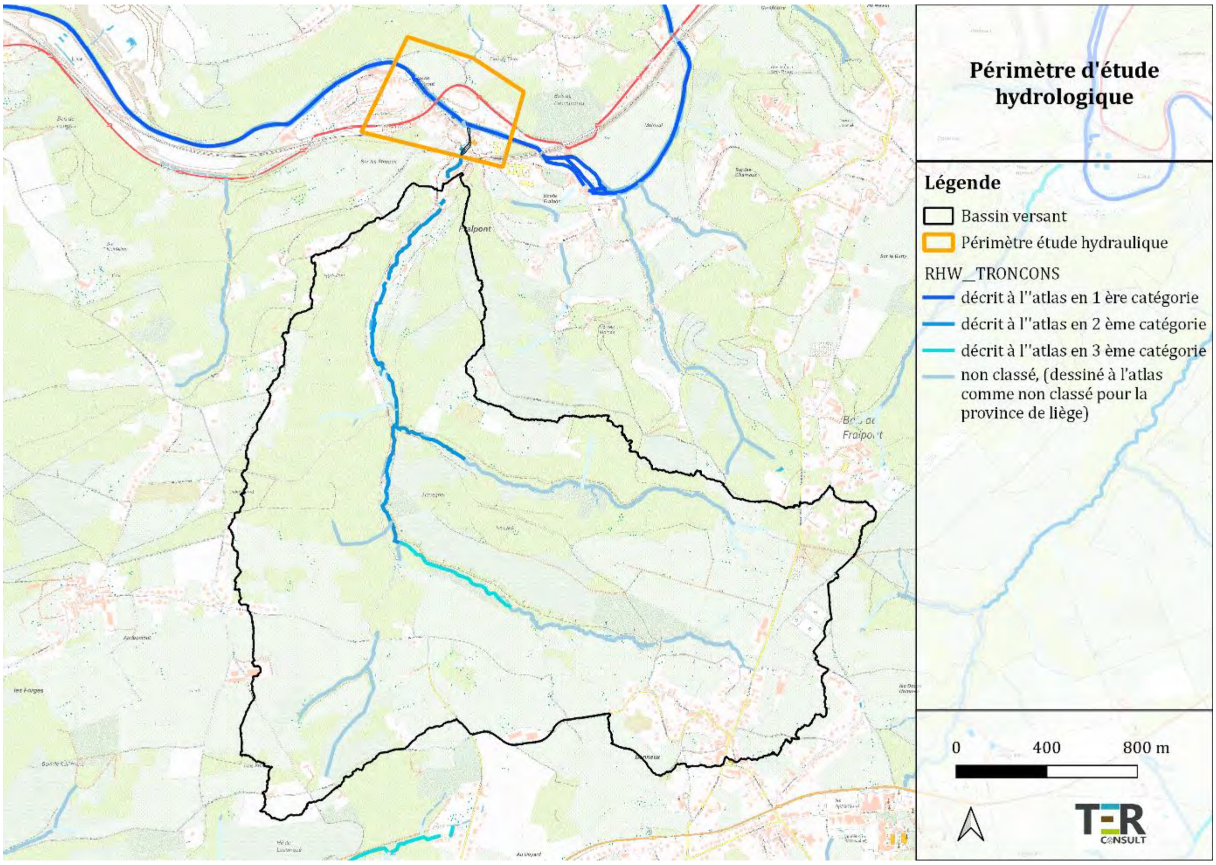


Figure 2 - Scope of the hydrological study (catchment area taking the inlet of the hydraulic model as its outlet)

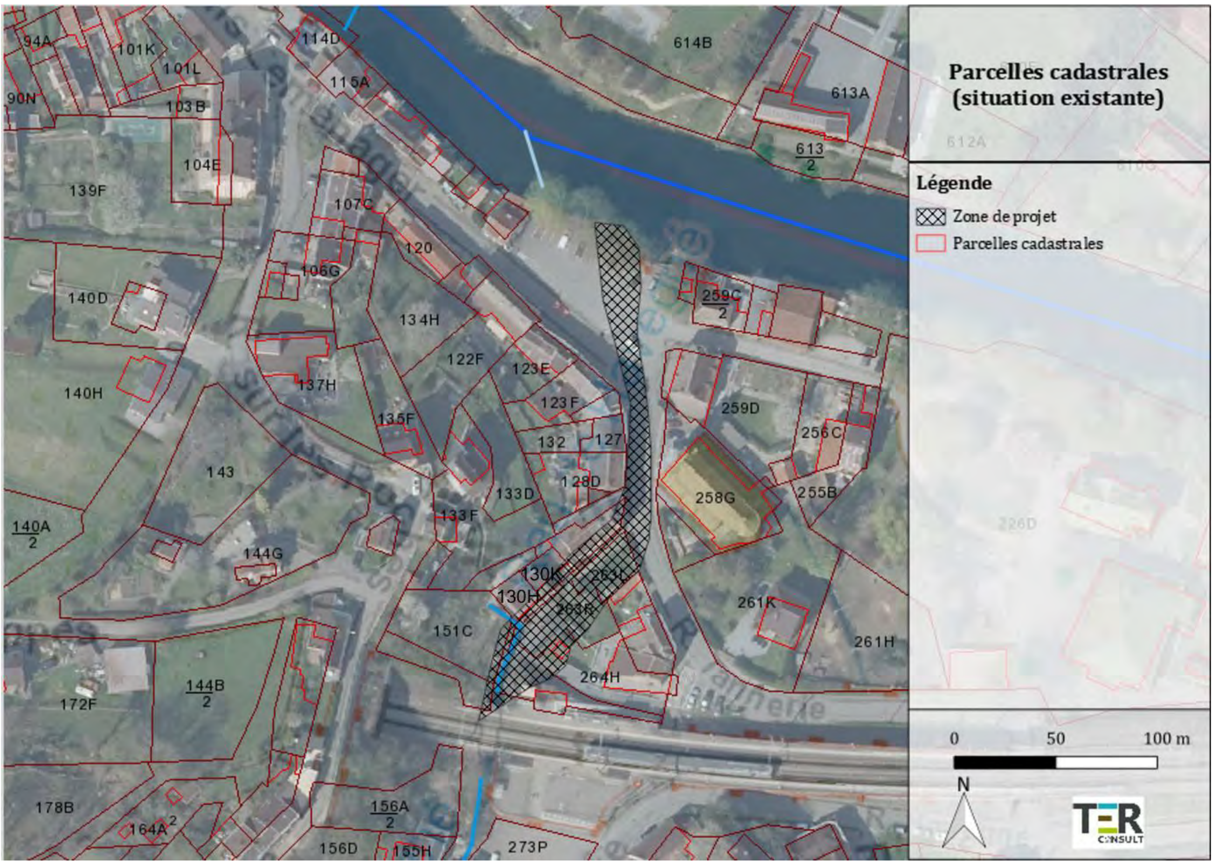


Figure 3 - Cadastral parcels in the project area

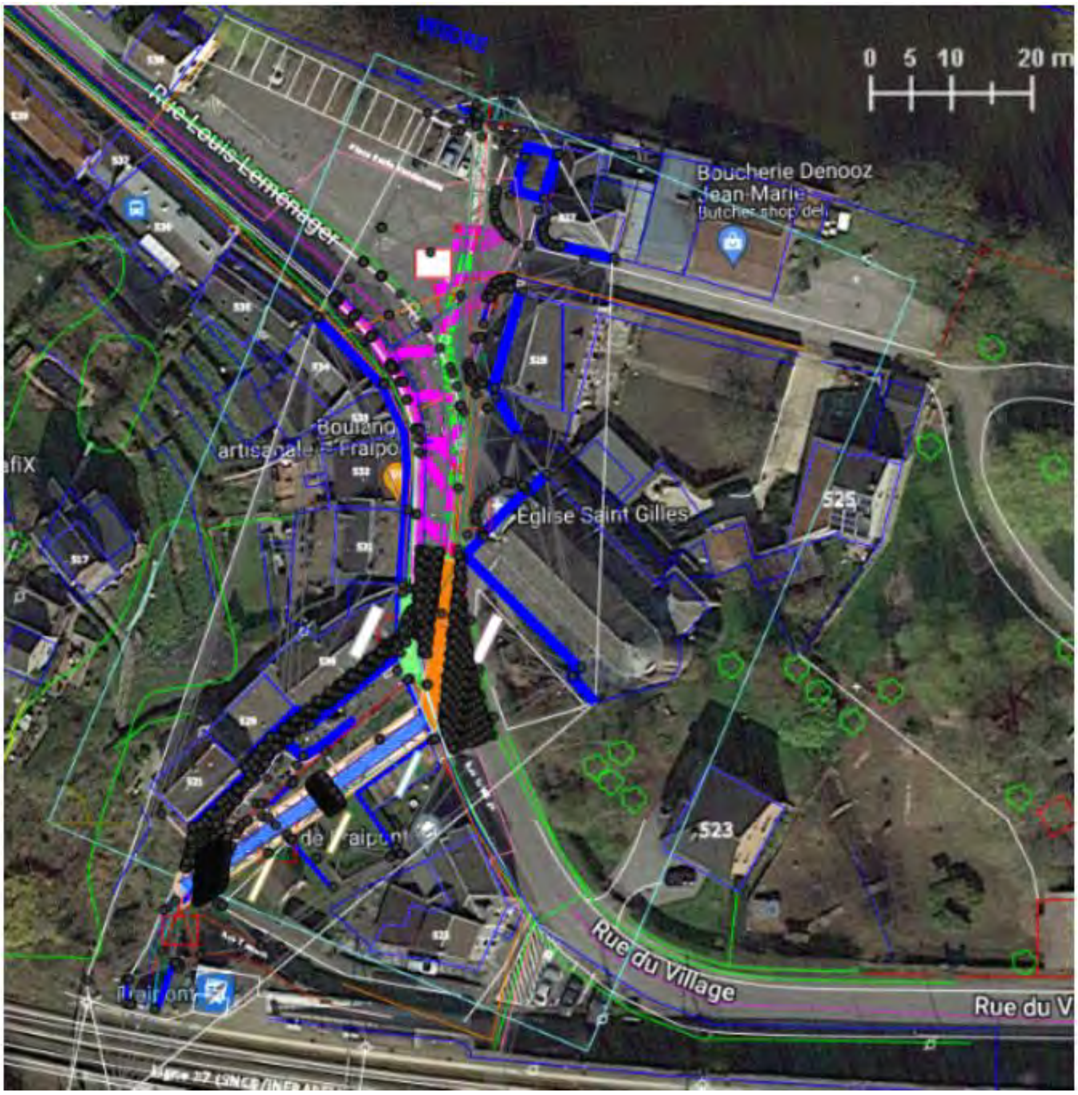


Figure 4 : Overview of the planned project: relocation of the upstream section and opening up and resizing of the downstream opening

3 METHODOLOGY

3.1 AVAILABLE DATE

3.1.1 General information

Figure 5 shows the Walloon Hydrographic Network and proposes a modelled approach to preferential runoff axes based on Walonmap's LIDAXE tool. Based on a model developed in collaboration with the SPW, these data remain indicative, but show a number of elements of interest to us:

1. A concentrated runoff axis draining 20 to 50 ha at the river Vesdre meander where the model ends downstream.
2. A concentrated runoff axis draining 20 to 50 ha and flowing into the river Vesdre upstream of the hydraulic model perimeter.
3. There are no runoff features referenced in the project area.

The project area is located in a low to high hazard zone (**Figure 6**), with disparities across the zone.
A classification map of water heights and velocities is available for return periods of 25, 50 and 100 years (**Figure 8**). It shows that the flood-prone areas are more extensive on the left bank in the upstream part of the hydraulic perimeter and on the right bank in the downstream part, where a flatter area appears after the N61 bridge and visible overflows upstream of the bridge.

Upstream, the left bank flows have a direct impact on the project area for all 3 return periods. Within the project area, there is a zone with a water height of 0 to 0.3 m and a zone of 0.3 to 1.3 m that will extend over the return periods, covering almost the entire project area for the 100-year return period.
The differences from one return period to the next within the hydraulic study perimeter are mainly visible in the form of the extent of the 0.3 to 1.3m zone for increasingly long return periods, for example: upstream of the project area along the left bank of the river Vesdre, downstream of the project area on the right bank of the river Vesdre and on the right bank of the river Vesdre after the bridge.
For all three return periods, areas with velocities in excess of 1 m/s can be observed at certain points on both sides of the river Vesdre, particularly downstream of the project area at the level of the car park on Louis Leménager street. This indicates a faster flow at these locations.
Figure 7 shows the extent of flooding in July 2021. The extent of the flooded areas is much greater than that of a 100-year rainfall, and the project area is largely flooded.

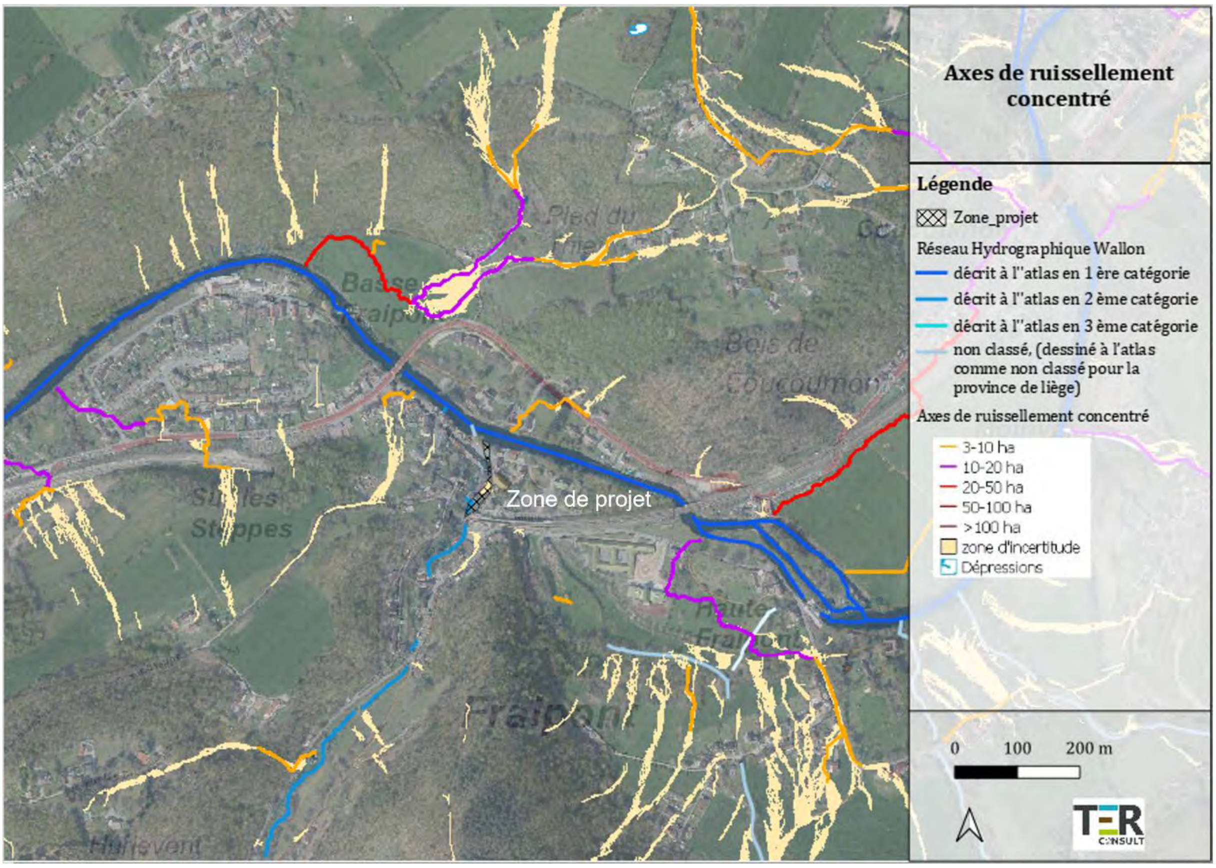


Figure 5 - Lidaxe model - Concentrated runoff axes - source: https://geoportail.wallonie.be).

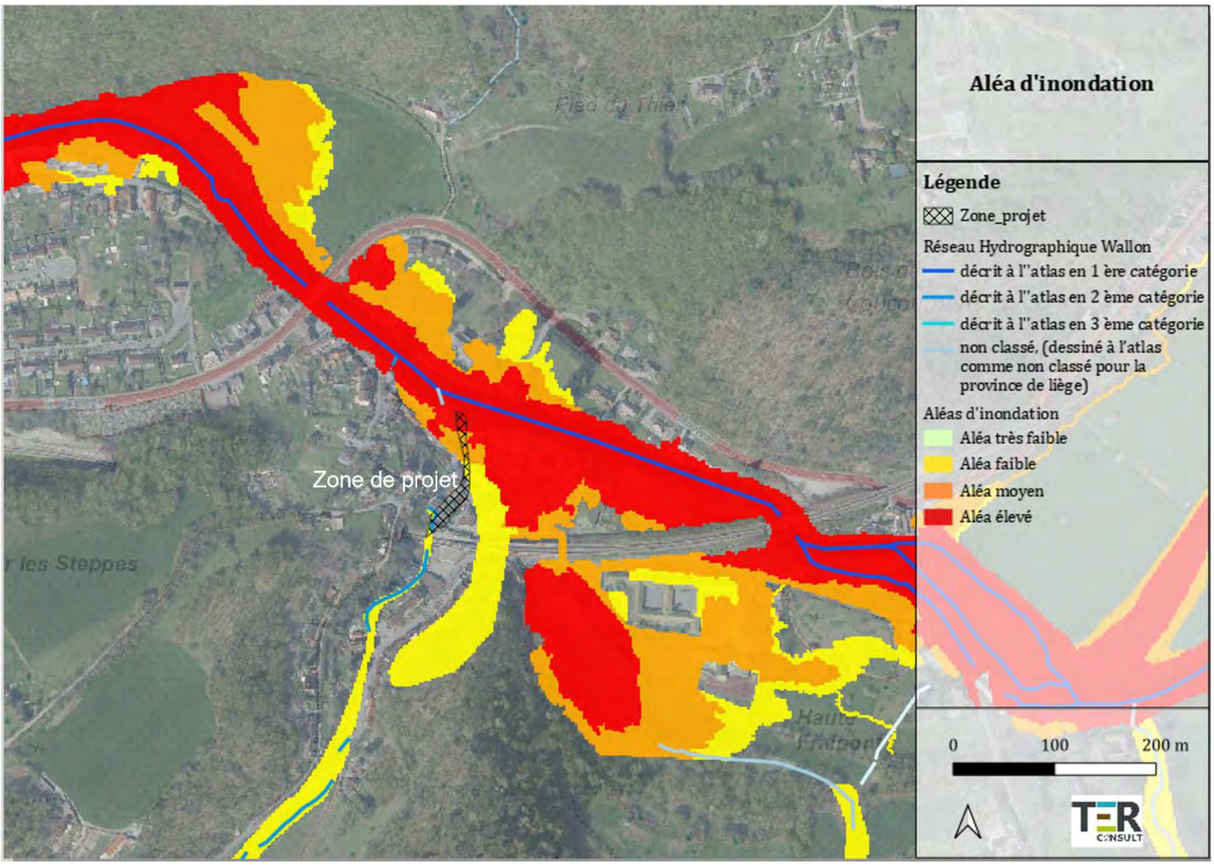


Figure 6 : Flood hazard mapping (source: geoportail.wallonie.be)

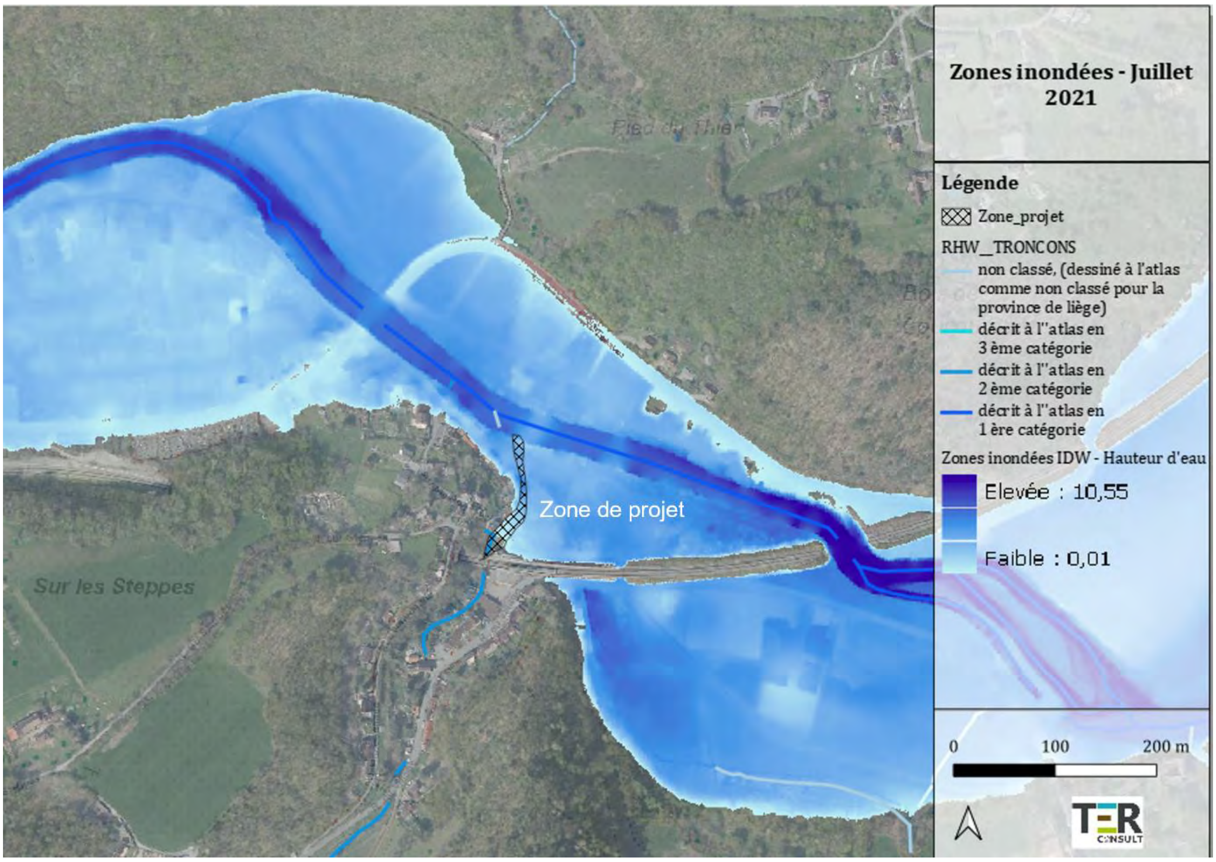


Figure 7 : Mapping of flooded areas in July 2021 by applying the IDW modelling method on the basis of "Survey Géomatique" field surveys carried out following the floods of July 2021 for the CSR by the Geomatics Department of the SPW (source: http://geoapps.wallonie.be/).

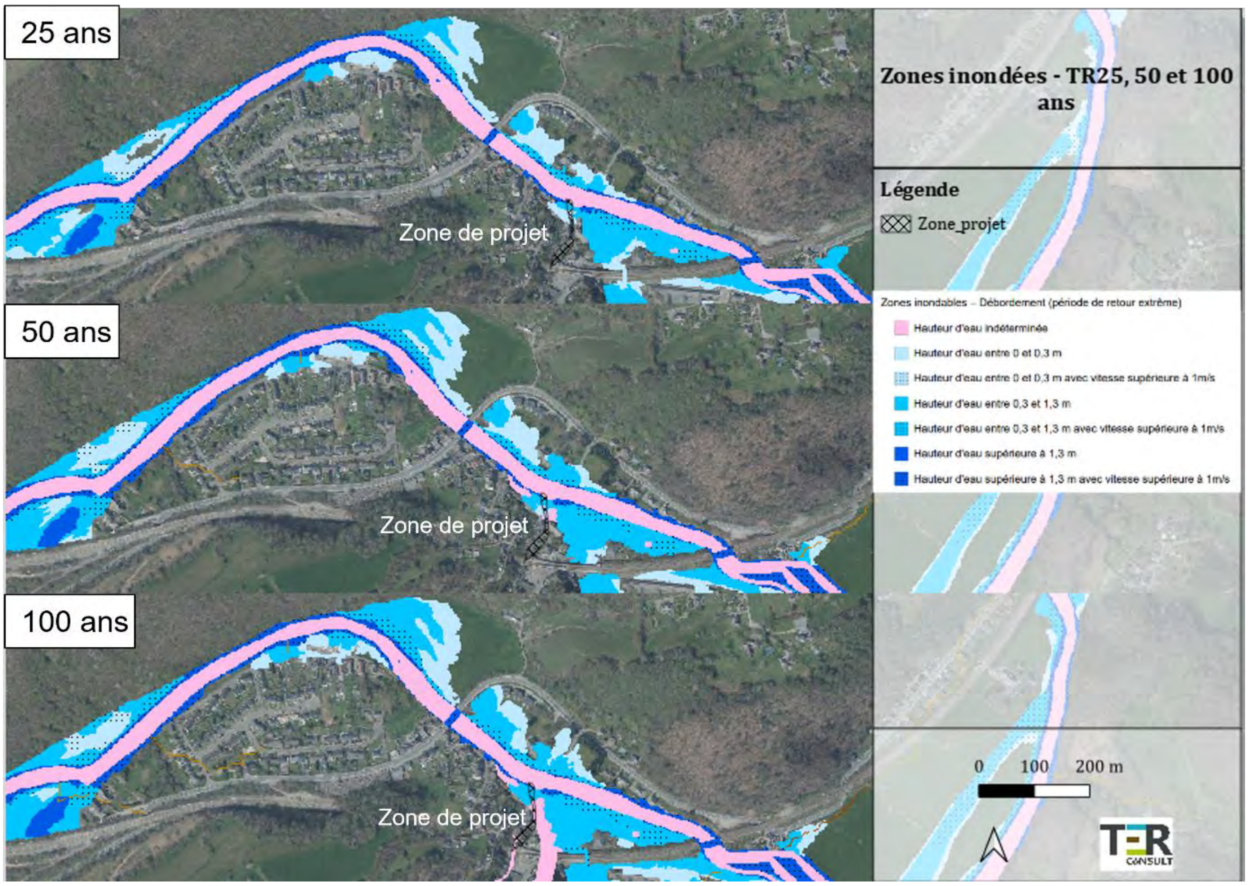


Figure 8 : Mapping of flood zones according to return periods of 25-50-100 years (source: http://geoapps.wallonie.be/) .

3.1.2 Description of the topographic surveys used.

The existing topography is taken from the SPW Digital Terrain Model (DTM), the data for which was acquired by a Lidar flight in 2013-2014. This DTM has 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.) and enables the major bed of the watercourse to be shown at the time the data is taken (**Figure 9**). However, this DTM cannot be used reliably for minor river beds for reasons inherent in the LIDAR technology used.

The bathymetry of the minor bed (from bank crest to bank crest) of the river Vesdre was then extrapolated from the profiles provided by the Direction des Cours d'eau Non Navigables (SPW). These profiles were taken from the Duchêne survey carried out in the 60s and 70s. It is important to emphasise that as these data are old, they should be treated with caution, as the numerous flooding events that followed (including that of July 2021) may have naturally modified the hydromorphological conditions of the minor bed.

For the Haveignée stream, the various profiles were surveyed by STP on 06/03/23 and completed by our team during a field visit on 15-03-23. The profiles were selected with a view to reproducing as closely as possible the configuration of the Haveignée stream in the project area. For the canalised sections, the elevation of the bottom, the invert of the opening and the ground were surveyed, as well as the widths. For the open sections, the bank crest, foot of bank and middle of the minor bed dimensions were surveyed.

Figure 10 shows the DTMs centred on the project area for the existing and projected situations. Along the creek, the DTM represents the elevation of the bottom of the channel in the channelled areas.

The plans of the structures in the river Vesdre which interest us are taken from the Duchêne surveys mentioned above (bridge N61) and from an update of the crest of the weir by means of a brief survey.

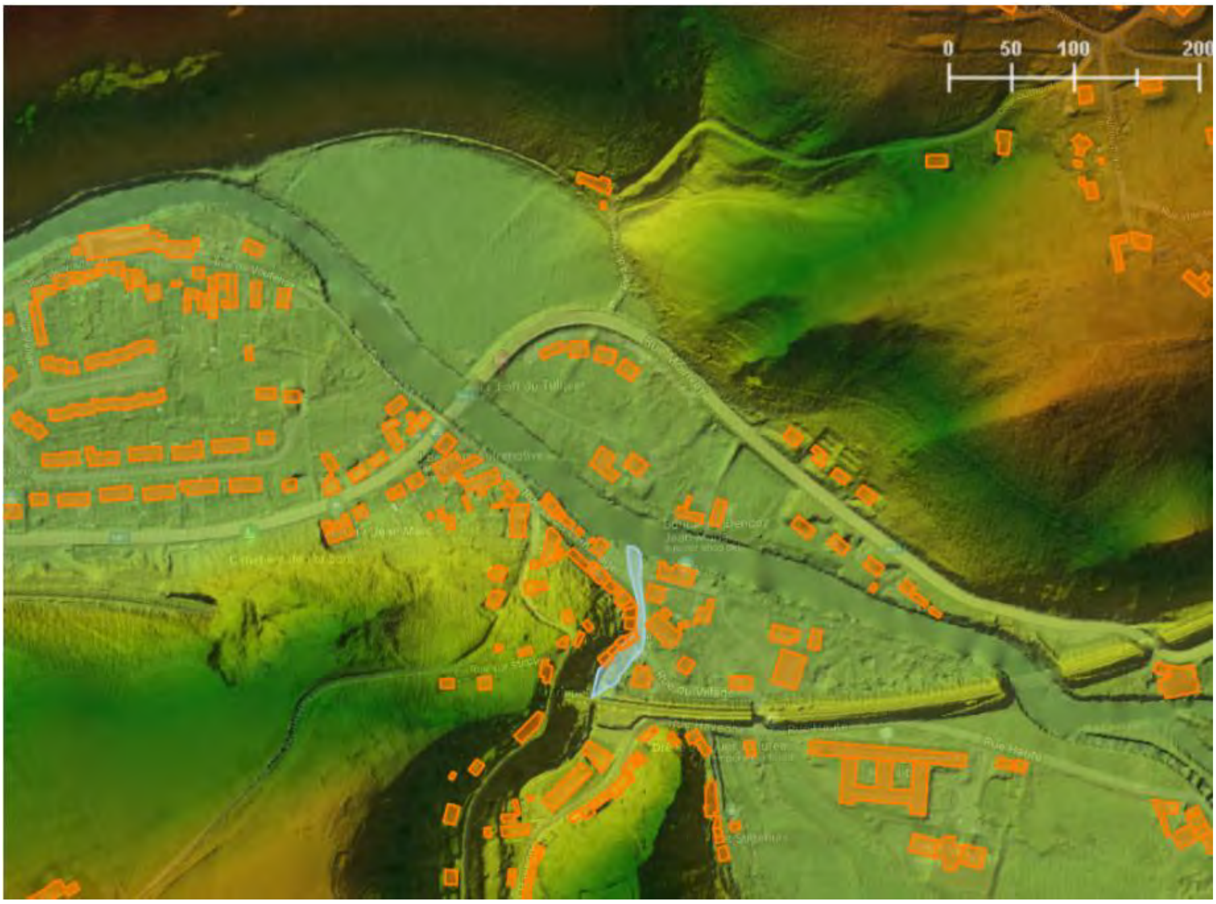
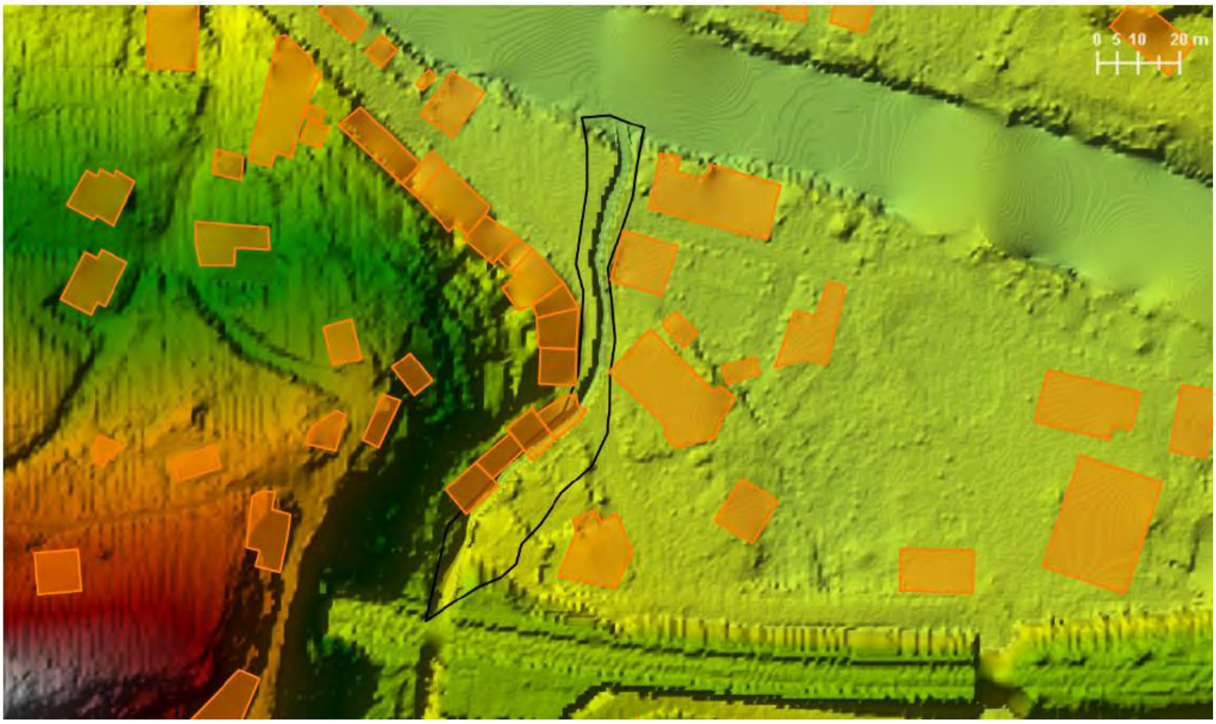


Figure 9 : Reconstructed DTM hydraulic study area (project area in blue)


0 5 10 20 m

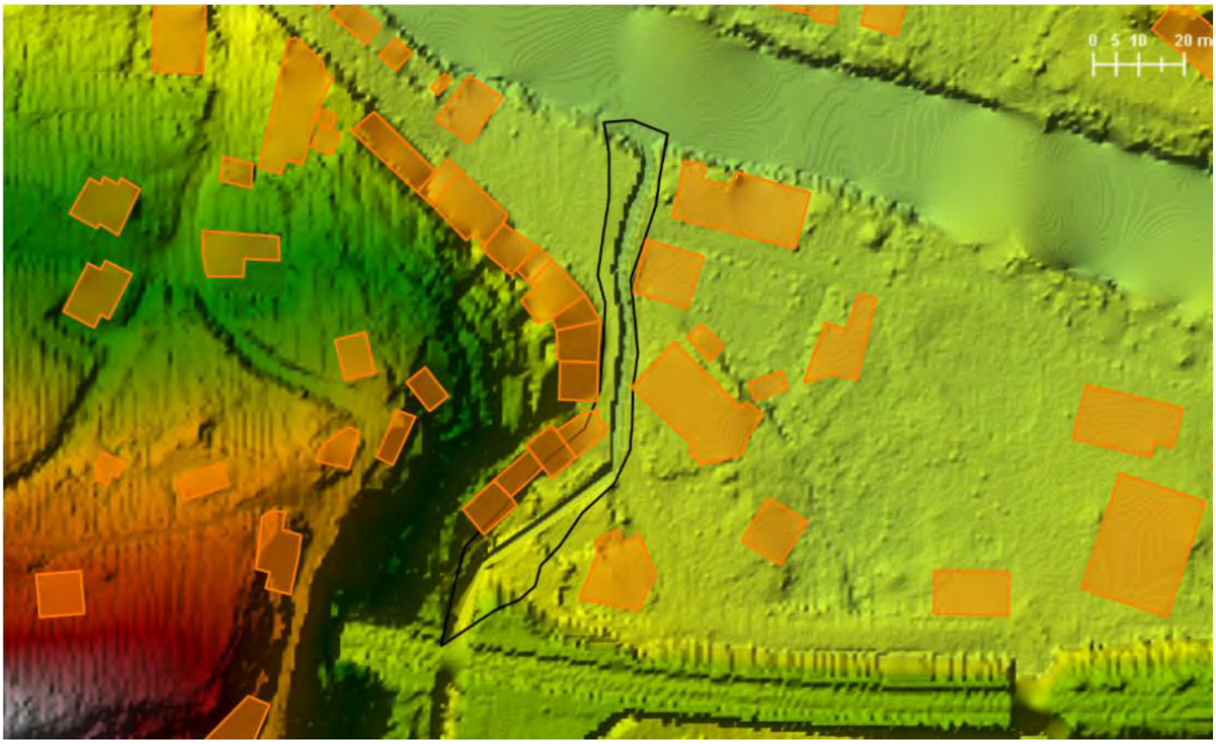


Figure 10 : Reconstructed DTM of the existing (above) and projected (below) situation in the project area

The changes taken into account for the projected situation are the adjustment of the dimensions of the opening in the downstream part of the project area (from the rue du Village at church level to the river Vesdre) and the opening up of the upstream part. The planned situation is taken from the plans provided by Province of Liège and shown in **Figure 4** and **Figure 11** and **Figure 12**, which show the cross-sections of the proposed new canalised section and the proposed open-cut section. It is important to specify that the last section of the watercourse in the project area should not be modified according to the wishes of the project owner (passage of electrical cables).

Based on this initial proposal, our objective was to carry out a sensitivity analysis of the problem of overflowing the watercourse as a function of several cross-sectional gauges of the watercourse.
In the topography, the buildings are considered to be obstructions to the flow of water.

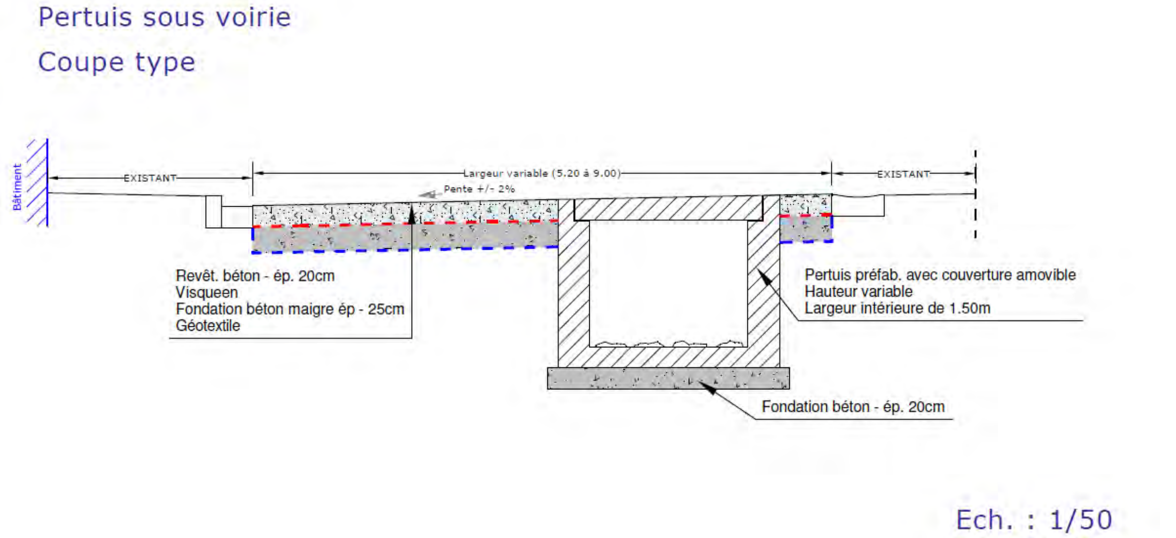


Figure 11 - Sectional drawing of the underpass required for the downstream section of the project area (source: STP)

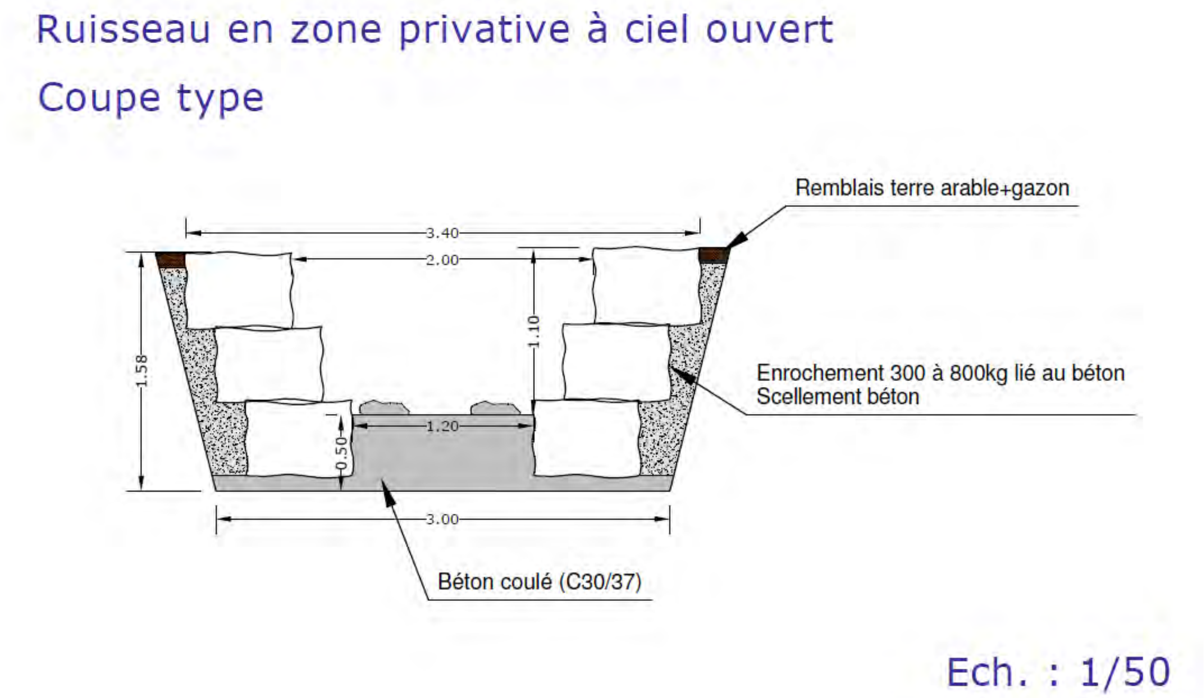


Figure 12 - Sectional drawing of the open-cast section (source: STP)

3.1.3 Hydrology

3.1.3.1 Haveignée stream

The Haveignée stream is the focus of this study, as it is its course within the project area that would be modified as part of this study. It is therefore essential to assess the inflow of water from this stream at the point of entry to the hydraulic model, i.e. at the exit of the railway bridge. In the absence of a measuring station in this catchment area, a hydrological model must be used to reconstitute these inflows.

The methodology used to estimate runoff is the Soil Conservation Service (SCS) method. This is a semi-empirical non-linear model for transforming rainfall into flow. The SCS method is widely used throughout the world and is recommended by the Walloon Region (GISER unit) for calculating flood flow reconstructions. The topography used for the hydrological study of la Gauche stream catchment is taken from the 2013-2014 Digital Terrain Model (DTM) (presented above). The data required for the hydrological modelling Soil Conservation Service (SCS) - Curve Number (CN) are as follows:

- Theoretical rainfall events defined by the Quantity-Duration-Frequency (QDF) curves for the municipality of Trooz, obtained from the Royal Meteorological Institute of Belgium (IRM) (https://www.meteo.be/fr/climat/climat-de-la-belgique/climat-dans-votre-commune).
- Observed rainfall event in July 2021. Hourly rainfall data from 1 to 31 July from the Louvegné station (identifier 6657, located approximately 4 km from the project area) are downloaded (hydrométrie.be).
- Land cover: based on the most recent land use map (WALOUS 2019, source: Geoportail.wallonie.be).
- The hydrological group: which defines the potential for water infiltration in soils, taken from the article by Demarcin et al. (2011).
- CN tables from the SCS describing the value of CN as a function of the hydrological group, soil cover and hydrological conditions.
- Tables describing the roughness coefficient according to land use (USACE, 1998, Appendix 1).
- The SCS synthetic unit hydrograph describing the response over time of the flow in a theoretical catchment caused by one unit of rainfall runoff.

3.1.3.2 Flow measurement station on the river Vesdre

The hydrological data are taken from the Chaudfontaine station (REF: 6228 Chaudfontaine) located approximately 6 km downstream of the project area. Hourly flows were used for this study. A transfer coefficient of 0.85 had to be applied in order to obtain the correct model input flows in the river Vesdre.

Table 1 : Parameter of the reference measuring station on the river Vesdre.

Measuring station	Chaudfontaine
Lambert coordinates (x,y)	240980, 142873
Catchment area	683.26 km^2
Put into service	2/06/1975

The statistical values of flood flows according to their return period are given in **Table 2**. They have been determined using Gumbel's law adjusted by the method of moments on the hourly data available at the Chaudfontaine station between 1975 and December 2020, thus excluding the exceptional data from July 2021 from the statistics. This choice was made in order to compare the results with the flood hazards in force, which do not take account of this exceptional event[1].

These flows will be used in the rest of the study to compare the existing situation with the planned situation.

[1] At present, discussions are underway within the river managers and the TFG (Transversal Flooding Group) to redefine the descriptive statistics of flood events and to take into account the flood of July 2021 in the definition of hazards.

The last flow (Q15/07/21) is a reference peak flow used in the rest of the study.

Table 2 : Estimation of hourly flows according to their return period.

Return period	Peak flow (m^3/s)
(Q2	100
(Q5	138
Q10	163
Q25	195
Q50	219
Q100	242
Q15/07/21	575

Given that the Chaudfontaine station was destroyed during the event of 13 to 15 July 2021, the peak flow for this event was reconstituted on the basis of values calculated in the Stucky report[2] and readjusted according to the transfer coefficient of 0.85.

3.2 PROPOSED APPROACH

3.2.1 Hydrological model of the Haveignée stream

The objective of the hydrological model is to reconstruct the flood hydrographs of the Haveignée stream, i.e. the flow rate over time following a rainfall event. We take the catchment area whose outlet is the model's input point (**Figure 2**).
Two types of simulations are applied to the catchment: simulations based on theoretical rainfall with defined return periods (2 years, 5 years, 10 years, 25 years, 50 years and 100 years), and simulation based on an observed rainfall event, the event of 13-15 July 2021.
Rainfall (theoretical or observed) is simulated in the catchment area of the Haveignée stream, the characteristics of which will determine the dynamics of water runoff and therefore the flow of the stream. The simulation time step is 5 minutes. The hydrological modelling stages are summarised below:

- Rainfall simulation: construction of theoretical hyetograms (rainfall height as a function of time) associated with each return period using the double triangles method.
- SCS production function: determination of the proportion of rain that runs off:
  - Calculation of the Curve Number (which can be likened to a runoff coefficient): CN is calculated per sub-basin, under average humidity conditions and is parameterised by land cover, hydrological group and hydrological conditions. **Figure 13** and **Figure 14** show, respectively, the land cover and hydrological groups used to calculate the CN at each point in the catchment, as shown in **Figure 15**.
  - The CN is then adjusted according to the slope of the catchment, using the Sharpley and Williams (1990) formula.
  - Finally, the runoff height is calculated using the following formulae:

[2] STUCKY, U.Liege (2021). Independent analysis of the management of waterways during bad weather in the week of 12 July 2021. Lot 1. Factualisation. Summary report.

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

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

o R: height of runoff [mm]
o P: rainfall [mm]
o Ia: initial interception [mm]
o Coeff_Ia: initial interception coefficient = 0.05 (Jiang R. (2001), Hawkins et al. (2002), Woodward et al. (2003)).
o S: Potential soil water retention [mm].

- SCS transfer function: transformation of runoff height into flow rate:
  - Calculation of the time of concentration using the SWRRB (Simulator for Water Resources in Rural Basins) method. The time of concentration is the factor that determines the maximum flow and corresponds to the time it takes for a particle of water coming from the part of the basin that is "hydrologically" furthest from the outlet to reach it (depending on the characteristics of the flow channels). It is made up of a surface concentration time and a channel concentration time. The formulas are as follows:
    - **Concentration time = surface concentration time + channel concentration time**
    - **Surface concentration time** =

$$t_{cs} = \frac{(\lambda n_v)^{0,6}}{18 S^{0,3}}$$

With:

λ = length of slope [m]
S = average slope of slope [m/m]
n_v = Manning's coefficient on the slope (weeds 0.13; grassland and gardens 0.24)

- **Channel concentration time** =

$$t_{cc} = \frac{0{,}62 L n^{0,75}}{A^{0,125} \Omega^{0,375}}$$

With :

n : Manning's coefficient in the channel
L : longest channel length in the basin [km]
A : average channel width [m]
Ω : average slope of the channel [m/m]

The time of concentration value obtained for the catchment area of the Haveignée stream is 136 minutes.

- Calculation of flood hydrographs (flow as a function of time) by convolution of SCS unit hydrographs.

It should be noted that no abatement coefficient has been taken into account for the catchment studied, given its small surface area, which represents a safe approach.

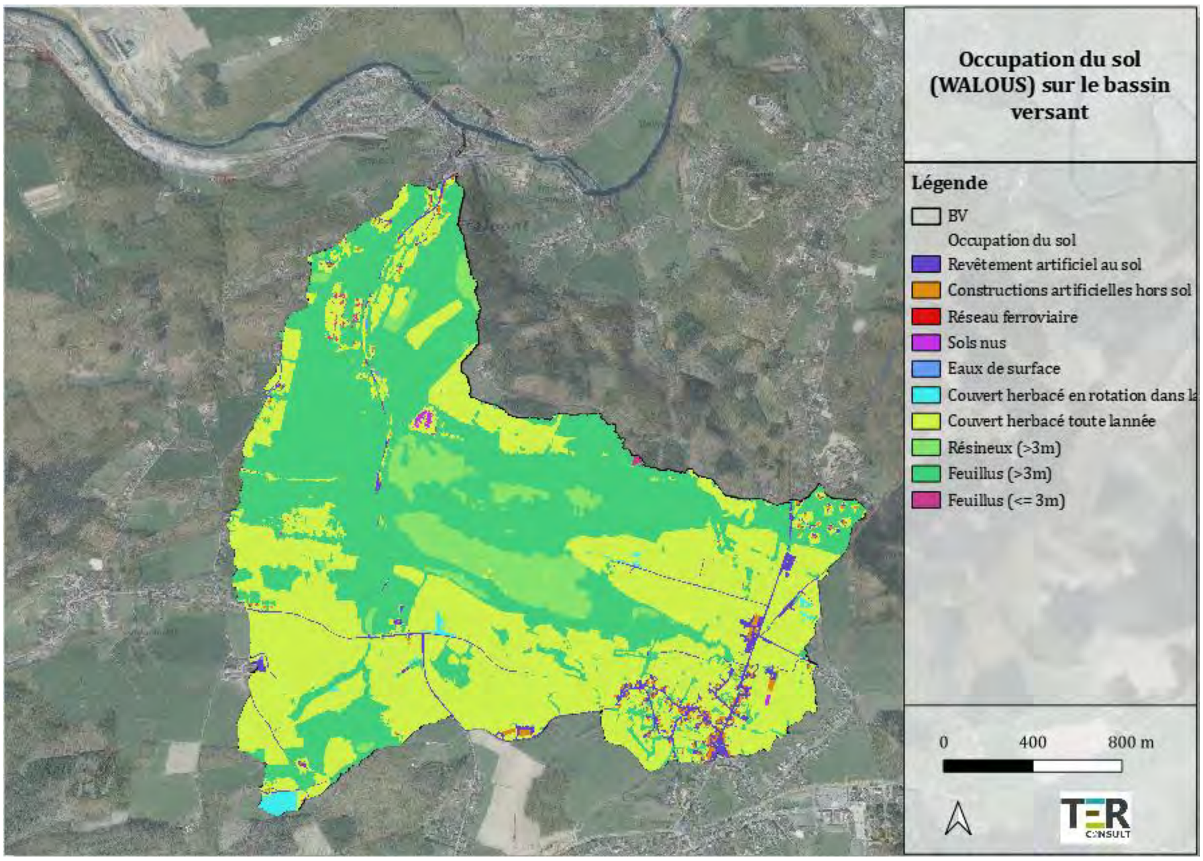


Figure 13 - WALOUS 2019 land use in the catchment area (source: geoportail Wallonie)

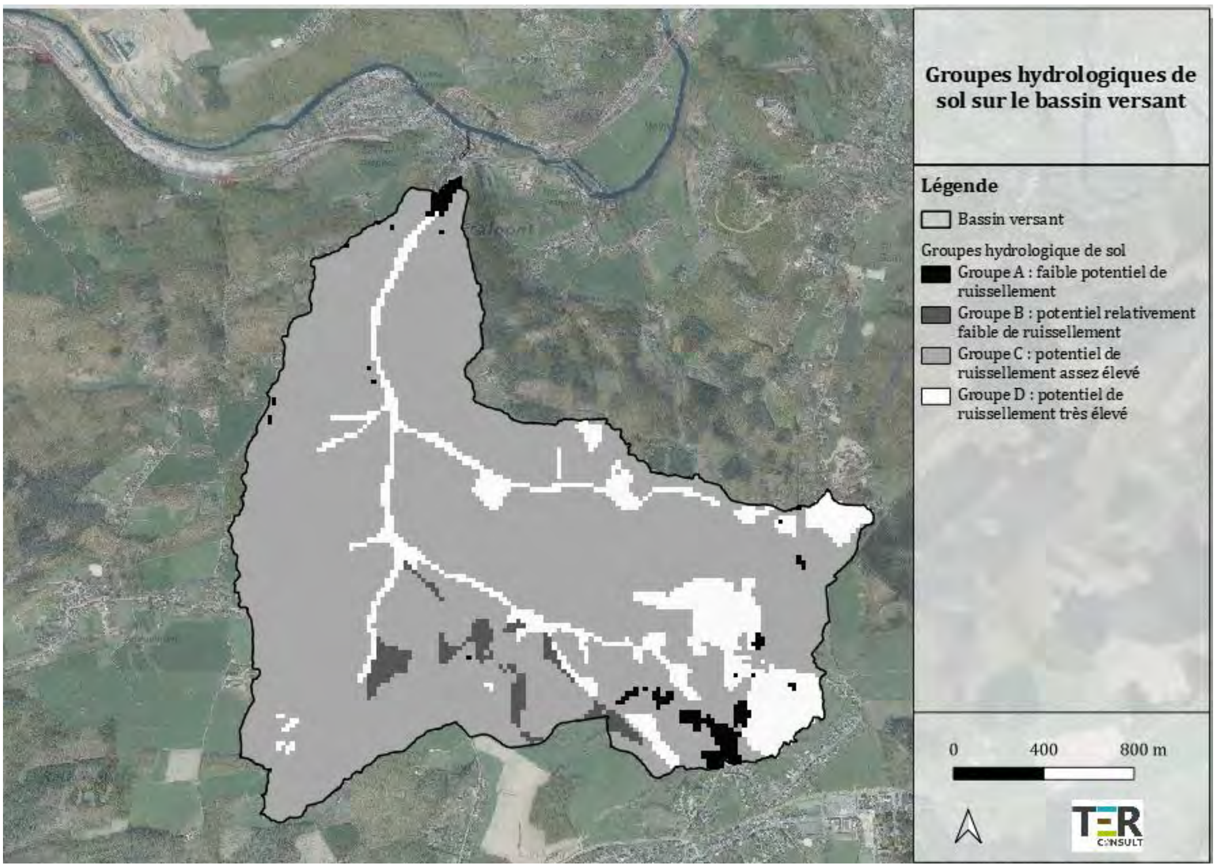


Figure 14 - Hydrological soil groups in the catchment (source: Demarcin et al., 2011)

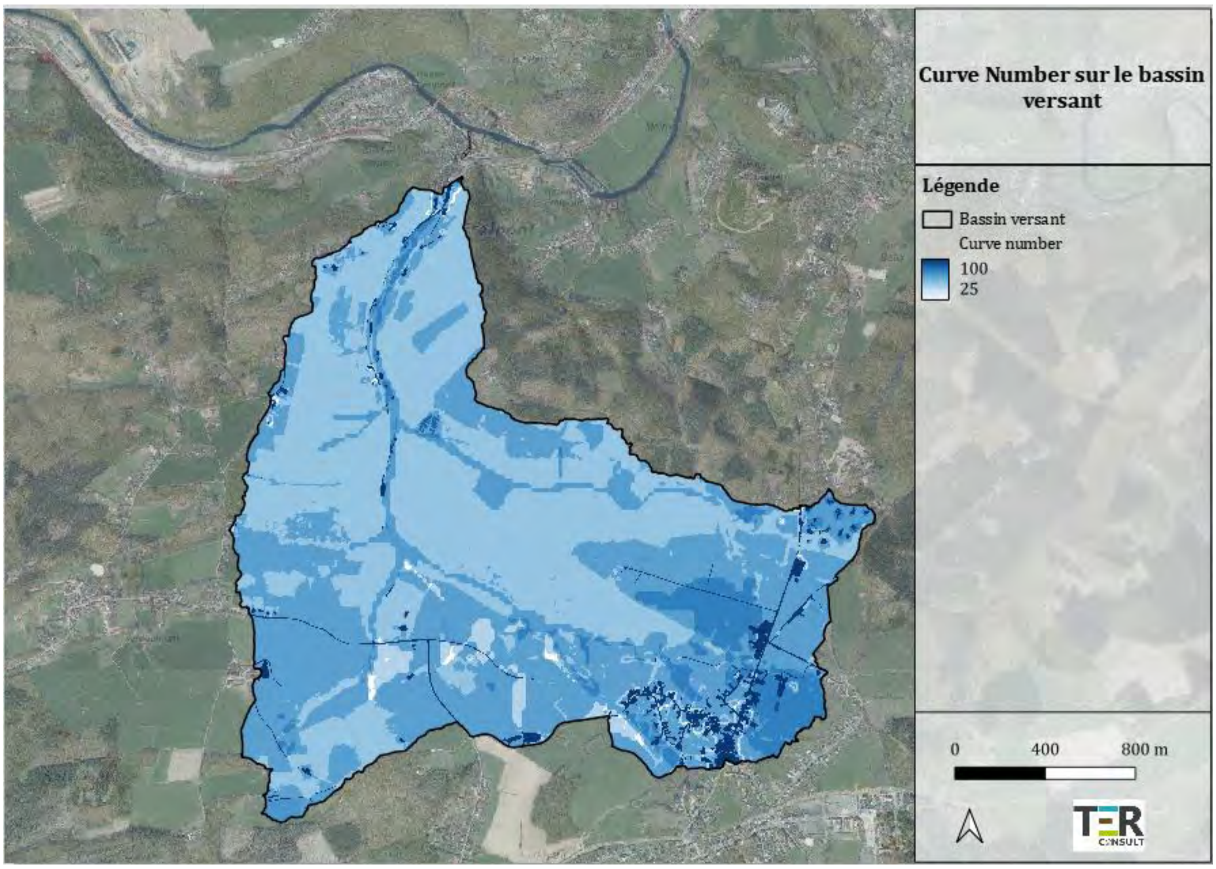


Figure 15 - Curve Number calculated for the catchment area

3.2.2 *Hydraulic model*

The general approach was to use the data obtained from the hydrological modelling and the topographic and bathymetric measurements of the perimeter to create a hydraulic model.

The modelling software used was GEOHECRAS 2D©, developed by CivilGeo and based on the HECRAS (Hydrologic Engineering Centers River Analysis System) model developed by the US Army Corps of Engineers. The skills of this software, coupled with the expertise of the modeller, are recognised internationally. It is also used by a large number of engineering consultancies and government departments responsible for hydraulic studies.

It has been developed and tested to produce 1D and 2D hydraulic calculation models for a complete natural or artificial hydrological network.

Once the geometric model and the various parameters have been integrated, a calibration process is used to refine these parameters and then apply them to the characteristic flows.

Simulations of steady-state and unsteady-state flows are then carried out.

Due to the configuration of the watercourse (canalised sections with variable profiles), a 1D geometric model was used.

The following steps were taken:

- Definition of the geometry of the bottom of the minor bed of the Haveignée stream and river Vesdre, the banks and hydraulic structures (dams, sluices, etc.) in the area under study, based on in situ measurements and available plans - creation of a DTM of the bathymetry;
- Definition of the geometry of the major bed based on the Walloon Region's LIDAR DTM beyond the bank crest;
- Representation of specific structures:
    - o Canalisation of the Haveignée stream from plot 130H (**Figure 3**) to the Vesdre river
    - o Low wall on the right bank of the Haveignée stream upstream of the canalised section
    - o SNCB bridge wall at the model entry point
    - o N61 bridge over the river Vesdre
    - o Weir on the river Vesdre
- Geometric coupling (building, obstruction, etc.) and parameter definition;
- Definition of coefficients (friction coefficients assigned on the basis of the WALOUS 2019 land use map and tables from Chow, 1959 and ASCE, 1982) and model boundary conditions (uniform heads downstream and peak flows upstream and at the confluence);
- Model calibration based on flood hazard maps (return periods of 25, 50 and 100 years) ;
- Sizing of structures for the planned situation for TR2, 5, 10 and 25;
- Exploitation and interpretation of results.

4 RESULTS

4.1 RECONSTRUCTION OF THE FLOOD HYDROGRAPHS FOR THE HAVEIGNÉE STREAM

Synthetic flood hydrographs are calculated for the different return periods.
The maximum simulated flows (peak flow) are presented in **Table 3**. **Figure 16** shows an example of the results obtained. The results for each return period are shown in **Appendix 2**.
The initial flow was considered to be zero in this case (a minimum initial flow could have been

envisaged), which does not alter the quality of the results since it is the flood peaks that are of interest in this context.

Table 3. Maximum flows modelled for different return period simulations for the Gauche catchment.

Return period	Peak flow (m³/s)
Q2	2.71
Q5	4.74
Q10	6.46
Q25	9.13
Q50	11.49
Q100	14.18
Q15/07/21	17.38

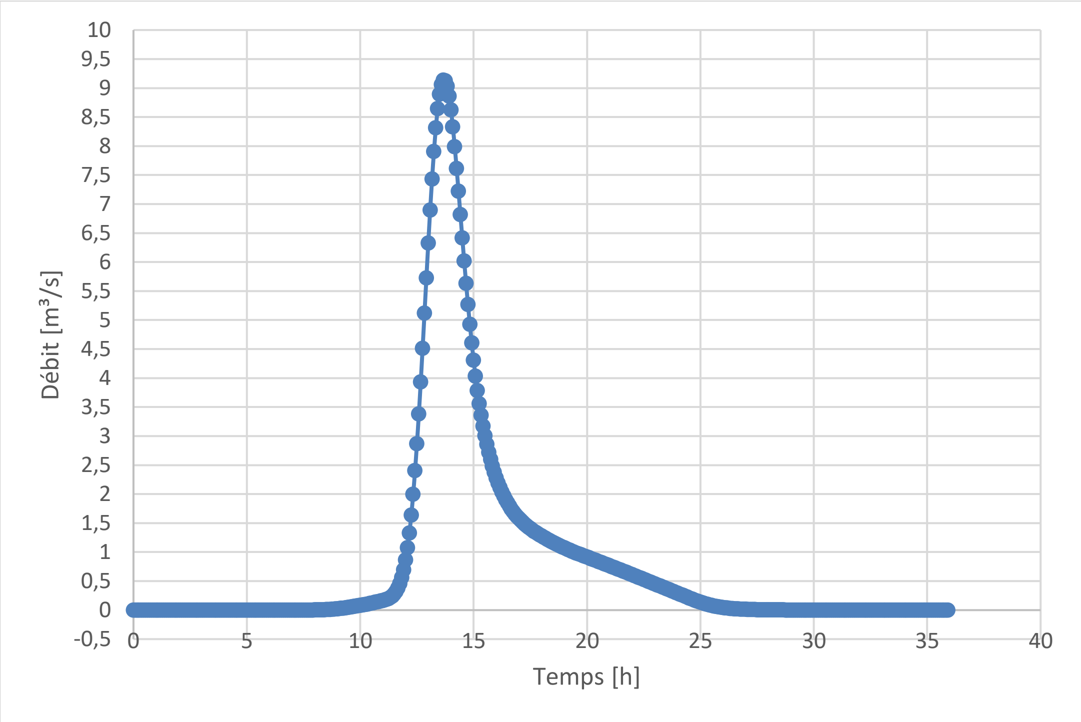


Figure 16 - Hydrograph reconstructed using the SCS method for the theoretical 25-year flood on the Haveignée stream at the model entry point.

4.2 CREATION OF THE GEOMETRIC MODEL

The geometric model is made up of 38 cross-sections based on the minor bed survey we carried out in the field for the Haveignée stream and those of the DCENN for the river Vesdre, extended by data from the Walloon Region's LIDAR DTM of 2013-2014 and the project's DTM. They are shown in brown in **Figure 17**, along with the structures across the watercourse and the position of the banks in question (red dots). Other key features in the hydraulic landscape, such as embankment dykes and ineffective flow zones, are also shown. The section of river studied is approximately 570 m long (120 m of the Haveignée stream and 450 m of the river Vesdre). **Figure 18** and **Figure 19** show the zoomed-in 1D geometric models of the study site for the existing and planned situations.

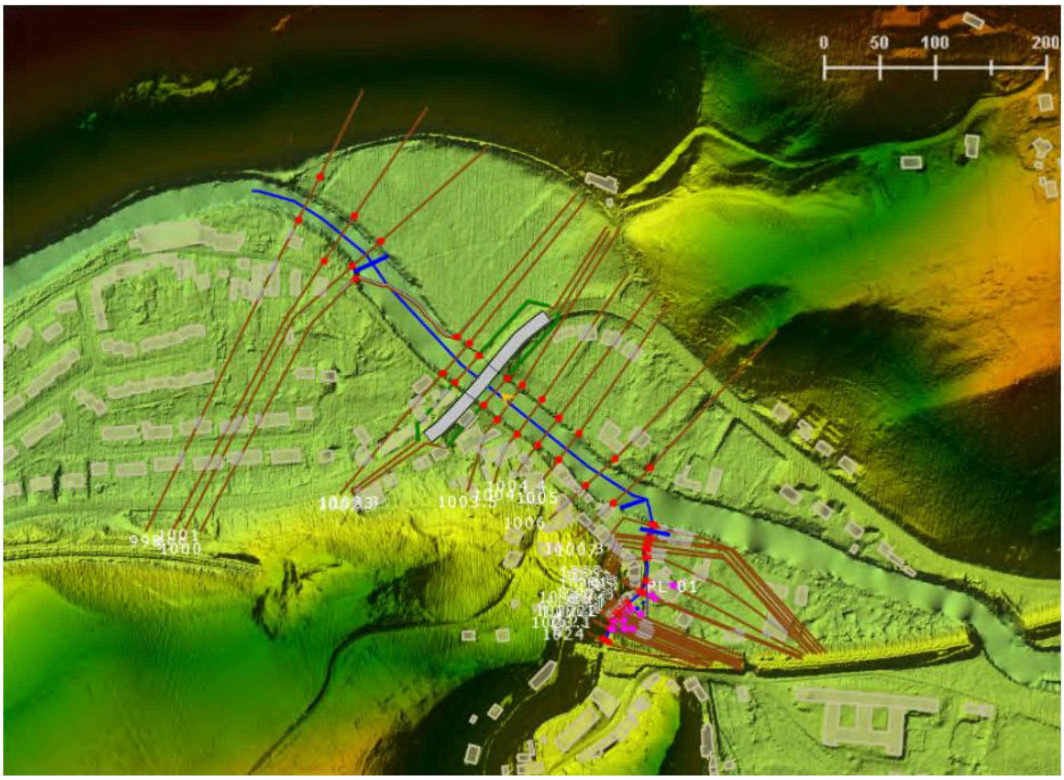


Figure 17 : 1D geometric model - general view of existing situation

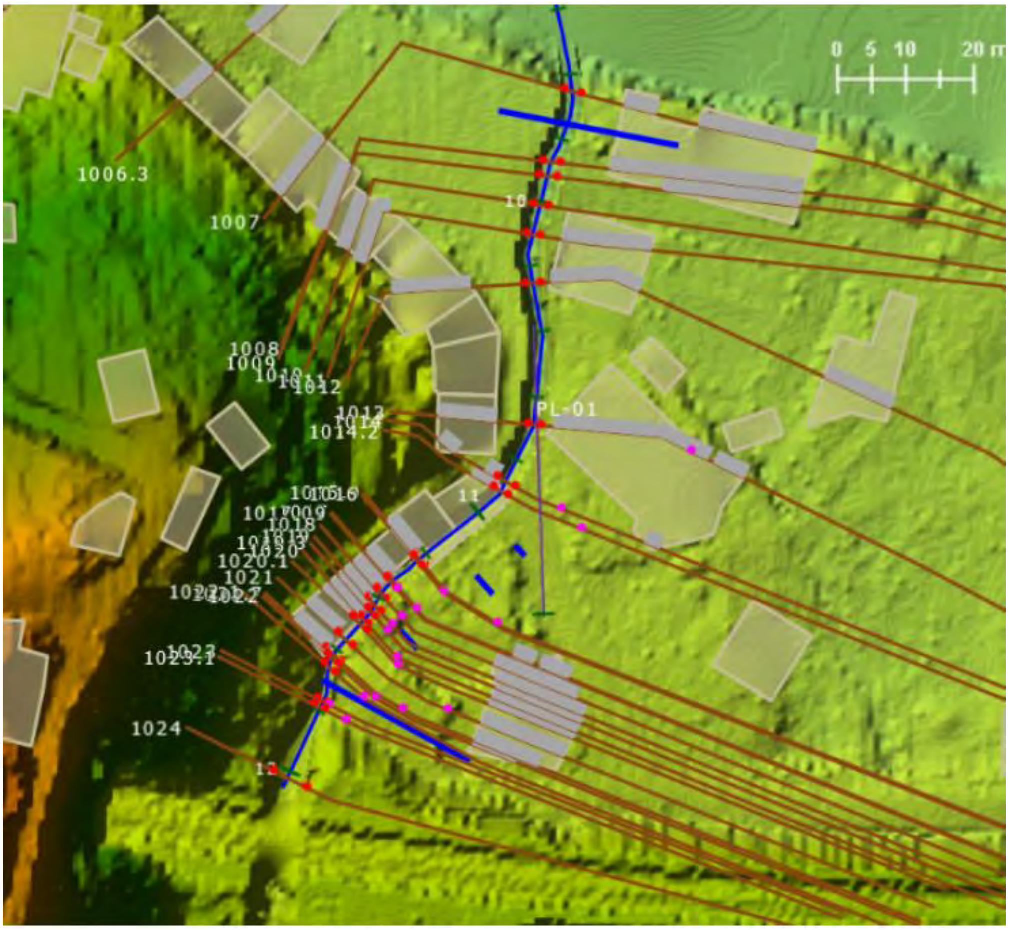


Figure 18 - 1D geometric model - zoom on the study site (existing situation)

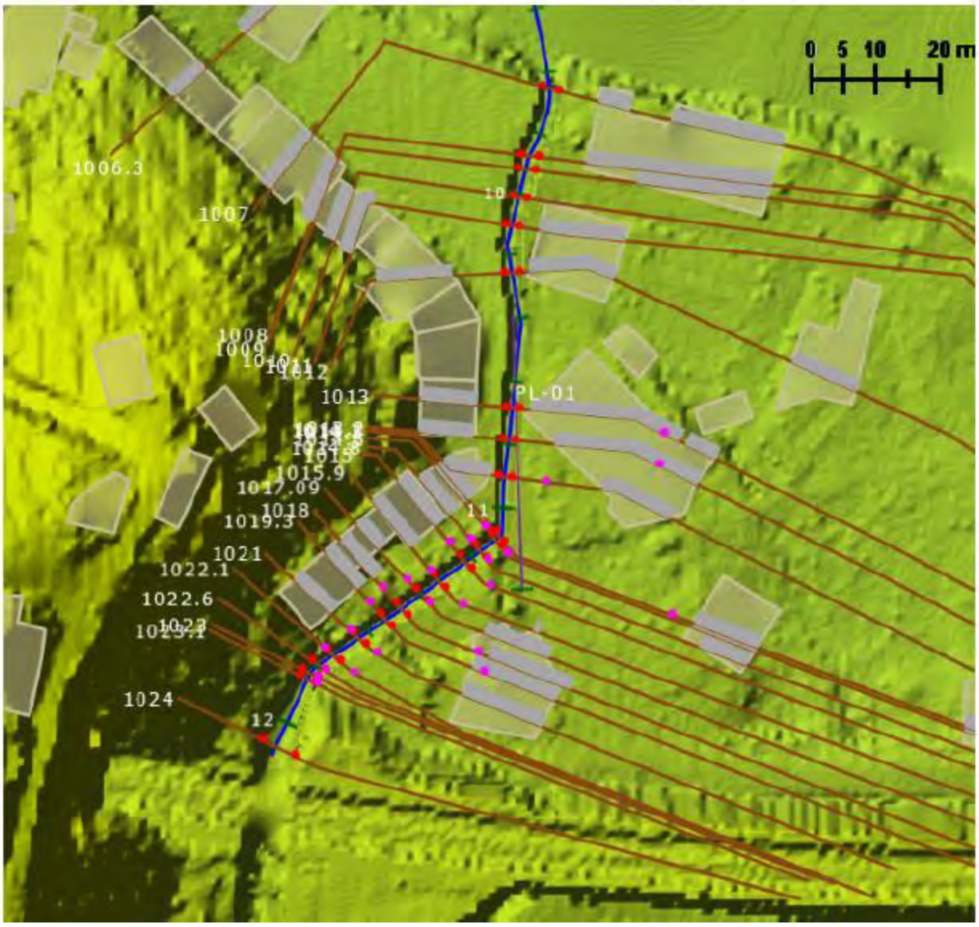


Figure 19 - 1D geometric model – zoom on the study site (planned situation)

3 structures are considered: the N61 bridge, the weir across the river Vesdre and the Haveignée stream pipe. **Figure 20** and **Figure 21** show how the N61 bridge and the weir are represented respectively. **Figure 22** shows the plan used to model the canalised part of the project area, subdivided into different sections. **Figure 23** gives an example of the section description sheet for section 5; it shows the length of the section, its average gradient and the shape of the upstream and downstream profiles as represented in GEOHECRAS. All the description sheets are available in **Appendix 3**.

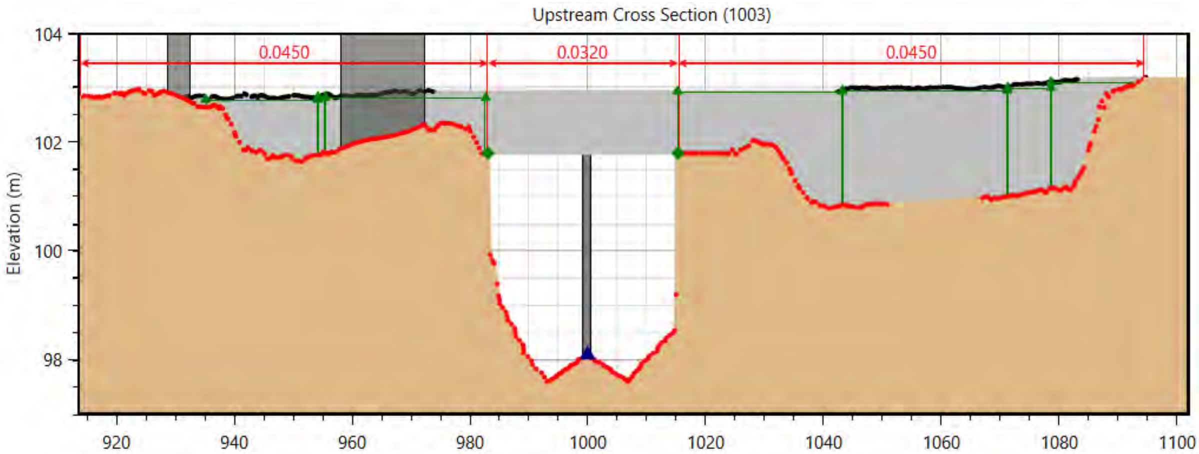


Figure 20 : N61 bridge (upstream view - in green : inefficient flow zones)

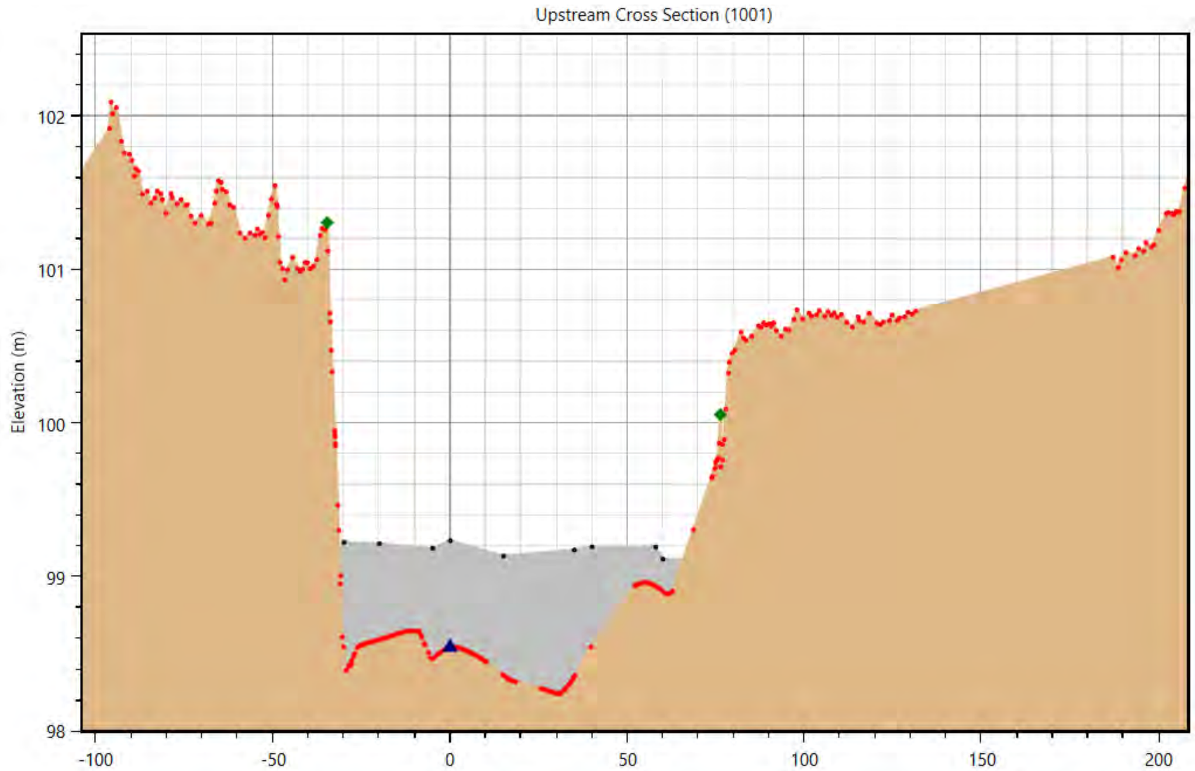


Figure 21 : Weir across the river Vesdre

Figure 22 - Plans of sections of the project area in the existing situation

Section 5

Longueur (m) : 16.5

Pente moyenne (%) : 4.4

Section amont

Section avale

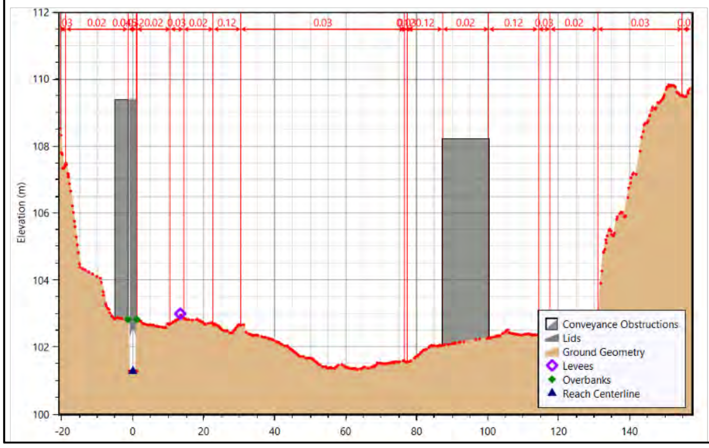


Figure 23 - Description of section 5 of the project area

Compared with the planned situation, the modified sections are sections 1 to 7 of the section plan (**Figure 22**) for the open-cut section and 8 to 12 for the modification of the downstream opening. As mentioned earlier, the last section of the project area (section 13) will not be modified. **Figure 24** and **Figure 25** show the typical profiles modelled for the new canalised section and for the open-cut section.

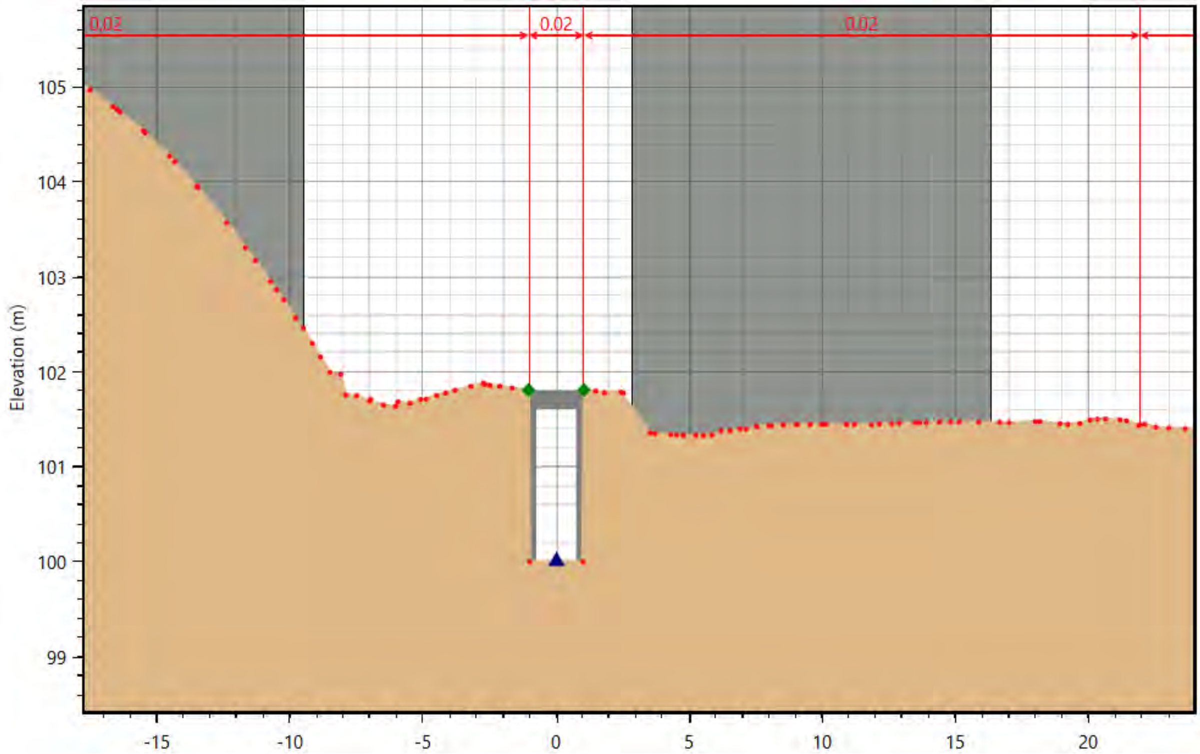


Figure 24 - standard profile taken into account for the new canalised section.

Figure 25 - standard profile taken into account for the new open-cast section.

The land cover was taken from Walonmap's WALOUS layer and reclassified according to their respective Manning coefficients, as shown in **Table 4**. The Manning coefficients for the major bed were then assigned in accordance with these values. This is illustrated in **Figure 26**. For the minor bed of the Haveignée stream and the river Vesdre, a coefficient of 0.032 was taken into account.

Table 4 - Manning coefficients by land use class WALOUS

Typology	Manning
Artificial floor covering	0.02
Artificial above-ground structures	0.02
Rail network	0.02
Bare floors	0.03
Surface waters	0.035
Herbaceous cover in rotation during the year (e.g. annual crop)	0.04
Grass cover all year round	0.03
Softwood (>3m)	0.12
Hardwood (>3m)	0.12
Hardwood (<= 3m)	0.15

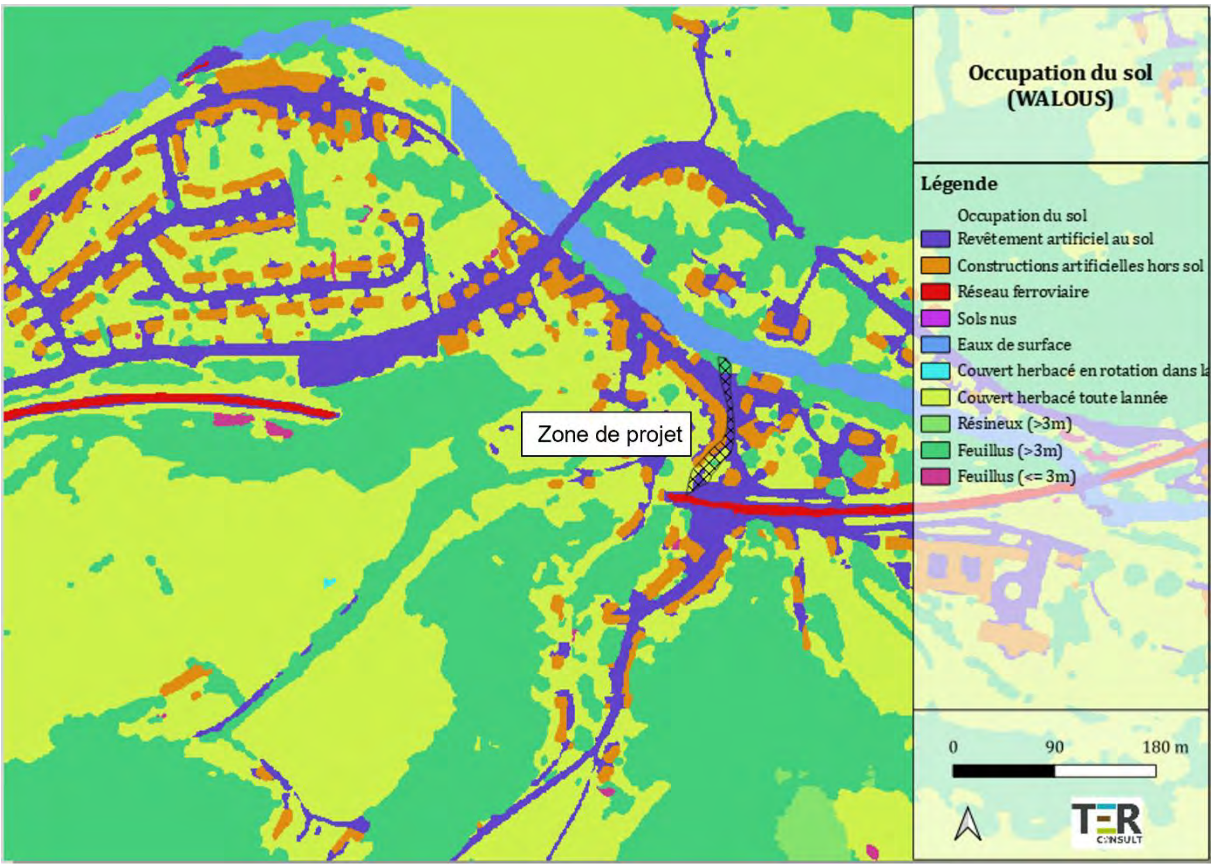


Figure 26 : Close-up view of land use at the study site.

4.3 MODEL DEVELOPMENT, CALIBRATION AND VALIDATION

One of the important phases in the construction of a hydraulic model is the verification, by all available means, that the results of the model are consistent with reality. In this case, there was little data available to calibrate the model, as there were no water level records or flood levels taken in situ. Nevertheless, we were able to compare our results with the following data sources:

- Maps of flood zones and water heights used to draw up the reference flood hazard maps (**Figure 8**);
- Maps of the extent of the July 2021 flood reconstructed on the basis of analysis of aerial views taken just after the flood or IDW reconstruction with field surveys (**Figure 7**).

<u>Comparison of hazards according to return periods</u>

As a preamble, it is important to note that the flows used to model these exceptional floods in GEOHECRAS are derived from the statistical approach, the results of which are presented in **Table 2**. The data used extends over a longer period (up to December 2020) than that considered when defining these hazards in the SPW study (different data set, see also stations). The results of these comparisons should therefore be treated with caution.

- 25-year return period

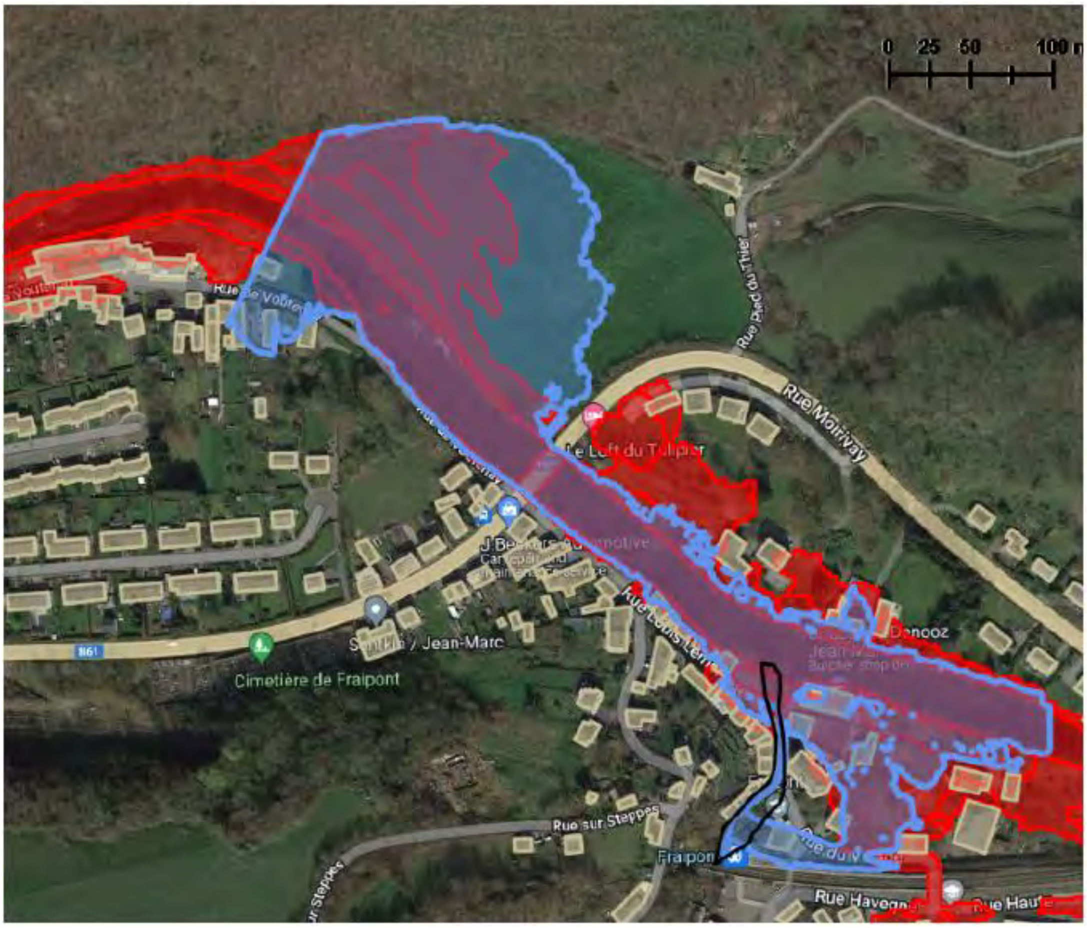


Figure 27 : Comparison between the hazard for the 25-year flood (red outline, source: https://geoportail.wallonie.be) and the result of the GEOHECRAS model for the same return period (blue outline).

- 50-year return period

Figure 28 : Comparison between the hazard for the 50-year flood (pink contour, source: https://geoportail.wallonie.be) and the result of the GEOHECRAS model for the same return period (blue contour).

- 100-year return period

Figure 29 : Comparison between the 100-year flood hazard (yellow outline, source: https://geoportail.wallonie.be) and the result of the GEOHECRAS model for the same return period (blue outline).

- Observations

In the vicinity of the project area, on the left bank of the river Vesdre, the model fits fairly well with the reference flood zone maps for the 3 return periods. On the right bank of the river Vesdre, there are a few differences, particularly upstream of the N61 bridge, where our model seems to underestimate overflows compared with the reference flood zones (TR25 in particular). Downstream of the bridge, where this time the modelling seems to overestimate the overflows of the river Vesdre compared with the reference flood zones. This is probably due to the short length of watercourse studied on the river Vesdre (compared with overall phenomena in the catchment area), to possible changes in ground level and to the approximations of the various studies. It should also be noted that the Haveignée stream only has a low hazard level in the official maps and that a hydraulic contribution to the non-negligible hazards comes from the Vivier stream, close to the Haveignée stream, which overflows and flows through the Rue du Vivier under the railway, merging the hazards at the level of Fraipont village.

Map approach to the reconstructed flood zone for the July 2021 flood

As previously stated, this reconstruction was carried out on the basis of field data collected by SPW teams just after the event. **Figure 30** compares the hazard map for the July 2021 flood (source: geoportail.wallonie.be) with the results of modelling this same flood at maximum extent. The results are very similar.

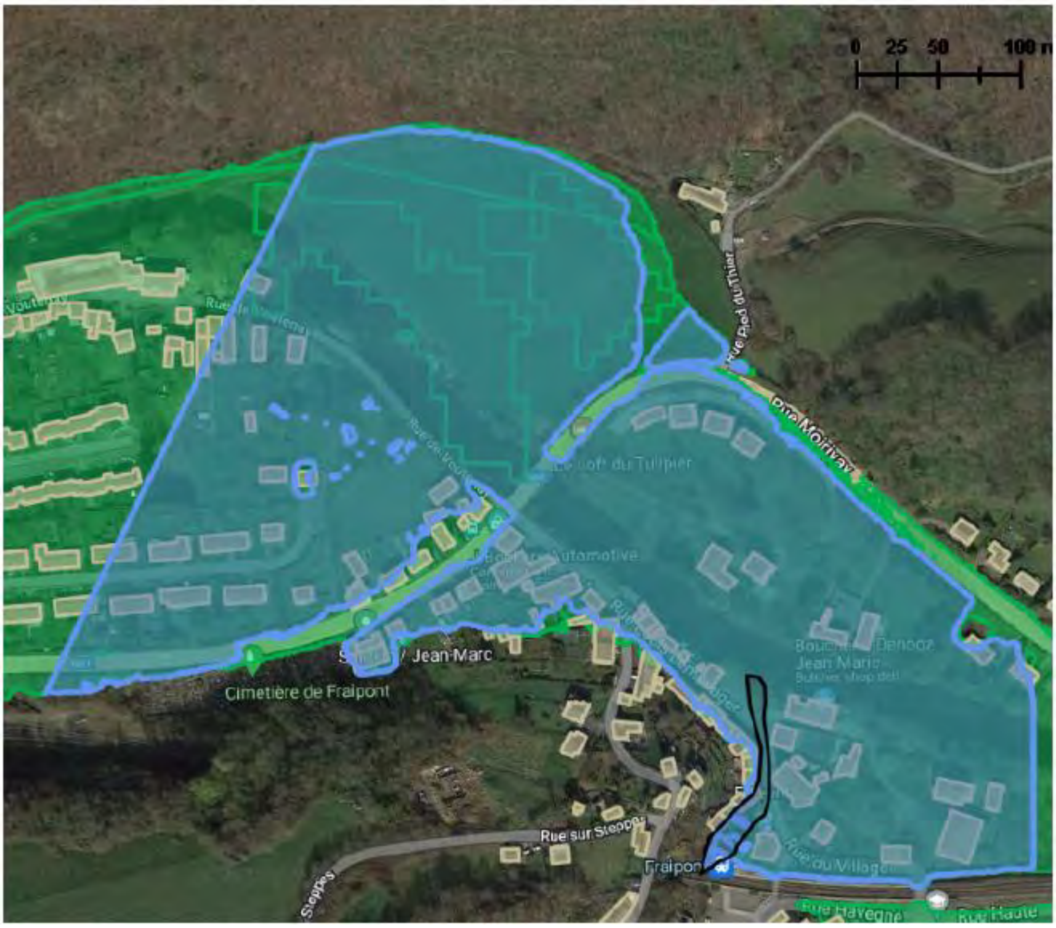


Figure 30 : Comparison between the flood zone obtained using the IDW method (green outline, source: https://geoportail.wallonie.be) and the maximum extent modelled (blue outline) for the July 2021 flood.

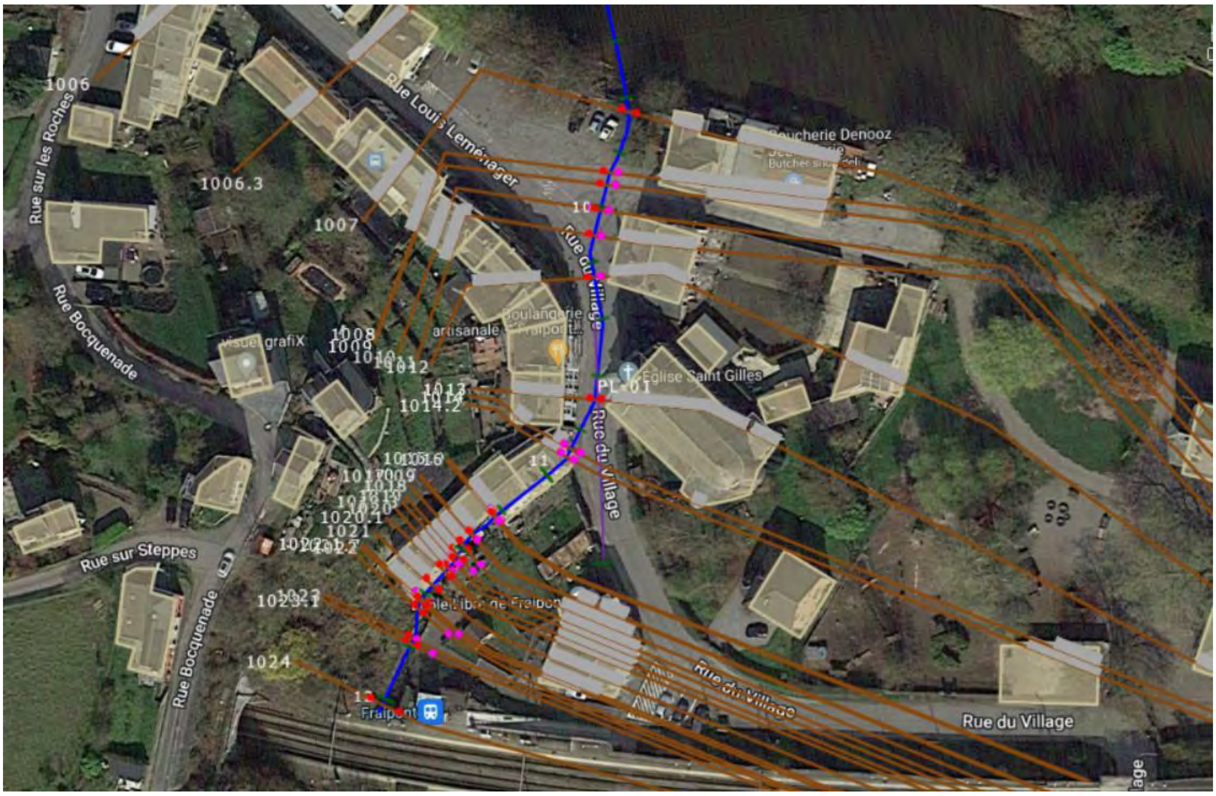
General conclusions

The different approaches show that the model developed in this study is robust. Indeed, for July 2021, the modelling showed good results compared with the IDW layer, and those proposed for the 25-50-100 year hazards are entirely acceptable. However, it should be noted that more precise quantitative calibration data, such as water level readings taken during reference rainfall events, would have enabled the model to be calibrated more accurately.
It is important to note that it was only possible to calibrate the river Vesdre section. In the case of the Haveignée stream, given that there were no historical data or overflow events, no calibration could be carried out. However, a neighbour's statement that the stream overflows almost every year enabled us to compare the results of the model, which seems to be a good representation of reality. The parameters for this tributary were set on the basis of the usual values found in the literature for this type of watercourse.
On the basis of the data compared here, the model can therefore be considered validated and calibrated for the floods studied.

4.4 1D MODELLING RESULTS

4.4.1 Analysis of the existing situation

Firstly, it is important to mention that it was observed during the simulations that the Vesdre had a significant influence on the flow within the project area. In fact, during floods, the rising water level and overflow of the river Vesdre creates a blockage effect within the Haveignée stream channel, amplifying flood phenomena within the project area. In this study, the main aim was to provide a refined dimensioning of the various channelled or open sections of the project area and to isolate the flows due to the Haveignée stream alone. It was therefore decided to set the flow rate within the river Vesdre at a low level (30 m³/s) and constant for all return periods for the sizing analysis of the structures. **Figure 31** shows the plan of the sections to be followed for the results of the existing situation and the planned situation scenario 1.


1006
1006.3
1007
Rue sur les Roches
Rue Louis Leménager
Rue du Village
Rue Bocquenade
visuel grafiX
1008
1009
1012
artisanale
1013
1014.2
PL-01
Eglise Saint Gilles
1021
Rue sur Steppes
1024
Rue du Village

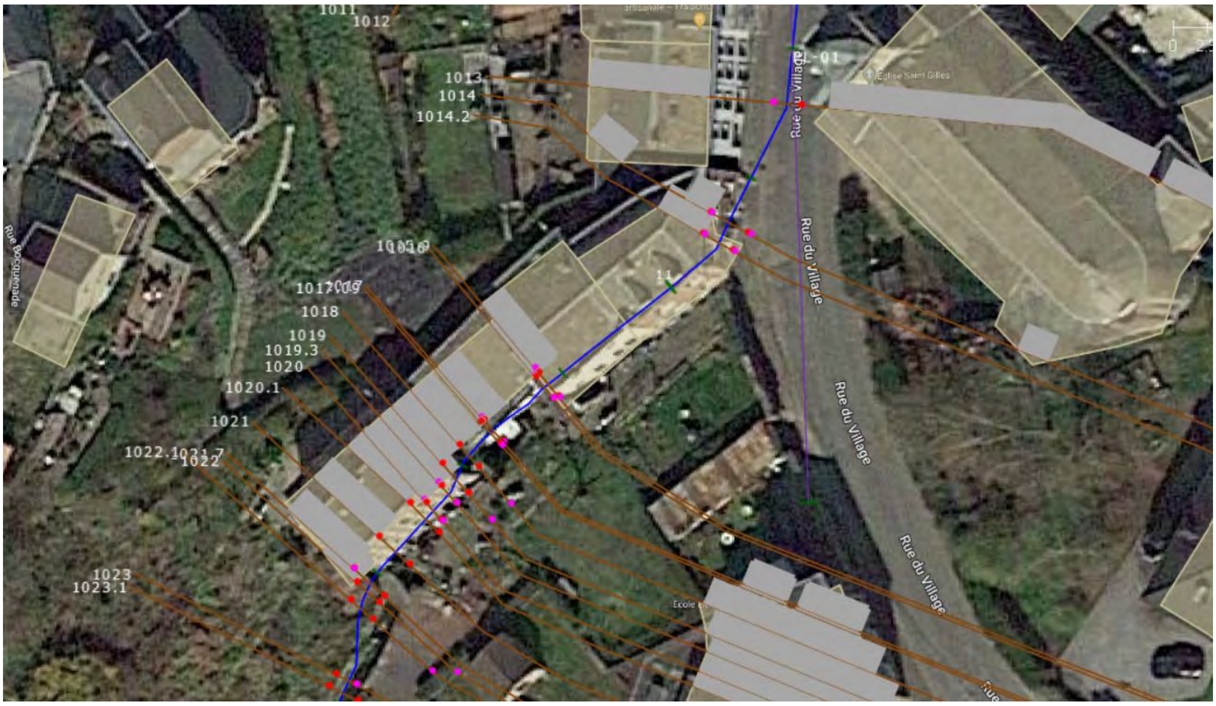


Figure 31 - Plan of existing situation + planned situation scenario 1.

The results for the existing situation are given in the form of water levels within the profiles in **Appendix 4**.

4.4.2 Analysis of the planned situation

This section presents the modelling results for the projected situation. In consultation with the project owner, it was decided to deal with two different scenarios for the planned situation: a first scenario in which the current upstream sections under the sluice under the dwellings are retained (sections 1 to 7 cf. plan of sections **Figure 22**) and the downstream sections under the village street are modified (sections 8 to 12 of the plan of sections) according to the principle plans proposed in **Figure 11** (width of the opening of 1.5 m), with a margin in width of 0.4 m additional and a second scenario where the upstream sections under the opening under the dwellings are replaced by an open channel (project plan **Figure 4** and typical section plan **Figure 12**) in addition to the changes of sections under the village street.

As already specified, the aim of the study is to provide a design for the structures (downstream pipe and open-cut section). For their design, we have adapted various elements in consultation with the project owner.

- For scenario 1 (modification of the downstream opening only): the width of 1.5m of the new section channelled downstream under the village street (also testing a section of 1.9m at most compared with the STP's pre-dimensioning).
- For scenario 2: adaptation of the downstream opening (see sc1) and adaptation of the width of the bottom of the open section: in this context, we are using the standard riprap profile which we are adapting in width as the overflow analysis progresses.

The bottom of the structures (invert dimensions) was not subjected to a sensitivity analysis, given that the pre-dimensioned slope, based on the connection with the bottom of the existing watercourse, could not be modified in order to limit sediment deposition and respect the current depths.

The overflow from TR2 in the existing situation at profile 1023.1 on the right bank (currently partly filled in by a flowerpot and sandbags, see **Figure 34**) means that this low point in the bank needs to be filled in for all the scenarios considered.

The sizing method used is based on trial-and-error, with different dimensions being tested until we obtain dimensions that do not overflow.

The dimensions adopted for scenario 2 are given in **Table 5**. These dimensions were determined in order to avoid overflows in the open section and saturation in the downstream opening.

Table 5 - Dimensions selected for scenario 2 (refer to Figure 32 for names)

	Open air section			Downstream canalised section
	Bottom width (m)	**Middle width (m)**	**Top width (m)**	**Width (m)**
TR2	1.2	1.6	2	1.5
TR5	1.8	2.2	2.6	1.5
TR10	2.5	2.9	3.3	1.9

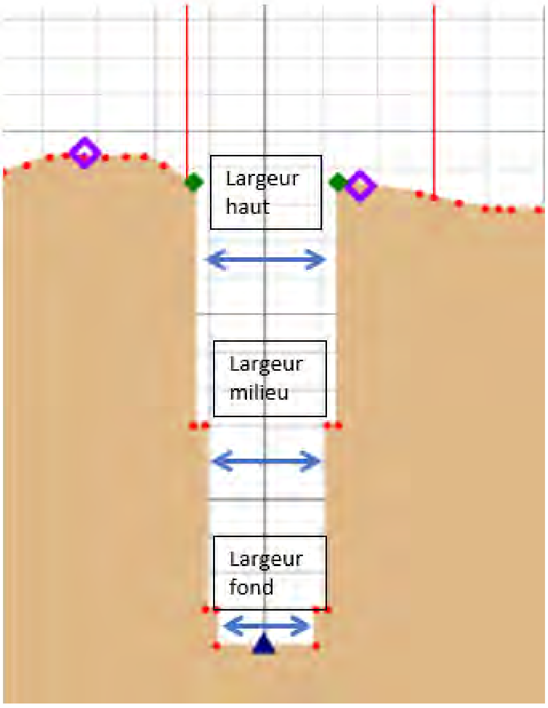


Figure 32 - Dimensions of open-air section for scenario 2

For the planned situation, the results are presented in the form of water levels on the profiles in **Appendices 5** and **6** (for profile numbers, refer to **Figure 33**). Tables defining the geometry of the profiles (bottom elevation, distance from one profile to another, slope from one profile to another) are presented in **Appendix 7** for scenarios 1 and 2.


1006
Rue sur les Roches
1006.3
1007
Rue Louis Lemeinager
Boucherie Denooz
10
Rue du Village
Rue Bocquenade
1008
1009
1012
1013
artisanale
Eglise Saint Gilles
PL-01
1015.9
1017.09
1018
1019.3
11
1021
1022.1
Rue sur Steppes
1022.6
1023.1
Ecole libre de Fraipont
1024
12
Fraipont
Rue du Village
Rue du Village

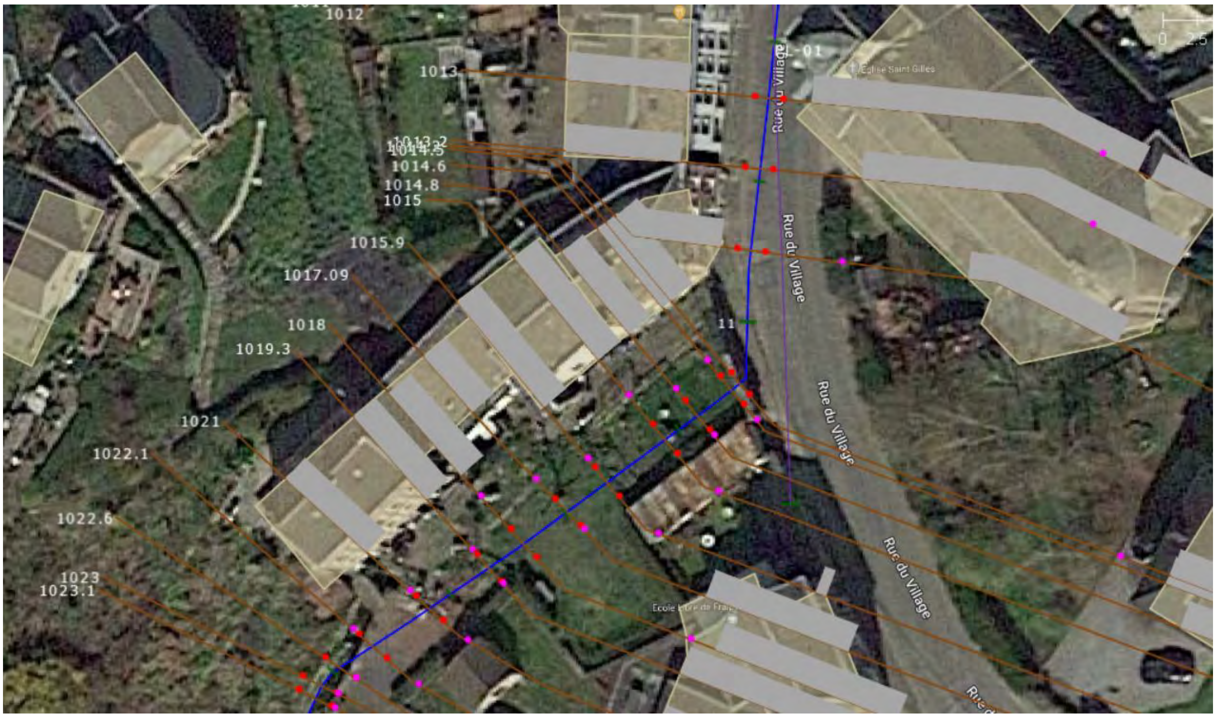


Figure 33 - Plan of the planned situation scenario 2

4.5 INTERPRETATIONS

4.5.1 Analysis of the results of the existing situation

The profile numbers are shown in **Figure 31**.
The calculated water line is shown in **Appendix 4**.
The following observations can be made:

- For a flood with a return time of 2 years, the flows follow the bed of the watercourse as far as the river Vesdre, with an overflow upstream of the project area. This starts at a low point on the right bank at profile 1023.1 (illustrated in **Figure 34**). This depression in the bank profile is a source of overflow from the 2-year flood. Profiles 1013, 1012 and 1010 (**Figure 35**), which lie under the village street, also show saturation of the openings for flood level TR2.

Thus, the existing situation already shows several areas of saturation of the vaults and an overflow point at the 2-year return period flood;

- Beyond the TR2, the overflows are more and more marked in the open areas, flooding the gardens of the houses and then the centre of Fraipont via the village street.

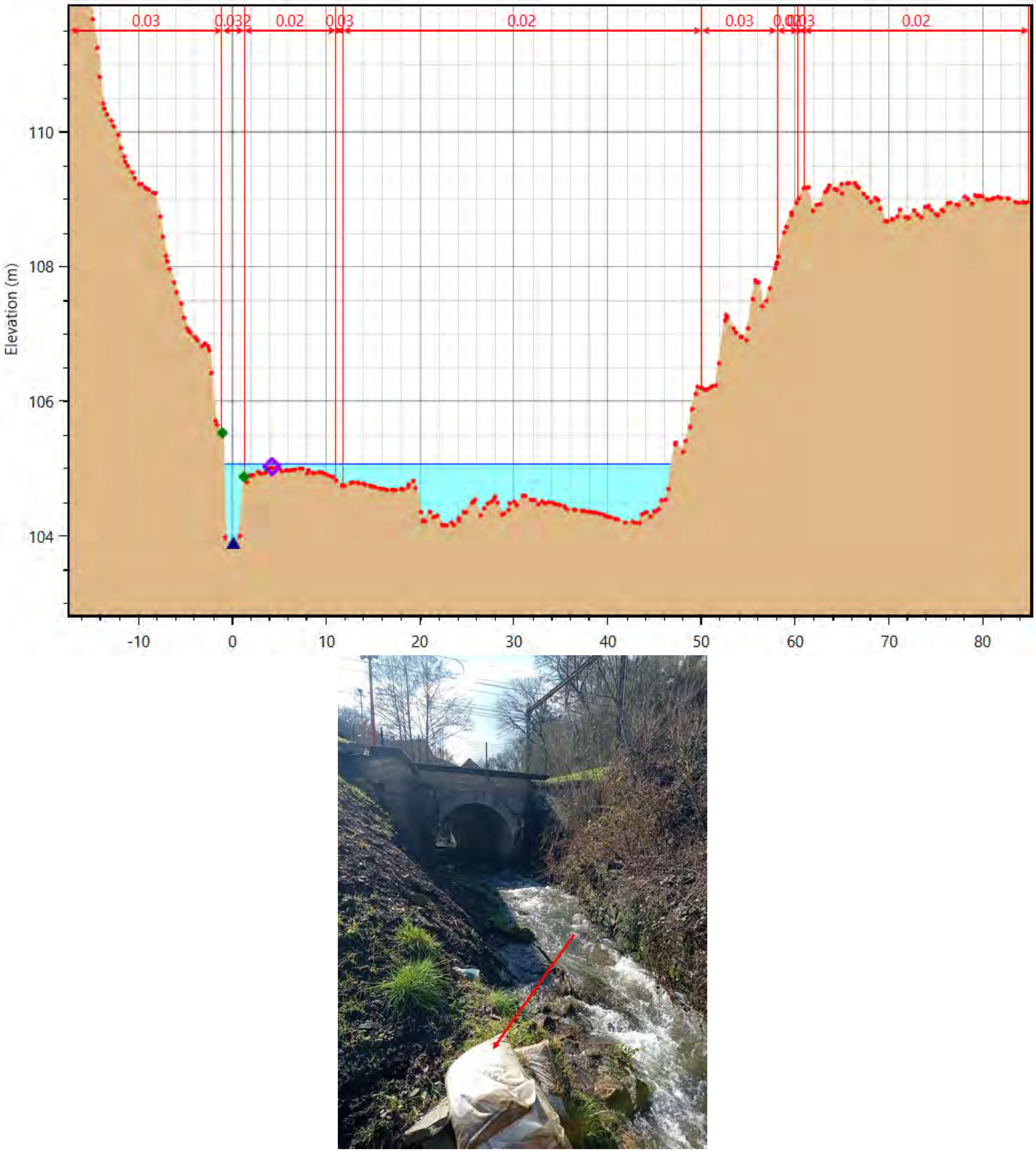
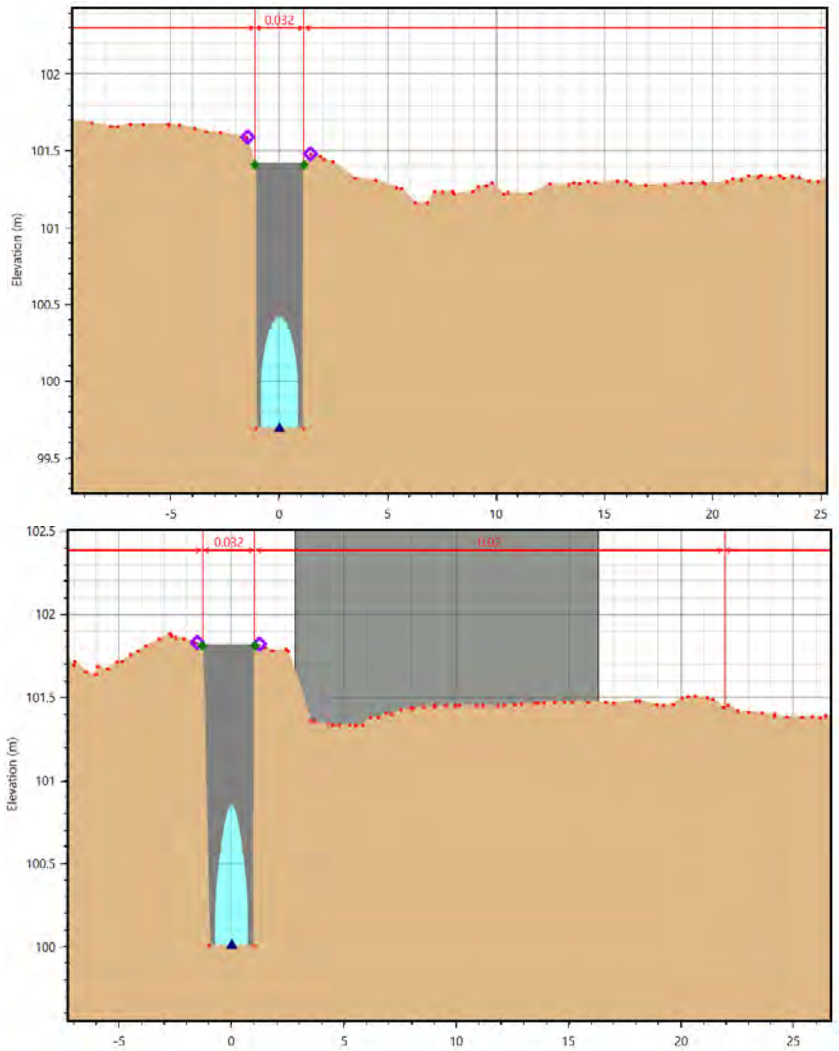


Figure 34 – Low point on the right bank in profile 1023.1 (top : GEOHECRAS TR2 profile, bottom: photo taken in the field)


0.032
102
101.5
101
100.5
100
99.5
Elevation (m)
-5
0
5
10
15
20
25
102.5
0.032
0.07
102
101.5
101
100.5
100
Elevation (m)
-5
0
5
10
15
20
25

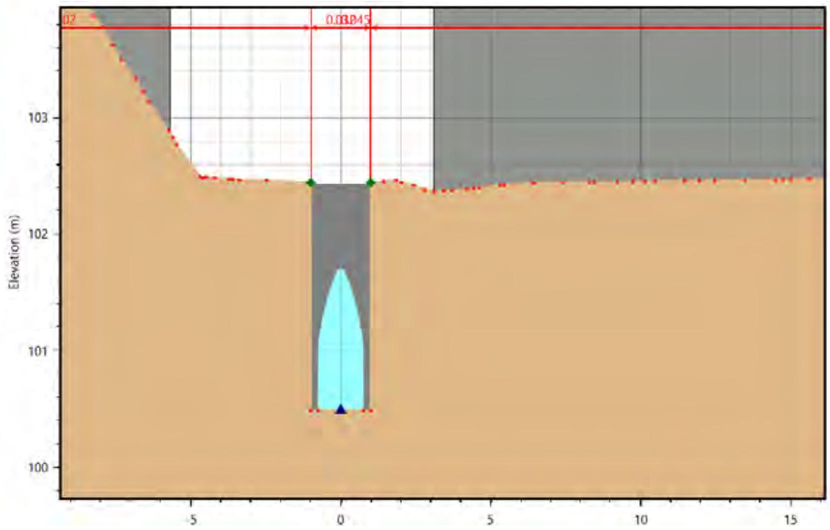


Figure 35 - Profiles 1010, 1012 and 1013 (from top to bottom) saturated at TR2 in the existing situation.

4.5.2 Analysis of the planned results (scenario 1 : replacement of the downstream gate)

The profile numbers are shown in **Figure 31.**

The calculated water line is shown in **Appendix 5**.

For this scenario, the final width dimensioned for the downstream canalised section is 1.5m.

The following observations can be made:

- For a flood with a return period of 2 years, the open section upstream of the start of the vaulting under the dwellings (1021.7) is sufficient to prevent overflows. However, as mentioned earlier, this is the case on condition that the subsidence on the right bank at profile 1023.1 is filled in, for example by extending the low wall of the following right bank profiles. This first measure would therefore be an improvement on the existing situation. By preventing overflow, the vaulted profile 1020 under the dwellings (critical sections) has a higher water load than in the existing situation and saturates the opening (**Figure 36**). It can also be seen from standard profile 1010 that the new canalised section downstream is not saturated at TR2 (**Figure 37**);
- For TR5, overflows are beginning to be observed at profile 1023.1 beyond the existing low wall (**Figure 38**). Profile 1020.1 is also saturated in addition to 1020. The new channelised opening downstream remains unsaturated;
- Beyond that, the vaulted passageways under the houses in the first section are saturated and overflows can be seen in the gardens towards the centre of Fraipont.

Figure 36 - Profile 1020 saturated at TR2 in planned situation 1.

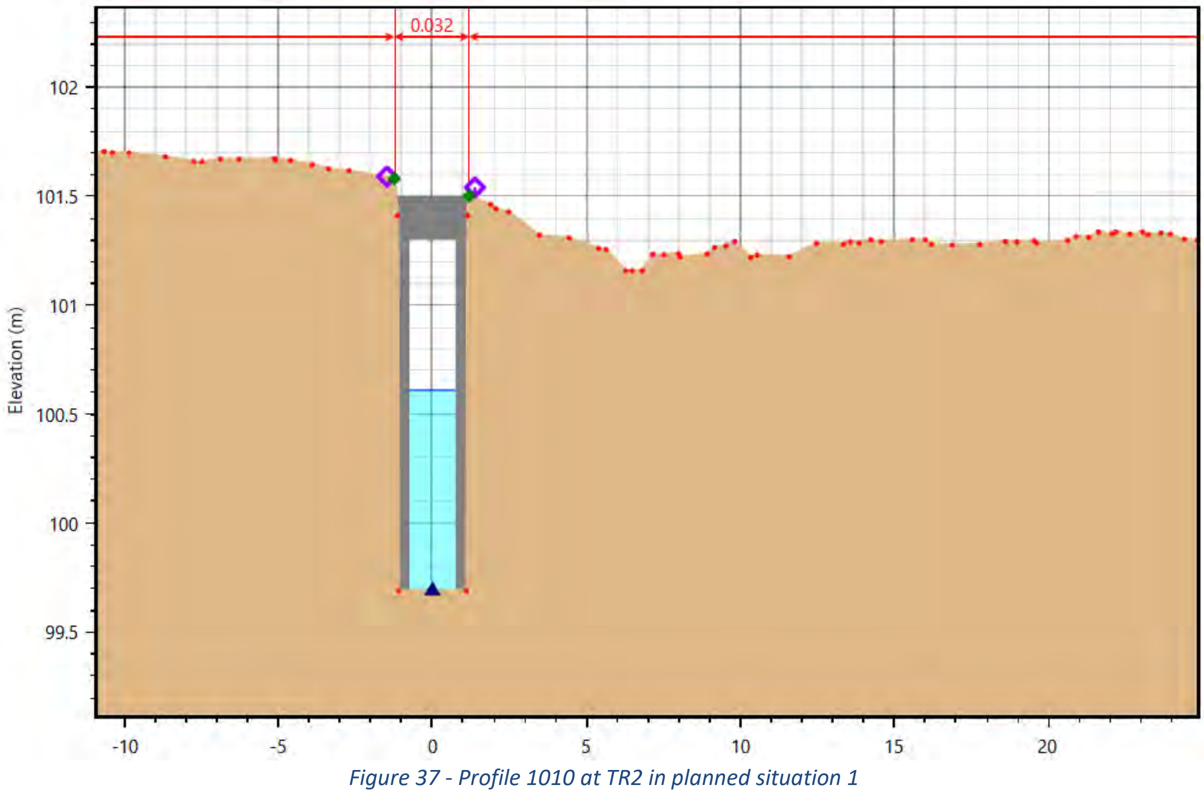


Figure 37 - Profile 1010 at TR2 in planned situation 1

Figure 38 - Profile 1023.1 overflow at TR5 in planned situation 1

4.5.3 *Analysis of the results of the planned situation (scenario 2 : replacement of the downstream sluiceway and reopening of the upstream section)*

The profile numbers are shown in **Figure 33**.

The calculated water line is shown in **Appendix 6**.

Beforehand, several simulations were carried out in which the width of the upstream open section was successively increased to avoid overflowing. The downstream opening under the rue du village was replaced.

The following observations can be made:

- To contain a TR2 without overflow on the open profiles (1024 to 1014.6), the basic dimensions provided by the STP are sufficient (1.2m width at foot).
- From TR5 onwards, the width at the foot of the open section must be increased to 1.8 m (e.g. profile 1021, **Figure 39**).
- From TR10, the canalised section downstream becomes saturated at profiles 1010, 1009 and 1008 (**Figure 40** for profile 1010 and **Figure 41** for profile 1008) but no overflow is observed.
- Beyond TR10, with the maximum dimensions imposed on the downstream vault under the Rue du Centre (1.5+0.4 m, i.e. 1.9 m wide - **Figure 40**), overflows will be observed in the upstream open-cut section. The increase in width from 1.5 to 1.9 m will delay saturation of the downstream opening.

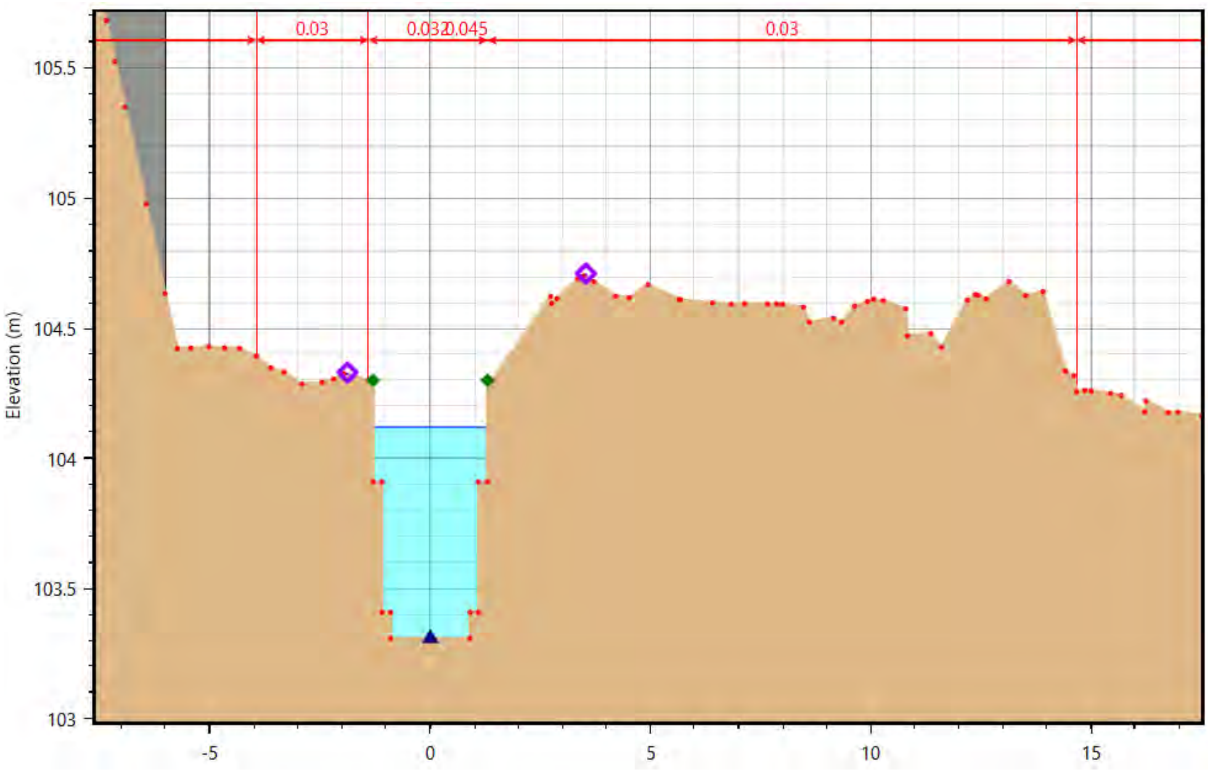
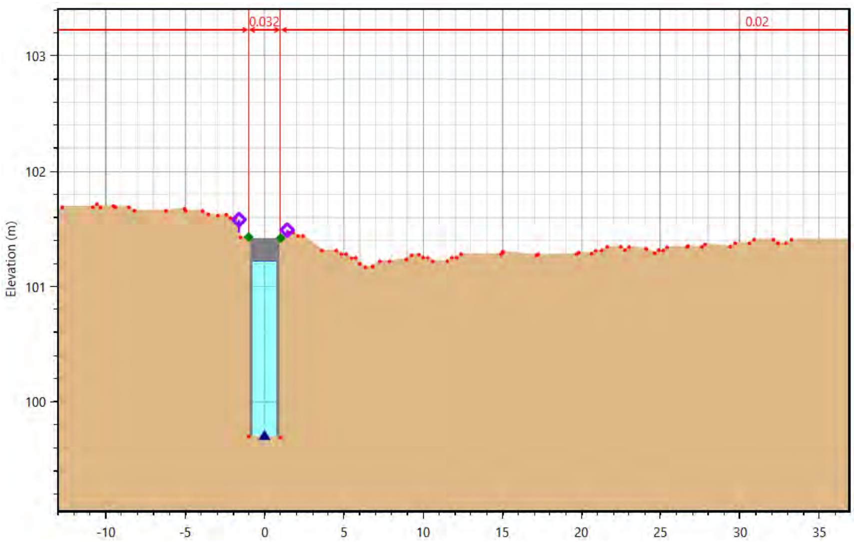


Figure 39 - Profile 1021 at TR5 for scenario 2. Dimensions adapted to contain TR5.



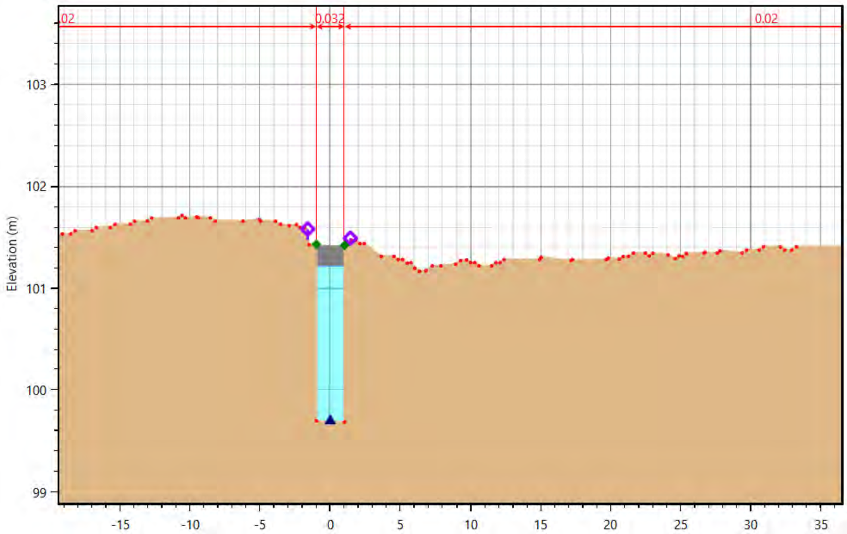


Figure 40 - Saturated profile 1010 at TR10 for scenario 2, top : with a width of 1.5m, bottom: with a width of 1.9m

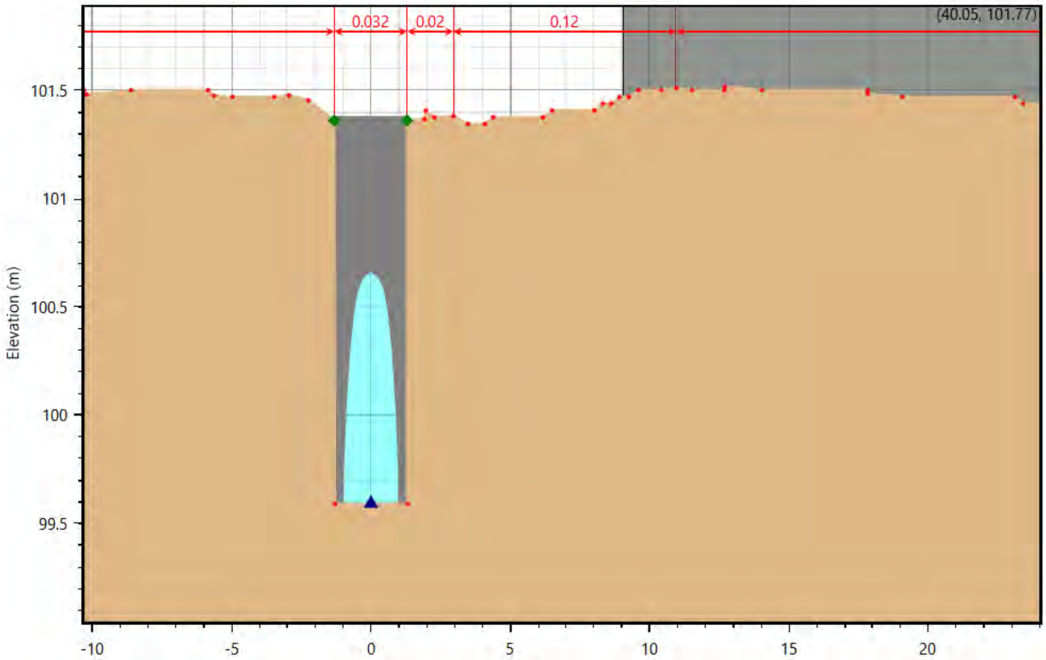


Figure 41 - Profile 1008 saturated at TR10 for scenario 2.

4.5.4 Conclusions from the analyses and possible solutions

The hydraulic study shows that both scenarios improve the hydraulic situation in the project area compared with the current situation.
In all cases, the lowering of the right bank at the exit of the bridge under the railway needs to be repaired.
For scenario 1, the existing situation is improved by replacing the arched opening under the village street with a wider opening, and an overflow can be observed between TR2 and TR5 at garden level (as opposed to before TR2 for the existing situation).
Although the cross-section of the downstream opening under the rue du village has been increased, overflow can be observed in the gardens upstream of the project area due to the existing critical cross-sections under the dwellings.

In scenario 2, the situation is improved by opening up the vaulted openings under the houses behind the gardens and adapting the dimensions of the open section. No overflow can be observed up to TR10, where the downstream opening reaches saturation point. Beyond TR10, overflows are observed.

Scenario 2 is therefore recommended to maximise improvements and protect residents over a larger TR:

- Work on the arched opening under the village street (inside dimension of the opening = 1.9m);
- Work on the upstream section under the dwellings to make it open to the air, with a bottom dimension of at least 2.5 m.

4.6 LIMITS OF THE STUDY

The results presented in this study are based on a simplified model of the actual situation. The simulations are carried out on the basis of systematic in situ surveys collected by the STP and our teams for the Haveignée stream and by the Direction des cours d'eau non navigables for the river Vesdre.
These surveys are considered to serve as a basis for developing the geometric model, calibrations and the formulation of working hypotheses and a parameterisation adjusted to reality. However, as the data dates from the 60s and 70s for the minor bed of the Vesdre, it is likely that more or less significant changes have taken place in the minor bed (sediment transit, floods of 1993, 2003, 2011, 2021, etc.). They should therefore be treated with caution. As the data for the major bed is more recent (lidar from 2013), as is that for the tributary, the error and uncertainty for this area is smaller.

The modeller's expertise makes it possible to optimise the uncertainties associated with the model construction data (topography and bathymetry), the modelling assumptions, the calibration assumptions (friction coefficients, intermediate inputs, hydraulic parameters) and the operational use of the hydraulic models (initial conditions, upstream and downstream boundary conditions, etc.). This hydraulic study is therefore indicative, and the authors cannot be held responsible for any discrepancies with reality. The results presented have been verified on the basis of the information available to us (calibration, etc.) and their interpretation is intended to guide the project owner and the public authorities in their choice of developments.

The following limitations may also affect the accuracy of the results:

- The hydraulic model was not calibrated on the basis of in situ measurements taken within the project area. The results obtained are consistent with the sources compared for calibration, but altimeter readings or high-water marks at different locations in the project area would have enabled the hydraulic model to be calibrated more accurately. Conventional" literature values for friction and flow coefficients and boundary conditions were applied to the model.
- In sizing the openings in the project area, it was assumed that the river Vesdre had no hydraulic influence on the project area, which is potentially not the case in a real flood event. The river Vesdre begins to overflow the project area at Place Emile Vandervelde at TR10. The rise in the water level in the river Vesdre and its subsequent overflow may lead to a blockage effect at the level of the sluice and present different results from the simulations carried out as part of this study.
- The model conditions are limited to the study area, without taking into account the possible influences of phenomena located upstream or downstream of the study area. Nevertheless, the boundary conditions were taken far enough away from the project area (a 450m section of the Vesdre was taken into account) to limit their influence on the model results as much as possible;
- Buildings are considered as obstructions to the flow that they influence. For the purposes of this study, all permanent buildings (listed in the OSM openstreet map database) are considered. “Light" or removable structures or buildings and any other equipment are not taken into account. They can nevertheless create point obstructions or accumulate, in the same way as vegetation, at the level of structures on watercourses by developing logjams.
- Logjam management is a crucial factor during floods. Given the presence of a single pile for the N61 bridge and the absence of piles for the other structures in the Vesdre minor bed, the modelling does not take into account the formation of logjams at the structures. An obstruction and sedimentation at the level of the Haveignée stream has also not been considered, but significant slopes have been retained in the project to limit these aspects.

5 CONCLUSIONS AND RECOMMENDATIONS

The project involves adapting the dimensions of the culvert under the Route du Village and Place Emile Vandervelde and the route of the Haveignée stream in the village of Fraipont, and opening up the part of the culvert under the dwellings. This follows the deterioration of the culverts caused by the exceptional rainfall event that took place from 13 to 15 July 2021, as well as the regular overflowing of the course of the stream caused by the undersizing of the culvert.

Once the geometric model had been produced, the results of the existing situation were compared with several observations available in the literature consulted (Calibration). The different approaches show that the model developed in this study is robust and can therefore be considered validated and calibrated for the floods studied.

The results were given for two different planned scenarios. The first scenario proposes to keep the current upstream sections partly under vaults and to replace the downstream sections with a wider opening (except for the last section). The second scenario proposes to add to the first one an open-cutting of the upstream sections up to the passage under the railway.

The sizing of these structures was proposed in order to avoid overflows as much as possible for flows associated with 2, 5 and 10-year return period floods. These results were compared with each other and with the existing situation.

Overall, it was found that the two scenarios in the projected situation improved the hydraulic situation in the study area.

Scenario 2 is particularly recommended and would enable a TR10 to be channelled over the entire project area without overflowing, but with the beginning of saturation of the downstream openings.

Finally, it is important to remember that it was considered in the design that the river Vesdre had little hydraulic influence on the project area (simulations with a low flow of 20 m3/s). This fact, and the possibility of ice jams at the N61 bridge, should be considered in future flood management.

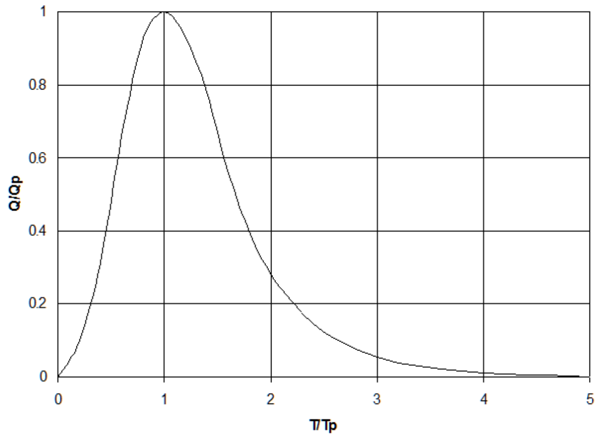
6 APPENDICES

6.1 APPENDIX 1. CN VALUES ACCORDING TO LAND USE AND HYDROLOGICAL GROUPS AND MANNING COEFFICIENT VALUES ACCORDING TO LAND USE.

Land use (WALOUS 2019)	TABLE CN SCS	Not known	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
Above-ground artificial structures		98	98	98	98	98	0.013
Rail network		90	90	90	90	90	0.033
Bare floors	fallow land (bare soil)	87	77	86	91	94	0.03
Surface waters	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
Herbaceous cover all year round	permanent grassland	59	30	58	71	78	0.04
Softwood (>3m)	favourable forest	57	25	55	70	77	0.1
Softwood (<3m)	unfavourable forest	68	45	66	77	83	0.05
Hardwood (>3m)	favourable forest	57	25	55	70	77	0.1
Hardwood (< 3m)	unfavourable forest	68	45	66	77	83	0.05

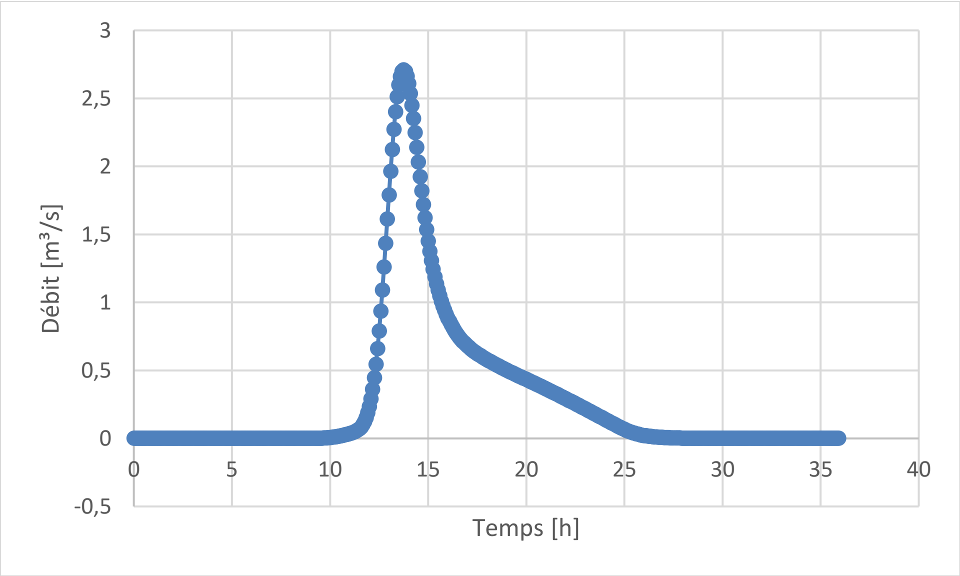
6.2 APPENDIX 2. A. ILLUSTRATION OF THE SCS SYNTHETIC UNIT HYDROGRAPH. Q (FLOW), QP (PEAK FLOW), T (TIME), TP (PEAK TIME) B. HYDROGRAPH RECONSTRUCTED USING THE SCS METHOD FOR THEORETICAL 2-,5-,10-,50- AND 100-YEAR FLOODS ON THE STREAM FROM THE LEFT AT THE MODEL ENTRY POINT

A

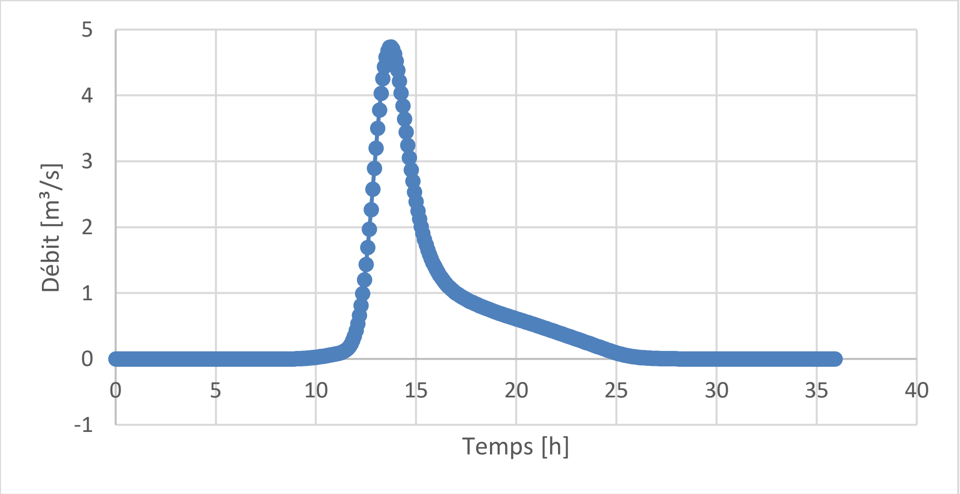


B

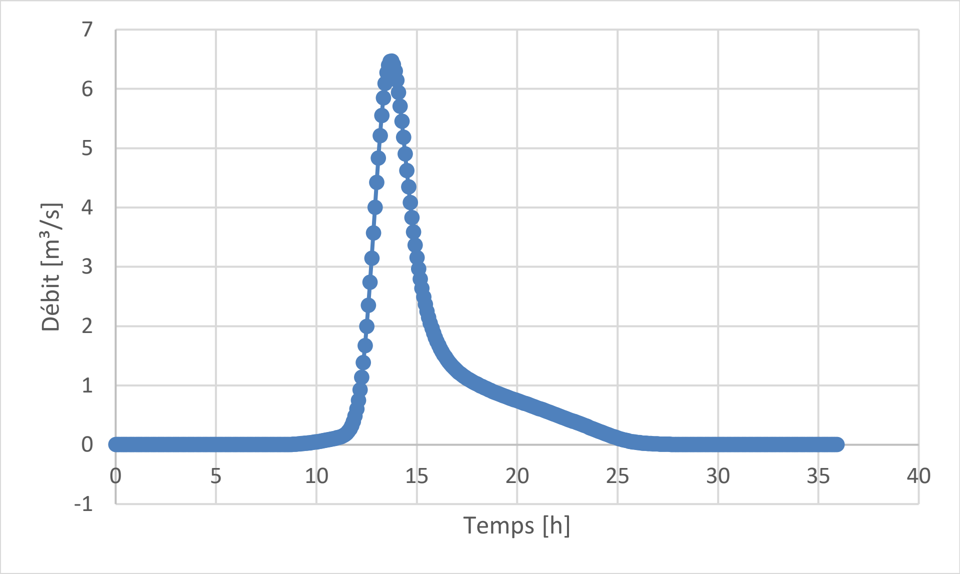
2 years



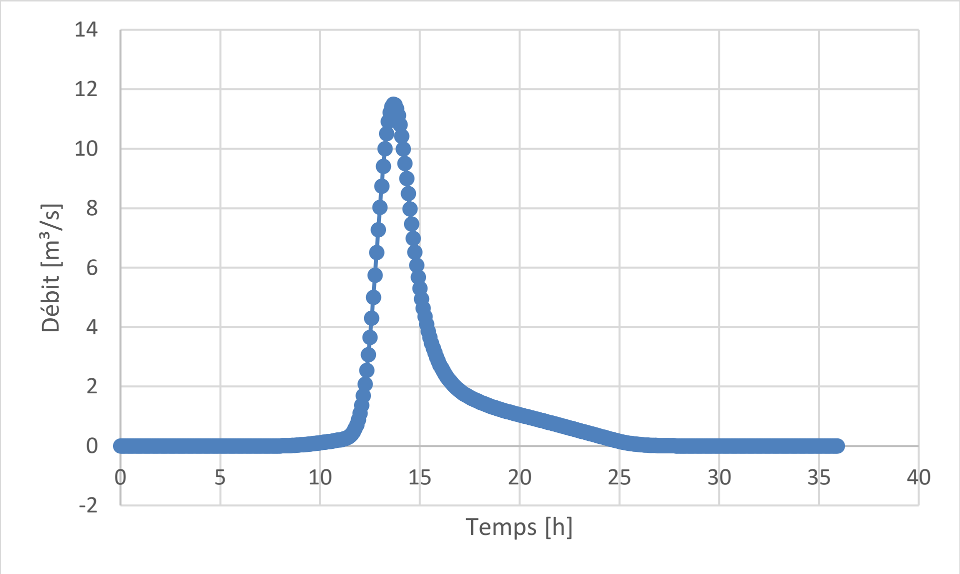
5 years



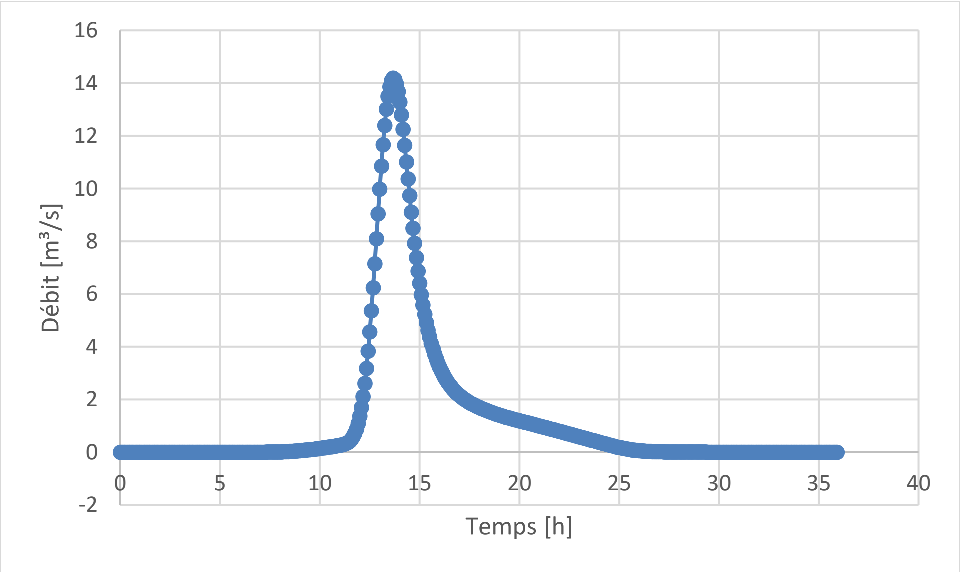
10 years



50 years



100 years



6.3 APPENDIX 3: DESCRIPTIVE SHEETS OF THE SECTIONS TAKEN INTO ACCOUNT IN THE PROJECT AREA ACCORDING TO THE SECTION PLAN (FIGURE 22)

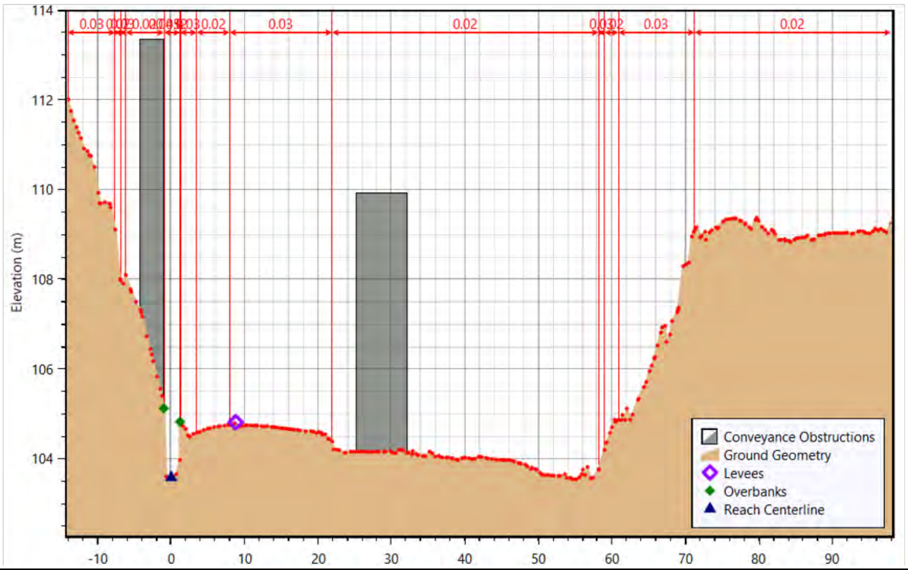
Profil ciel ouvert

Longueur (m) : 19.5

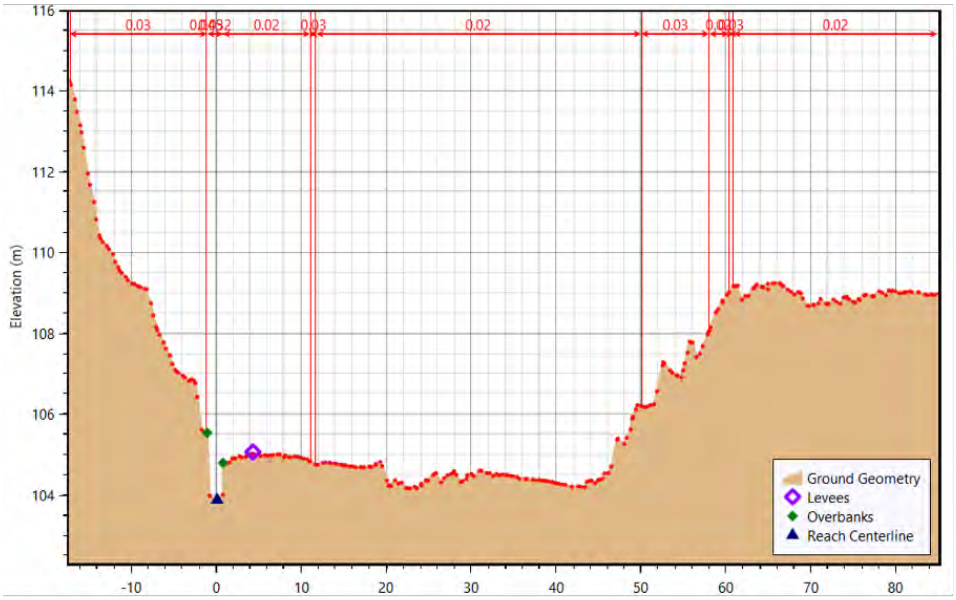
Pente moyenne (%) : 4.3

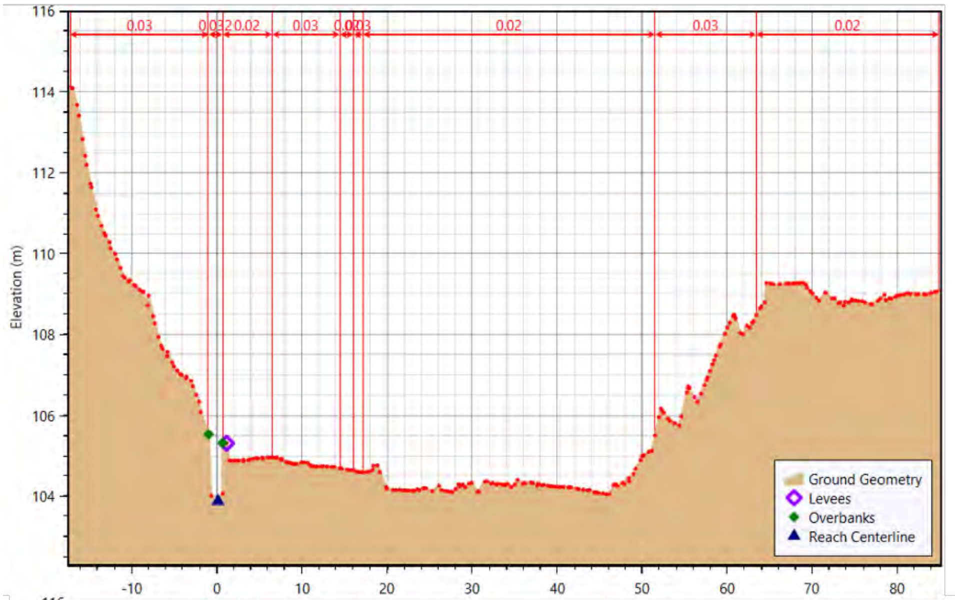
Profil amont

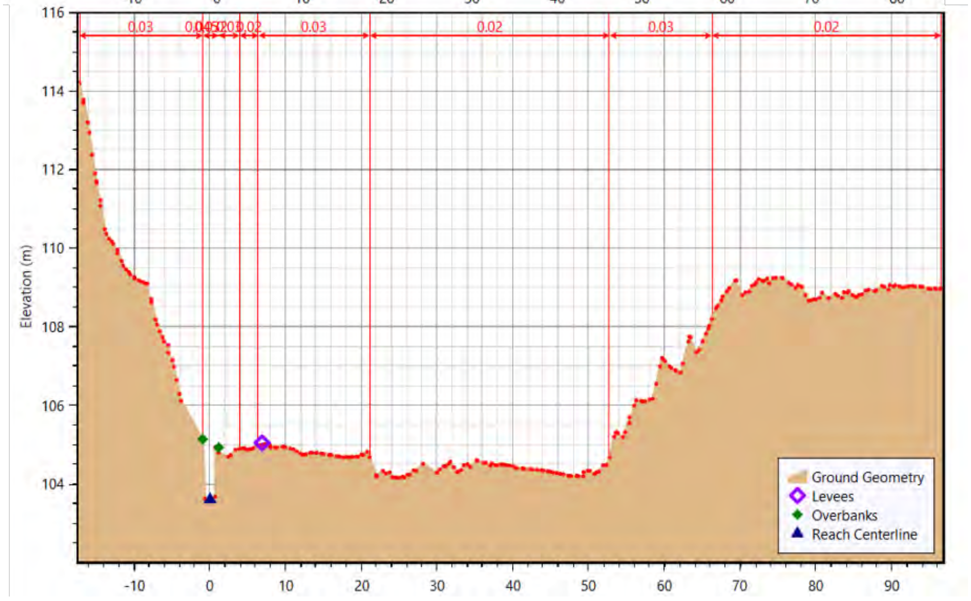
Profil aval



Profils intermédiaires





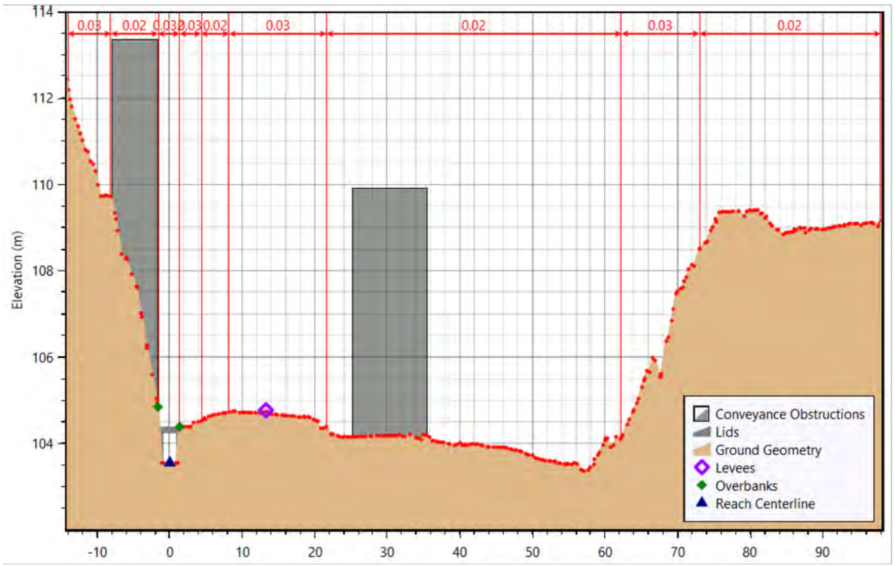


Section 1

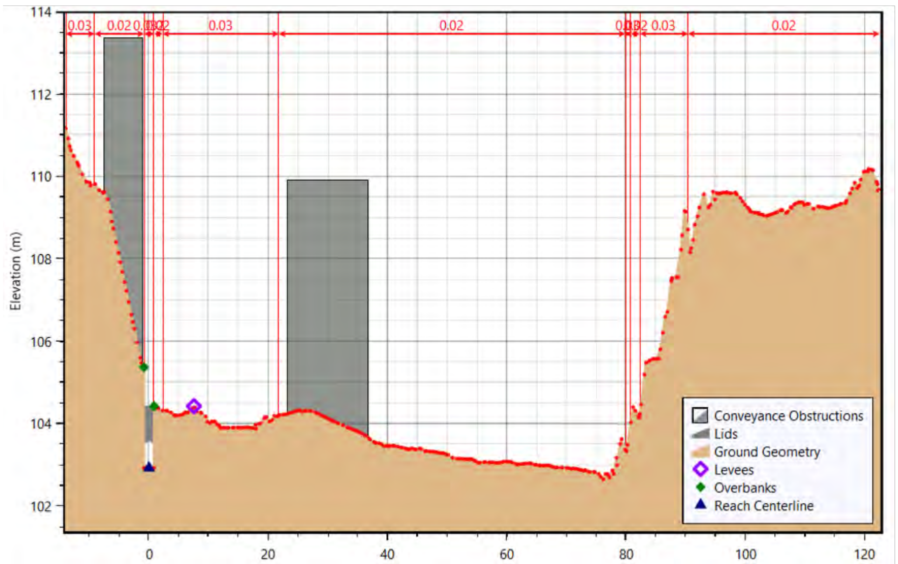
Longueur (m) : 7.1

Pente moyenne (%) : 8.8

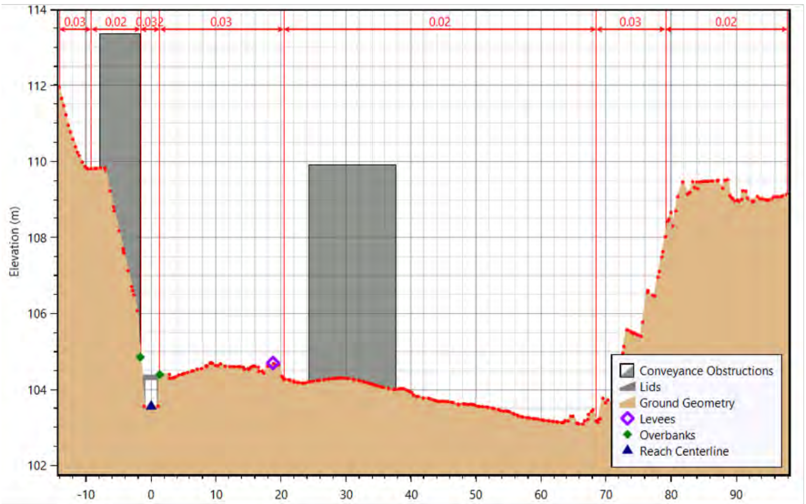
Profil amont

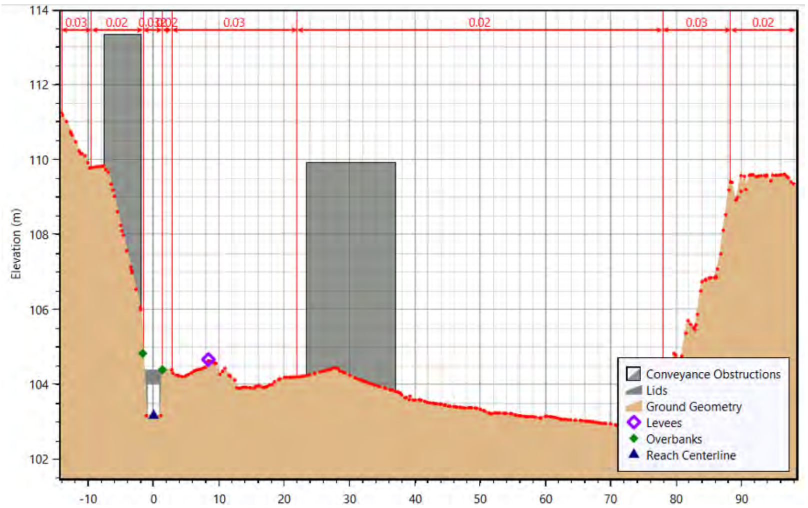


Profil aval



Profils intermédiaires



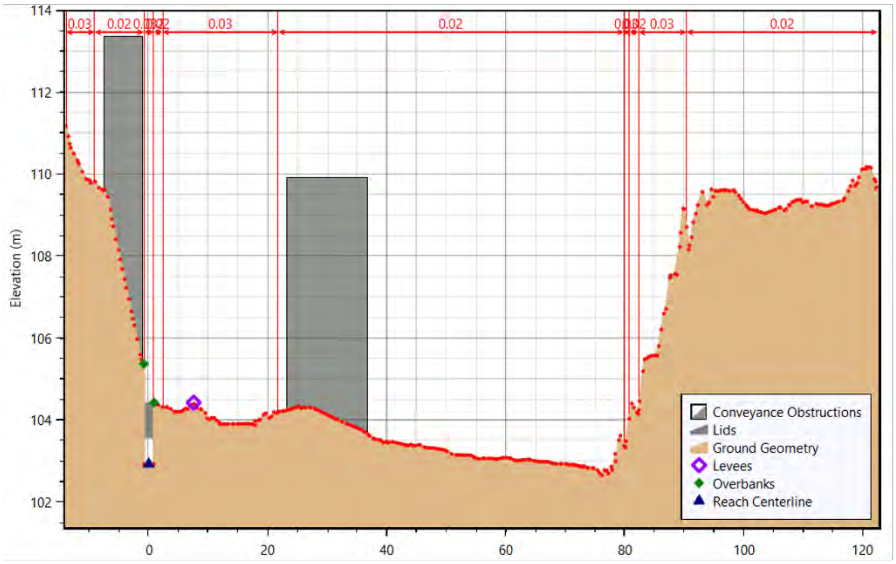


Section 2

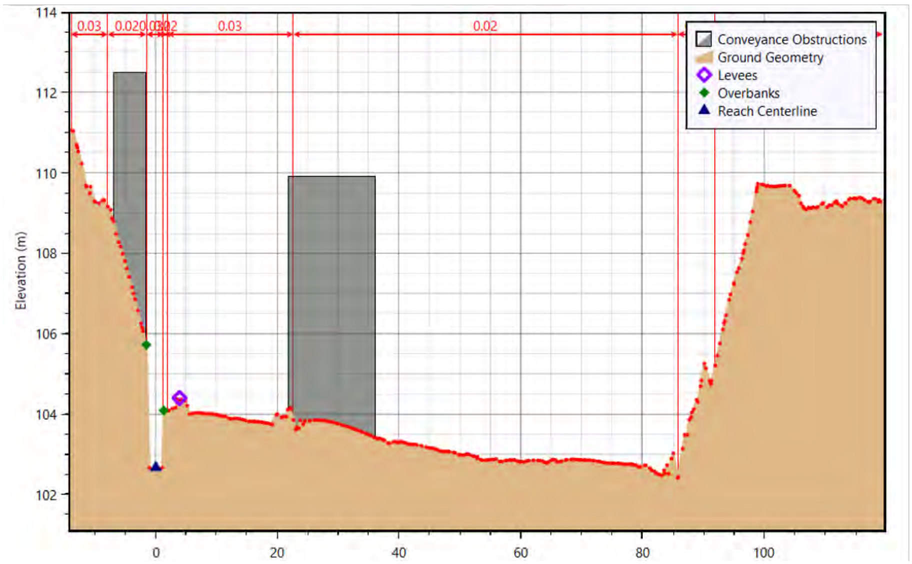
Longueur (m) : 2.8

Pente moyenne (%) : 8.9

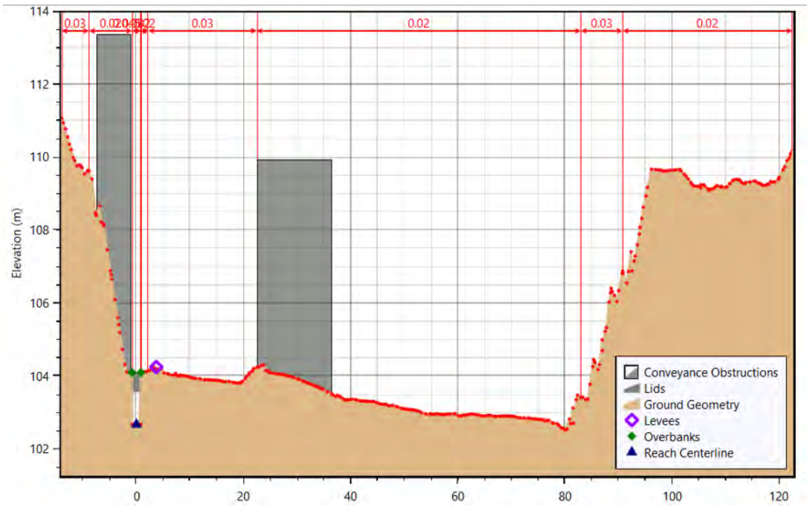
Profil amont



Profil aval



Profils intermédiaires

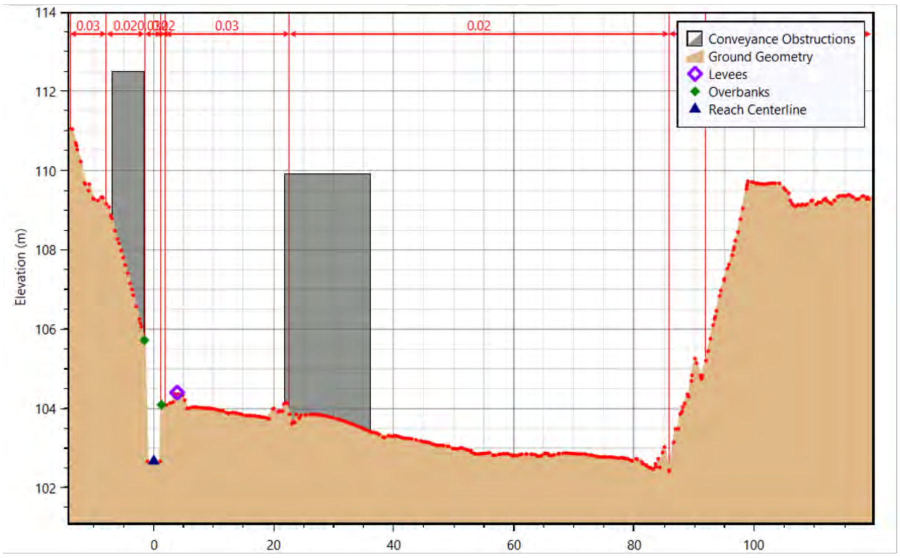


Section 3

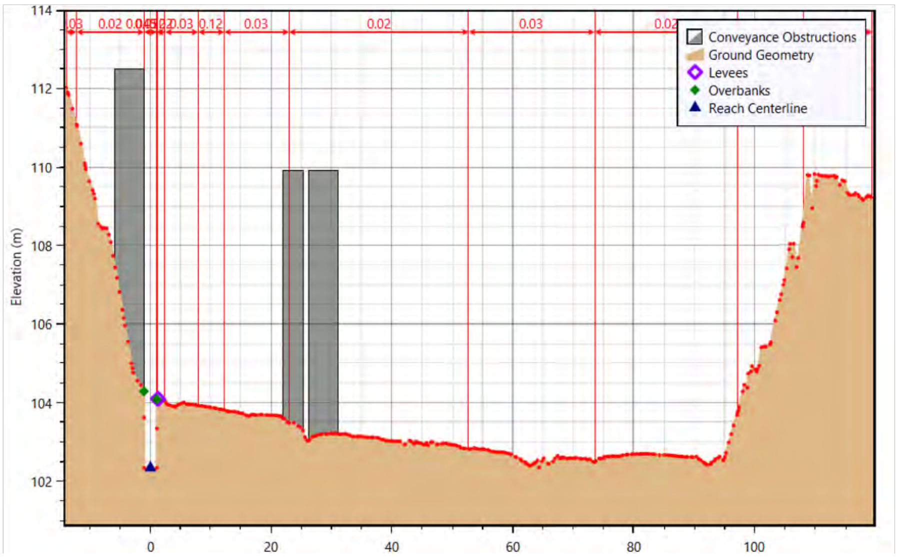
Longueur (m) : 4.3

Pente moyenne (%) : 7.6

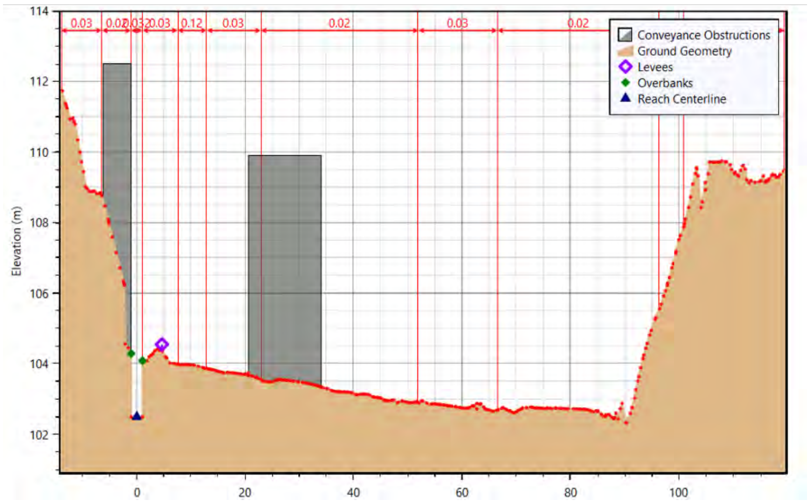
Profil amont



Profil aval



Profils intermédiaires

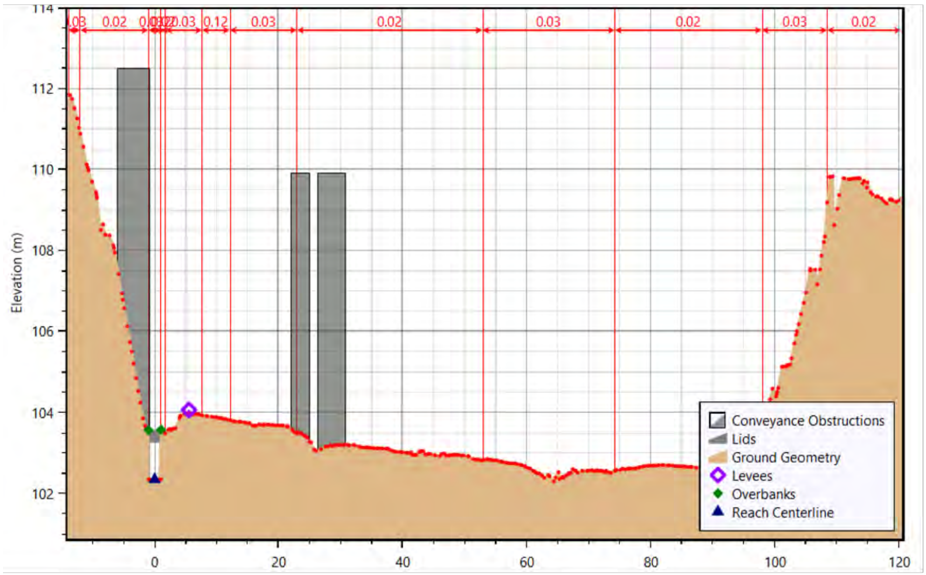


Section 4

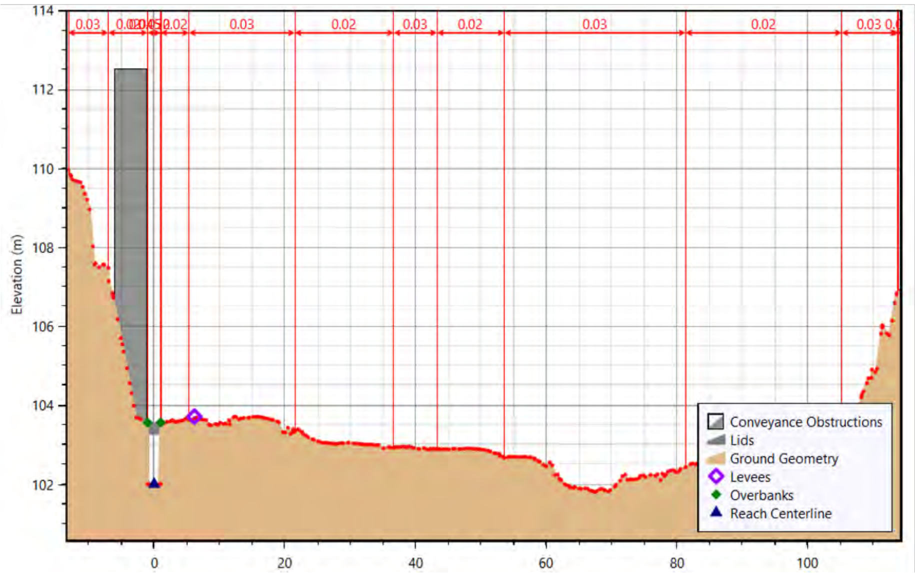
Longueur (m) : 4.9

Pente moyenne (%) : 6.9

Profil amont



Profil aval

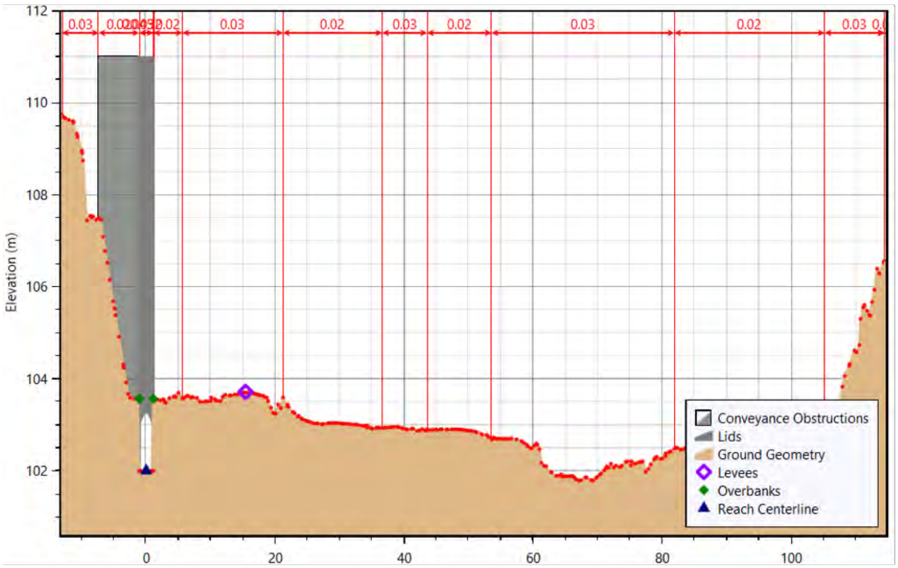


Section 5

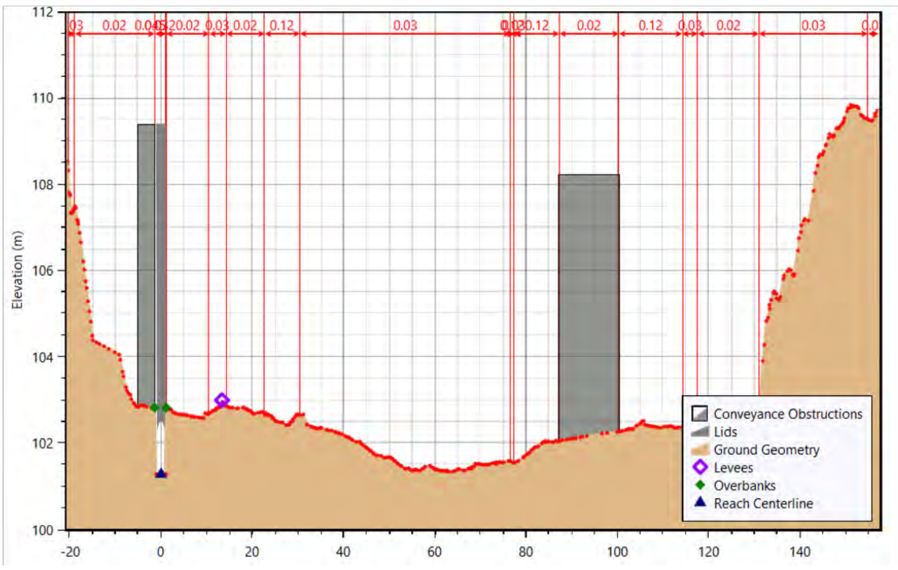
Longueur (m) : 16.5

Pente moyenne (%) : 4.4

Profil amont



Profil aval

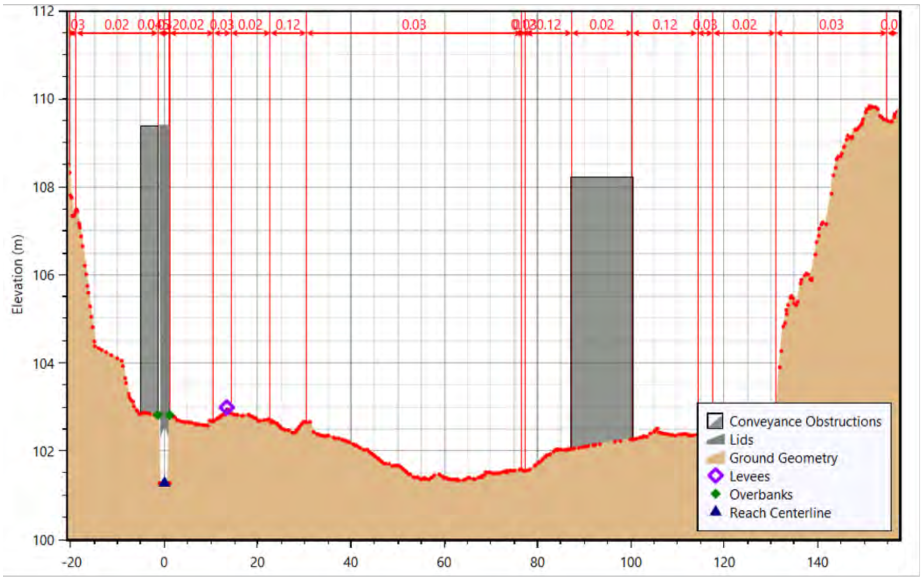


Section 6

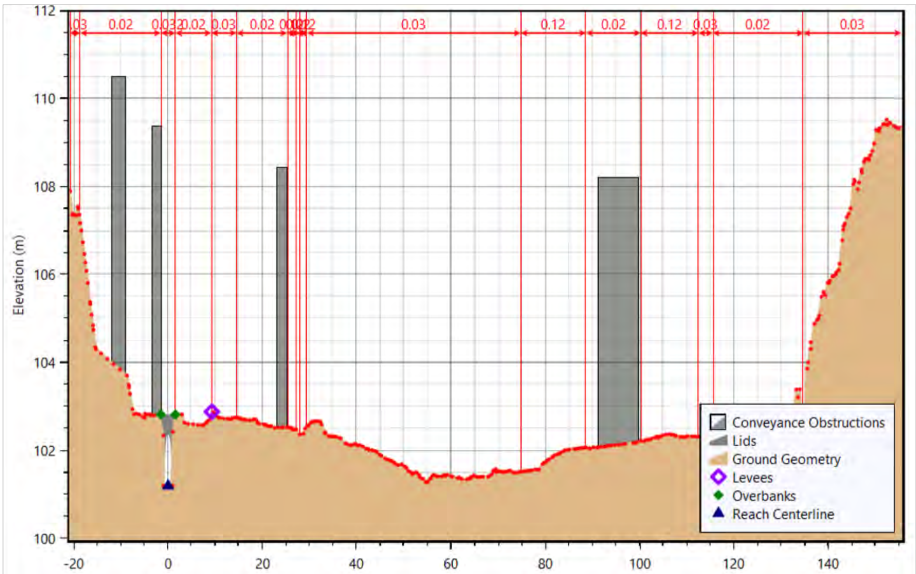
Longueur (m) : 1.6

Pente moyenne (%) : 5

Profil amont



Profil aval

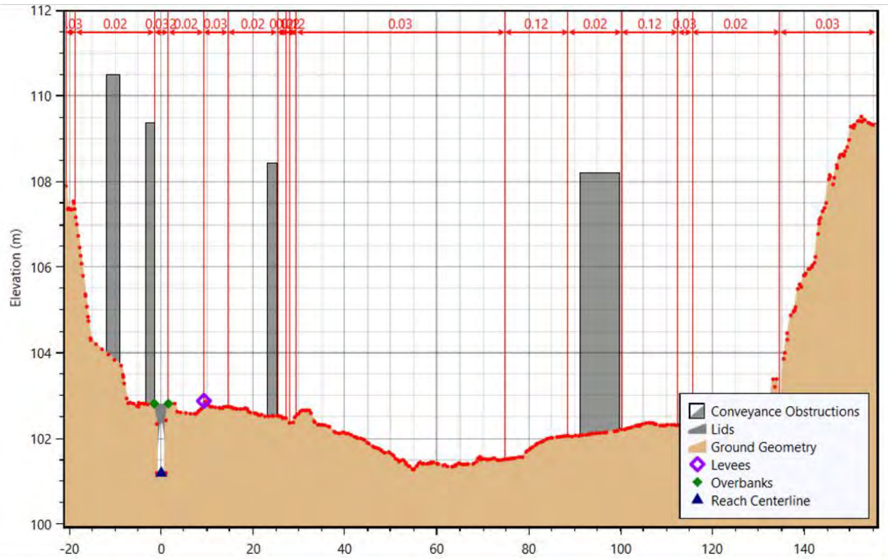


Section 7

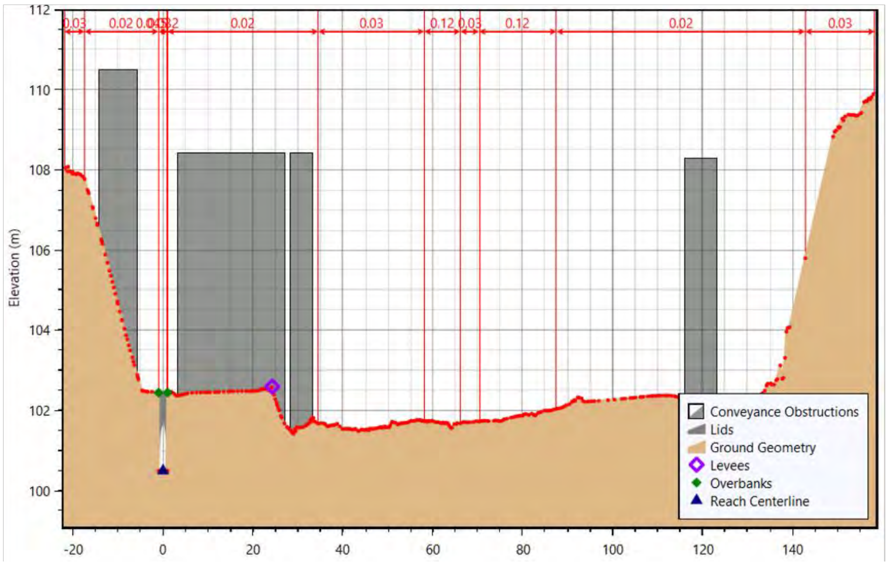
Longueur (m) : 9.6

Pente moyenne (%) : 7.4

Profil amont



Profil aval

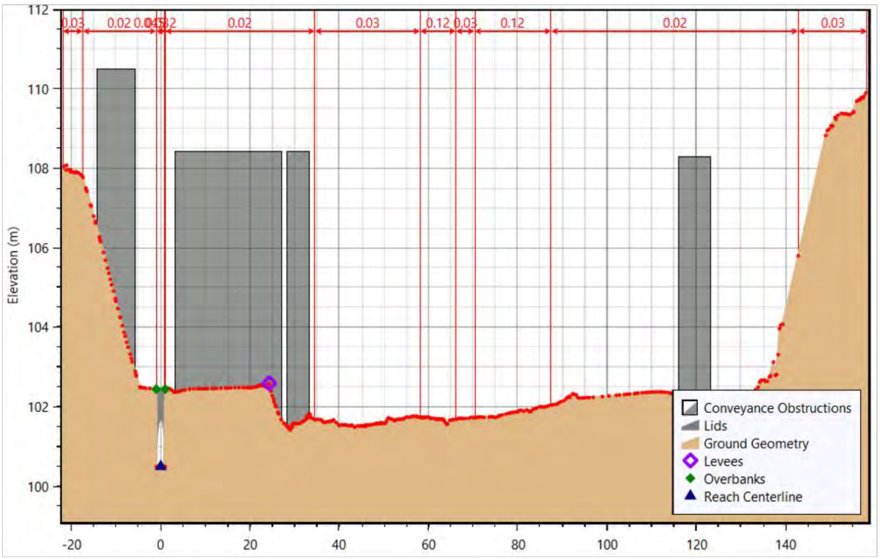


Section 8

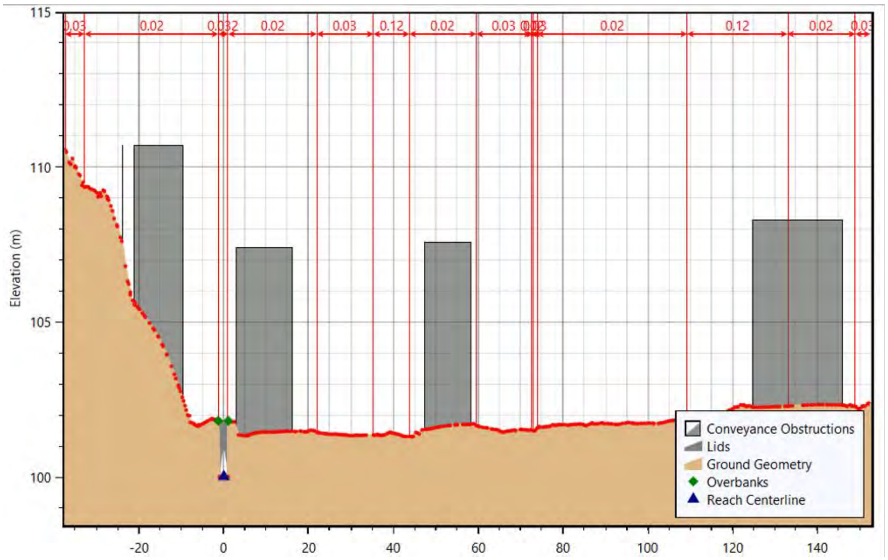
Longueur (m) : 20.5

Pente moyenne (%) : 2.3

Profil amont



Profil aval

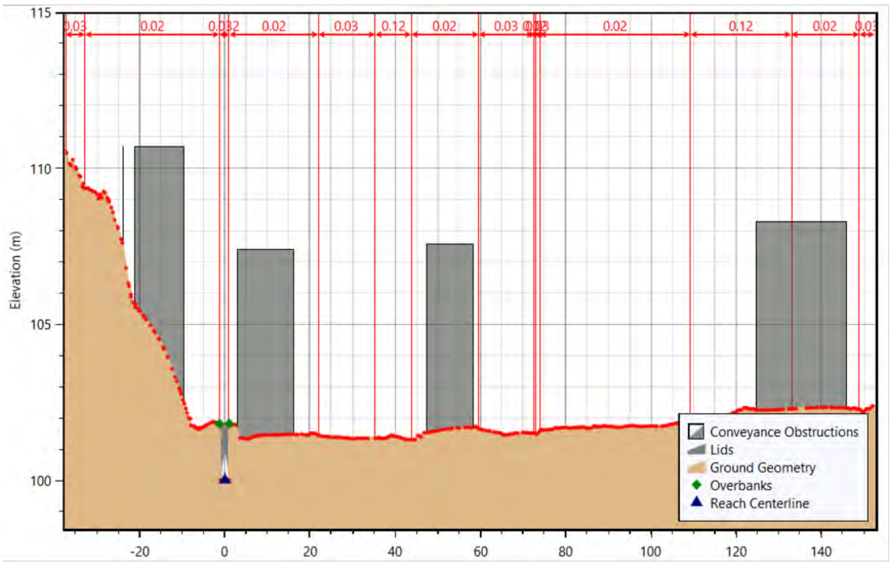


Section 9

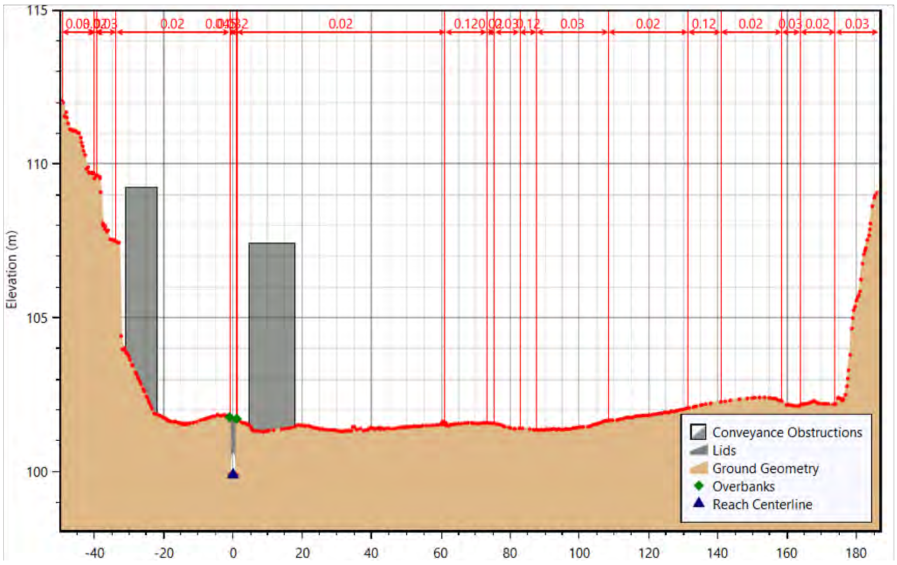
Longueur (m) : 7.5

Pente moyenne (%) : 1.6

Profil amont



Profil aval

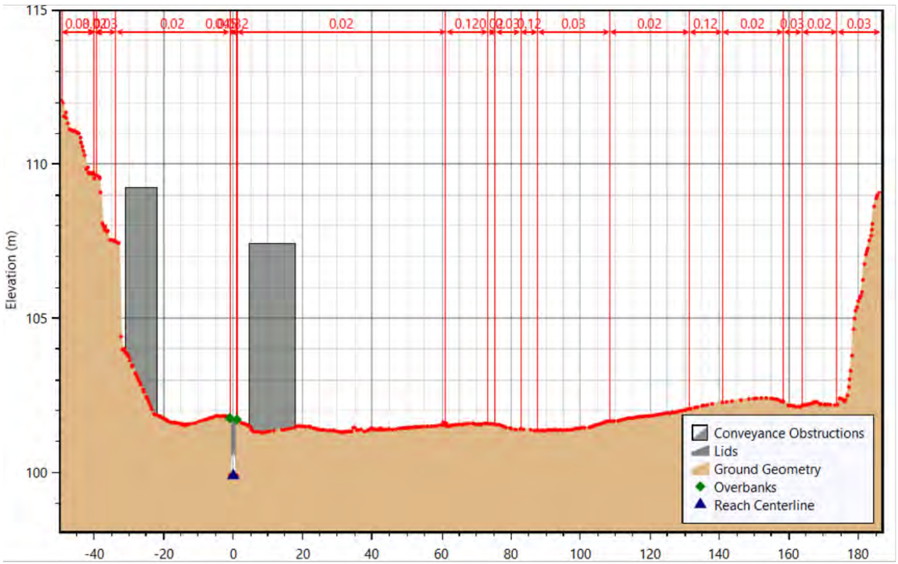


Section 10

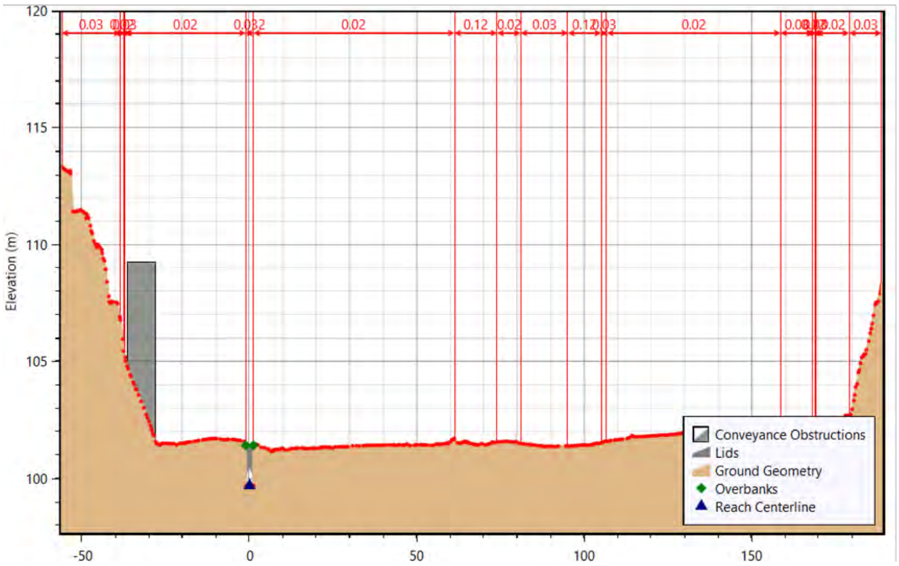
Longueur (m) : 4.6

Pente moyenne (%) : 4.3

Profil amont



Profil aval

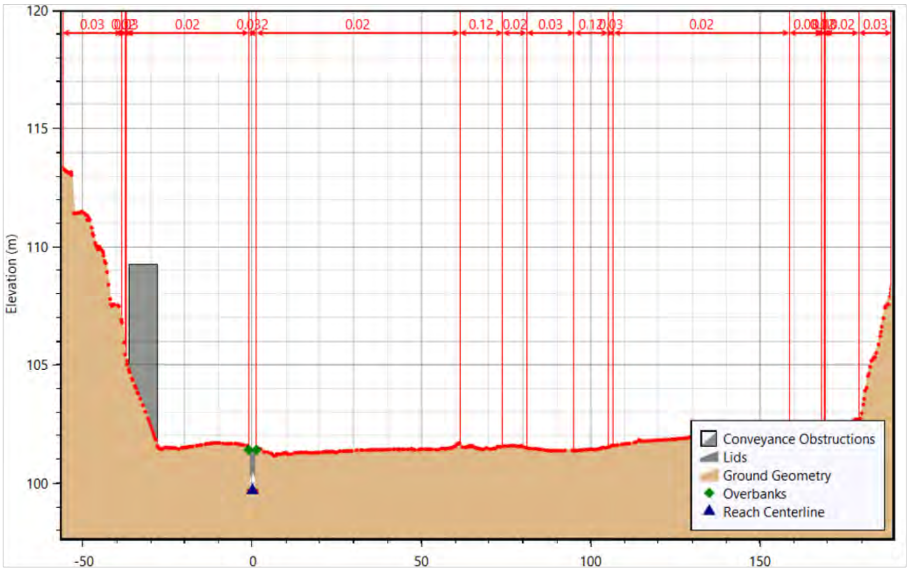


Section 11

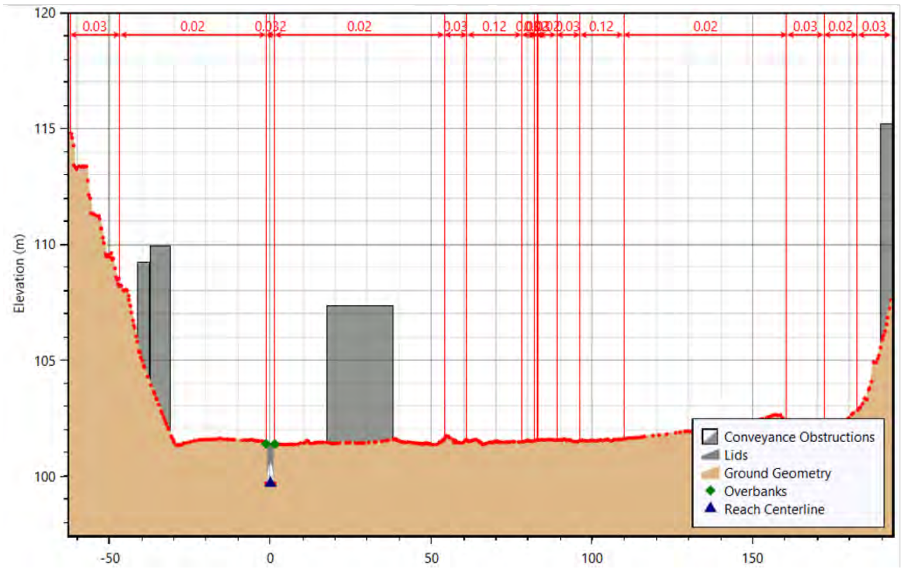
Longueur (m) : 4.5

Pente moyenne (%) : 0.2

Profil amont



Profil aval

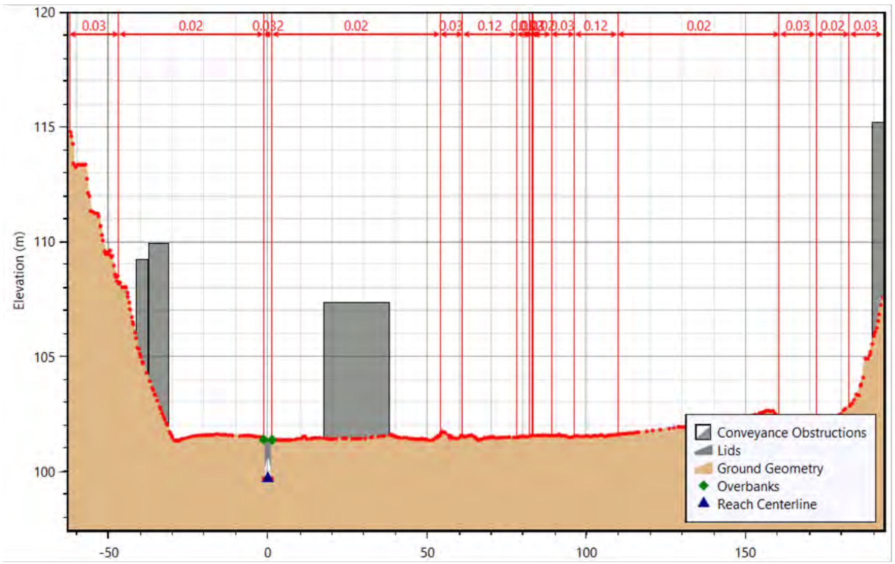


Section 12

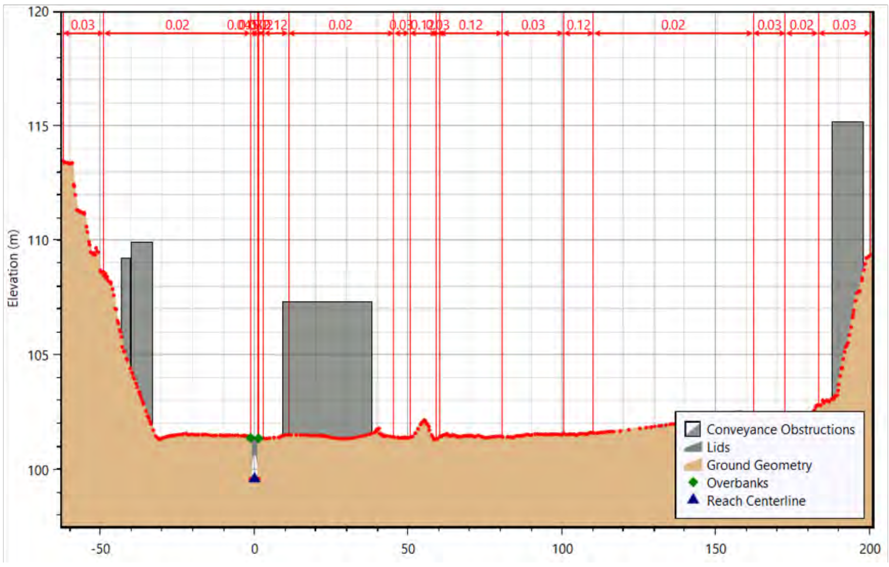
Longueur (m) : 2.2

Pente moyenne (%) : 4.1

Profil amont



Profil aval

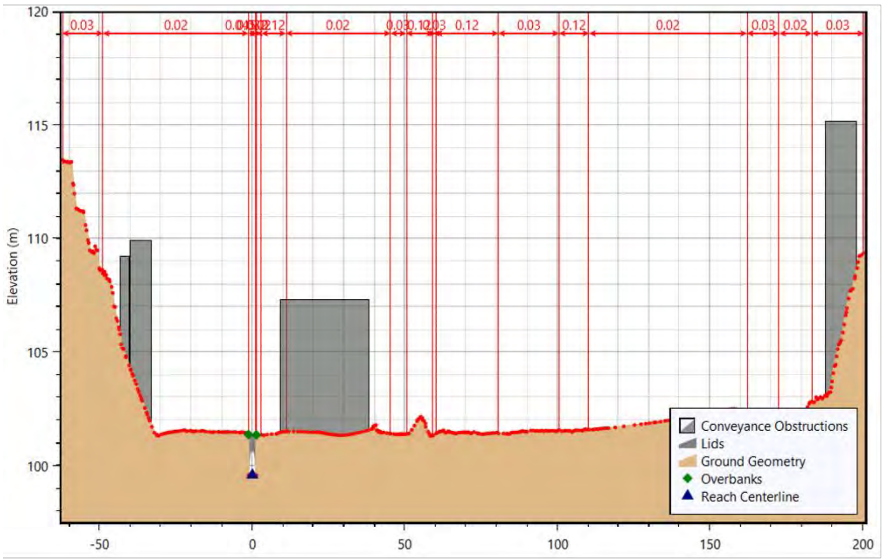


Section 13

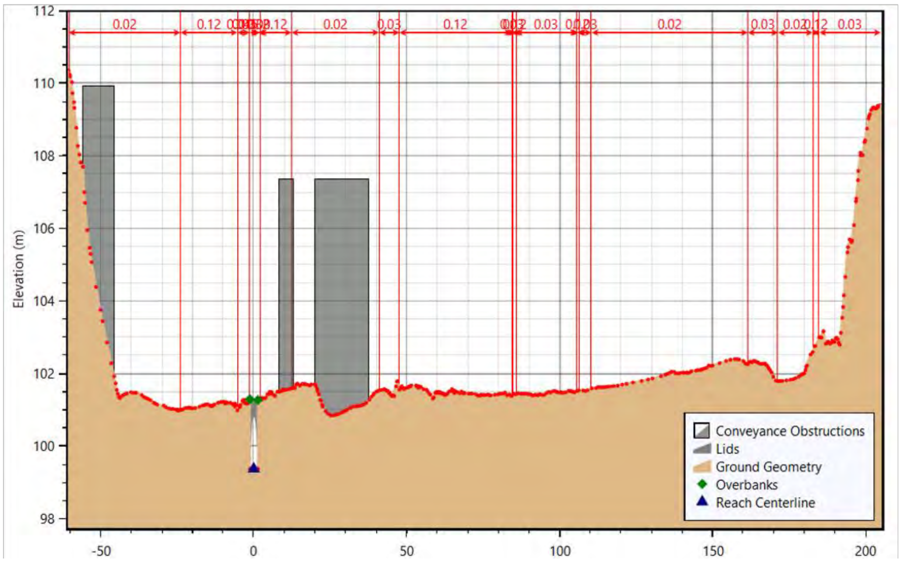
Longueur (m) : 11.3

Pente moyenne (%) : 2.0

Profil amont

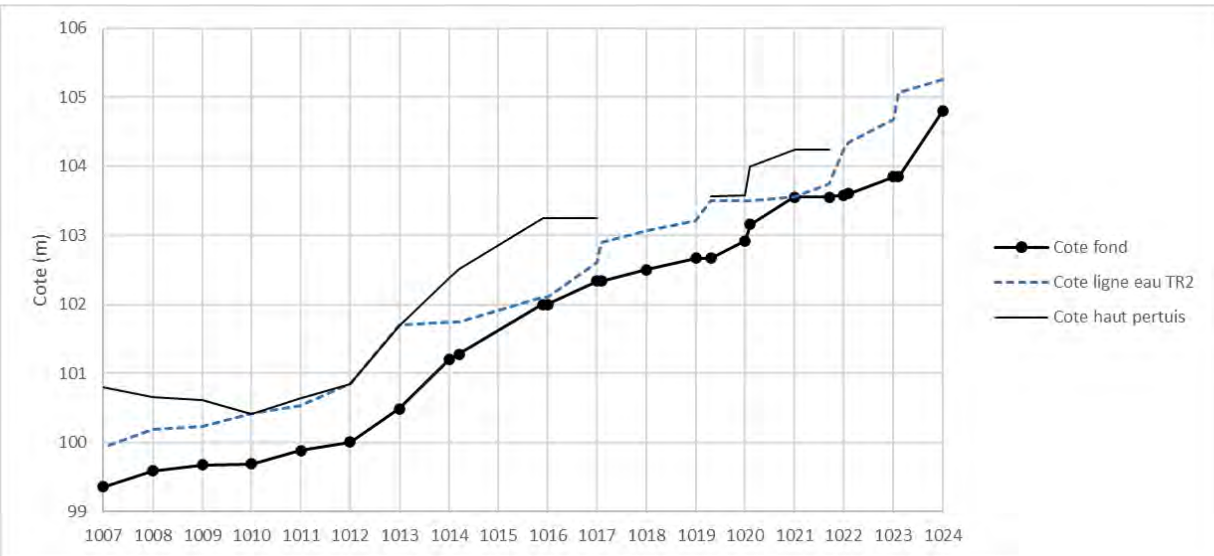


Profil aval



6.4 APPENDIX 4: WATER LEVELS EXISTING SITUATION TABLE + GRAPH (TR2)

Profile number	**Rating Bottom side (m)**	**Rating Water surface TR2 (m)**
1024	104.8	105.25
1023.1	103.85	**Débordement**
1023	103.85	104.68
1022.1	103.6	104.35
1022	103.58	104.25
1021.7	103.55	103.74
1021	103.55	103.56
1020.1	103.16	103.5
1020	102.92	103.5
1019.3	102.67	103.5
1019	102.67	103.21
1018	102.5	103.07
1017.09	102.34	102.9
1017	102.34	102.61
1016	102	102.11
1015.9	102	102.1
1014.2	101.28	101.74
1014	101.2	101.74
1013	100.49	**Saturation**
1012	100.01	**Saturation**
1011	99.89	100.54
1010	99.69	**Saturation**
1009	99.68	100.24
1008	99.59	100.19
1007	99.36	99.94

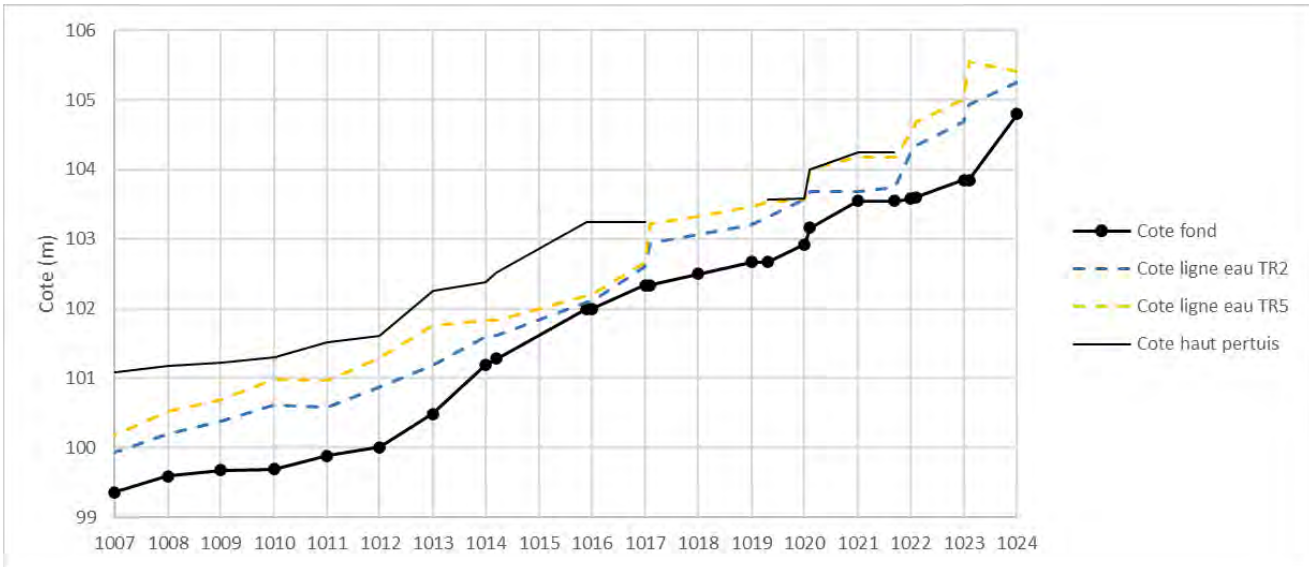


Note: the absence of a line representing the opening means that the watercourse is open water.

6.5 APPENDIX 5: WATER LEVELS PLANNED SITUATION SCENARIO 1 TABLE + GRAPH (TR2 + TR5)

Profile number	**Rating Bottom side (m)**	**Rating Water surface TR2 (m)**	**Rating Water surface TR5 (m)**
1024	104.8	105.25	105.41
1023.1	103.85	104.93	**Overflow**
1023	103.85	104.68	105.01
1022.1	103.6	104.35	104.68
1022	103.58	104.25	104.53
1021.7	103.55	103.74	104.17
1021	103.55	103.68	104.17
1020.1	103.16	103.68	**Saturation**
1020	102.92	**Saturation**	**Saturation**
1019.3	102.67	103.32	103.53
1019	102.67	103.21	103.46
1018	102.5	103.07	103.33

1017.09	102.34	102.95	103.22
1017	102.34	102.61	102.67
1016	102	102.11	102.21
1015.9	102	102.1	102.18
1014.2	101.28	101.61	101.84
1014	101.2	101.6	101.84
1013	100.49	101.19	101.76
1012	100.01	100.88	101.29
1011	99.89	100.58	100.97
1010	99.69	100.61	100.99
1009	99.68	100.38	100.69
1008	99.59	100.2	100.52
1007	99.36	99.94	100.18

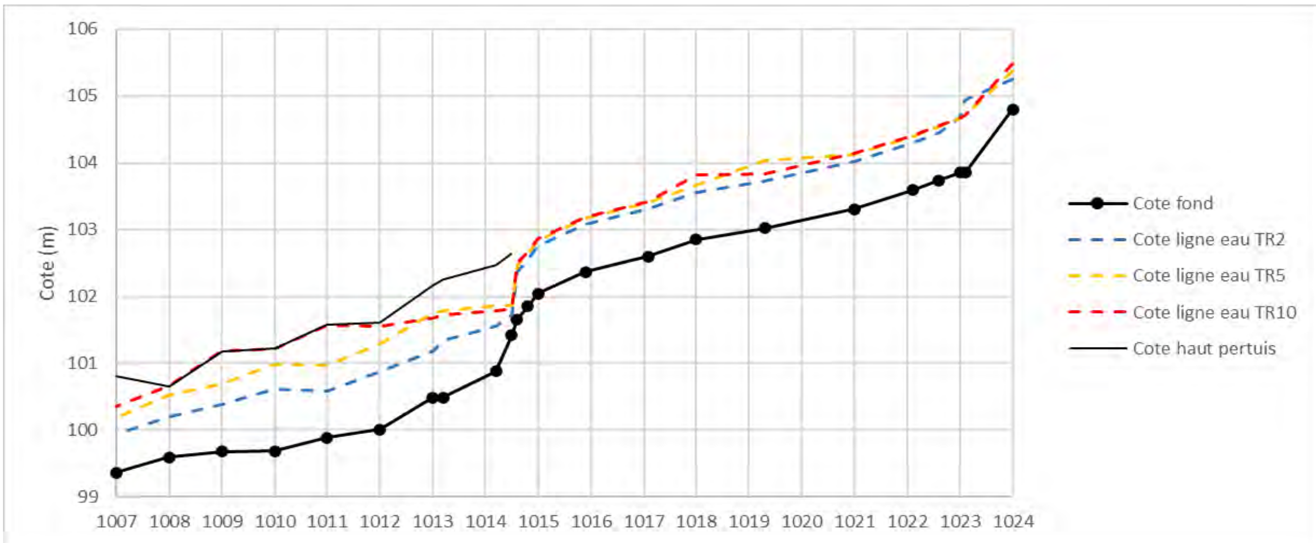


Note: the absence of a line representing the opening means that the watercourse is open water.

6.6 APPENDIX 6 : WATER LEVELS PLANNED SITUATION SCENARIO 2 TABLE + GRAPH (TR2 + TR5+TR10)

c	Rating Bottom side (m)	Rating Water surface TR2 (m)	Rating Water surface TR5 (m)	Rating Water surface TR10 (m)	Rating Water surface TR10 + 40cm (m)
1024	104.8	105.25	105.38	105.49	105.49
1023.1	103.85	104.94	104.72	104.72	104.72
1023	103.85	104.68	104.66	104.67	104.67
1022.6	103.74	104.45	104.55	104.56	104.56
1022.1	103.59	104.3	104.39	104.41	104.41
1021	103.31	104.02	104.12	104.14	104.14
1019.3	103.02	103.73	104.04	103.84	103.84
1018	102.85	103.56	103.66	103.82	103.82

1017.09	102.6	103.31	103.41	103.42	103.42
1015.9	102.37	103.08	103.17	103.19	103.19
1015	102.04	102.75	102.84	102.86	102.86
1014.8	101.85	102.56	102.65	102.67	102.67
1014.6	101.66	102.37	102.46	102.51	102.51
1014.5	101.42	101.73	101.86	101.82	101.82
1014.2	100.88	101.55	101.86	101.78	101.78
1013.2	100.49	101.34	101.79	101.72	101.66
1013	100.49	101.19	101.76	101.68	101.62
1012	100.01	100.88	101.29	101.55	101.49
1011	99.89	100.58	100.97	101.57	101.51
1010	99.69	100.61	100.99	**Saturation**	**Saturation**
1009	99.68	100.38	100.69	**Saturation**	**Saturation**
1008	99.59	100.2	100.52	**Saturation**	**Saturation**
1007	99.36	99.94	100.18	100.36	100.3



Note: the absence of a line representing the opening means that the watercourse is open water.

6.7 APPENDIX 7: GEOMETRIES OF SCENARIO 1 AND 2 PROFILES (BOTTOM ELEVATION, FROM ONE PROFILE TO THE OTHER AND SLOPE FROM ONE PROFILE TO THE OTHER)

Scenario 1

Profile number	Rating Bottom side (m)	Distance (m)	Slope (m/m)
1024	104.8	0	
1023.1	103.85	11.7	-0.08
1023	103.85	12.7	0.00
1022.1	103.6	18.06	-0.05

1022	103.58	19.41	-0.01
1021.7	103.55	20.03	-0.05
1021	103.55	22.98	0.00
1020.1	103.16	26.18	-0.12
1020	102.92	27.01	-0.29
1019.3	102.67	28.64	-0.15
1019	102.67	29.85	0.00
1018	102.5	31.82	-0.09
1017.09	102.34	34.12	-0.07
1017	102.34	34.37	0.00
1016	102	39.23	-0.07
1015.9	102	39.6	0.00
1014.2	101.28	55.73	-0.04
1014	101.2	57.36	-0.05
1013	100.49	66.94	-0.07
1012	100.01	88.46	-0.02
1011	99.89	95.93	-0.02
1010	99.69	100.54	-0.04
1009	99.68	105.05	0.00
1008	99.59	107.26	-0.04
1007	99.36	118.52	-0.02

Scenario 2

Profile number	Rating Bottom side (m)	Distance (m)	Slope (m/m)
1024	104.8	0	
1023.1	103.85	11.7	-0.08
1023	103.85	12.7	0.00
1022.6	103.74	14.55	-0.06
1022.1	103.59	17.31	-0.05
1021	103.31	22.15	-0.06
1019.3	103.02	27.26	-0.06
1018	102.85	30.28	-0.06
1017.09	102.6	34.11	-0.07
1015.9	102.37	37.65	-0.06
1015	102.04	42.76	-0.06
1014.8	101.85	45.62	-0.07
1014.6	101.66	48.64	-0.06
1014.5	101.42	49.4	-0.32
1014.2	100.88	59.07	-0.06
1013.2	100.49	64.91	-0.07
1013	100.49	69.97	0.00
1012	100.01	91.49	-0.02
1011	99.89	98.96	-0.02
1010	99.69	103.57	-0.04

1009	99.68	108.08	0.00
1008	99.59	110.29	-0.04
1007	99.36	121.55	-0.02