Hydraulic study of the Ruyff stream

Interreg-EMR228-EMfloodResilience
Report

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1 Introduction

The present hydrological and hydraulic study entrusted to HydroScan SA/NV by the Province of Liège is part of the reduction of flood risks on the 2nd category watercourse, the Ruyff, a right bank tributary of the Vesdre. The catchment area and the length of the watercourse are illustrated in Figure 1.1. The aim of the study is ultimately to determine the most appropriate development proposals for reducing flooding, in particular in the downstream sector at the level of the Vieux Moulin district, which is frequently affected by overflows from the river. This study is also part of the INTERREG - EMR228 - EMfloodResilience project, which aims to subsidise various projects and actions relating to flood risk reduction in cross-border regions, including the Province of Liege.

The specific objectives of the study are multiple:

- To carry out a detailed hydrological and hydraulic study of the Ruyff stream (2nd category) in the area shown in Figure 1.2 between a potential future TIA site and the confluence with the Vesdre. The study also includes more simplified modelling of the watercourse between the Welkenraedt storm water basin so that the effects of this existing development and the transfer time between upstream and downstream of the catchment can be considered;

- Determine the most appropriate flood reduction measures for the Ruyff stream, particularly in the Vieux Moulin area (see Figure 1.3) which is the main known critical area downstream of the catchment. This analysis forms the core of the contract and is based on a thorough analysis of the hydraulic model taking into account the field situation. It includes a brief cost-benefit analysis to determine the most relevant list of schemes

The following parts are detailed in this report:

- Location of the study river;
- Analysis of the context and objectives of the study;
- Description of the topographic surveys used
- Explanation of the development, calibration and validation of the models;
- Analysis of the existing situation through a diagnosis of the hydraulic functioning;
- Identification and/or optimisation of relevant solutions

In order to meet the objectives of the study, our design office implemented a hydrological model (quantification of the contributions to the watercourse) and a hydraulic model (estimation of the water heights in the minor and major bed according to the flows in the watercourse). The model was parameterised in order to establish a reference situation allowing a diagnosis of the hydraulic functioning of the watercourse and to dimension hydraulic installations according to the inputs and limiting structures on the section studied. The dimensioning of the solutions was carried out based on discussions with the watercourse manager. The solutions were tested in the model to evaluate their impact on the flooding of areas at stake further downstream (Vieux Moulin district) in Dolhain-Limbourg (see Figure 1.3).

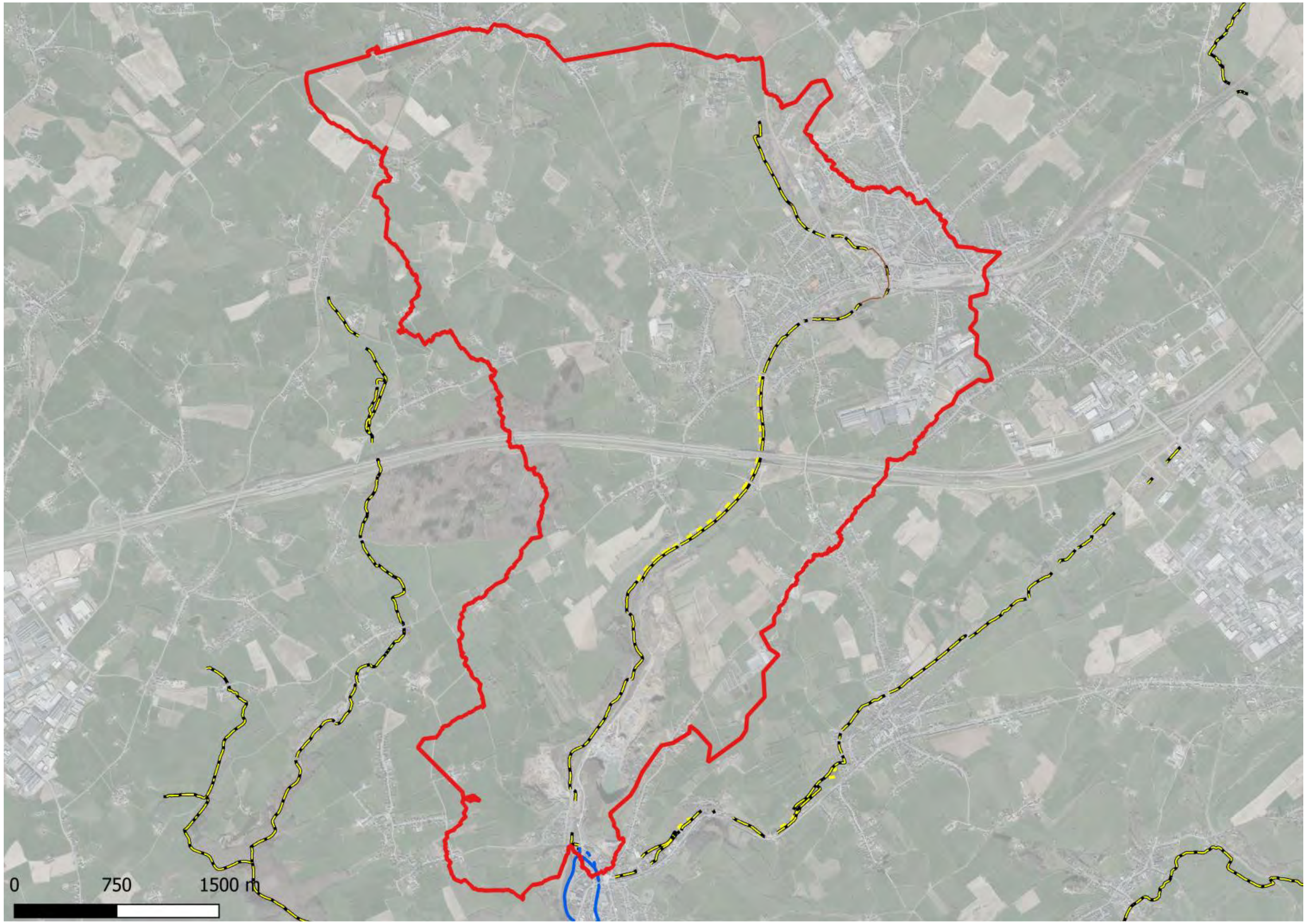

Figure 1.1 – Catchment area of the Ruyff stream and length of the watercourse

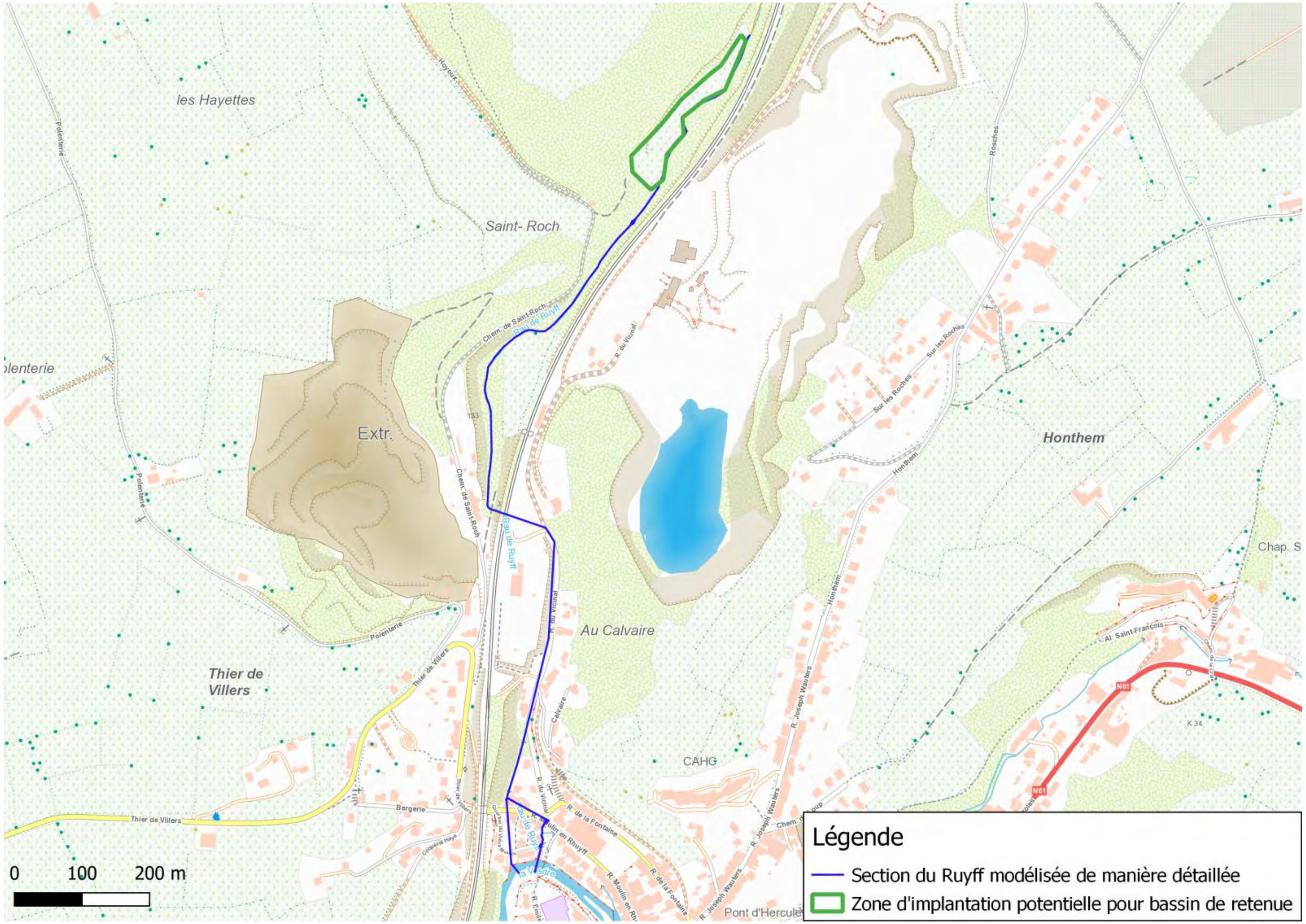

Figure 1.2: Length of watercourse subject to detailed modelling

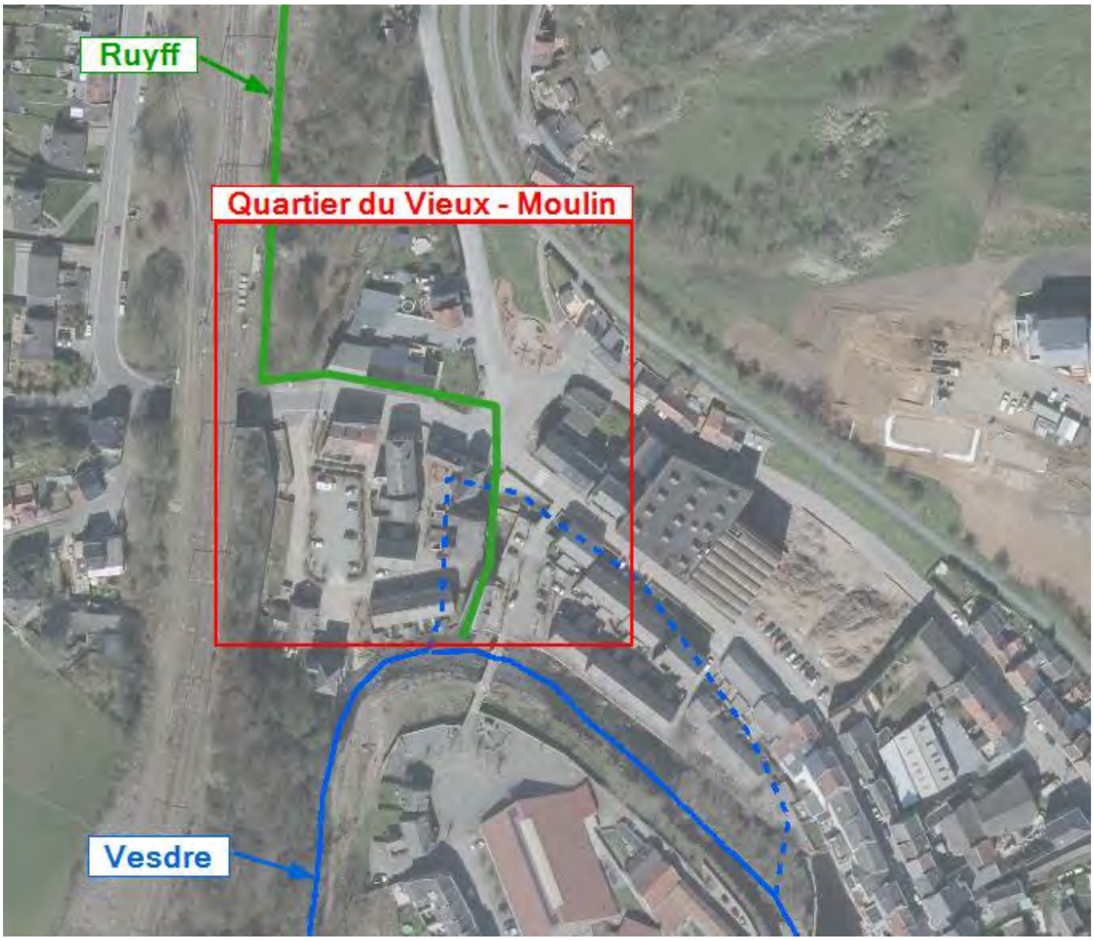

Figure 1.3: Critical area from the Vieux-Moulin district to the confluence of the Ruyff with the Vesdre (Dolhain-Limbourg)

2 Overall analysis of the study area

2.1 Situation of the studied river

The catchment area of the Ruyff covers an area of approximately 17 km². The river has its source upstream of the town of Welkenraedt. It first flows through the town of Welkenraedt (by means of an opening), then passes under the E40 motorway and then runs along the railway line to the Vieux-Moulin district in Dolhain-Limbourg where it flows into the Vesdre. The length of the watercourse in the 2nd category to be modelled in this study is approximately 7 km, but only the downstream part is modelled in detail.

The catchment area is predominantly rural in character as shown in Figure 2.1 and Figure 2.2. It is mainly made up of agricultural areas (73%) and man-made areas (13.6%). Urban areas are concentrated in the town of Welkenraedt (with its industrial estate) and the city of Limburg. Forest occupies 10% of the catchment area, bare soil about 3% and wetlands and water areas less than 0.4%.

Figure 2.1: Percentage of land use in the study area

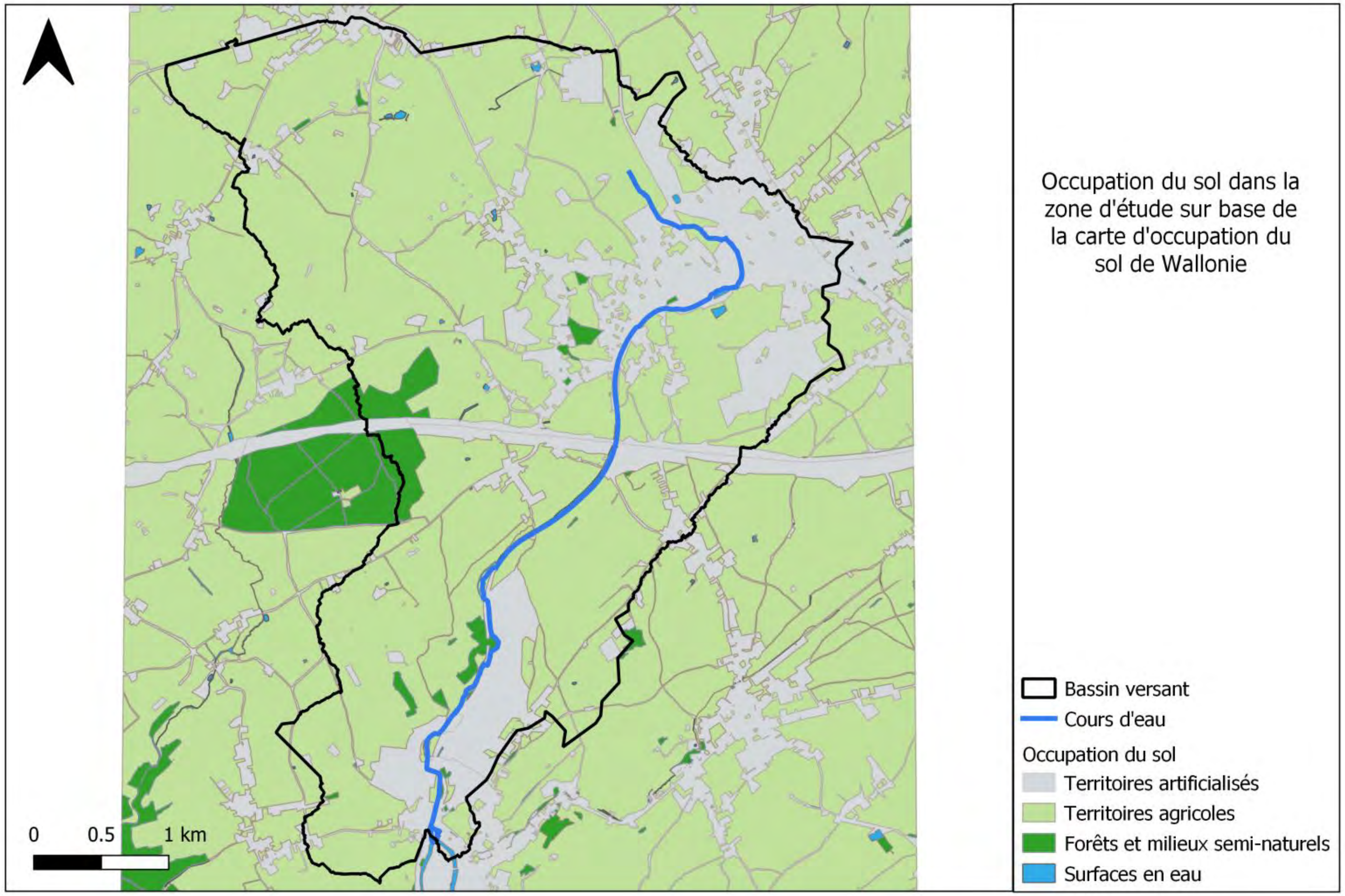

Figure 2.2 : Land use in the study area based on the Walloon land use map

2.2 Available data

2.2.1 Description of the topographic surveys used for the minor bed

There is no accurate topographic survey of the minor bed in the study area. Therefore, topographic surveys were carried out to allow the construction and validation of the model.

Two types of topographic surveys were carried out in the study area in order to build and operate the hydraulic model:

- Surveys for Type S (Simplified) stretches, applying to the stretch between the Welkenraedt storm water basin and the beginning of the potential area for the reservoir. For this section of the watercourse, a few representative flow cross-sections are raised to consider the flow capacity. Structures are not included in this approach, as the principle is mainly to be able to represent the transfer of flows between the upstream and downstream parts of the catchment area (hydraulic routing).

- Surveys for the Type S+S (Simplified with structures) sections, applied to the downstream section between the potential area for the reservoir and the confluence with the Vesdre. The structures are in this case well considered in order to take into account the effect of these structures on the flows and to have a detailed representation of the overflow of the watercourse.

The cross-sections were surveyed using a differential GPS. A cross-section is characterised by the following points: top left bank, bottom left bank, talweg, bottom right bank and top right bank. These sections form the basis for the construction of the 1D river model and were surveyed at a step size defined during the first field visit and which varies according to the section considered. In the upstream section (type S), only a few cross sections were considered. The watercourse has a globally uniform and modified gauge compared to its natural state (rectangular or trapezoidal shape). In the downstream section (type S+S section), the cross-sections are measured at intervals of between 25 and 100 m depending on the structures present and the uniformity of the cross-sections.

The accuracy of the surveys is of the order of 5 cm (or less) in open areas. However, due to the heavy vegetation and buildings in the study area, the reception of the GPS network may be less good. In this case, an optical level was used to measure cross-sections in areas not covered by the GPS.

Each surveyed cross-section is checked and compared with the DTM to verify the consistency of the topographic data. Particular attention is also paid to the presence of low walls along the watercourse. Measured sections were cut or extended at their ends to delineate the boundaries of the 1D model, if necessary. An example of a cross-section is available in Figure 2.3.

	Offset (m)	X coordinate (m)	Y coordinate (m)	Bed level (m AD)	Roughness Manning's n	New panel
1	0.000	261315.064	147933.239	217.565	0.0400	☐
2	0.609	261314.525	147933.523	217.244	0.0400	☐
3	1.589	261313.659	147933.981	217.106	0.0400	☐
4	2.412	261312.931	147934.365	216.672	0.0400	☐
5	3.543	261311.931	147934.894	216.470	0.0400	☐
6	4.662	261310.942	147935.416	216.396	0.0400	☐
7	6.286	261309.506	147936.175	216.483	0.0400	☐
8	6.477	261309.337	147936.264	216.531	0.0400	☐
9	6.987	261308.886	147936.503	217.360	0.0400	☐
10	7.334	261308.579	147936.665	217.450	0.0400	☐
11	8.292	261307.732	147937.112	217.650	0.0400	☐
*						☐

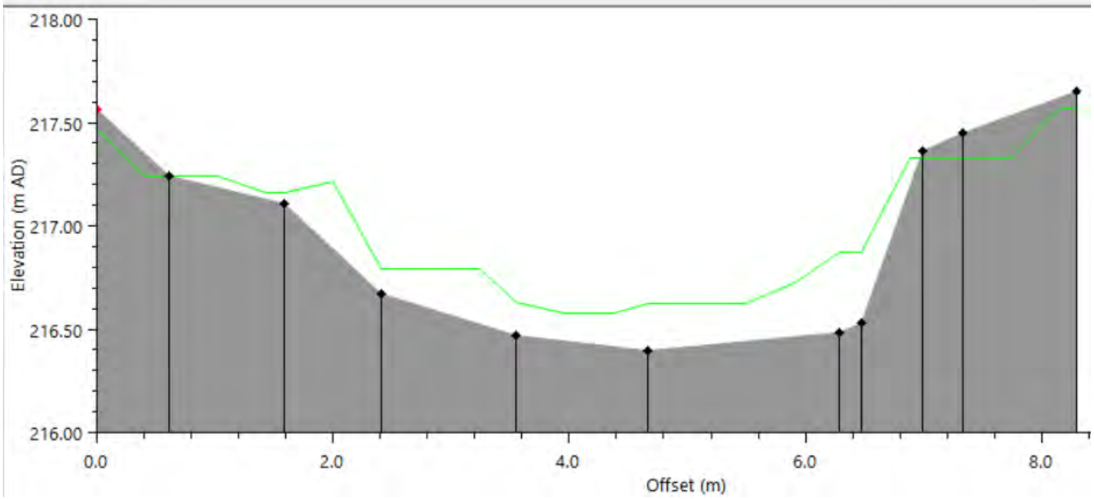

Figure 2.3: Cross-section of the Ruyff watercourse reconstructed on the basis of point surveys taken during the field visit. The green line represents the Digital Terrain Model of the Walloon Region.

With regard to the structures, only those structures that have a hydraulic effect on the flow are surveyed. The upstream and downstream dimensions, dimensions (width, height) and photos of the structures are recorded. The sheets of the structures surveyed in the field are available in Appendix 7.1.

Finally, some data were recovered from old existing data:

- At the level of the passage of the Ruyff under the sluice at Welkenraedt, data were recovered from a hydraulic model built by the AIDE in 2019 on the basis of cadastral surveys ;

- At the level of the Vesdre, a section of this watercourse was integrated on the basis of old surveys from 1960 provided by the SPW-DCENN. The purpose of this addition is to be able to analyse a potential effect of the Vesdre on the flow of the Ruyff.

The topographic surveys therefore come from several different sources. In summary, for each section, from upstream to downstream :

- Section of the Ruyff under the town of Welkenraedt: The topographic data for this section was provided by the AIDE and comes from the cadastral database 2019
- Simplified Ruyff section type S: The topographic surveys of this section were measured with precision GPS or optical level in July 2022.
- Simplified section of the Ruyff with structures type S+S: The topographic surveys of this section were measured using precision GPS or optical level in July 2022.
- Vesdre section: Topographic surveys of the Vesdre in 1960 were provided by the SPW.

Figure 2.4 summarises the different sources used for the topographic data.

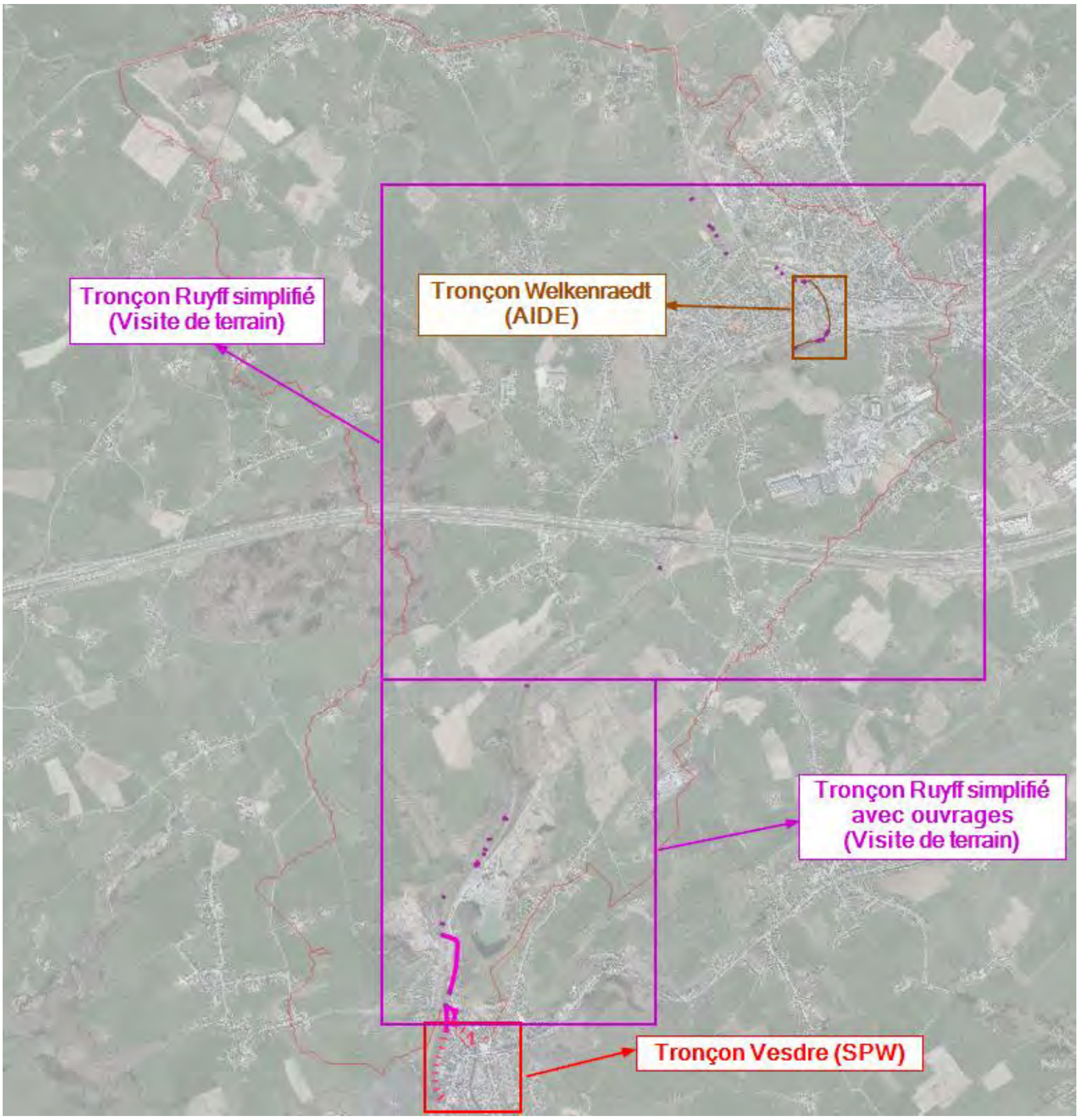

Figure 2.4: Map of the different sources of topographic data

2.2.2 Other topographic data

The following data are also used for the construction of the hydrological and hydraulic model

- Plan of the Welkenraedt storm water basin provided by the Province of Liège;
- Topographic surveys of the Vesdre river provided by the Public Service of Wallonia;
- Hydrological and hydraulic models of the Welkenraedt sewerage system provided in a transportable database with simulations in the InfoWorks ICM modelling software. These data are useful for the integration of the vaulted part of the Ruyff at Welkenraedt. In addition, these data make it possible to obtain the locations of the storm overflows and the urban areas of Welkenraedt.
- The Digital Terrain Model of the Walloon region with a 1m x 1m grid;
- Cartographic data (orthophoto, PICC, Watercourse Atlas, Walloon Land Use Map, WALOUS 2018);

2.2.3 Hydrological data

There is no water level data available directly to the right or downstream of the section of the river studied which is in the 2nd category. The hydrological data are therefore essentially made up of rainfall data.

The statistical data available on the IRM site for the municipality of Limburg, using the Intensity-Duration-Frequency (IDF) curves or Montana coefficients, were collected to construct project rainfall for different return periods. Projected rainfall for the return periods 2 years, 5 years, 10 years, 25 years, 50 years, 100 years, and 100 years with 30% increased flow will be constructed according to the CSC. The data from the city of Limburg was used because there is no significant difference with the data from the city of Welkenraedt.

Furthermore, the observed rainfall of the Battice rain gauge available at a time step of 5 min and hourly was collected at the beginning of the study. The latter is not located directly in the catchment area, but these data could be useful for use as reference rainfall that can be tested as model input.

2.2.4 Historical data

The main historical information consists of

- Photographs and videos of historical floods provided by residents living in the Vieux Moulin area (see Appendix 7.2). This information, dating from June 2016 and July 2021, shows the extent of the overflows at the downstream sluiceway with consequences for the entire downstream section as far as the Vesdre. The frequency of flooding at this critical location just upstream of the downstream floodgate is estimated to be around 5 years, some of the photographs are also in Appendix 7.2;
- Photographs and videos of historical floods provided by the Directorate for Non-Navigable Watercourses, SPW for the Vesdre ;
- The hydraulic study and simulations of the Welkenraedt model carried out by AIDE in 2019.

2.2.5 In-depth site visit

A field visit took place with the project owner at the beginning of the study in June 2022 in order to fully understand the situation on the ground and the context of the study. During this visit, the study area was covered from upstream to downstream, with particular attention paid to known critical areas. Some important elements to be retained from this field visit are:

- The storm water basin upstream of Welkenraedt has a storage capacity of 32,000 m³. The control structure currently consists of a rectangular concrete sluice with dimensions of 1 m x 1.5 m. This stormwater basin was included in the model because it controls the flows to Welkenraedt and downstream of the catchment area. A nozzle of 1.5 m³/s will be added in the future and will be included in the model for future scenarios. This value considered for the nozzle comes from a hydraulic note carried out by AIDE for the reservoir on the Ruyff upstream of the rue de Fromenteau;
- The Ruyff passes through a channel at Welkenraedt with variable dimensions over the sector (detailed information available in the AIDE model). This opening has been integrated into the model because it is known to be a limiting factor that can affect the flows downstream of the study area;;

- The Ruyff is largely modified on its downstream section with a reworked rectangular gauge (see Figure 2.5), which certainly allows rapid transfer of flows downstream due to the low roughness and the absence of a meander.

Figure 2.5: Photograph of the part of the redesigned rectangular build

- The potential area for a retention pond was included in the field visit. This area is a meadow located north of St. Roch Street and on the right bank of the river (X=261280; Y=147920). The topography of this meadow available from the DTM was compared with measurements taken with the precision GPS. This comparison shows that the DTM is very accurate at this location.

- The critical zone of the Vieux Moulin (Moulin-en-Ruyff district). This area experiences frequent overflows approximately every 5 years with a significant impact on the houses adjacent to the watercourse. Some photos and videos were collected during the field visit in relation to the critical flood events of June 2016 and July 2021. These elements are included in a file attached to this report (Appendix 7.2).

- The site visit shows an unfavourable configuration of the downstream opening oriented perpendicular to the flow. In addition, the latter is subdivided at another bend into 4 parallel openings, again adding a configuration that is not very favourable to flows. According to the residents, the response time between a storm and the overflow of the Ruyff at this point seems to be quite short, which may underline the relatively low concentration time of the catchment area and in particular of its urban parts. Certain structural elements present in the Moulin-en-Ruyff neighbourhood, such as low walls and buildings, can also impact the direction of flows once they overflow into the major river bed.

3 Explanation of model development, calibration, and validation

3.1 Hydrological modelling

3.1.1 Catchments and hydrological responses

Surface hydrology models allow the simulation of the surface runoff generated for a given rainfall at the scale of a catchment. In this study, two types of hydrological models are combined in order to best reproduce the hydrological response of the catchment area including urban and rural parts. This requires the definition of two types of sub-catchments, rural and urban (see Figure 3.1 below).

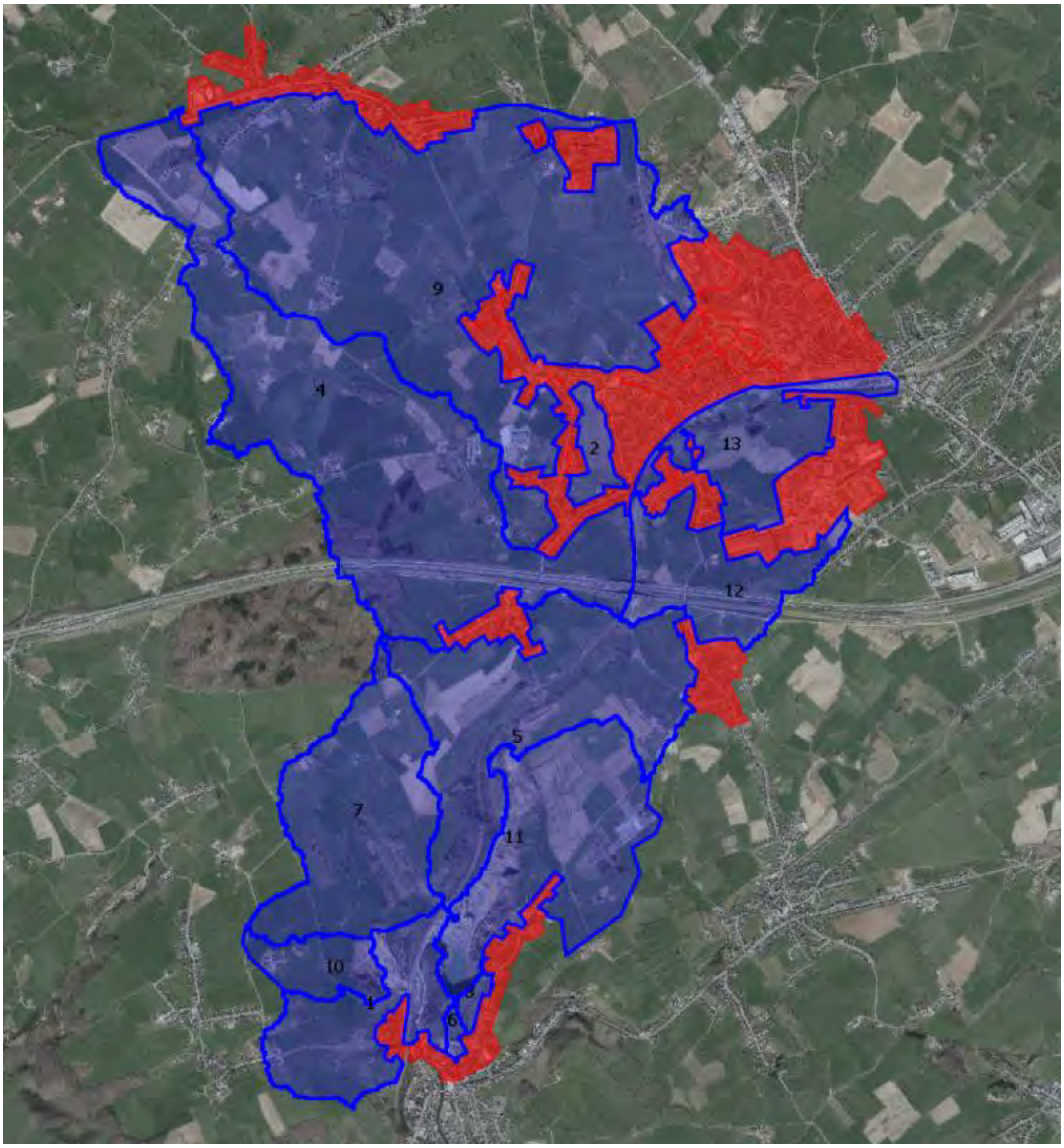

Figure 3.1: Delineation of urban (red) and rural (blue) sub-basins for the study area

The available hydrologically continuous DTM is used for the determination of the river basin and its sub-basins through different GIS procedures (flow direction, flow accumulation, etc.). This division takes into consideration the configuration of the river network, its physiographic characteristics (land use, soil type and topography), structural elements (structures, roads) and modelling objectives. Figure 3.1 shows the sub-basins delineated by the DTM.

Rural runoff is estimated using the SCS-CN hydrological model, which is widely validated in the Walloon region for small rural catchments. The production function of the SCS method is based on the calculation of the Curve Number (CN), which is a function of the hydrological group and the land use. The transfer function associated with the SCS method is a function of the time of concentration calculated for each sub-basin. For each contributing sub-basin, a weighted average NC is calculated based on land use and hydrological groups. Similarly, the time of concentration of each sub-catchment is estimated from several empirical formulas: Kirpich, Passini. Ventura. A summary of the NC and time of concentration values is given in Annex 7.3 and Table 7.1

Runoff from urban areas with the fixed coefficient method

The delimitation of the urban sub-basins in Welkenraedt and Limburg was done on the basis of the PASH and the hydraulic model when available. Figure 3.1 shows the delimited urban sub-basins. It should be noted that there is no overlap between the urban and rural sub-basins so that the contributing areas are not double counted.

The runoff of each urban sub-basin is also estimated thanks to the combination of a hydrological model with a fixed runoff coefficient combined with a Wallingford transfer function (double reservoir in series) which is a function of a routing coefficient. This hydrological model is well suited to simulate the hydrological response of impervious surfaces with a fast response time to rainfall. The different surfaces of the catchment area thus generate different contributions according to their type (see table below, roofs and roads coefficient of 0.8, grassland 0.1, etc.). For each urban sub-catchment, the percentage contribution of each type of surface is estimated according to the land use map of Wallonia (Walous 2018). A summary of the main characteristics of the urban sub-basins is given in Annex 7.3 and Table 7.2.

Table 3.1 – Fixed runoff coefficient according to land use.

Type of soil	Flow coefficient
Impervious surface (streets, roofs)	0.8
Tarmac, road	0.5
Fields, meadow	0.15
Forest	0.05
Water surface	1

3.1.2 Composite rainfall input to the model

The rainfall used in the study is a composite rainfall to produce synthetic hydrographs in the river. Observed rainfall data is not used as there are no rain gauges in the study area and there are no water level data downstream to validate the results.

The definition of project rainfall (composite or synthetic rainfall) is necessary to carry out a detailed hydraulic diagnosis and to design flood reduction measures. These rainfalls are constructed on the basis of the statistical data available on the IRM site for the municipality of Limburg, including the Intensity-Duration-Frequency (IDF) curves and Montana coefficients for different return periods. The Montana coefficients used for the composite rainfall are shown in the table below.

Table 3.2: Montana coefficients of the city of Limburg for different return periods (IRM)

Return period	a	b
2 years	297.6	0.6990
5 years	471.2	0.7272
10 years	611.1	0.7426
25 years	821.6	0.7599
50 years	1006.2	0.7718
100 years	1217	0.783

The composite rainfall applied as input to the model is of the double triangle type, allowing the modelling of both intense events, which are more representative of summer thunderstorm events, and less intense but longer events, which are more representative of winter and spring volume events. The return periods considered for the analysis are, in accordance with the CSC, 5 years, 10 years, 25 years, 50 years and 100 years (see example in Figure 3-2) for a duration of 4 hours. This duration is greater than the critical time of the catchment area, which is often equivalent to the time of concentration, which in this case is about 3 hours. The extreme scenario associated with a return period of 100 +30% is obtained on the basis of the hydrographs generated downstream of each sub-catchment with simulation for a return period of 100 years, multiplied by 1.3. These are then injected into the river system as an injection.

The composite rainfall is applied uniformly across the sub-basins. This assumption therefore does not take into account an abatement coefficient that could take into account the non-simultaneity of rainfall over the whole catchment area. The application of a rainfall uniformly over the contributing catchment area of this size is nevertheless reasonable due to the contributing surface area but is also intended to be safe.

Concerning the recurrence of rainfall and the notion of return period as mentioned in the study, it is important to mention that it is a return period of rainfall and not of flow, which is therefore safer in relation to the results produced by the model. The rainfall return period is based on IDF records whereas the flow return period is based on measured flow data (which is not available for this study). <u>It is important to note that a rainfall with a given return period applied to the whole of a large catchment area such as that of the Ruyfff generates in return a flow at the outlet with a higher return period</u>. Thus, the application of a rainfall with a return period of 2 years at the input of the model generates a flow with a return period greater than 2 years at the outlet. This means that by working with composite rainfall, the design of the structures and facilities planned in the future situation will be safe in terms of protection objectives.

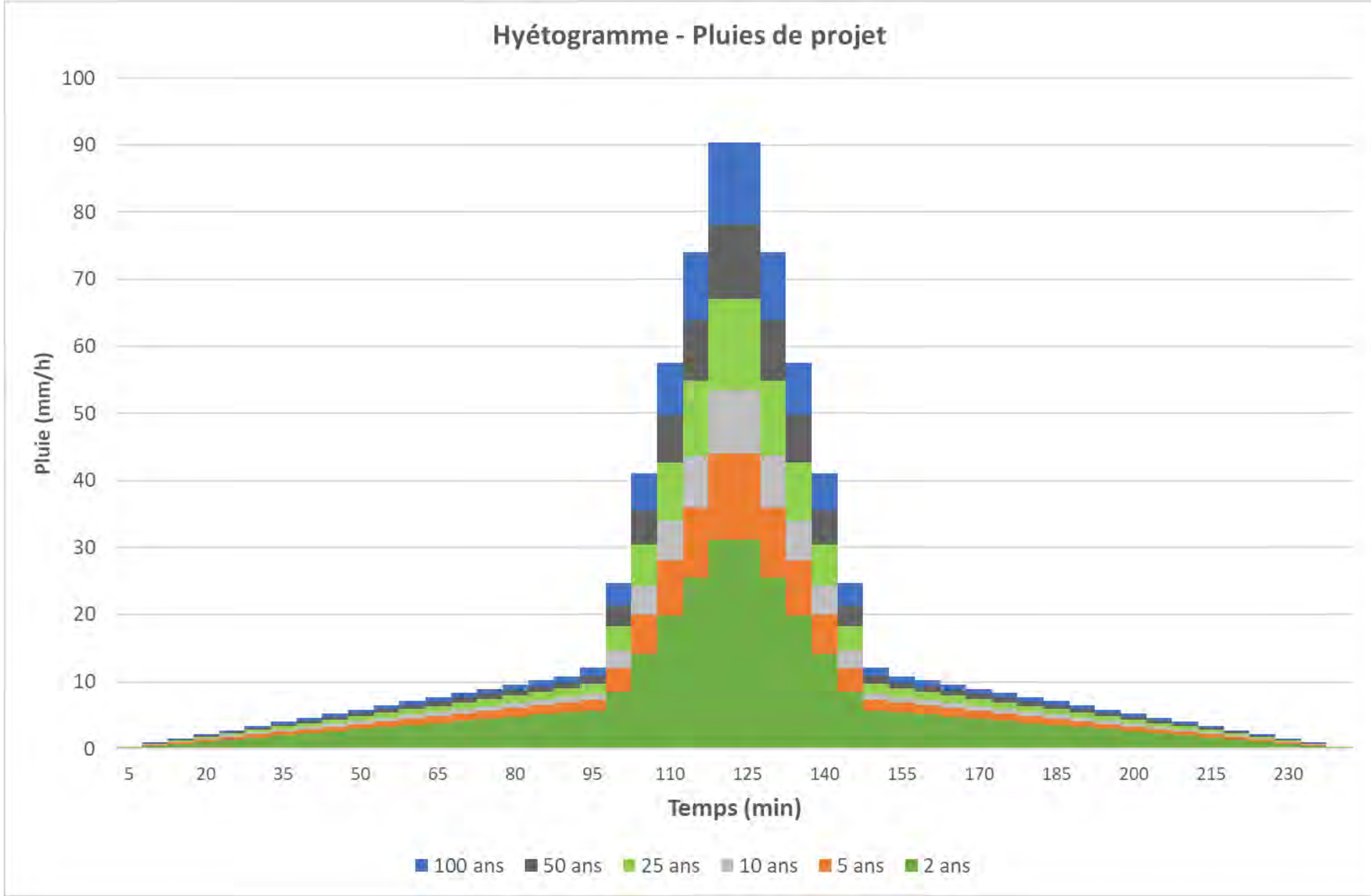

Figure 3.2: Double triangle composite rain applied as input to the model

3.2 Hydraulic modelling

The creation of the hydraulic model is divided into 2 parts depending on the objective:

- On the upstream section of type S between the Welkenraedt storm water basin and the potential location of the reservoir, the model is only built in 1D (only one-dimensional flow) and does not consider the presence of any structures. The objective is to carry out a hydraulic routing of the upstream sub-basins to the downstream section which must be modelled in more detail;

- On the downstream section of type S+S between the potential siting area for the reservoir and the confluence with the Vesdre, the model is of type 1D/2D (one-dimensional flow in the minor bed and two-dimensional once the watercourse overflows). This part also includes the hydraulically relevant structures. The objective is to model the hydraulic functioning of the sector in detail. The coupled 1D/2D approach allows a fine characterization of the flood hazard at any point of the flooded areas.

3.2.1 Simplified 1D hydraulic modelling for hydraulic routing of the upper catchment

Some of the catchment areas in the upstream part of the catchment are not directly connected to the river network modelled in detail. Therefore, in order to take into account the transfer in the river between these sub-catchments and the beginning of the detailed modelled section, we will consider a hydraulic routing. This is done in the modelling software via the implementation of a simplified 1D modelling integrating a few cross-sections representative of the river gauge, then completed by a few interpolated profiles in order to have sufficient elements for hydraulic calculations. The objective is to obtain an approximation of the transfer time by obtaining parameters such as the average gauge, the average slopes, etc. This 1D model then receives contributions from the upstream sub-catchments of the catchment.

3.2.2 1D modelling of the river bed in the downstream section

The construction of part 1D is related to the minor bed in order to take into account the hydraulic capacity of the watercourse via the cross-sections measured at regular intervals and the capacity of the structures via the integration in the model of the characteristics of each structure (dimension, shape, etc.).

From downstream to upstream, 5 structures were surveyed and integrated into the model:

- OA01 - Downstream bridge at the confluence with the Vesdre
- V01 - Gate and weir of the Old Mill
- OA02 - Narrows passing under the Rue Moulin en Rhuyff (critical zone of the study)
- OA02bis - Pipe passing under the Vieux-Moulin district joining the Vesdre
- OA03 - Pertuis passing under the railway and station of Dolhain-Gileppe
- OA04 - Stone bridge at the right of the possible future storm basin (North of Chemin de Saint-Roch)

The five most downstream structures were integrated into the model using the information obtained during the field visit. The upstream structures (in the area of Welkenraedt) were integrated into the model with the data obtained by AIDE. The location of the 5 downstream structures is shown in Figure 3.3.

Downstream of the Ruyff model, a section of the Vesdre of a few hundred metres on either side of the confluence was modelled in order to analyse the interactions between the two rivers. Topographic surveys of the Vesdre were provided by the SPW (surveys dating from 1960). The Vesdre section is therefore connected to the section built for the Ruyff via structure OA01 so that a consistent boundary condition can be applied at the confluence with the Ruyff.

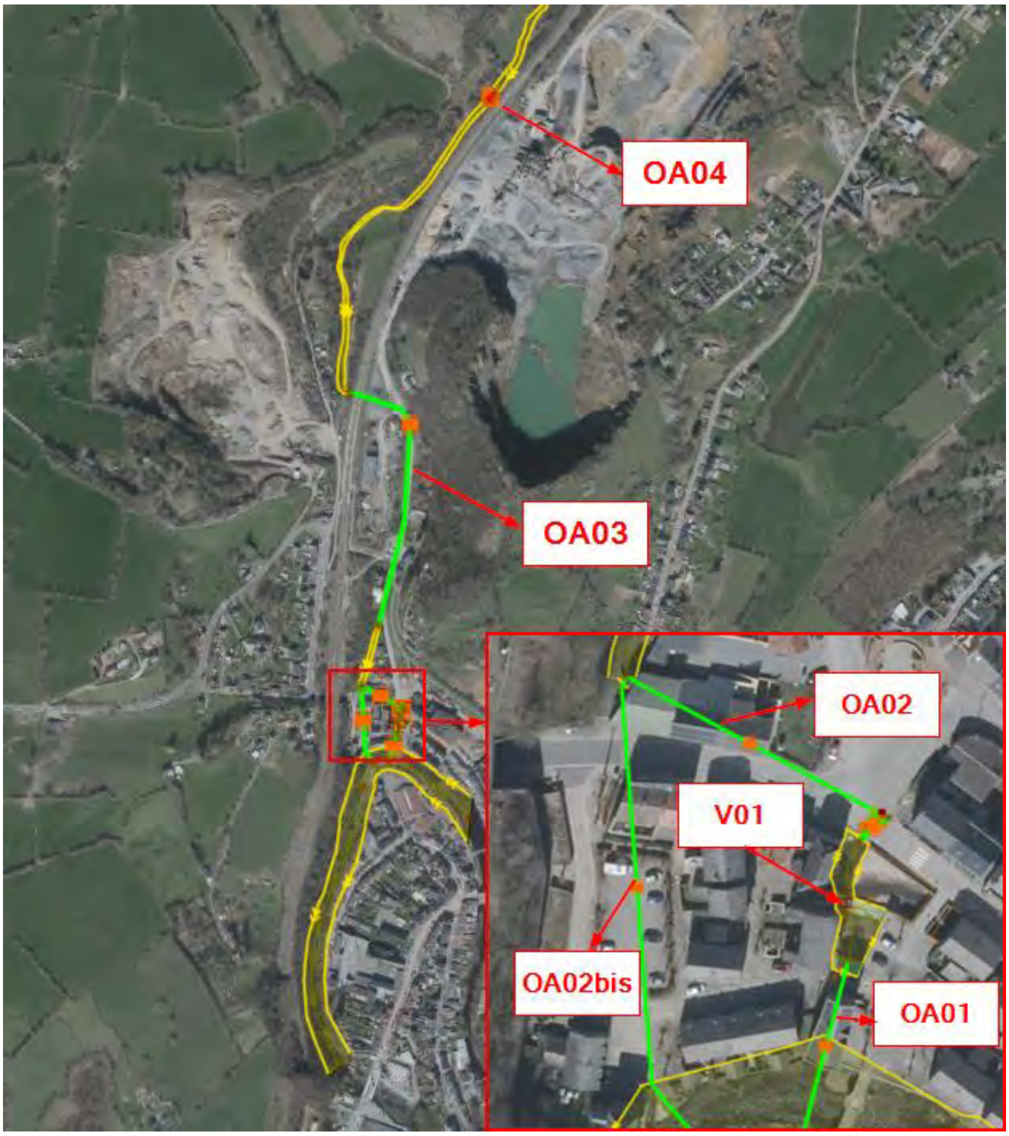

Figure 3.3: Location of the various structures downstream, particularly in the Vieux-Moulin district

OA02 is a critical structure in the study area due to its two unfavourable bends in the flow, as shown in Figure 3.4. These two bends cause localized head losses due to the turbulence that can be caused by such a configuration, which prevents the water from following its course easily. These localized pressure losses are taken into account in the modelling via the application of an additional pressure loss coefficient applied to the ends of the sections and depending on the angle of attack between the inlet and outlet of the flows. The pressure drop equation used in the software and an illustrative diagram showing the coefficient values as a function of the angle of attack are shown in Figure 3.5.

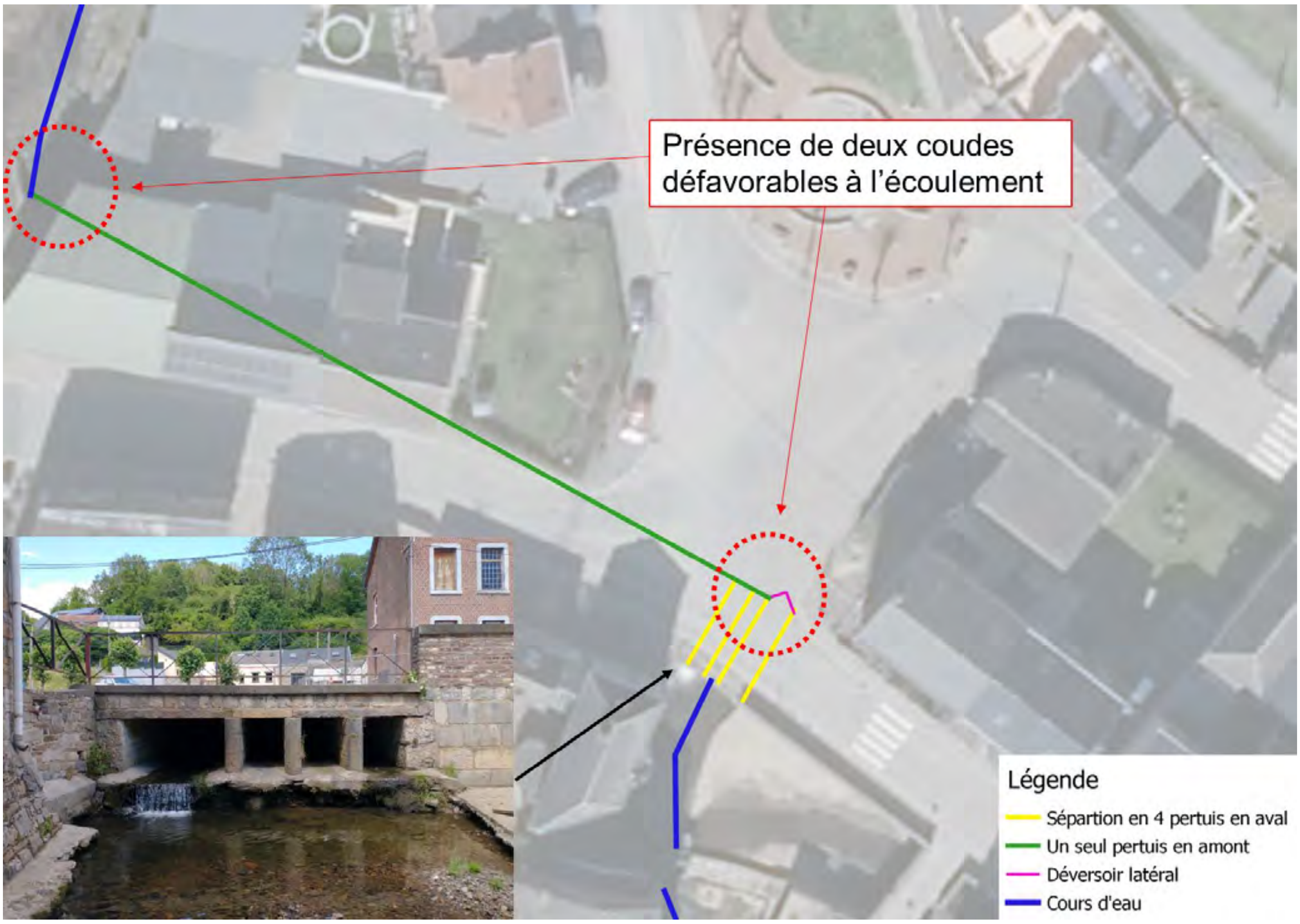

Figure 3.4: Diagram and photograph of the OA02 structure

$$\Delta h = ku * ks * kv * (v^2/2g) \quad (1)$$

where:

- Δh = headloss
- ku = user defined headloss factor
- ks = surcharge ratio coefficient
- kv = velocity coefficient
- v = flow velocity (m/s)
- g = acceleration due to gravity (m/s^2)

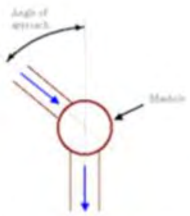

Figure 1 Angle of Approach

Angle of Approach	Ku
30	3.3
60	6.0
90	6.6
>90	8.0

Table 1 Angle of Approach

Figure 3.5: Pressure drop formula and approach angle factor (ICM Infoworks)

3.2.3 2D modelling in the major bed

The major river bed is modelled by a two-dimensional mesh. The 2D zone is constructed on the basis of the topography of the terrain, the information of which is contained in the DTM. The mesh is refined with the integration of some buildings, low walls, roads and a variable roughness depending on the zones (roads and parking, meadows and zones with vegetation). The boundary condition for the 2D area is a normal condition.

3.2.4 Boundary conditions

A hydraulic model always requires the definition of boundary conditions.

For each simulation, the upstream boundary condition is constituted by a synthetic rainfall relative to a given return period injected into the model and applied uniformly to all the sub-basins.

Two upstream boundary conditions are applied at the confluence of the Vesdre via injections upstream of the section in the hydraulic model on the Vesdre so as to be able to generate a downstream boundary condition on the Ruyff model:

- o Vesdre at normal level (Q = 3 m³/s) ;
- o Vesdre in flood (Q = 90 m³/s). This last value is set so as to have a water level in the Vesdre that can create an influence on the potentially limiting channel located in the Moulin-en-Ruyff district

3.2.5 Validation of the combined 1D/2D model of the existing situation

The model must be validated to ensure that the results are representative. Calibration essentially concerns the modification of the parameters of the hydrological model (NC, time of concentration) and the hydraulic model (roughness parameters, head loss coefficients) in order to reduce the difference between simulations and observations.

There is no water level data to obtain water level and flow data in the catchment area of the study zone. There are also no observed high-water marks downstream of the area with rainfall data to be injected upstream since there are no rain gauges within the catchment area. As a result, it is not possible to validate the model in a conventional way.

In view of the available data, the validation of the model is however possible thanks to the calculation of the average runoff coefficient calculating the ratio between the volume of water runoff and the volume of water precipitated. This average runoff coefficient value is generally between 10% and 30% depending on the type of catchment area (land use, slope, etc.) and the return period. The runoff coefficient (see formula below) has been calculated for the different return periods and the values are shown in Table 3.3.

$$CR_{moyen} = \frac{\Sigma Q_{ruisselé}}{\Sigma Q_{précipité}}$$

Table 3.3: Average runoff coefficient for different rainfall return periods.

Return period	Runoff coefficient
2 years	16.5%
5 years	14.9%
10 years	17.3%
25 years	20.6%
50 years	26.4%
100 years	33.4%

The values of the average runoff coefficient calculated are between 10% and 30%, which is consistent with the study area and also represents well the urbanised side in the upper part of the catchment area at the level of the cities of Limburg and Welkenraedt.

Furthermore, the historical data and the information obtained from the residents (see annex 7.2) indicate that there have been approximately 5 floods in 30 years, which makes it possible to estimate the recurrence of flooding in the Old Mill district in Rhuyff. The return period of the overflows in the critical zone is therefore close to 5 years. The known critical zone in the Old Mill district is well modelled as an overflow zone for a rainfall of 5 years in the existing situation, which is therefore consistent with the observed return period of overflow (this result is obtained with the absence of a downstream effect from the Vesdre).

Moreover, according to the residents, the maximum water level observed over the last 30 years upstream of the sluice is approximately 204.8 mDNG. The latter level is well above the extreme floods such as the one with a return period of 100 years.

Finally, the overflow behaviour and the areas affected also seem consistent once the minor bed overflows towards the Vicinal and Moulin en Rhuyff streets. In this sense, an illustration of the flood map obtained for a return period of 25 years is shown in Figure 3.6. Photos of historical floods in the Vieux-Moulin area are also available in Figure 3.6.

In particular, the following can be observed

- Overflows on the left bank upstream of the double sluiceway at no. 70 Rue du Moulin en Rhuyff and at no. 3 Rue du Vicinal;
- Transfers into the major bed between the courtyards of the houses and on the rue du Vicinal;
- Overflows from the watercourse to Rue du Moulin en Rhuyff with transfer to the road and further downstream to the Vesdre;
- A general saturation of this area by water that is already significant, despite the fact that the effect of the Vesdre river has not been taken into account in the result presented (an overflow of the Vesdre into its major bed on the right bank would increase the saturation effect, as was observed in July 2021).

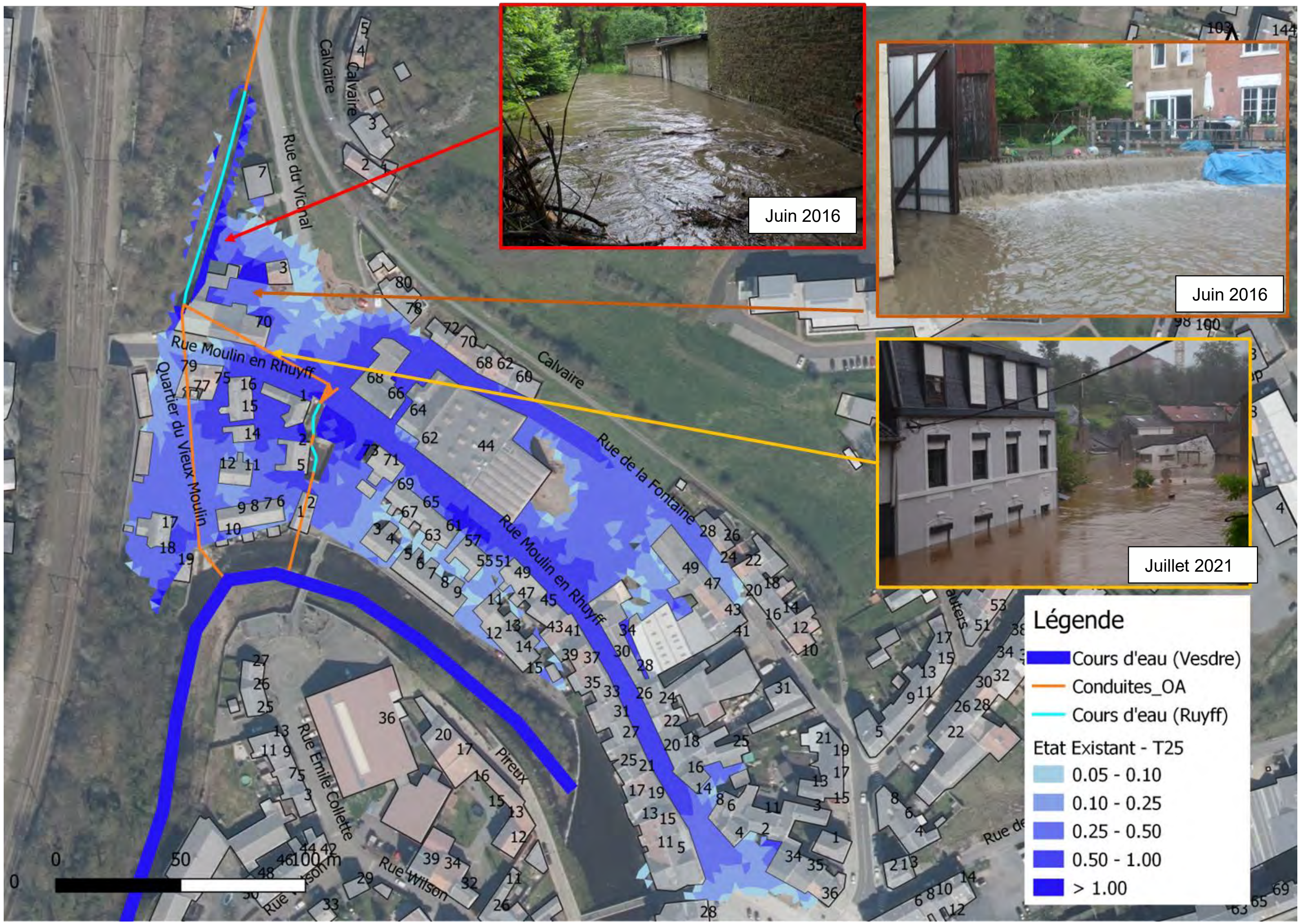

Figure 3.6: Modelled water level at the most critical time for a return period of 25 years.

4 Analysis of the existing situation

The diagnosis of the hydraulic functioning for the reference situation is carried out on the basis of the analysis of the results of the simulations with composite rainfall for return periods of 2 years, 5 years, 10 years, 25 years, 50 years and 100 years and for the extreme scenario with a return period of 100 +30%. The detailed hydraulic analysis makes it possible to determine the critical areas, the places where the first overflows occur and the most critical structures. The analysis of the hydraulic functioning is first carried out without the effect of the Vesdre and then focuses on the effect of the Vesdre on the flows in the Rhuyff.

4.1 No effect analysis downstream of the Vesdre

4.1.1 Hydraulic diagnosis

The simulated flood hydrographs upstream of the critical area of the Moulin-en-Rhuyff district are presented in Figure 4.2 for simulated return periods of 2 to 25 years.

The relevant hydraulic structures were analysed in order to determine an admissible flow for each structure (maximum capacity of the structure when under pressure) and also a critical return period for which the structure would be limiting. This information is summarised in the table below.

Table 4 :1 : Allowable flows and critical return periods of hydraulic structures

ID Structure	Location	Type	Allowable flow (m³/s)	Critical return period*
OA01	Bridge at the confluence with the Vesdre	Brick bridge	20	N/A
OA02	Tunnel under the Rue Moulin en Rhuyff	Brick gate	8	T2
OA02bis	Pipe under the Vieux Moulin district	Concrete pipe	2	
OA03	Bridge under the railway	Concrete bridge	20	T50
OA04	Bridge north of Chemin de Saint-Roch	Stone bridge	2x10	T50

* Return period for which the sluice is limiting, but no overflow has yet occurred

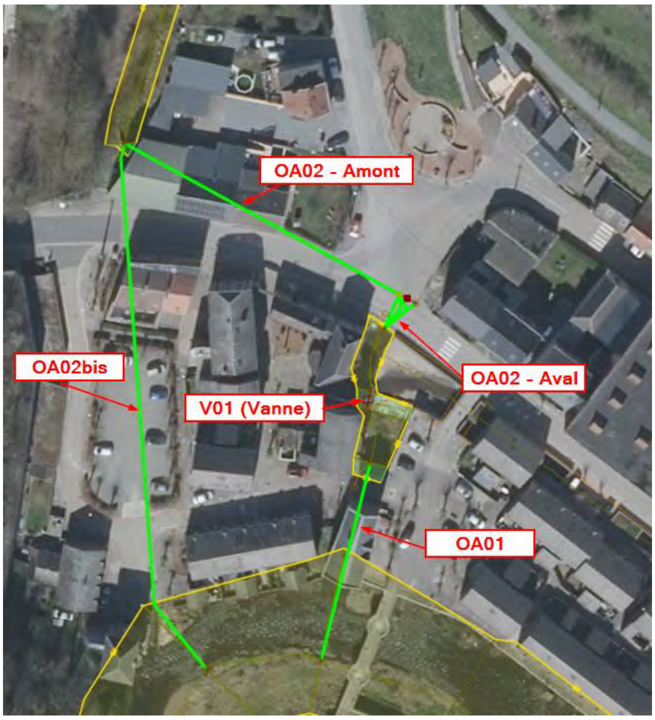

Figure 4.1: Localisation des ouvrages dans le quartier du Vieux-Moulin

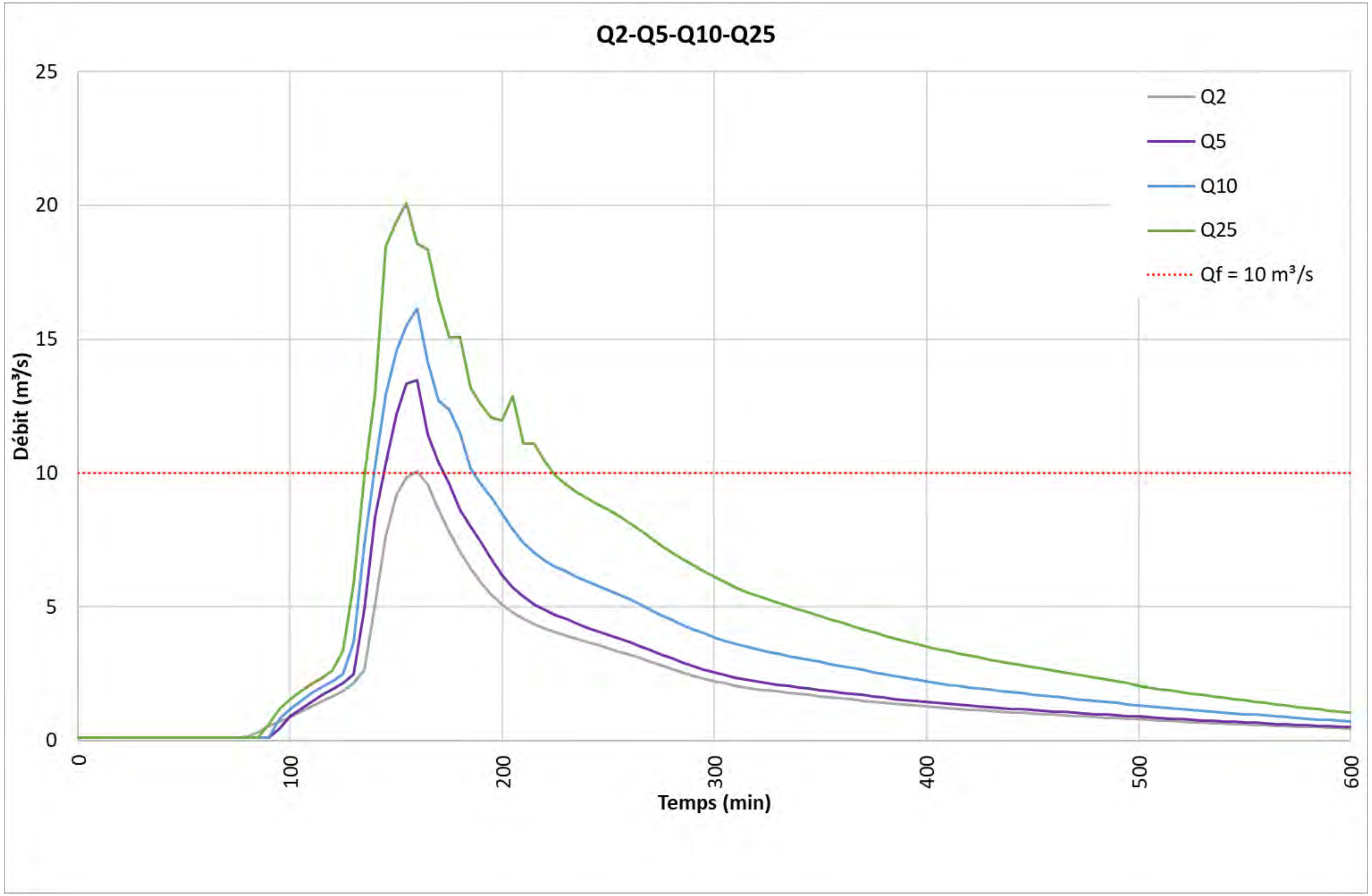

Figure 4.2: Simulated flood hydrographs upstream of the critical zone for return periods of 2 to 25 years. The flow rate of 10 m³/s corresponds to the admissible flow rate of the structure OA02 and OA02bis.

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The critical point in the area of one from a hydraulic point of view is the channelled part of the watercourse which is located under the buildings and the road and has several abrupt changes of direction. The other structures in the area are not limiting.

In the critical zone, the Ruyff is open and can join the Vesdre via 2 structures under the openings OA02 and OA02bis which are not in good alignment with the watercourse, which is responsible for the creation of significant localized head losses at the entrance to these structures. Moreover, concerning the OA02 structure, it is separated into 2 sections and the junction between the two is rather unfavourable with again a 90° angle which can cause new localized head losses. The hydraulic brake linked to the junction between the rectangular opening and the 4 downstream openings can be observed in the hydraulic simulations by the "fall" of the water line at the junction (see Figure 4.3). This unfavourable transition causes the first rectangular part of the structure to be pressurised, while the downstream part, consisting of the four openings, is not pressurised.

The set of structures constituted by the OA02 and OA02bis allows a cumulative admissible capacity of about 10 m³/s, which corresponds to a critical return period of 2 years, as can also be seen in Figure 4.2. The hydraulic diagnosis shows the following sequence:

- The opening OA02 has an admissible capacity of about 8 m³/s. The junction between the rectangular upstream part and the downstream part consisting of 4 parallel openings plays an important role, since significant head losses apply at this level (fall of the water line at the function as visible in Figure 4.3 for the time when the flow in the river is about 6 m³/s).
- Structure OA02 is the first to be loaded, followed by the ring main OA02bis which has an admissible capacity of about 2.0 m³/s ;
- The total admissible capacity of the 2 structures is therefore about 10 m³/s. Once the two structures are pressurised, the water level rises upstream as a result of the increased flow in the watercourse until the watercourse overflows the minor bed. Given the height of the banks at this point, the first overflows occur at a flow rate of approximately 11 m³/s (see Figure 4.4).
- The first overflows occur at the level of the apron and spread towards the Moulin-en-Rhuyff street, followed almost immediately by overflows of the watercourse on the left bank. These last overflows are reflected in the yards of the houses and in the Rue du Vicinal;
- The overflows in the Moulin-en-Rhuyff and Vicinal Streets progress towards the Vesdre, flooding the entire district, as shown in Figure 4.5.

The two structures OA02 and OA02bis are therefore the main cause of the overflows observed in the Vieux-Moulin area since their cumulative admissible capacity is already reached for a return period of 2 years. Based on this analysis, it is decided that the admissible flow to be respected downstream to avoid flooding problems must be a maximum of 10 m³/s in the current configuration of structures OA02 and OA02bis.

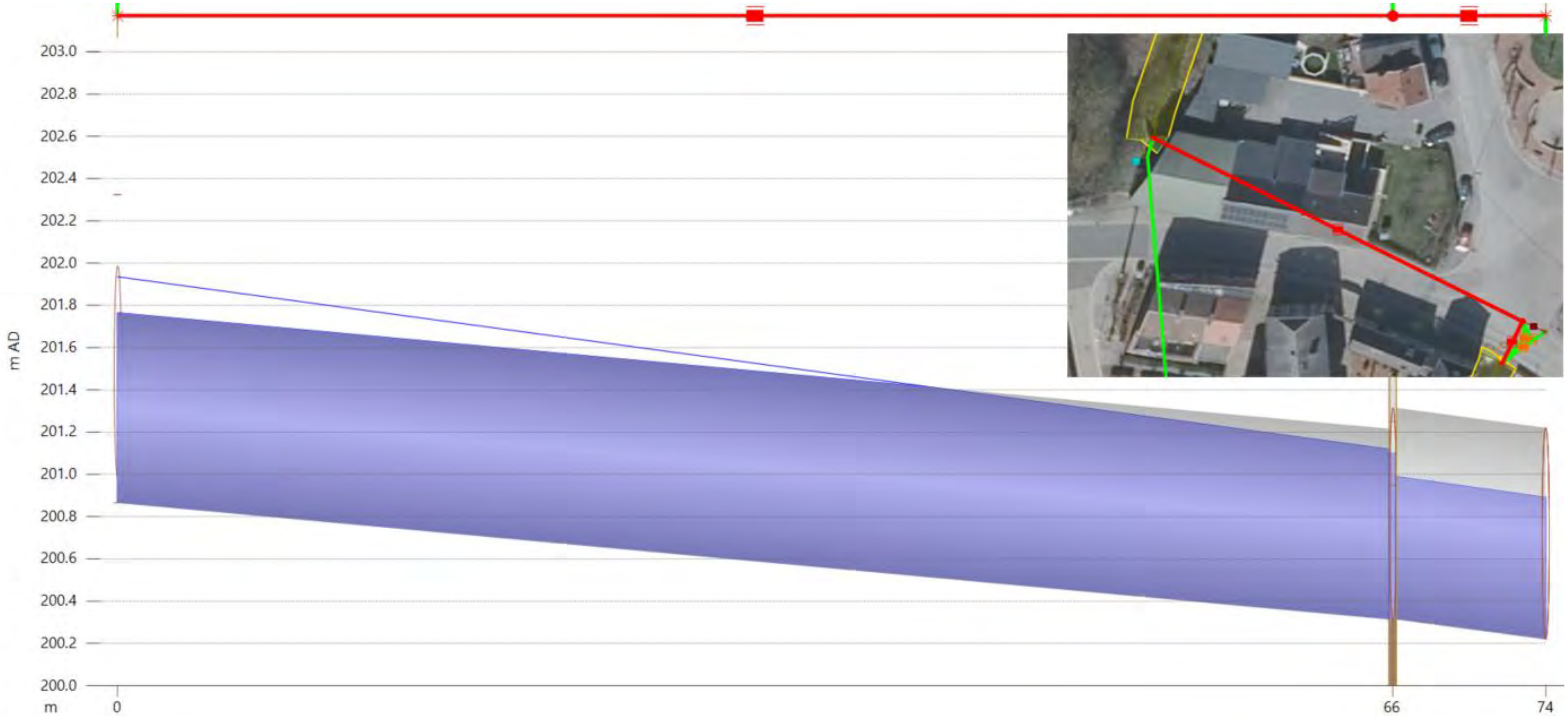


Figure 4.3 : Longitudinal profile of the passage under the Rue du Moulin en Rhuyff for a flow of 6m³/s (flow for which the upstream part of the structure becomes pressurised).

The junction between the rectangular upstream part and the downstream part composed of 4 openings in parallel presents an unfavourable hydraulic configuration which causes a loss of head at the junction and in return the pressurisation of the rectangular upstream part of the structure.

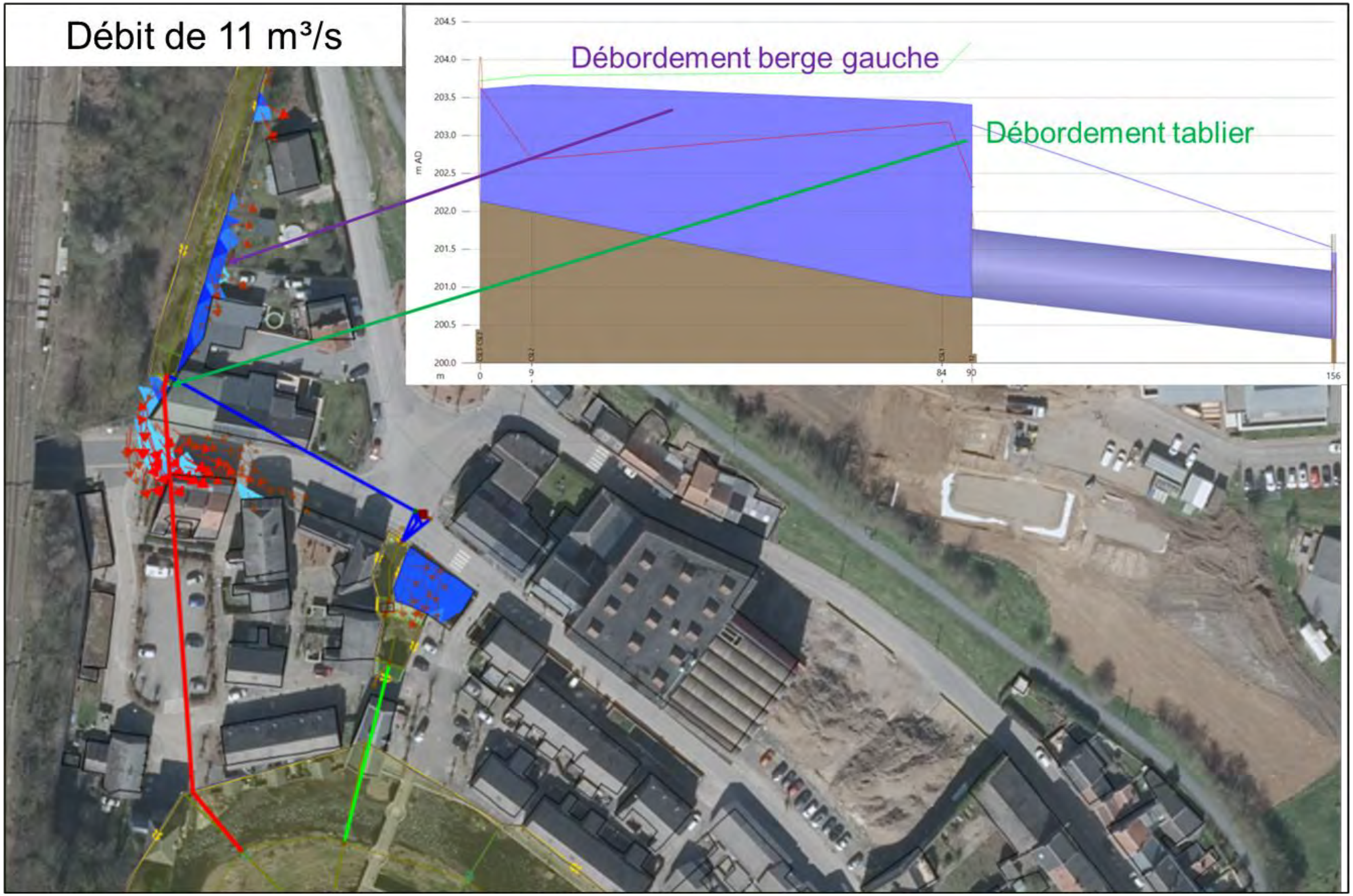


Figure 4.4: First overflows at the level of the deck and the left bank in the critical zone for a flow of 11 m³/s

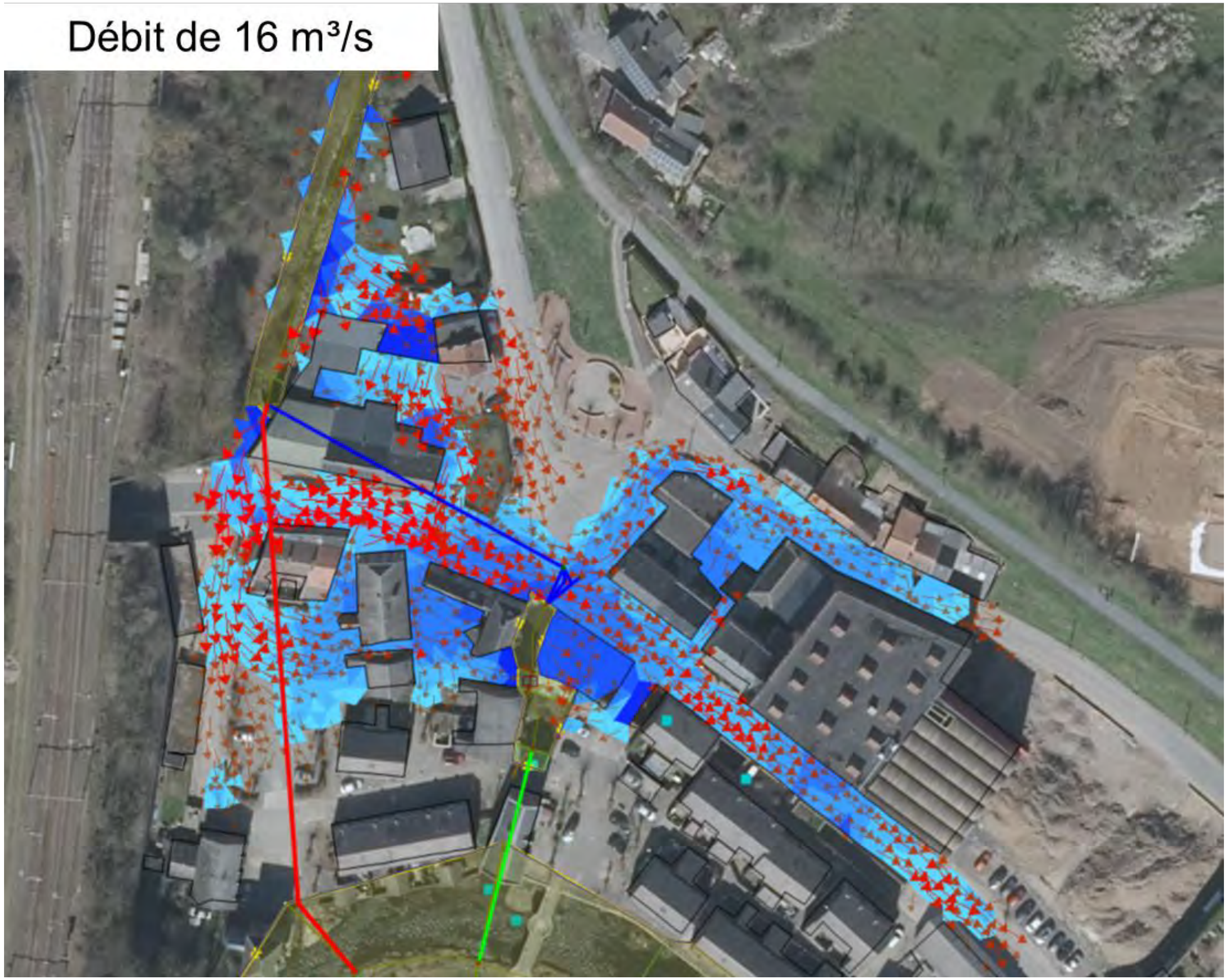


Figure 4.5 -: Propagation of the water in the Vieux-Moulin district once the water has entered the major bed (flow rate of 16 m³/s).

4.1.2 Flood maps for the existing situation

The flood maps for the return periods of 5 years, 10 years, 25 years and 100 years are presented in the following figures. These maps are based on the results of simulations without downstream effects of the Vesdre. All the flood maps for the existing situation for all the simulated return periods are shown in the appendix.

These flood maps highlight the frequent nature of overflows at this location and the impact that these overflows can have on the Moulin-en-Rhuyff and Vicinal streets, but also on other streets such as Rue de la Fontaine.

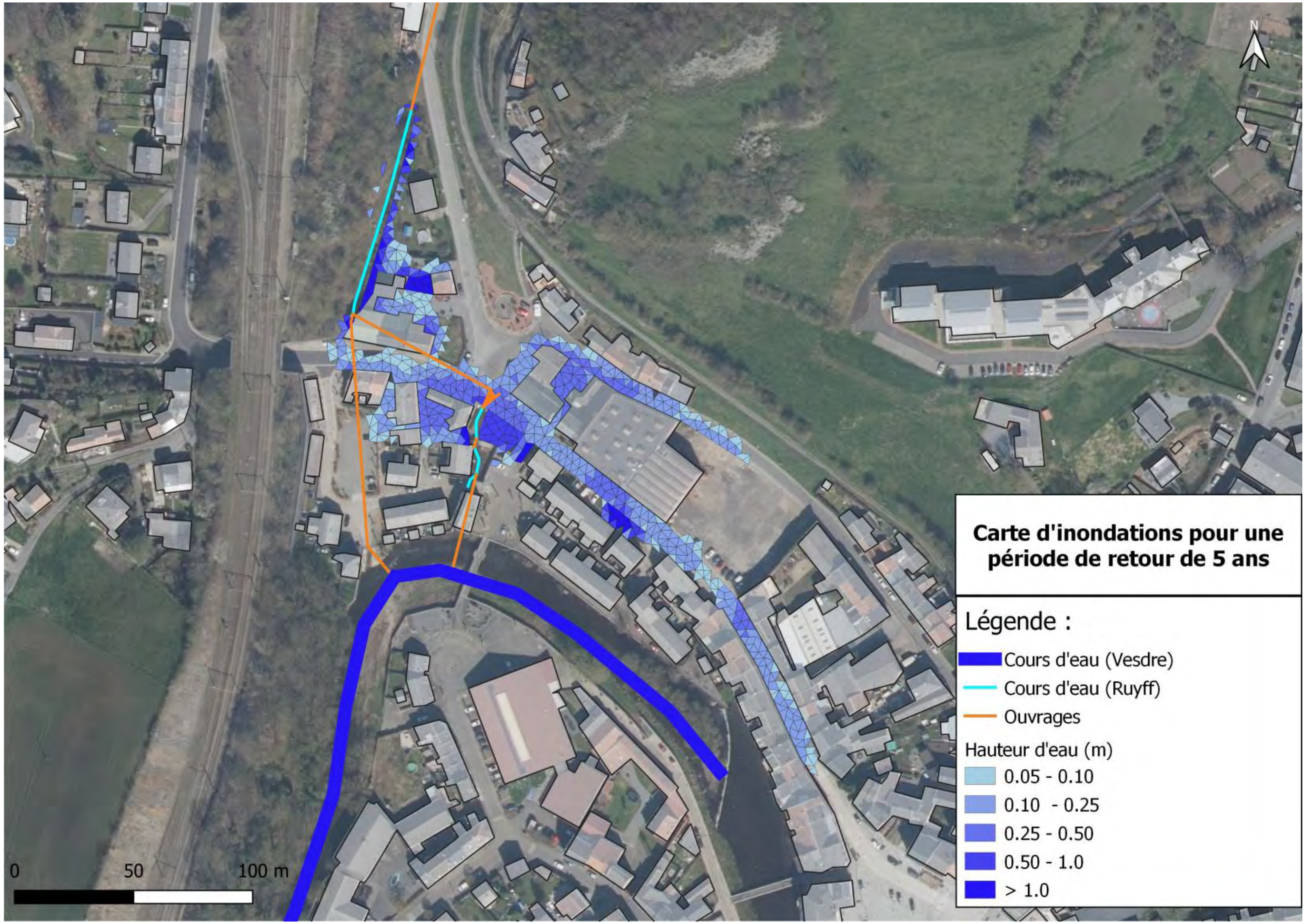


Figure 4.6 - Flood map of the existing situation at the most critical time of the flood for a return period of 5 years.

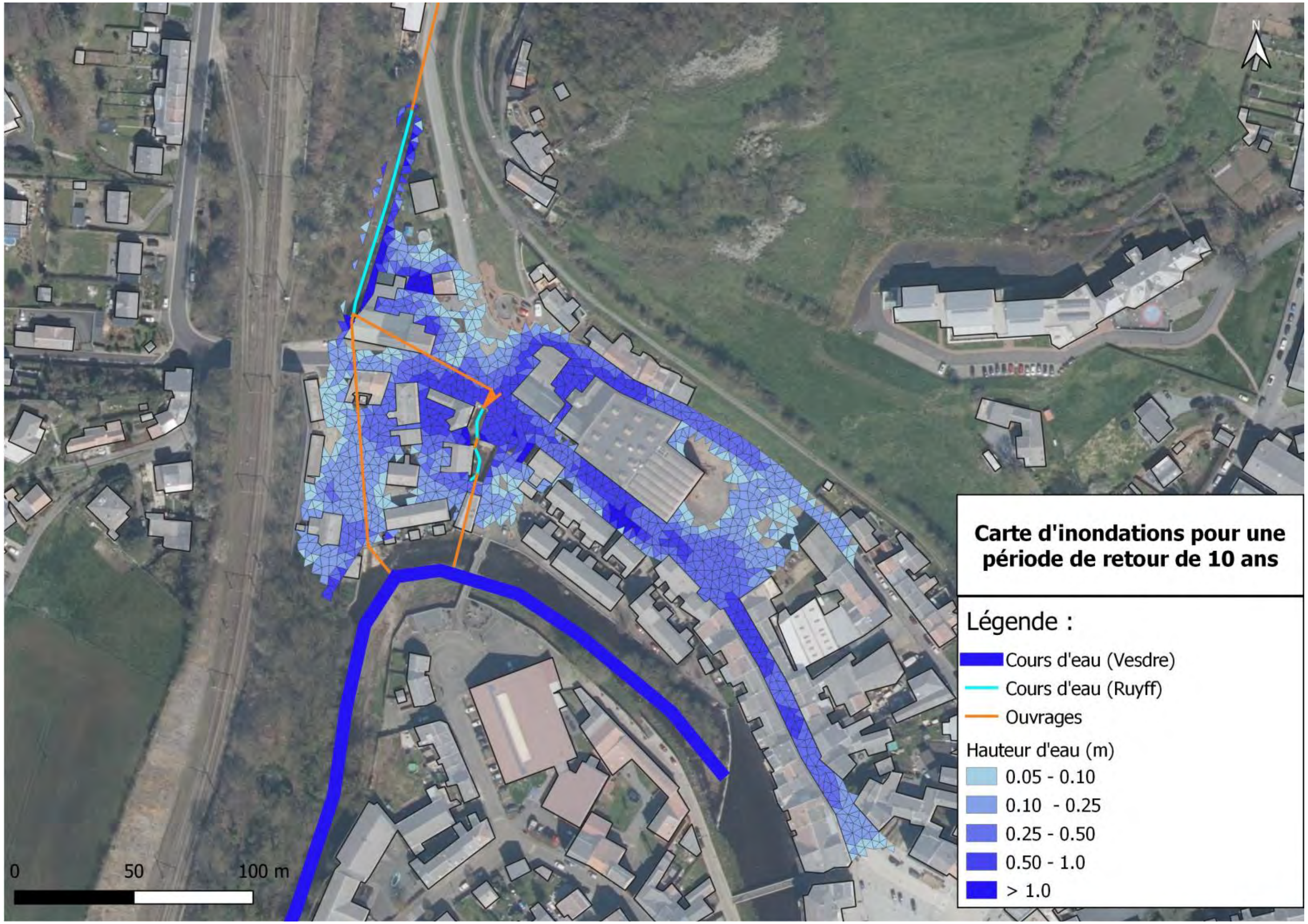


Figure 4.7 - Flood map of the existing situation at the most critical time of the flood for a return period of 10 years.

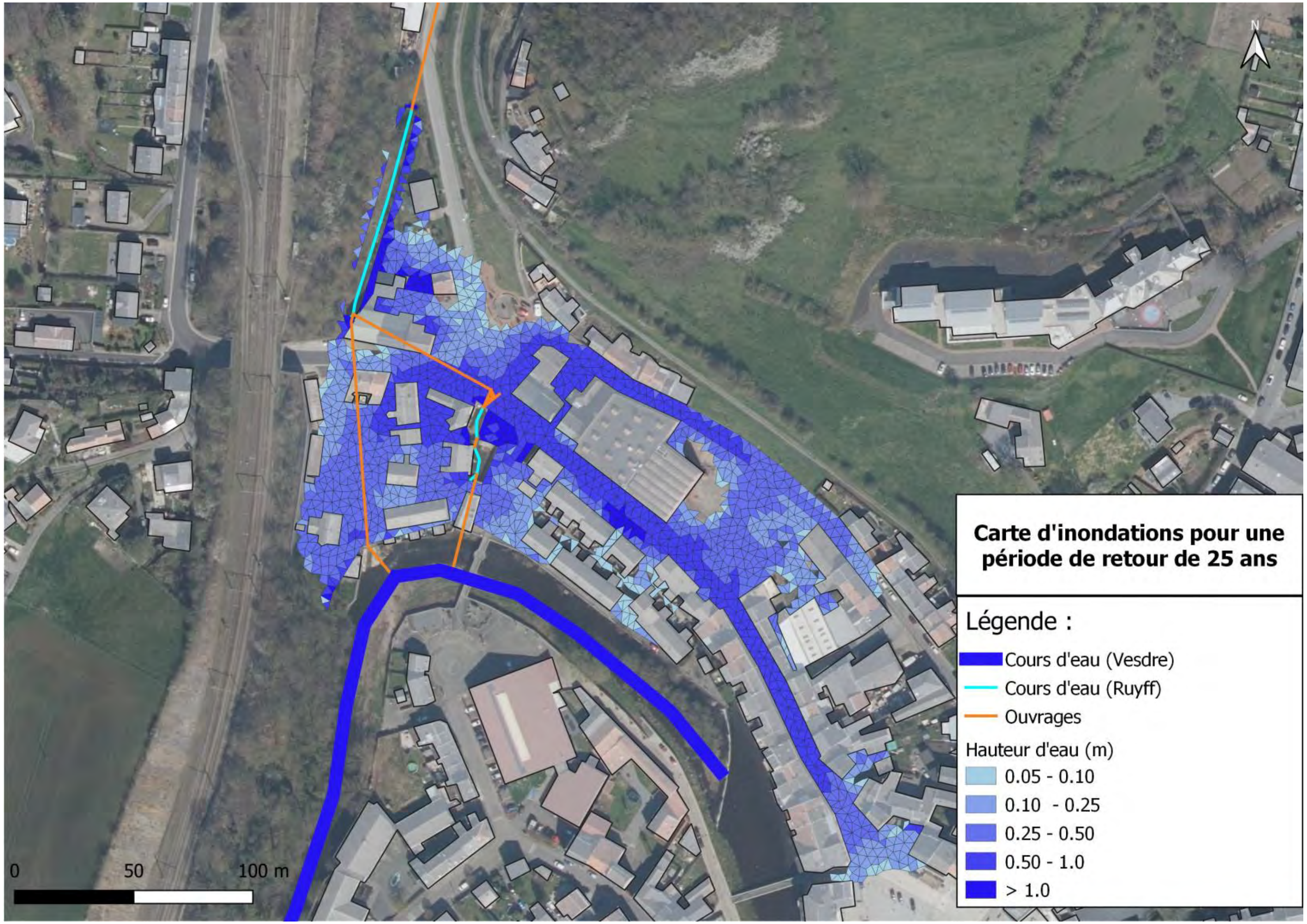


Figure 4.8 - Flood map of the existing situation at the most critical time of the flood for a return period of 25 years.

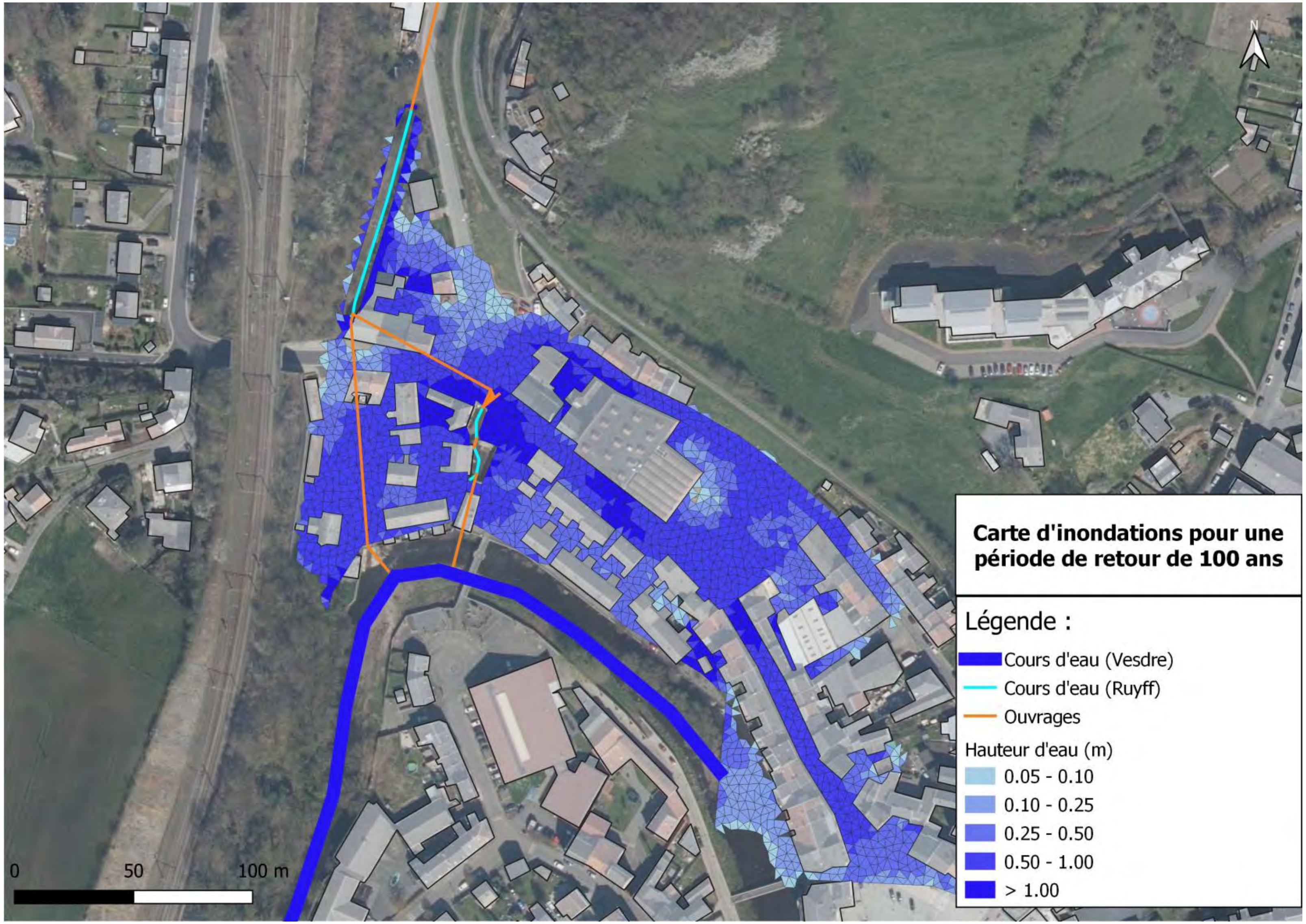


Figure 4.9 - Flood map of the existing situation at the most critical time of the flood for a return period of 100 years.

4.2 Influence of the Vesdre on the Rhuyff floods

An analysis of the influence of the Vesdre on the limiting structures is carried out by injecting a flood flow of 90 m³/s upstream of the Vesdre reach, which makes it possible to obtain a water level that puts pressure on the downstream side of the OA02 sluiceway (without causing an overflow of the Vesdre into its major bed, which would obviously worsen the situation).

Figure 4.10 shows the longitudinal profiles from the critical zone upstream of Rue du Moulin en Rhuyff to the confluence with the Vesdre (via structures OA02 and OA02bis) for simulation with a rainfall of 5 years return period. The difference in water level in the river upstream of Rue du Moulin-en-Rhuyff is approximately 20 cm. This small difference in water level nevertheless implies a significant difference in the maximum flood extents as illustrated in Figure 4.11.

The same analysis is performed for a return period of 25 years and is illustrated in Figure 4.12 and Figure 4.13. The differences observed for a 5-year return period are still observed for a 25-year return period but are smaller.

It can therefore be concluded that the Vesdre does exert an influence on the Ruyff, even for small return periods. The differences in water level compared to the situation without the effect of the Vesdre decrease with the return period, which can also be observed for the maximum flooding areas.

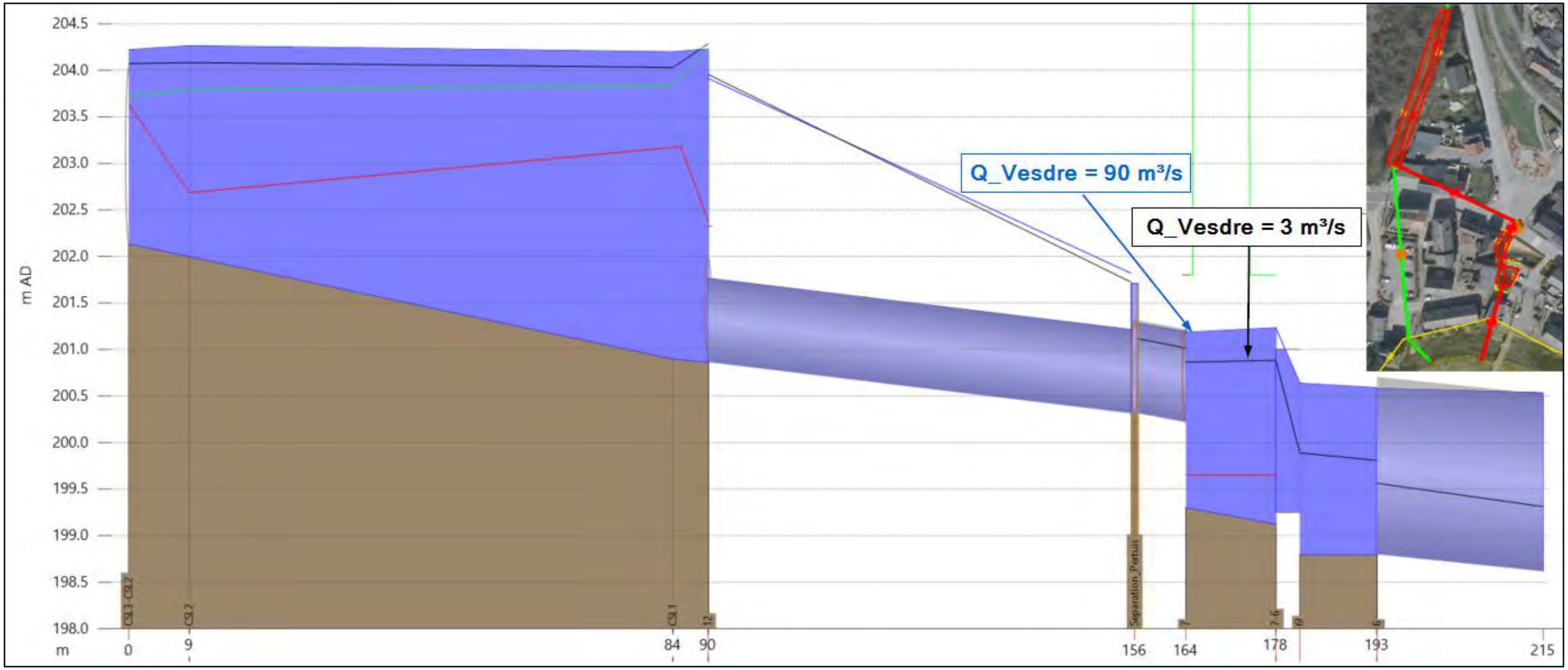


Figure 4.10: Comparison of the longitudinal profile of the critical zone upstream of the rue du Moulin en Rhuyff up to the confluence of the Vesdre (rectangular opening - OA2) for a return period of 5 years for a flow of the Vesdre of 3 m³/s (in black) and 90 m³/s (in blue)

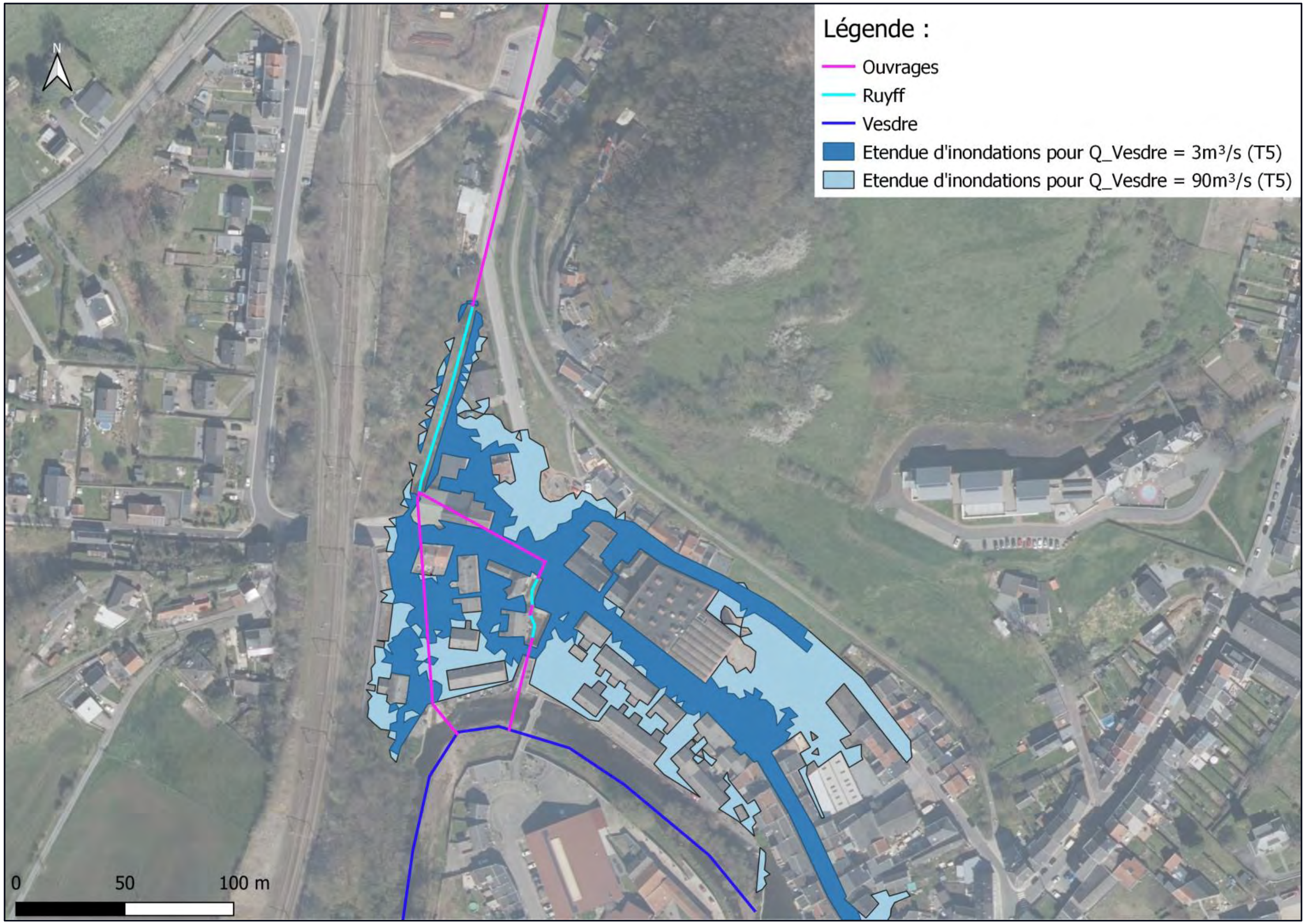


Figure 4.11: Comparison of the flood extent at the most critical time of the flood for a return period of 5 years and for two flows of the Vesdre (3 and 90 m³/s)

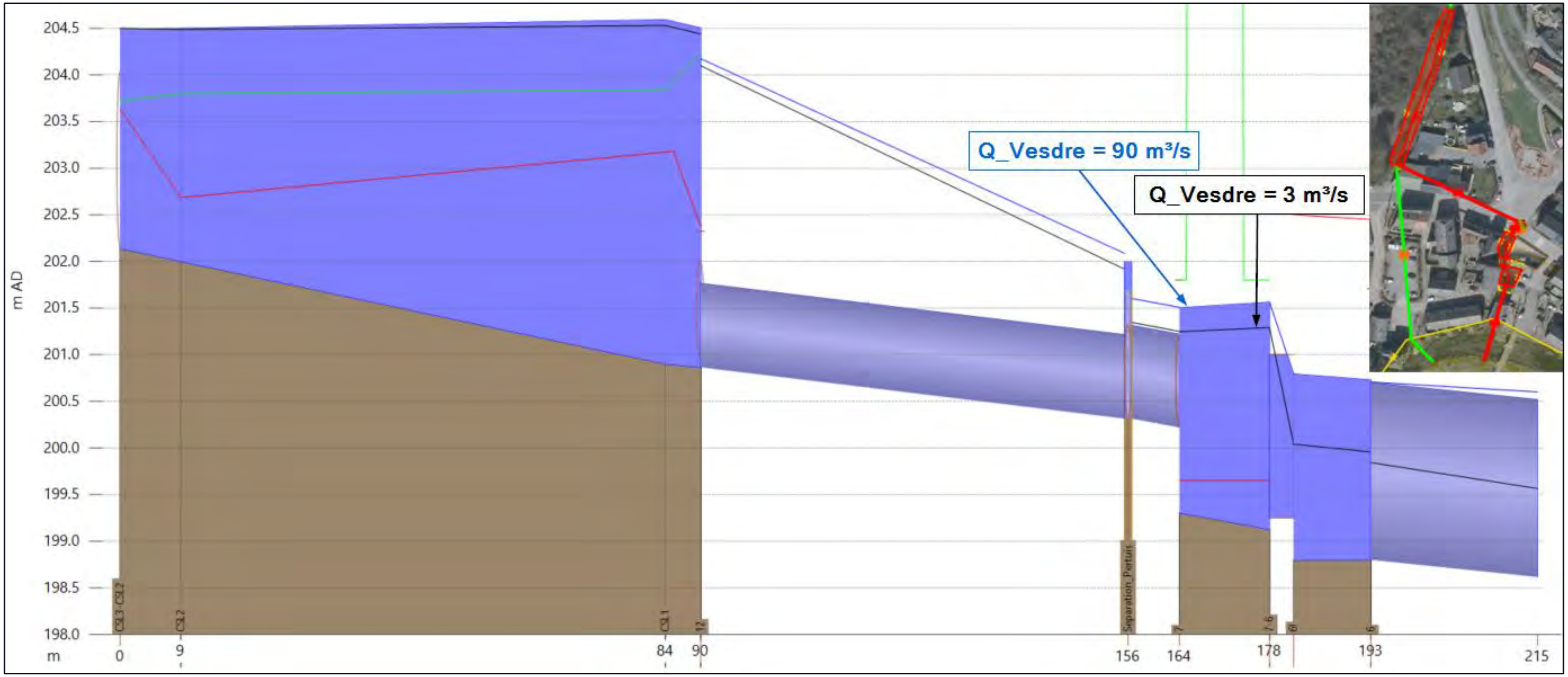


Figure 4.12: Comparison of the longitudinal profile of the critical zone upstream of Rue du Moulin en Rhuyff up to the confluence of the Vesdre (rectangular opening - OA2) for a return period of 25 years for a flow of the Vesdre of 3 m³/s (in black) and 90 m³/s (in blue).

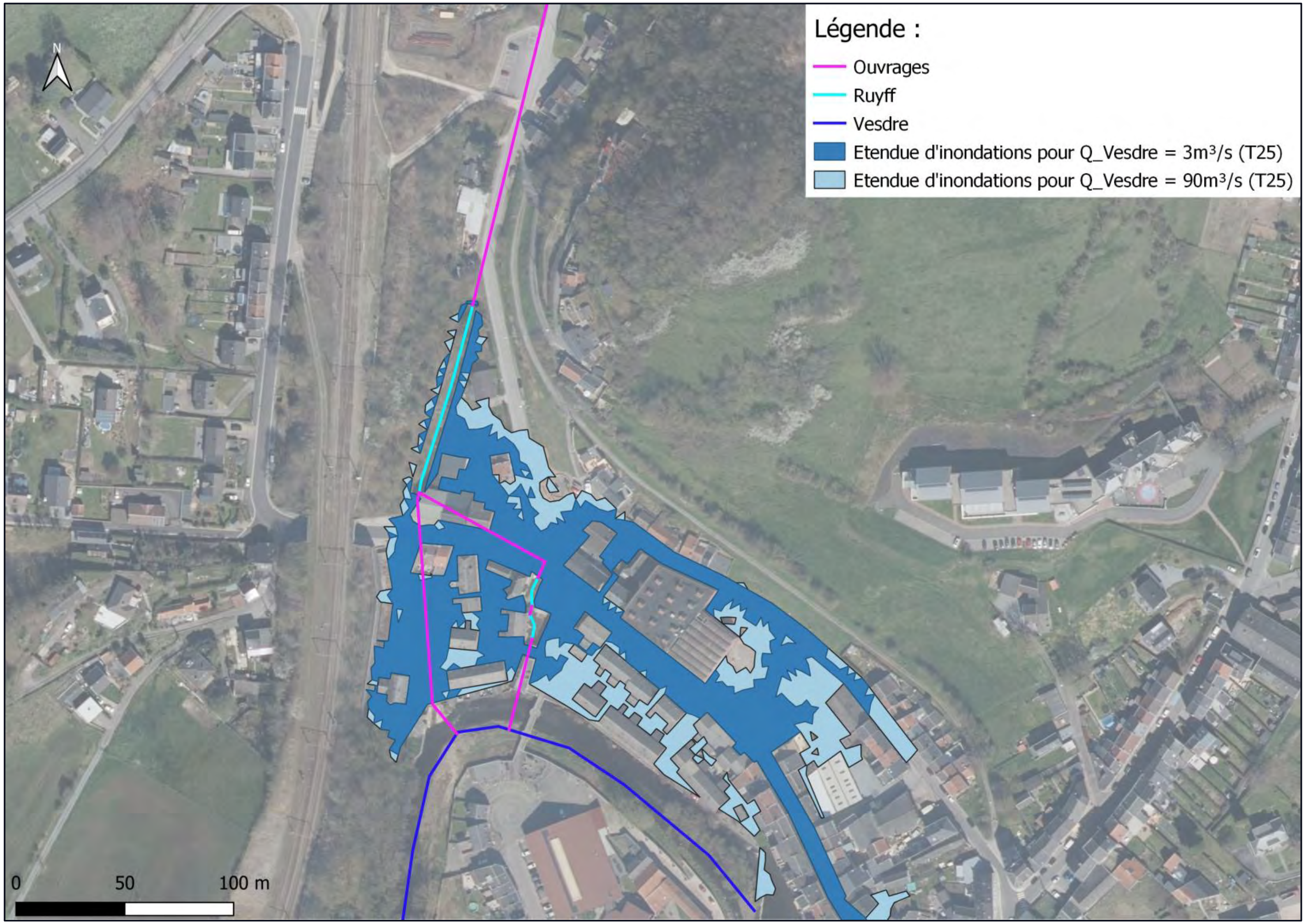


Figure 4.13: Comparison of the flood extent at the most critical time of the flood for a return period of 25 years and for two Vesdre flows (3 and 90 m³/s).

5 Identification and/or optimisation of relevant solutions

This section describes proposed solutions and the study of their impact on the flooding of the Vieux-Moulin at Dolhain-Limbourg. The reduction of flooding downstream of the study area can be achieved mainly by creating additional storage upstream of the area to be protected, i.e. by increasing the flow capacity at the critical bottleneck area.
In view of the flooding in the Vieux-Moulin area and the nature of the problem, two solutions have been put forward in this respect. Firstly, the retention of a volume of water upstream of the critical area by allowing a certain defined flow rate to pass through in order to avoid flooding downstream. A second solution is to reduce or eliminate the bottleneck that is unfavourable to the proper flow of the Ruyff at the level of the opening under the Rue du Moulin en Ruyff. These solutions are described in more detail in the following paragraphs.

With regard to the retention of a volume of water, the study of the topography did not make it possible to identify any other favourable zones than the one described and studied in the following section. The possibility of using the quarries as a retention area had been considered. However, this scenario was quickly ruled out in view of the topography (quarry crest approximately 20 m above the riverbed) and the scale of the work and bank heights that this would have entailed. In addition, the water diverted to the quarry would have had to be pumped back into the river once the flooding was over. This pumping would have resulted in high maintenance, energy and financial costs.

As for the reduction or elimination of the bottleneck, the scenario of opening up the watercourse on a section is studied below. Another scenario was considered, which consisted of installing a sluiceway parallel to the existing one in order to increase the capacity at this location. This scenario was discarded as it did not solve the problem of the double bends in the river described earlier in this report. In addition, the works required for this scenario would have been of the same magnitude as those for the open-cut scenario, but with less efficiency. Finally, solutions such as individual protections were not considered to be really relevant in this study area due to the extent of flooding once the river overflows and the proven problem of limiting the flow through.

Two scenarios were therefore considered on the basis of discussions with the watercourse manager:

- The dimensioning of a storm water basin upstream of the area to be protected;
- The opening of a section of the watercourse.

A combination of the two previous scenarios was not studied, as each of these scenarios involves major and costly works.

5.1 Design of the storm water basin

The first solution to be studied in the framework of flood reduction downstream of the study area is a first sizing of a storm water basin whose location has already been identified by the project owner at the level of a retention area in a meadow located to the north of the Saint Roch street (X= 261280; Y= 147920) and is illustrated in Figure 5.1. This dimensioning is based on the model built in the existing situation without modification of the downstream openings (structures OA02 and OA02bis).

The steps that have been implemented to carry out the sizing of this basin are

- Estimation of the admissible downstream flow to avoid overflows in the Moulin-en-Ruyff neighbourhood according to the information collected in the study area and the field visit ;
- Application of a safety coefficient to the admissible downstream flow to take into account the intermediate hydrological contributions between the basin and the critical zone to be protected in order to obtain an admissible flow at the basin outlet;
- Determination of the retention volume based on the graphical method (hydrograph cut at a flow value) using fractions of the admissible flow;
- Coupling several leakage rates with the return period allows to obtain a series of volumes to be retained;
- The volume to be stored at the future site is then compared to the currently available volume estimated on the basis of the DTM in order to determine the realistic leakage rate to be applied according to the protection objective, the height of the embankment and the corresponding right-of-way;

The most realistic scenario is implemented in the hydraulic model to analyse the impact compared to the existing situation and to conclude on the effect of the implementation of the storm water basin.

5.1.1 Estimating the volume to be stored

The first phase of the analysis consists in determining an admissible flow rate according to the zones at stake further downstream. This admissible flow (Qadm) is determined by analysing the available hydraulic model and determining the value of the flow to be respected to avoid flooding in the downstream critical zone. The analysis carried out previously shows that the admissible flow to be respected is about **10 m³/s** corresponding to the admissible flow of the structures OA02 and OA02bis.

As the storm basin is located upstream of the sector to be protected, the flow rate to be applied must be lower than this value and take into account the additional contributing surfaces between the temporary immersion zone and the critical structure. After a few iterations, it was determined that the reduction factor was about 30%, so that the flow rate to be applied at the outlet of the basin should be about **7 m³/s**.

The flows used in the following analysis to determine the volume to be buffered according to the return period are the permissible flow of 7 m³/s and flow values depending on this. The following flows are considered for the rest of the analysis:

- Q1 = Qadm = 7 m³/s
- Q2 = 0.8x Qadm = 5.6 m³/s
- Q3 = 0.6 x Qadm = 4.2 m³/s
- Q4 = 0.4 x Qadm = 2.8 m³/s

The evaluation of the volumes to be retained on the site is based on the simulated hydrograph in the area of the future storm water basin. Figure 5.2 shows the simulated hydrographs for the different return periods analysed according to the 4 theoretical leakage flows considered previously. The storage volume for the different return periods as a function of the considered downstream leakage flow is shown in Table 5.1. On the basis of this table, it appears that approximately 25,000 m³ should be retained on the future site in order to avoid downstream flooding for a return period of 25 years.

Table 5.1: Volume to be buffered (m³) depending on leakage rate and return period

Q leakage (m³/s)	V sup Q10 (m³)	V sup Q25 (m³)	V sup Q50 (m³)	V sup Q100 (m³)
7.0	14 523	25 167	39 653	62 460
5.6	20 150	34 901	54 901	85 188
4.2	27 589	50 349	77 242	113 790
2.8	40 587	73 261	106 639	148 183

It is important to remember that the estimates of the volumes to be stored are intended to be safe given certain modelling assumptions that have been made:

- Uniform application of project rainfall as an input to the model without application of an abatement coefficient depending on the size of the catchment area;
- Application of a factor that takes into account the intermediate contributions in order to reduce the leakage rate;

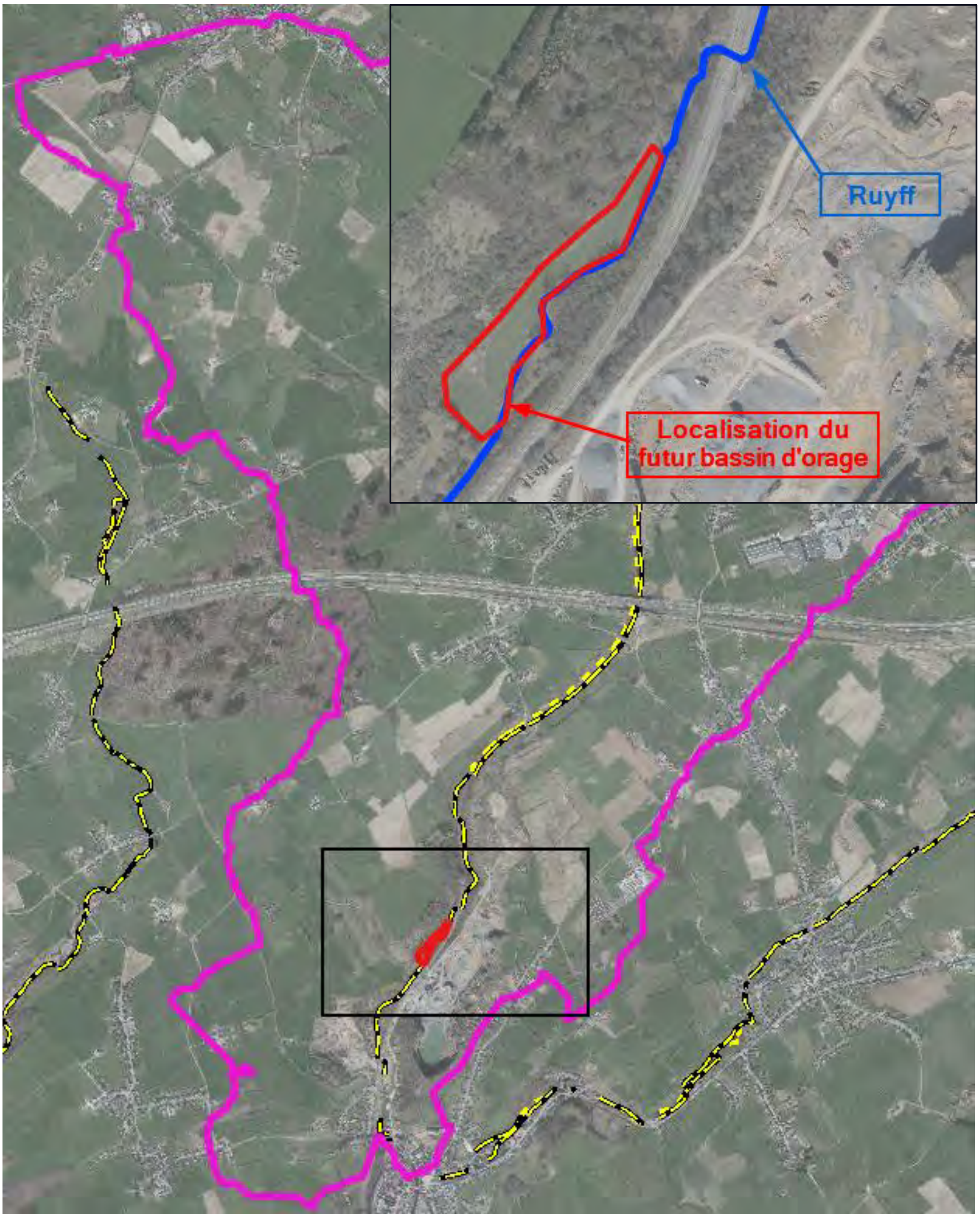


Figure 5.1: Location of the storm water basin to the north of Rue Saint Roch

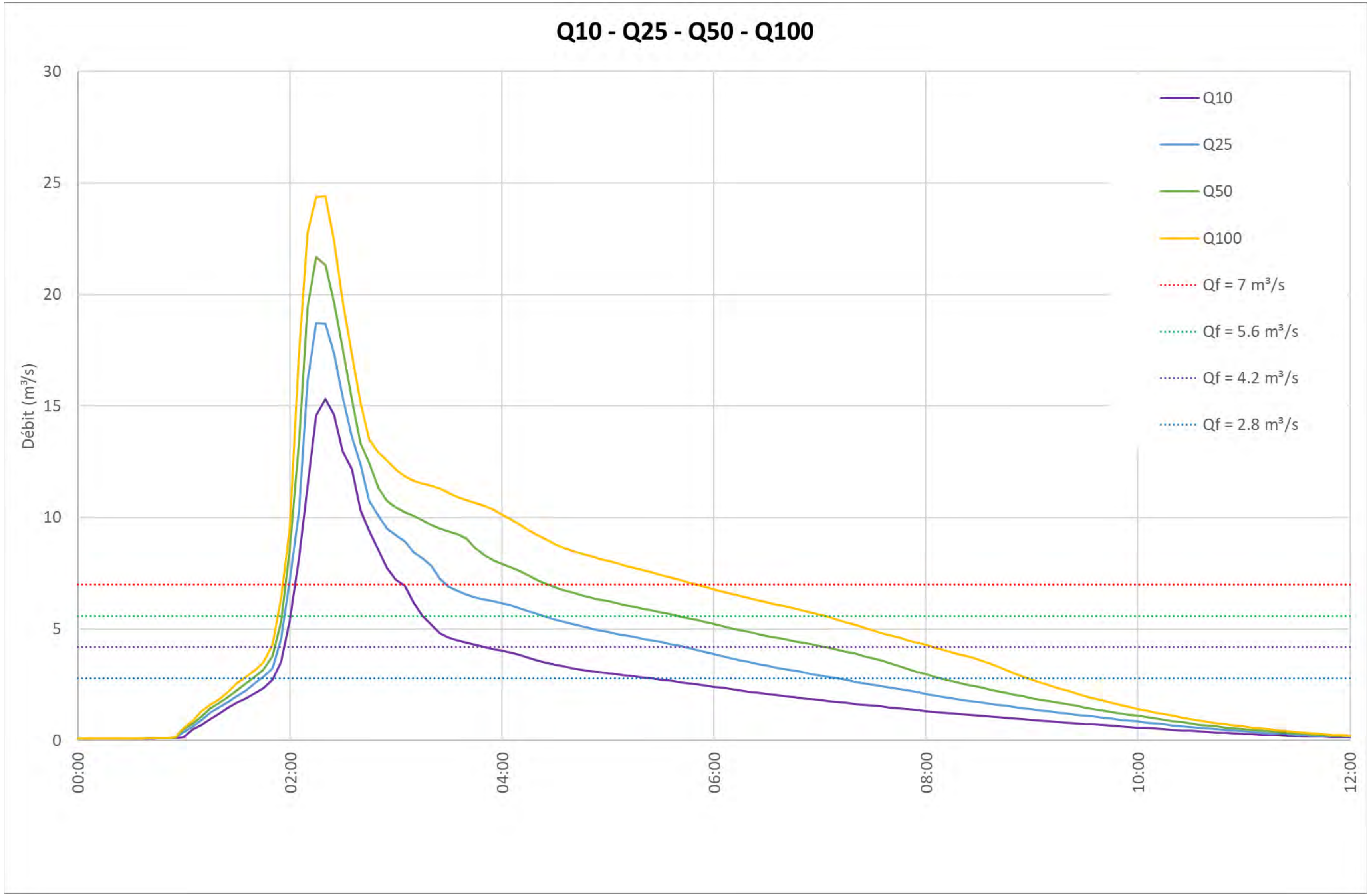


Figure 5.2: Simulated hydrographs in the area of the future storm water basin and leakage rates used to calculate the volume to be buffered.

5.1.2 Comparison of the volume to be stored and the volume available

The total volume to be reclaimed calculated in the previous section is compared to the volume theoretically available in the storm water basin right-of-way if only the currently available natural terrain is considered. The theoretically available storage volume in the storm water basin is determined by sampling the DTM in 0.1 m steps according to the estimated catchment area of the storm water basin in order to obtain a relationship between the level (mDNG) and the volume (m³) for the case where the existing natural ground is kept. This relationship between the storage volume and the level in the storm water basin is given in Figure 5.5. **The analysis of this curve shows that the level of the storm surge weir should be at a level of about 221 mDNG (i.e. about 4.5 m above the base of the river in this area) to store a volume of about 25,000 m³ associated with a 25 year protection objective with a leakage rate of 7 m³/s**

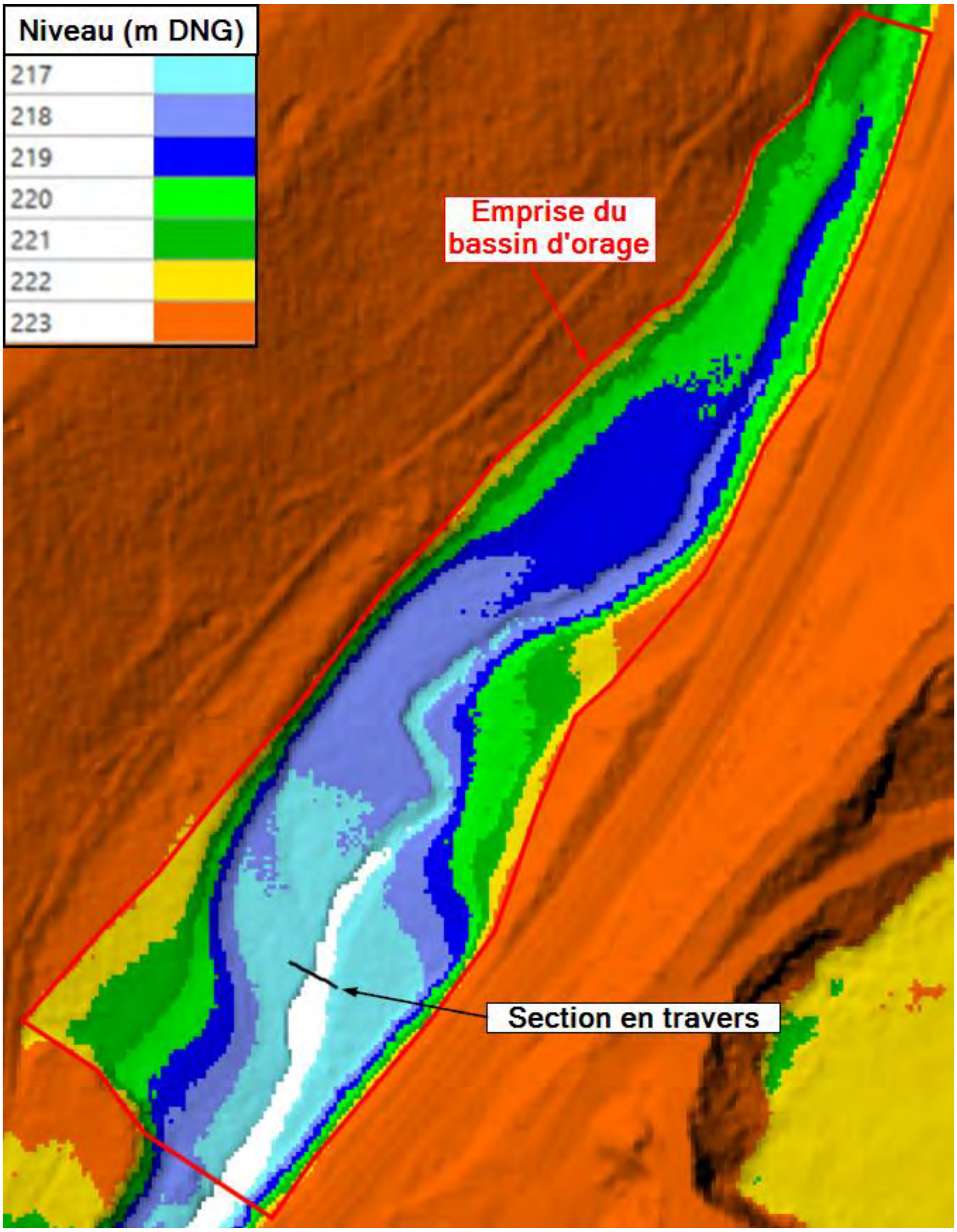


Figure 5.3: DTM levels (in m DNG) at the location of the future storm water basin

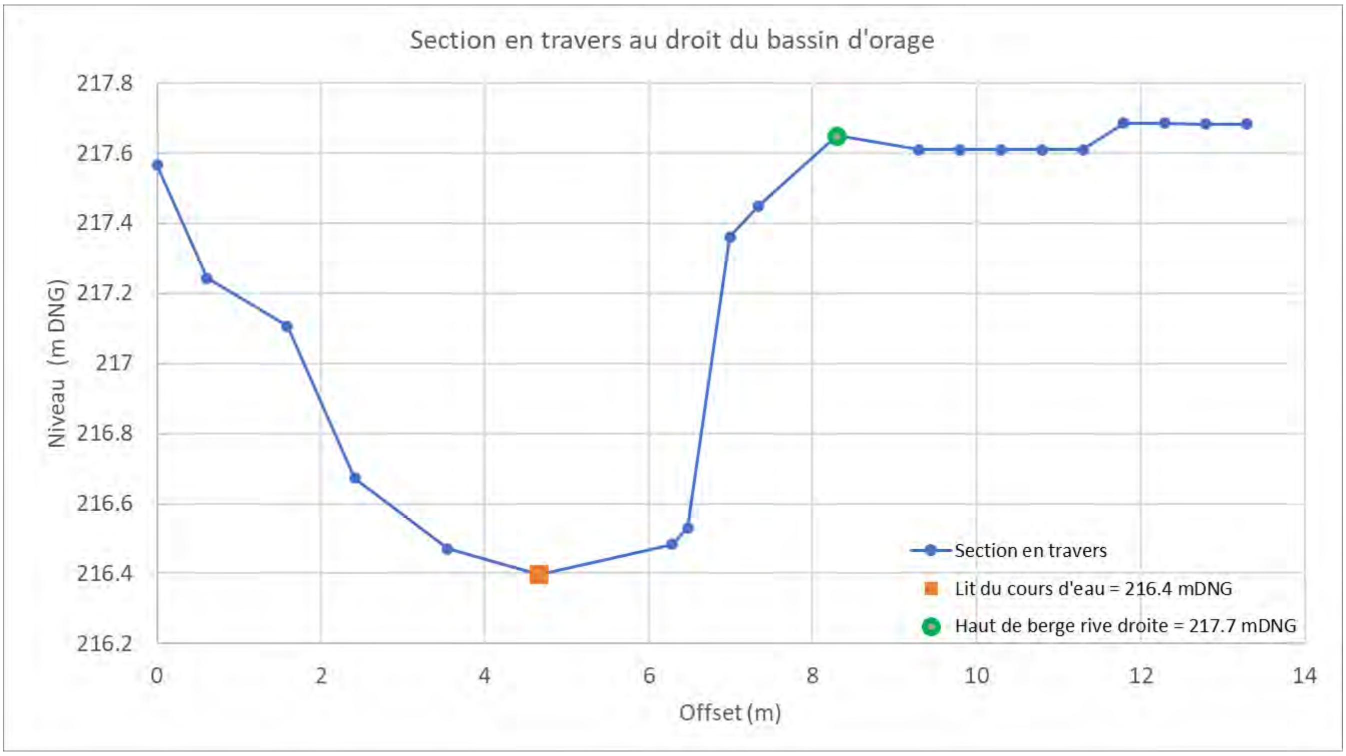


Figure 5.4: Cross-section at the storm water basin (top of bank and river bed)

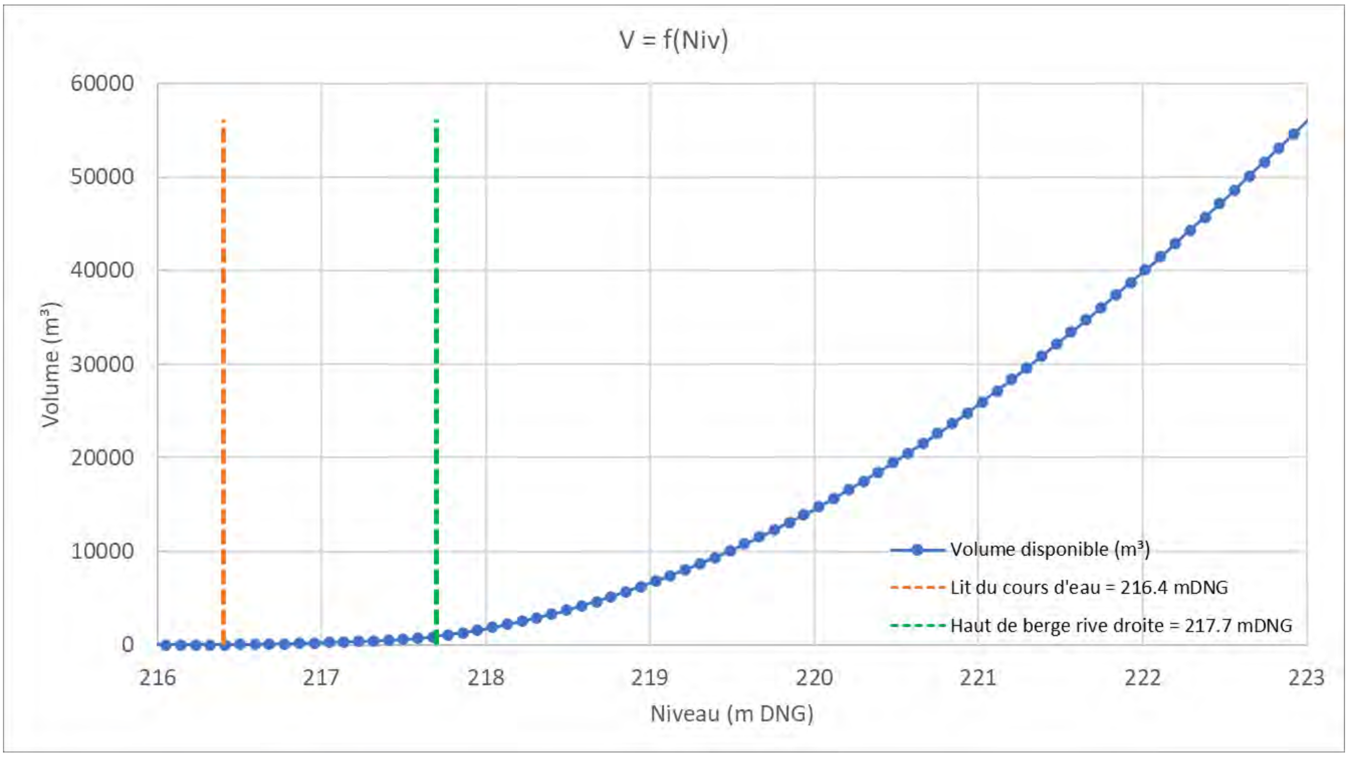


Figure 5.5: Storage volume of the storm water basin according to the level.

5.1.3 Optimisation of the design with the hydraulic model

The first analyses in relation to the volumes to be stored and the volume available allow the first sizing parameters to be considered (leakage rate, level of the safety spillway, level of the protection dam). The hydraulic model is used to carry out successive iterations and to optimise the design of the control structures. The result of the design of the storm water basin to guarantee a 25 year protection objective are in fine

- Leakage rate of 7 m³/s;
- Safety weir at a level of 221.2 mDNG, length of 16 m (dimensioning based on a 25-year flood);
- Protective embankment of approximately 50 m:
    - o at a level of 221.5 mDNG over a length of 20 m (dimensioning based on a 100-year flood)
    - o at a level of 222 m DNG over a length of 30 m

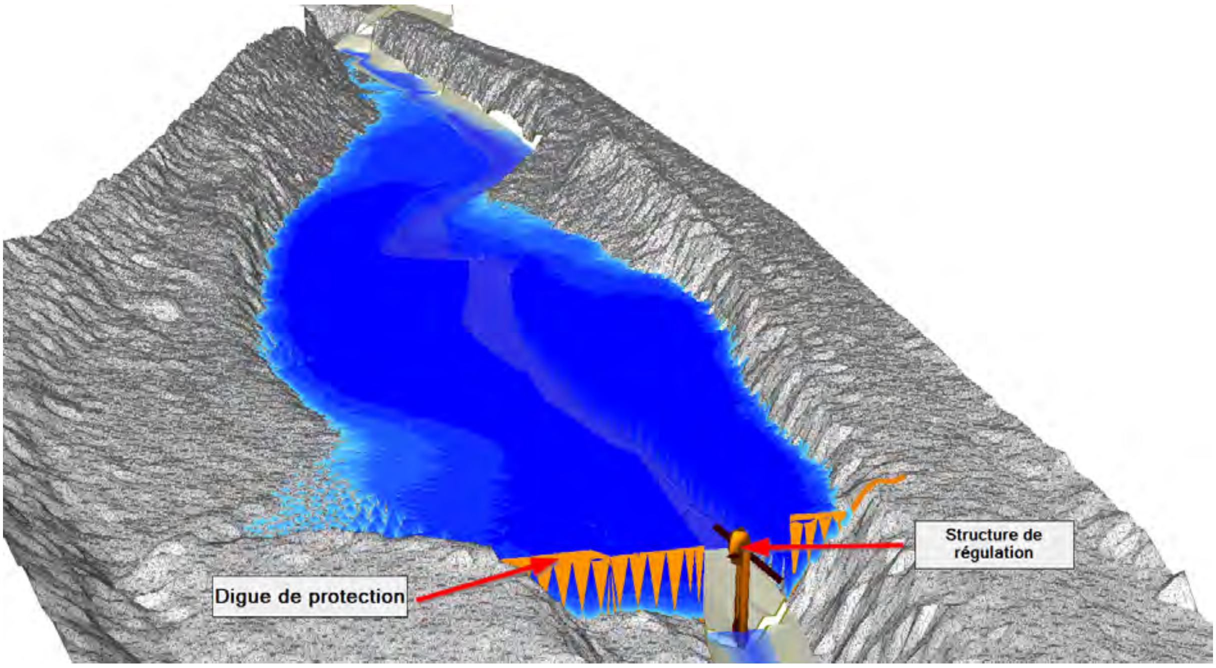


Figure 5.6 - Illustration of the integration of the storm water basin with its regulation structure and protection dam into the model.

5.1.4 Effects of the development on downstream flooding

Analysis of the effect of the TIA on the downstream

The simulated hydrographs for the return periods of 2 years, 5 years, 10 years, 25 years, 50 years and 100 years for the existing condition and after the implementation of the storm water basin with the previously defined dimensions are presented in Figure 5.7 to Figure 5.9.

These results highlight the significant effect of flow regulation on the downstream flood hydrograph, reducing the peak flow and smoothing the flood hydrograph. With the installation of the basin, the protection objective of approximately 10 m³/s downstream is respected up to a return period of 25 years, which has the effect of preventing the overflow of the watercourse in the Vieux-Moulin district. Beyond this return period, the threshold value of 11 m³/s is exceeded, and this leads to overflowing of the river (see flood maps for the future situation for return periods of 50 years and 100 years given in Figure 5 10 and Figure 5 11 respectively). Nevertheless, the effect of the pond is still evident in the flood hydrographs, but the activation of the pond's safety spillway results in an increase in the pond's outflow above the control value of 7 m³/s. The attenuation of the downstream hydrographs for the 50-year and 100-year return periods also results in a reduction of the flooded area as can be seen in Figure 5.12 and Figure 5.13. The differences observed between the modelling results of the existing and future situation for an exceptional 100-year flood are less significant, which is logical given the smaller effect of hydrograph attenuation between the 2 situations.

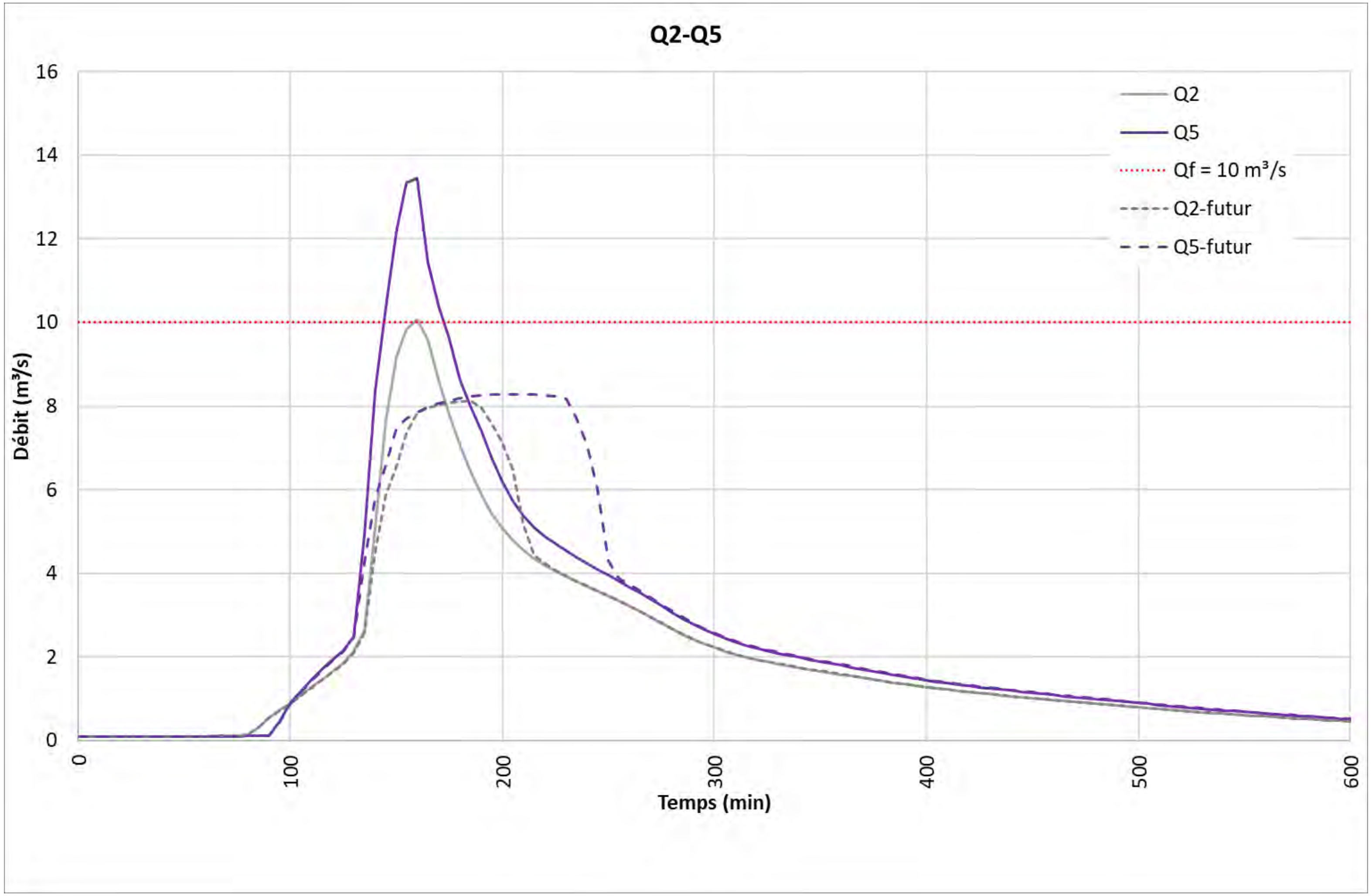


Figure 5.7 - Simulated hydrographs upstream of the critical zone of the structures OA02 and OA02bis for the existing and future situation after installation of the storm water basin for the return periods of 2 and 5 years.

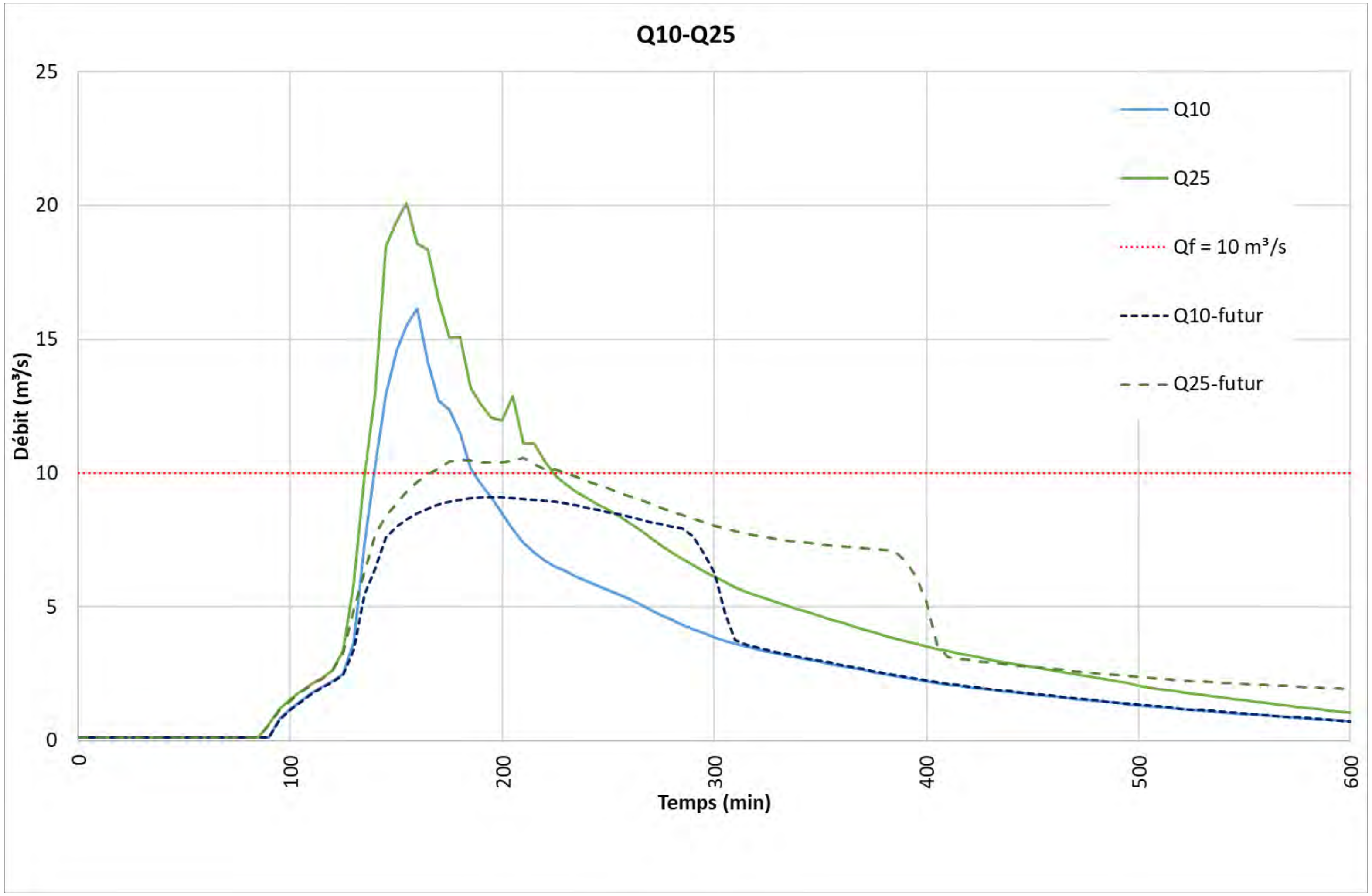


Figure 5.8 - Simulated hydrographs upstream of the critical zone of structures OA02 and OA02bis for the existing and future situation after installation of the storm water basin for return periods of 10 and 25 years.

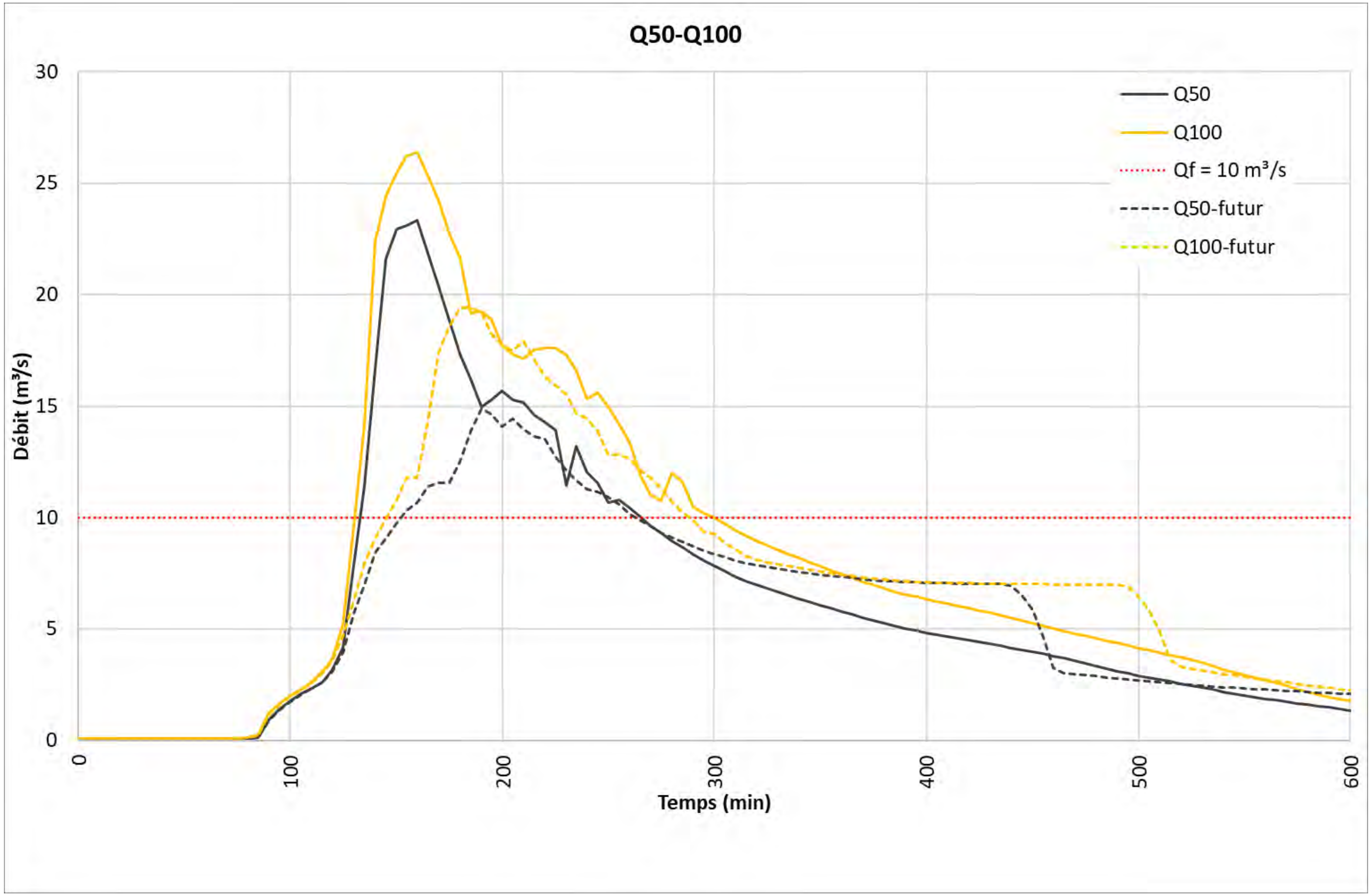


Figure 5.9 - Simulated hydrographs upstream of the critical zone of structures OA02 and OA02bis for the existing and future situation after installation of the storm water basin for return periods of 50 and 100 years.

Figure 5.10 - Flood map of the future situation at the most critical time of the flood for a return period of 50 years.

Figure 5.11 - Flood map of the future situation at the most critical time of the flood for a return period of 100 years

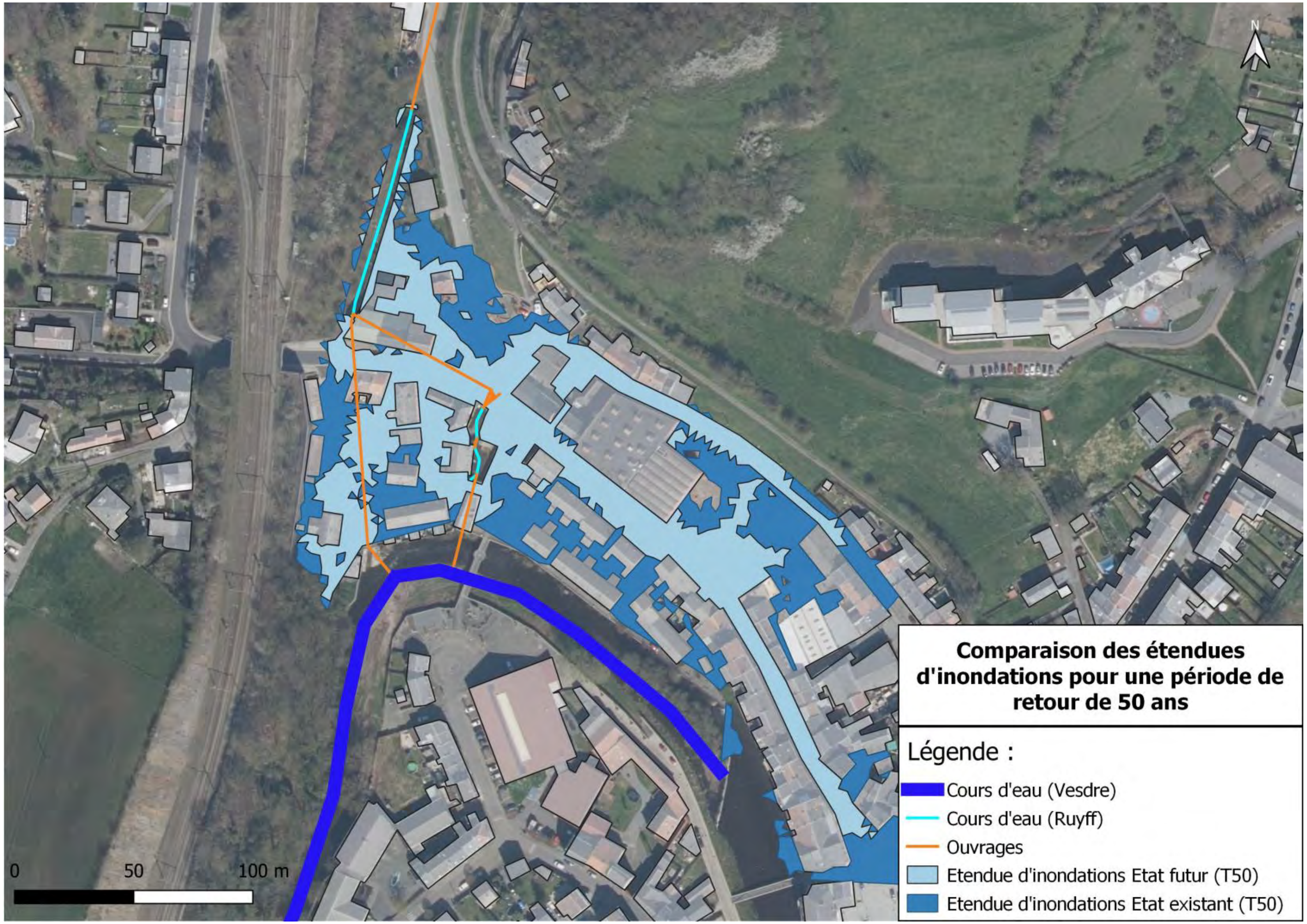


Figure 5.12 – Map comparing the flood extent of the existing and future situation at the most critical moments of the floods for a return period of 50 years.

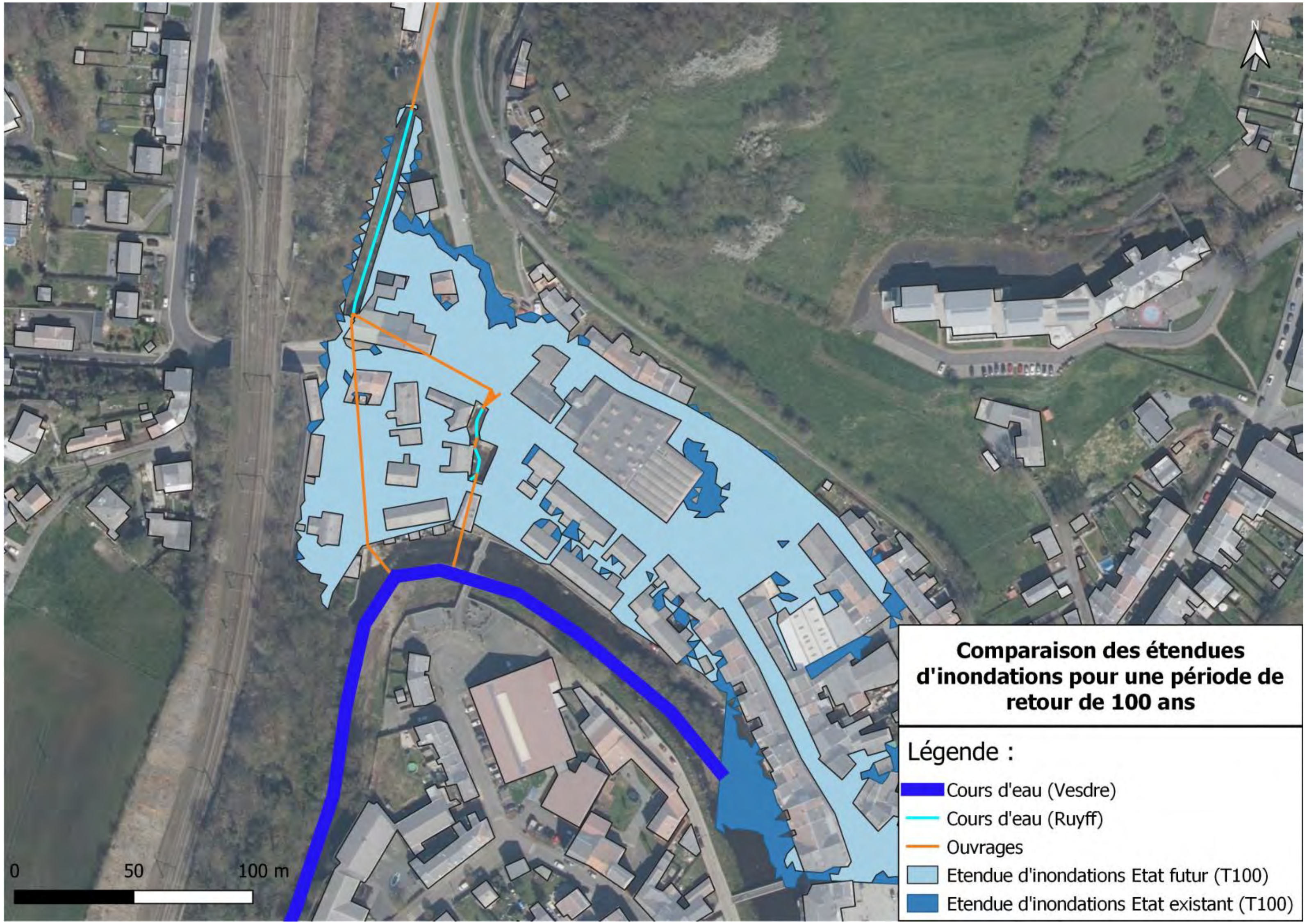


Figure 5.13 – Map comparing the flood extent of the existing and future situation at the most critical moments of the floods for a return period of 100 years.

Flooded area upstream of the storm water basin

There is little overflow of the watercourse in the existing condition in the area of the future development. The reduction in flow at the stormwater basin results in an increase in the floodplain upstream of the control structure. The simulated flood plots at the location of the retention basin are presented in the following figures for the 10-year, 25-year and 100-year return periods. The effect of the basin is marked upstream for the 100-year return period up to the downstream of the passage under the railway line. In view of the levels designed for the protective embankment at a level of 221.5 mDNG, there are no consequences for the areas at stake which would be present upstream, which is essential for the feasibility of the installation of such a structure.

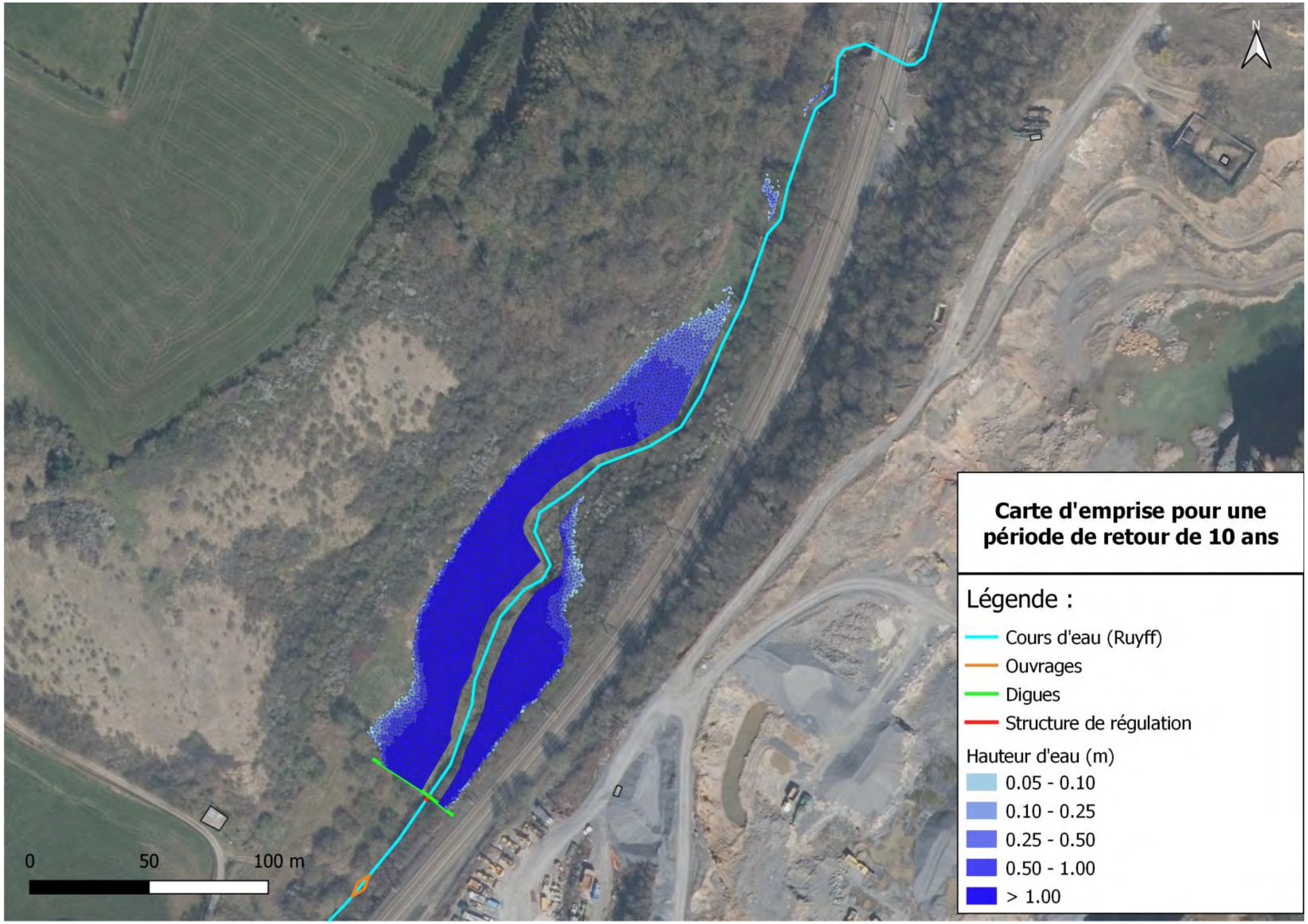


Figure 5.14 – Maximum flood extent for a 10-year return period once the storm water basin is in place.

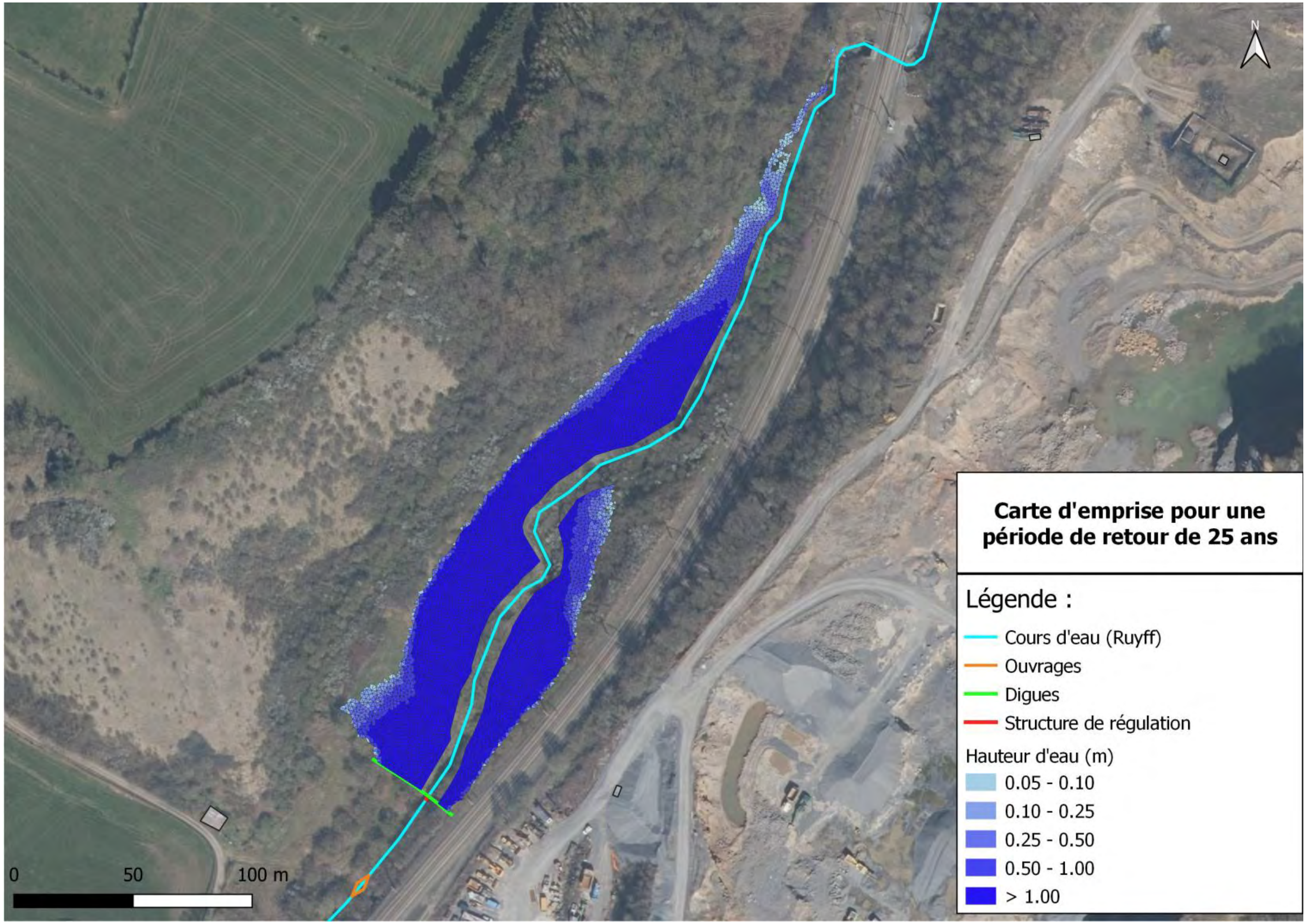


Figure 5.15 - Maximum flood extent for a return period of 25 years once the storm water basin is in place.

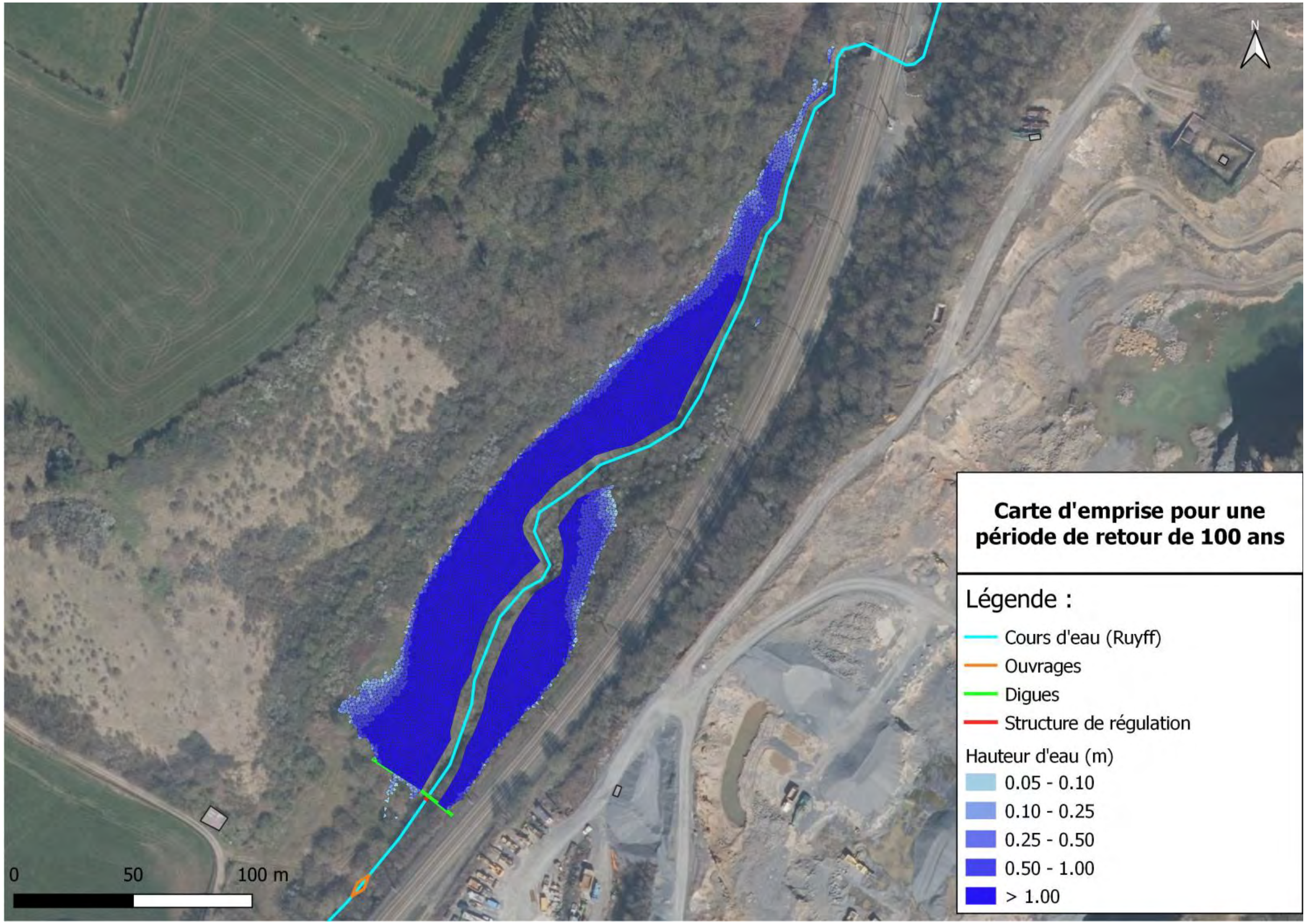


Figure 5.16 - Maximum flood extent for a return period of 100 years once the storm water basin is in place.

5.2 Opening the watercourse in one section

The hydraulic analysis of the existing situation in the critical area showed that the channelled part of the watercourse with a double change of direction constituted a hydraulic bottleneck which was the cause of the frequent overflows observed in this sector. One of the solutions to this problem is therefore to increase the flow capacity in the area.

Following discussions with the watercourse manager and also following ideas from the overall development plan for the Vesdre catchment area in the sector, a solution of opening up the watercourse on a section downstream from the study area in the Vieux-Moulin district was proposed and studied. The watercourse would now join the Vesdre in a straight line and would be opened up over the majority of its length, the only passage under aperture being necessary at the level of the passage under the road. The situation as it would exist in the future is shown in Figure 5.17. It should be noted that in this future situation, it is assumed that the current Ruyff passage (underpass under the houses with double change of direction) would be removed.

The biggest constraint of this scenario is the problem of the impetus passing under the Rue du Moulin en Ruyff. In addition to gas, telephone and electricity, a 630 mm diameter combined sewer pipe (rainwater and wastewater) crosses the opening of the watercourse and is not deep enough for the watercourse to pass over it (see Figure 5.18). The existing storm spillway is located on the left bank of the potential stream opening in the Old Mill area. The possibility of constructing a new storm overflow on the right bank of the watercourse, in order to discharge stormwater into the new part of the Ruyff, has been discussed with the watercourse manager and AIDE but will have to be studied in detail during the implementation of the project. At this stage, a pre-analysis of the feasibility of creating a weir and the impact of the Ruyff on drainage has been carried out and is available in Appendix 7.6. This analysis is based on certain assumptions such as the diameter of the flow reduction, the level of the weir, the location of the weir, the location of the connection between the weir and the watercourse, etc. These assumptions are considered satisfactory for the project. These assumptions are considered satisfactory to answer the question of the effect of the watercourse on the existing network but will need to be confirmed or modified during the detailed study of the modification of the sewerage network at this location. In view of the existing sewerage levels, the slopes present in the area and the model results, the important point to be retained from this analysis is that the water levels in the Ruyff in the future situation would have no impact on the discharge of stormwater from the weir into the watercourse up to return periods of 25 years.

In addition to the problem of impingement, the area has a steep slope. As the flows in the stream are already high in the area, the reduction in longitudinal length with the presence of existing steep slopes potentially leads to flows with high velocities in the stream even for small return periods. This element must be taken into account in the practical implementation of the project, in particular by means of appropriate measures (bank protection, etc.)

The steps that have been implemented to carry out the analysis of the river opening scenario are :

- The dimensioning of the opening under the Rue du Moulin en Ruyff road;
- The dimensioning of the open-cut section (gauge, slope, etc.). Concerning the gauge, several types of cross-section were considered;
- Integration in the hydraulic model of the most realistic scenario and analysis of the impact compared to the existing situation to conclude on the effect of the developments.

Figure 5.17: Open river scenario

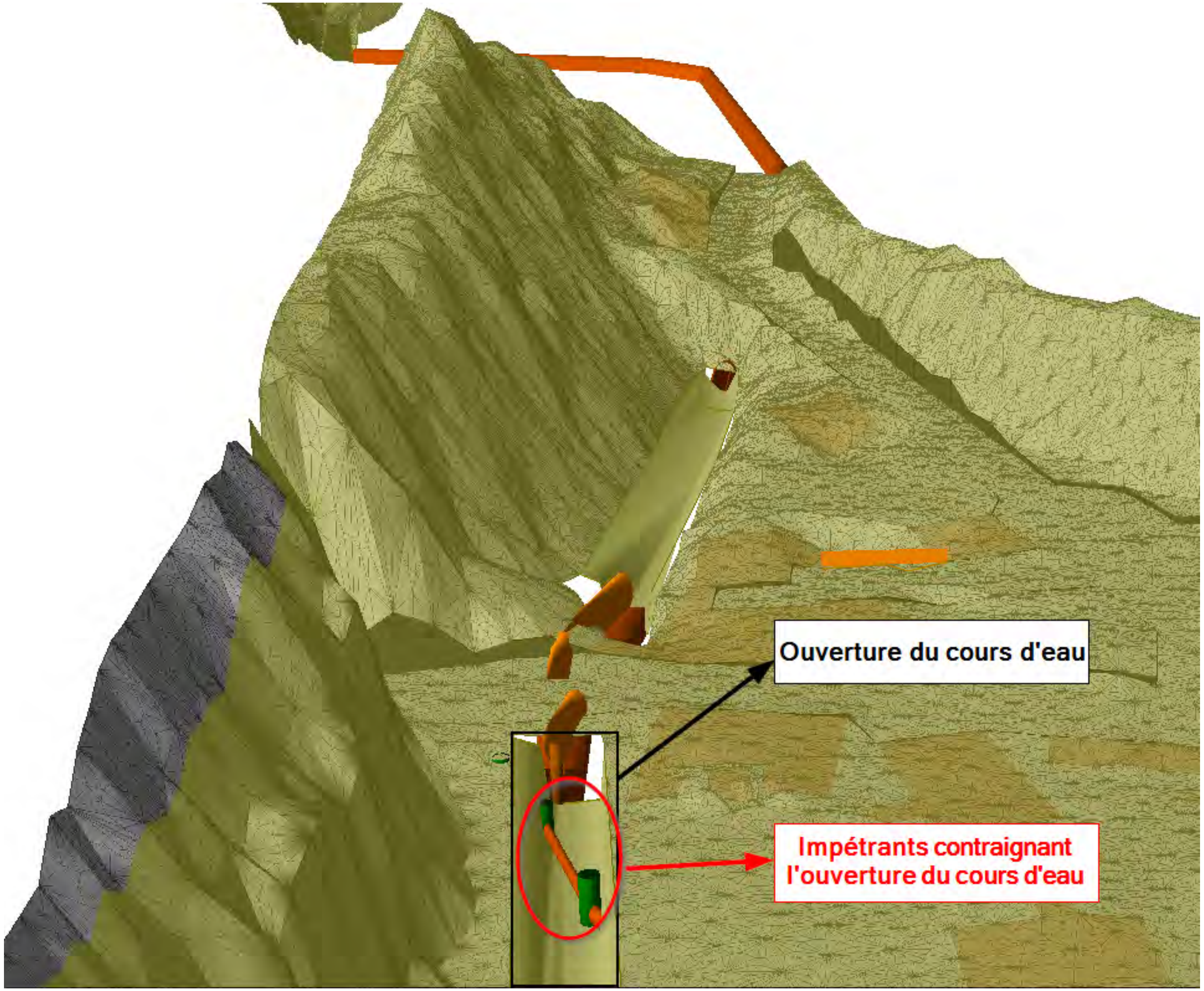


Figure 5.18: 3D view of the river opening scenario with emphasis on the constraint of the impediments

5.2.1 Stream sizing

Watercourse slope

In the future scenario considered, the watercourse is opened in a straight line from the Rue du Moulin en Ruyff over a distance of approximately 100 m to join the Vesdre. The slope of the watercourse on the new section is assumed to be the same as the existing pipe (O2bis), i.e. approximately 1.8%.

Passageway under the road

An opening under the Rue du Moulin en Ruyff is to be considered. A dimensioning was carried out, taking into account the gauge of the river upstream and the various local constraints of the site (level of the road, etc).

The dimensions of the new sluice would be :

- Upstream shelf: 200.87 m DNG
- Downstream apron: 200.35 m DNG
- Length: about 30 m
- Width x Height: 4000 x 1500 mm (6 m²)

Cross-section of the watercourse

For the part downstream of the opening under the road, we have designed several typical sections:

- A cross-section just downstream of the opening, with a smaller width at the top due to the space available
- A cross-section for the part closest to the Vesdre with a more natural profile and a larger top width. For the latter, 2 section proposals are given according to the available width that will be desired.

The dimensioning of the gauge downstream considers :

- A similar gauge to the one upstream because a reduction of the cross-section at the outlet of the channel is not desirable in order not to create a bottleneck with respect to the dimensions of the channel. This opening has been sized to allow sufficient passage under the road (especially for impediment) and not to increase the already high slope in this sector.
- The dimensions also include the desire to have a more natural gauge, i.e. with lesser slopes for the banks when possible. In view of the dimensions at the base of the section, the width at the top is in fact relatively high, even with slopes of the type 1:1 which are not yet close to the objective of a natural watercourse section.
- Increasing the gauge also allows for a minimum reduction in velocities which are not negligible given the flows to be handled and the slopes involved (explained in more detail later in this report).
- The parameters of the cross-sections (shape, width at the base, width at the top, passage area, slope of the banks, etc.) are average parameters, i.e. they are globally representative of the hydraulic characteristics of the future sections. Of course, the operational implementation of the project will deviate significantly from these values due to the addition of further local variations (roughness, micro-topography, local riprap etc).

The cross-section of the open-cut stream at the outlet of the sluiceway as dimensioned is shown in Figure 5.19. This proposal retains the same width of the watercourse upstream and follows the existing topography based on a 1:1 bank profile. The width of this section remains quite small in order to keep a reasonable distance between the watercourse and the Infrabel wall on the right bank and the house on the left bank (79 rue du Moulin en Ruyff). The maximum area of this section is 12 m² for a water height of 2 m.

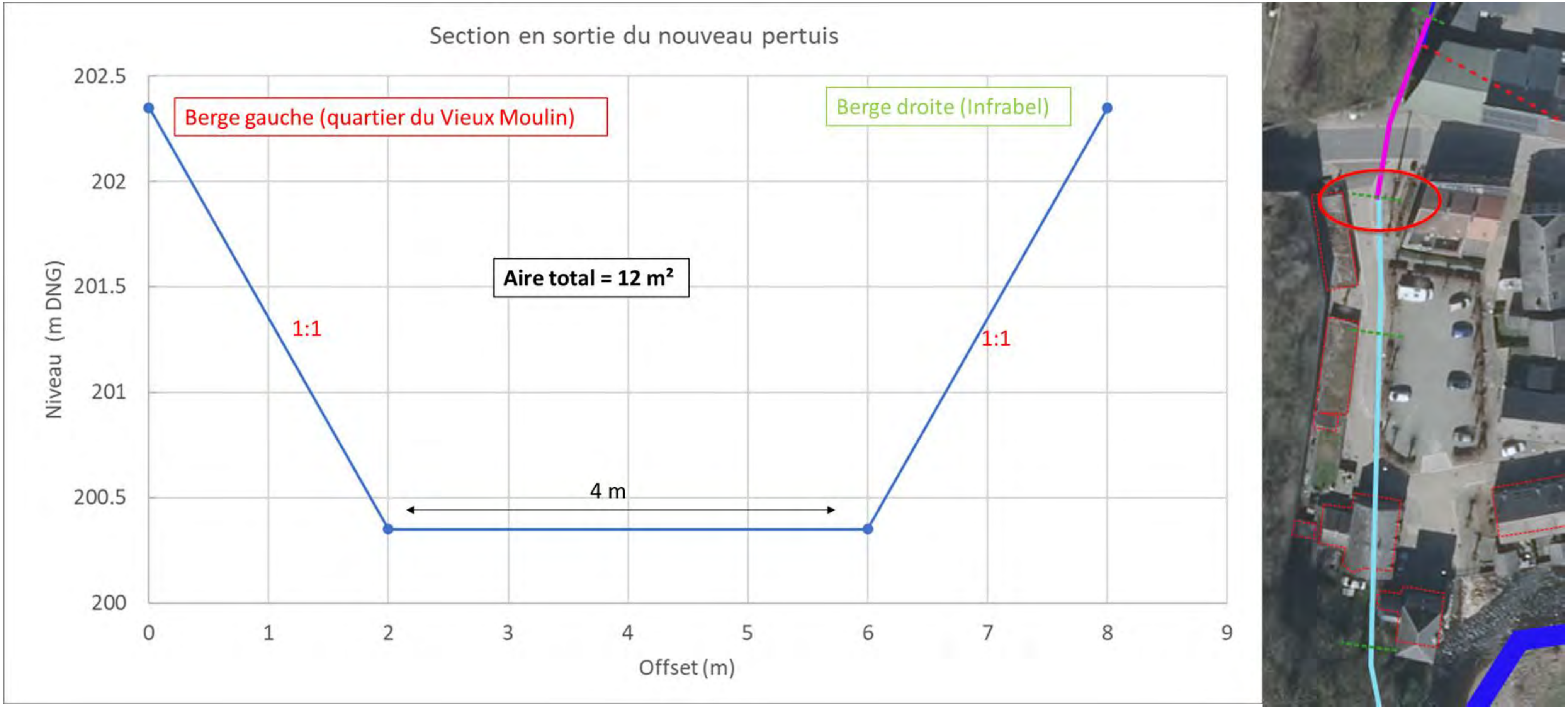


Figure 5.19: Proposed cross-section at the exit of the future tunnel passing under the Rue du Moulien en Ruyff.

Several proposed cross-sections for the second part of the open watercourse are presented below:

- Scenario 2A: Cross-section width of 10m and maximum area of 14m² at a water height of 2m, as shown in Figure 5.20
- Scenario 2B: Cross-section width of 15m and maximum area of 20m² for a water height of 2m, as shown in Figure 5.21

The interest of these two scenarios is to be able to analyse the sensitivity of these two different cross-sections on the height and velocities of the water in this new open cut. The Manning's coefficient used for these cross sections in the model is 0.05, corresponding to a natural section of a river with riprap.

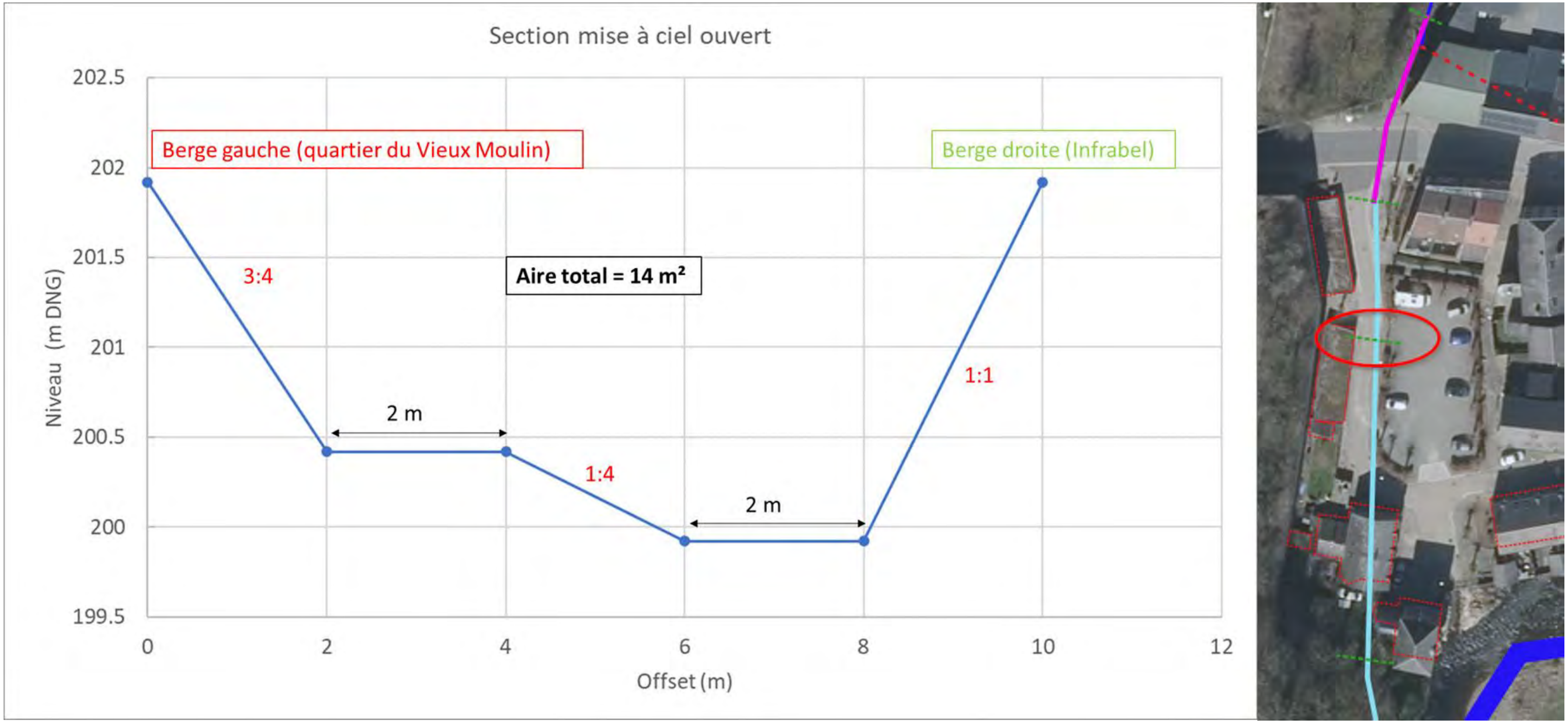


Figure 5.20: Scenario A: 10m-wide cross section

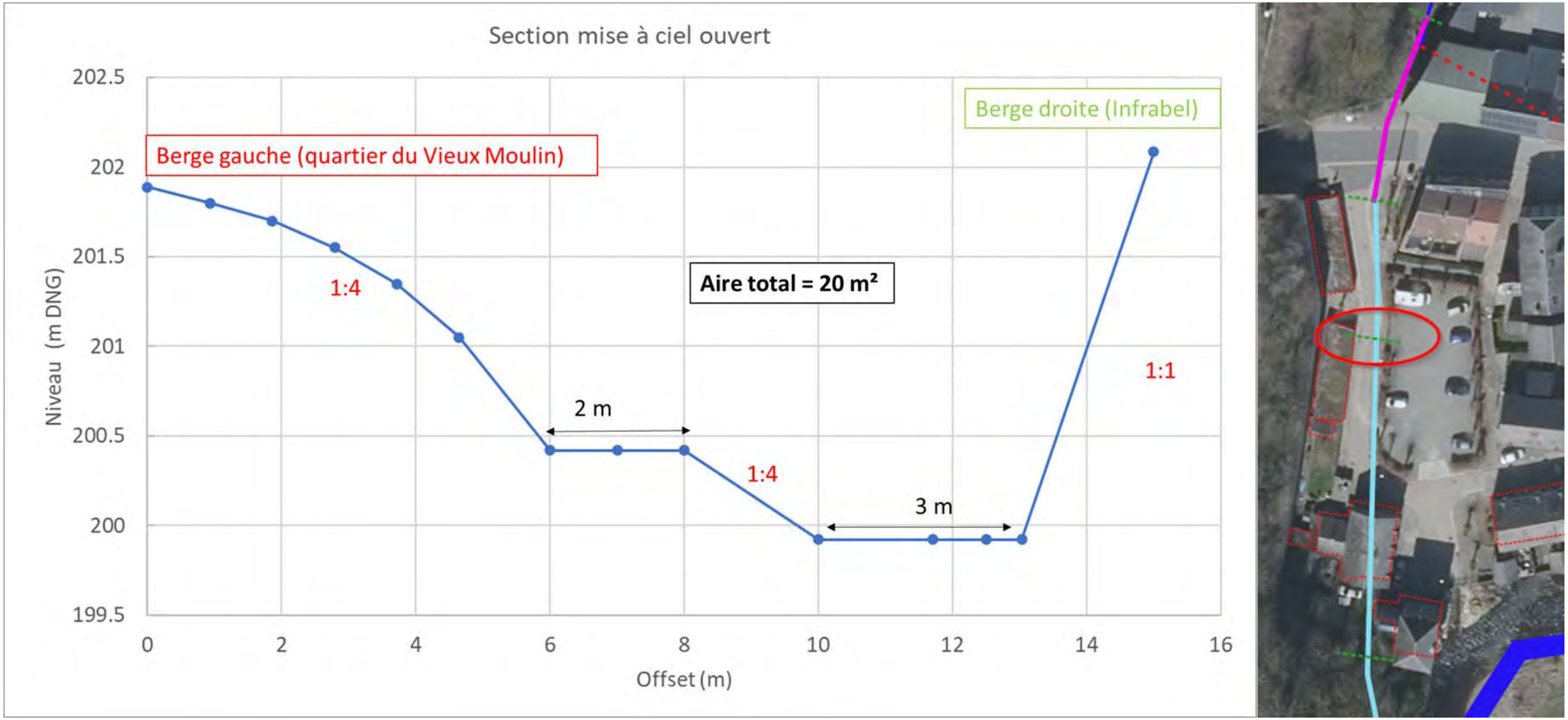


Figure 5.21: Scenario 2B: 15m-wide cross section

5.2.2 Effects of the development on downstream flooding

Scenario 2A - 10m-width at top of cross section

Figures 5.22, 5.23 and 5.24 illustrate the water level of two cross sections (at the most critical time of the simulation) for return periods of 2, 25 and 100 years. Table 5.2 shows the peak flows, maximum heights and velocities for these different return periods.

Table 5.2: Simulation results according to the return period for scenario 2A

Return period	Max. flow (m^3/s)	Max water level (m)	Max. speed (m/s)
2 years	10.0	1.0	2.0
25 years	20.4	1.4	2.5
100 years	25.5	1.5	2.7

These results indicate that the water level does not exceed the capacity of the watercourse, even for high return periods. This development therefore strongly reduces, if not totally, flooding in the critical downstream sector. However, high velocities (> 2 m/s) are observed for low return periods. Erosion protection and bank stabilisation measures should therefore be considered during the implementation of the project.

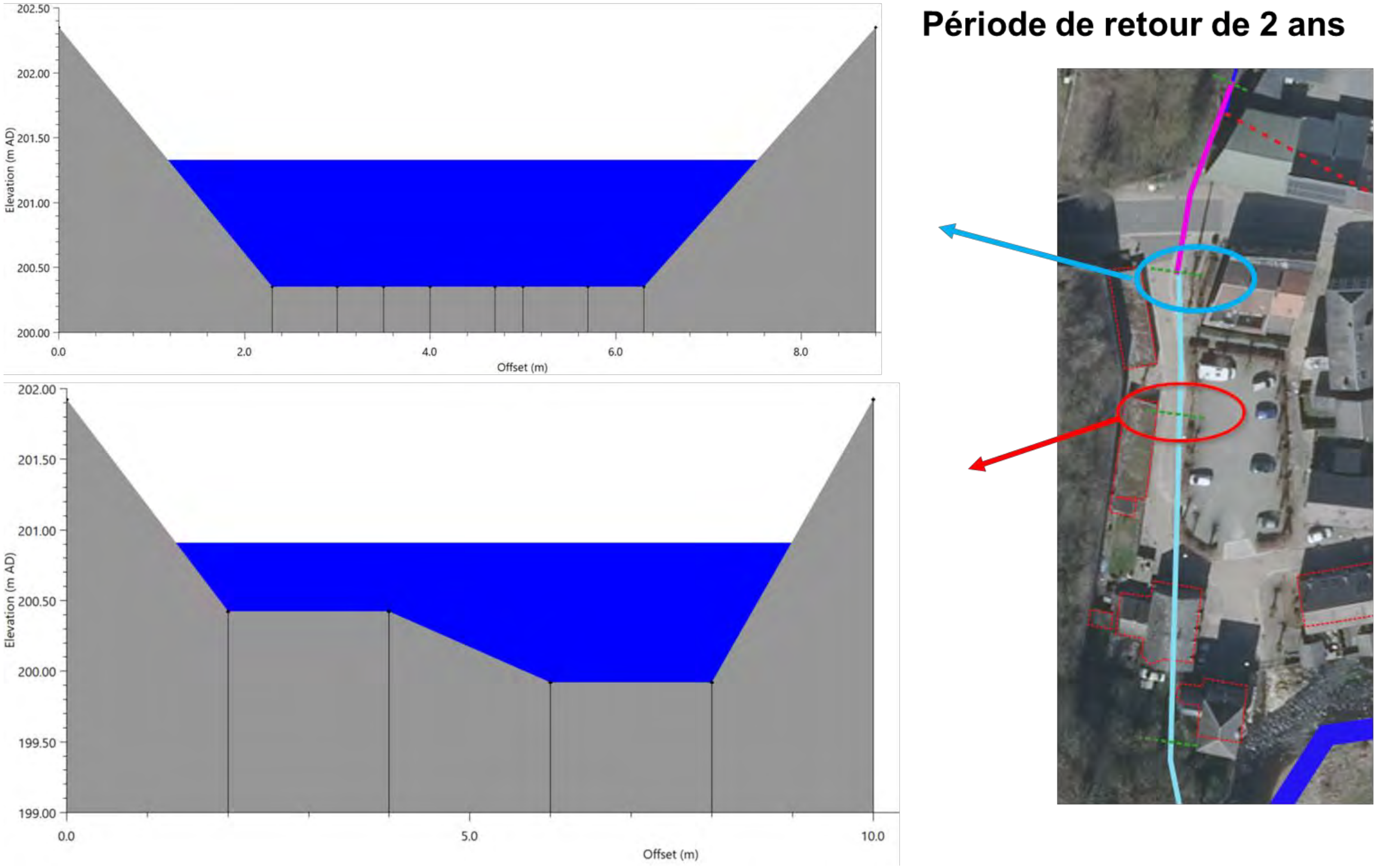


Figure 5.22: Water level in the cross-sections at the river opening for a return period of 2 years for scenario 2A.

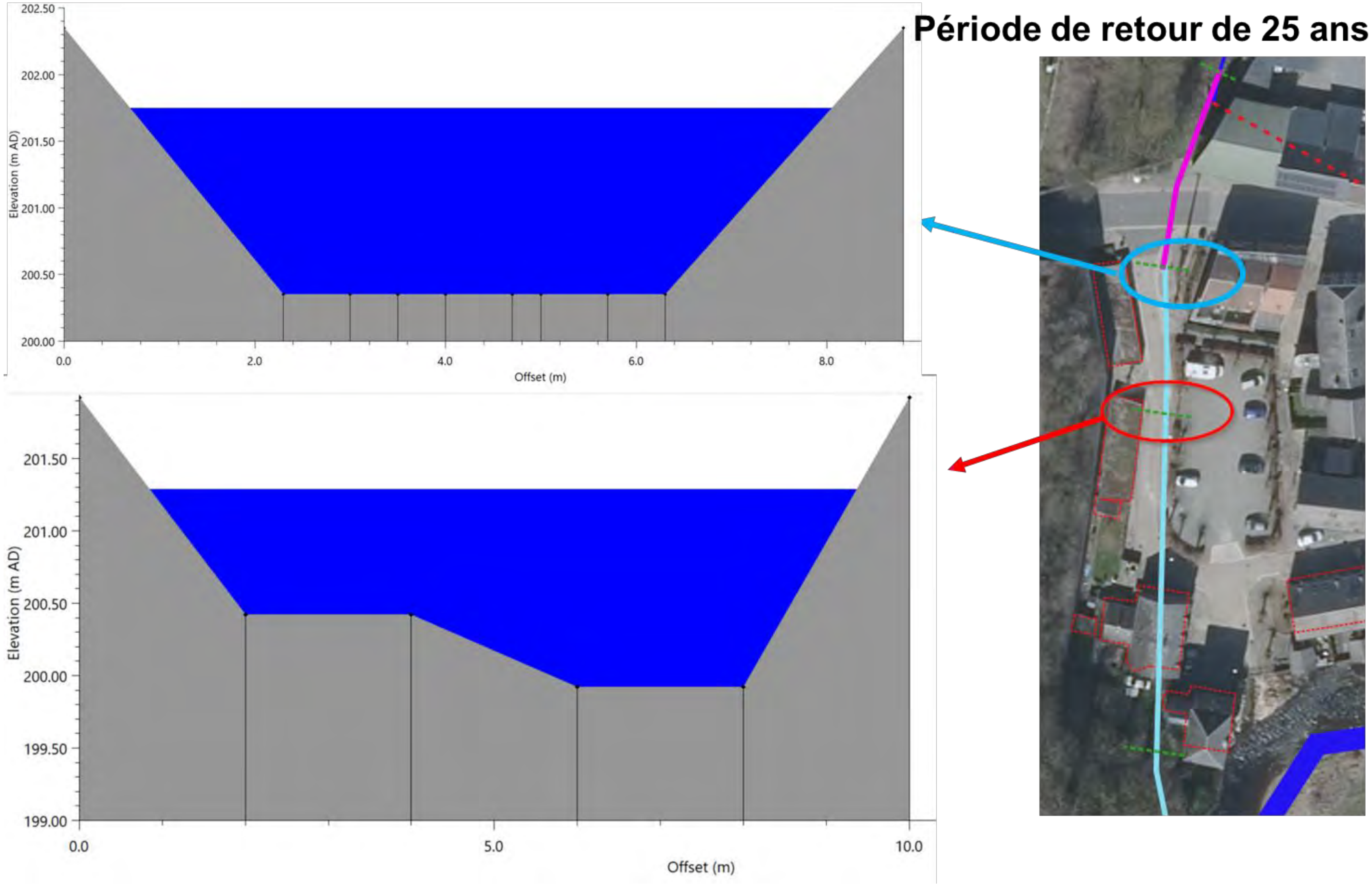


Figure 5.23 : Water level in the cross sections at the river opening for a 25-year return period for scenario 2A

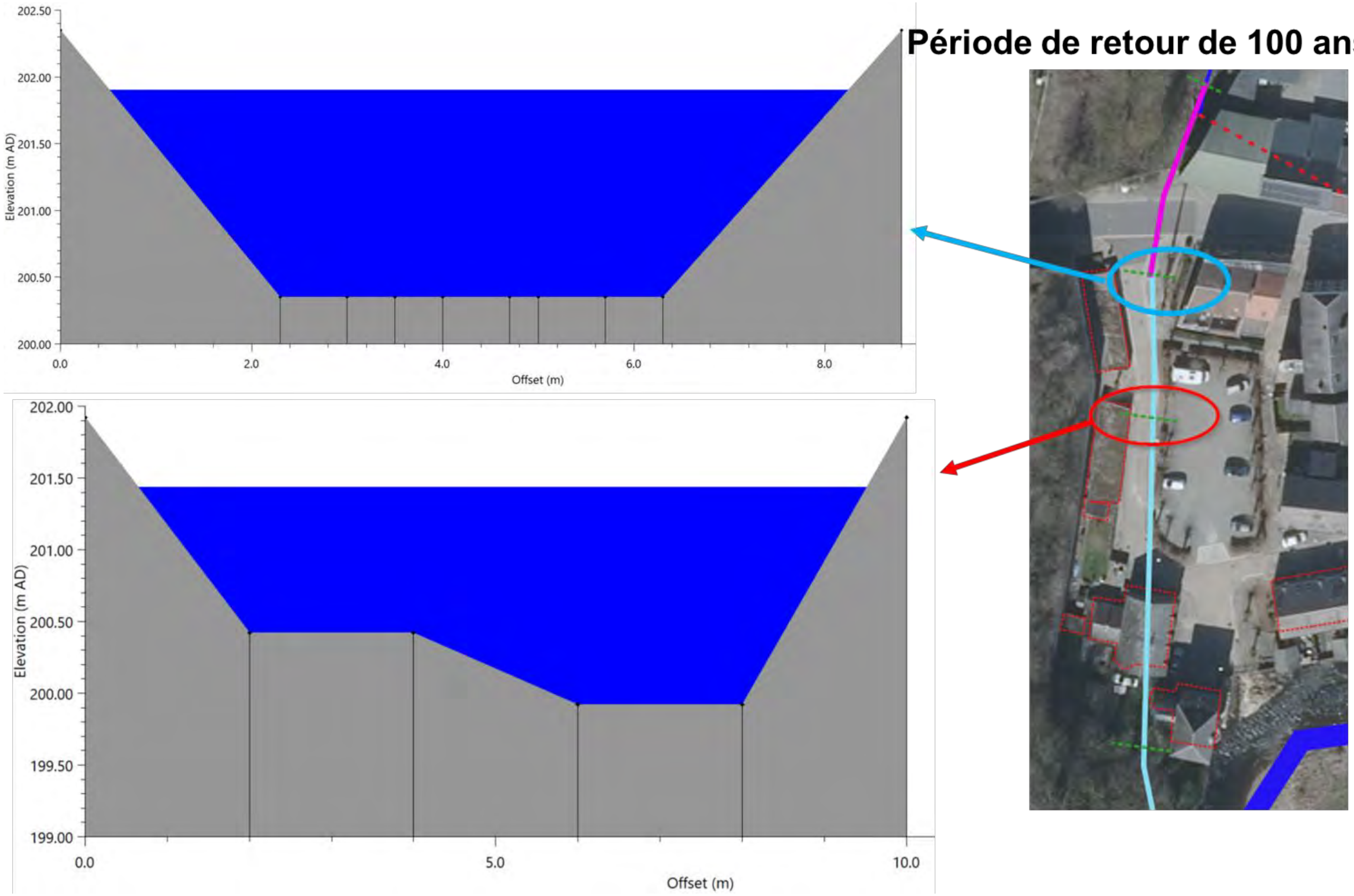


Figure 5.24 : Water level in the cross-sections at the river opening for a return period of 100 years for scenario 2A

Scenario 2B - 15 m-width at top of cross section

Figure 5.25, 5.26 and 5.27 illustrate the water level of two cross sections (at the most critical time of the simulation) for return periods of 2, 25 and 100 years. Table 5.2 shows the peak flows, maximum heights and velocities for these different return periods.

Table 5.3: Simulation results according to the return period for scenario 2B

Return period	Max. flow (m^3/s)	Max water level (m)	Max. speed (m/s)
2 years	10.0	0.9	1.9
25 years	20.4	1.2	2.4
100 years	25.5	1.4	2.5

As in the previous scenario, these results indicate that the water level does not exceed the capacity of the river, even for high return periods. This development therefore greatly reduces, if not completely, flooding in the critical downstream sector. As the cross-section is wider and has a different profile, a reduction in speed is observed. This reduction is not so significant and erosion protection and bank stabilisation measures should certainly be considered during the implementation of the project.

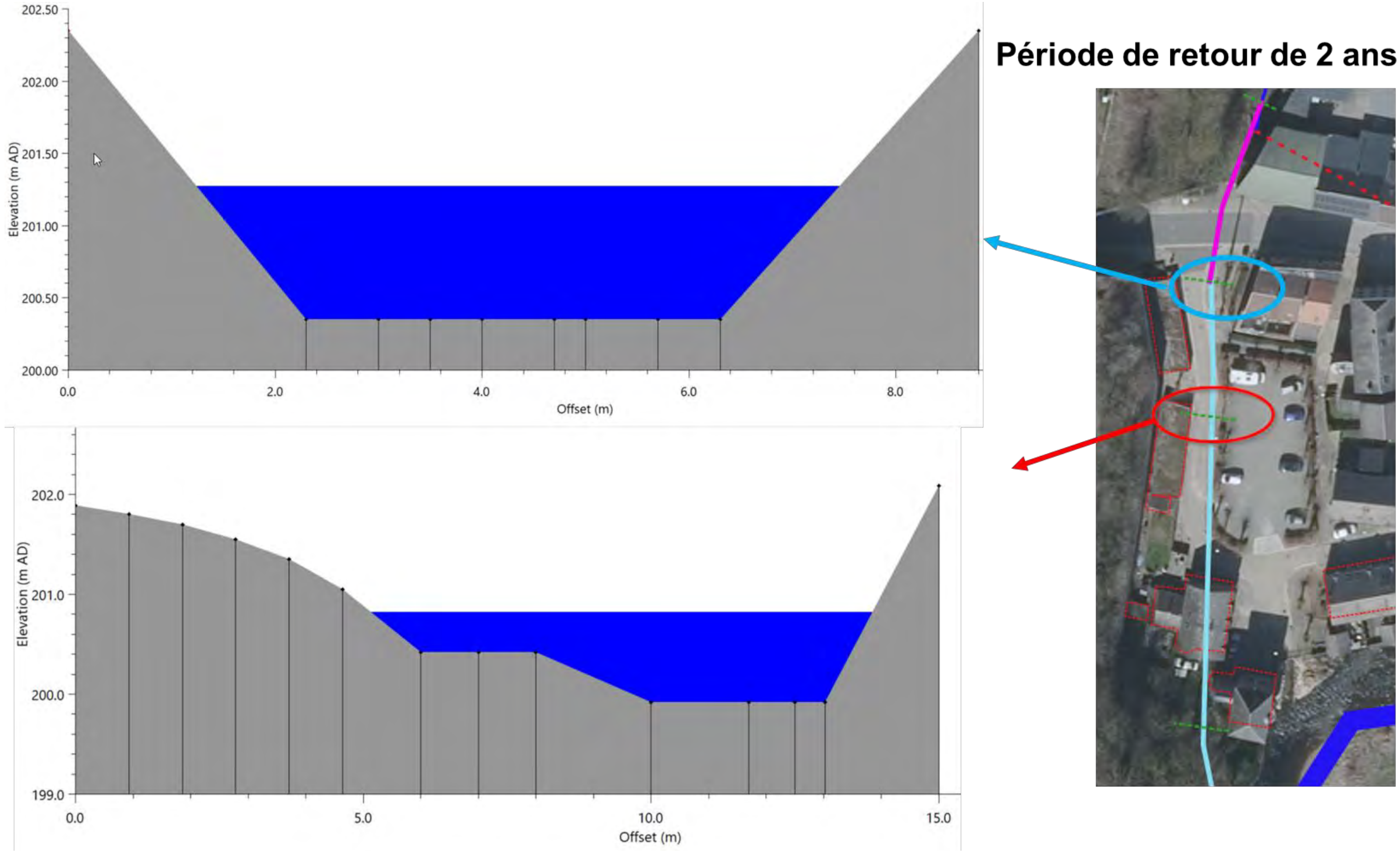


Figure 5.25: Water level in the cross-sections at the river opening for a 2-year return period for scenario 2B

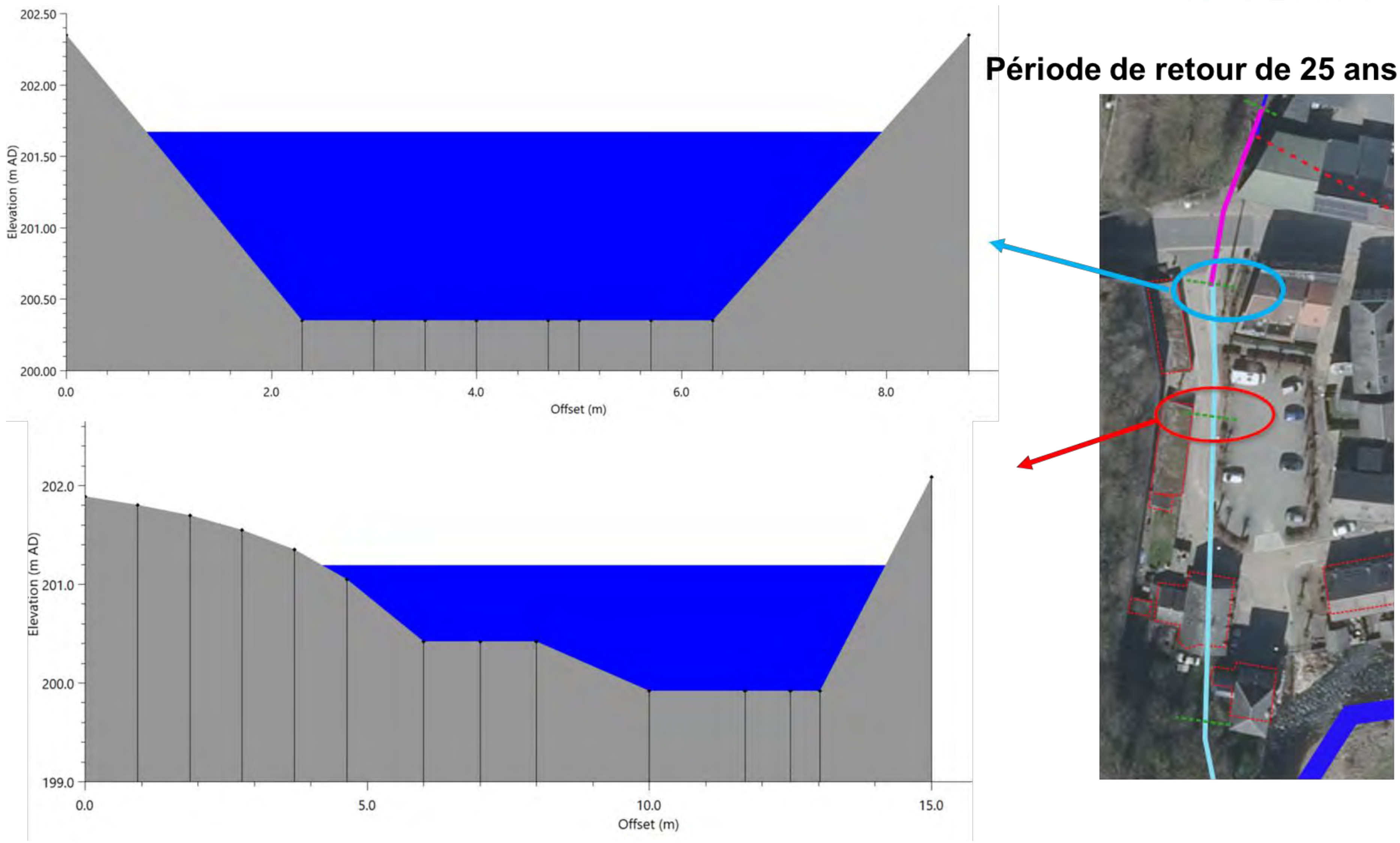


Figure 5.26 : Water level in the cross sections at the river opening for a 25-year return period for scenario 2B

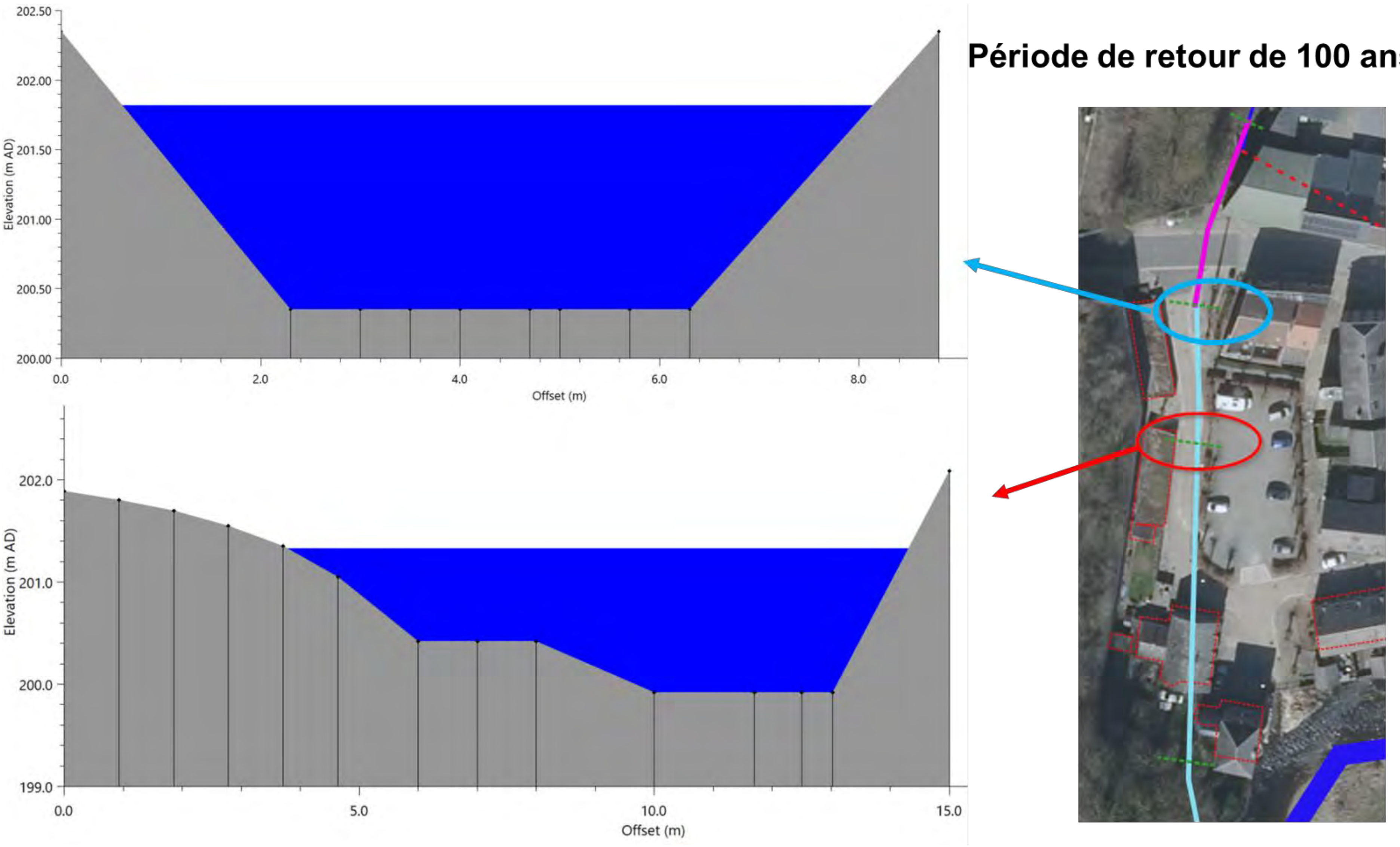


Figure 5.27 : Water level in the cross sections at the river opening for a 100-year return period for scenario 2B

Comparison of the future situation with the existing situation

The analysis of the future situation is based on the future scenario 2A for which the cross-sectional widths are the smallest in the last sector of the river (10m-width at the top of the cross-section).

Figure 5.28, 5.29 and 5.30 illustrate the longitudinal profiles of the future scenario for return periods of 2, 25 and 100 years. These figures also compare the water line between the existing and the future situations upstream of Rue du Moulin en Ruyff. **The modelling results indicate a significant reduction of the maximum water line compared to the existing situation and an almost total reduction of the overflow of the watercourse for extreme return periods. For the future scenario 2A, there is no more simulated overflow for a return period of 100 years.**

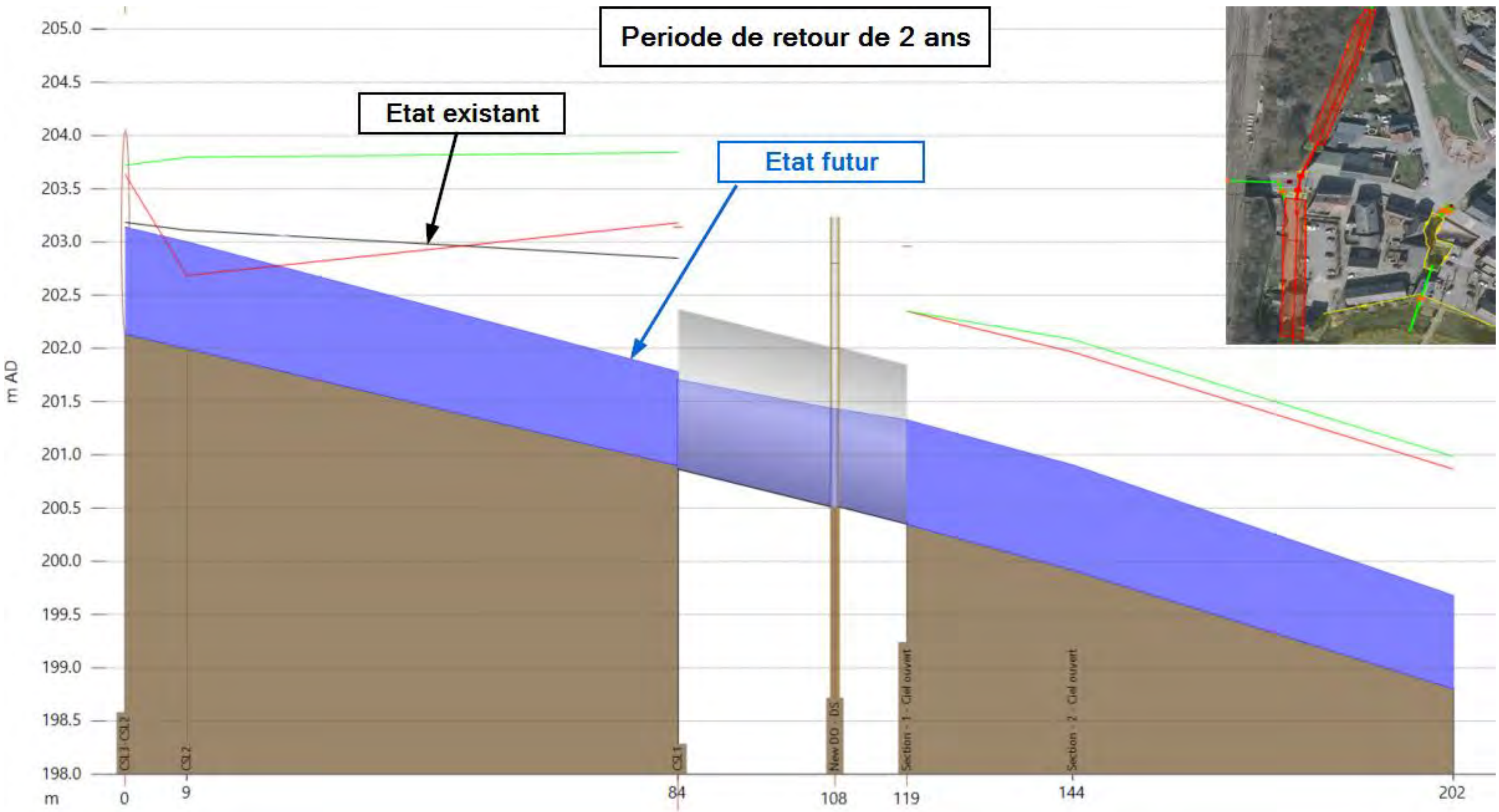


Figure 5.28: Longitudinal profile of the future situation (in blue) for the scenario of the opening of the watercourse for a 2-year return period. A comparison with the existing one is shown (in black) for the forebay

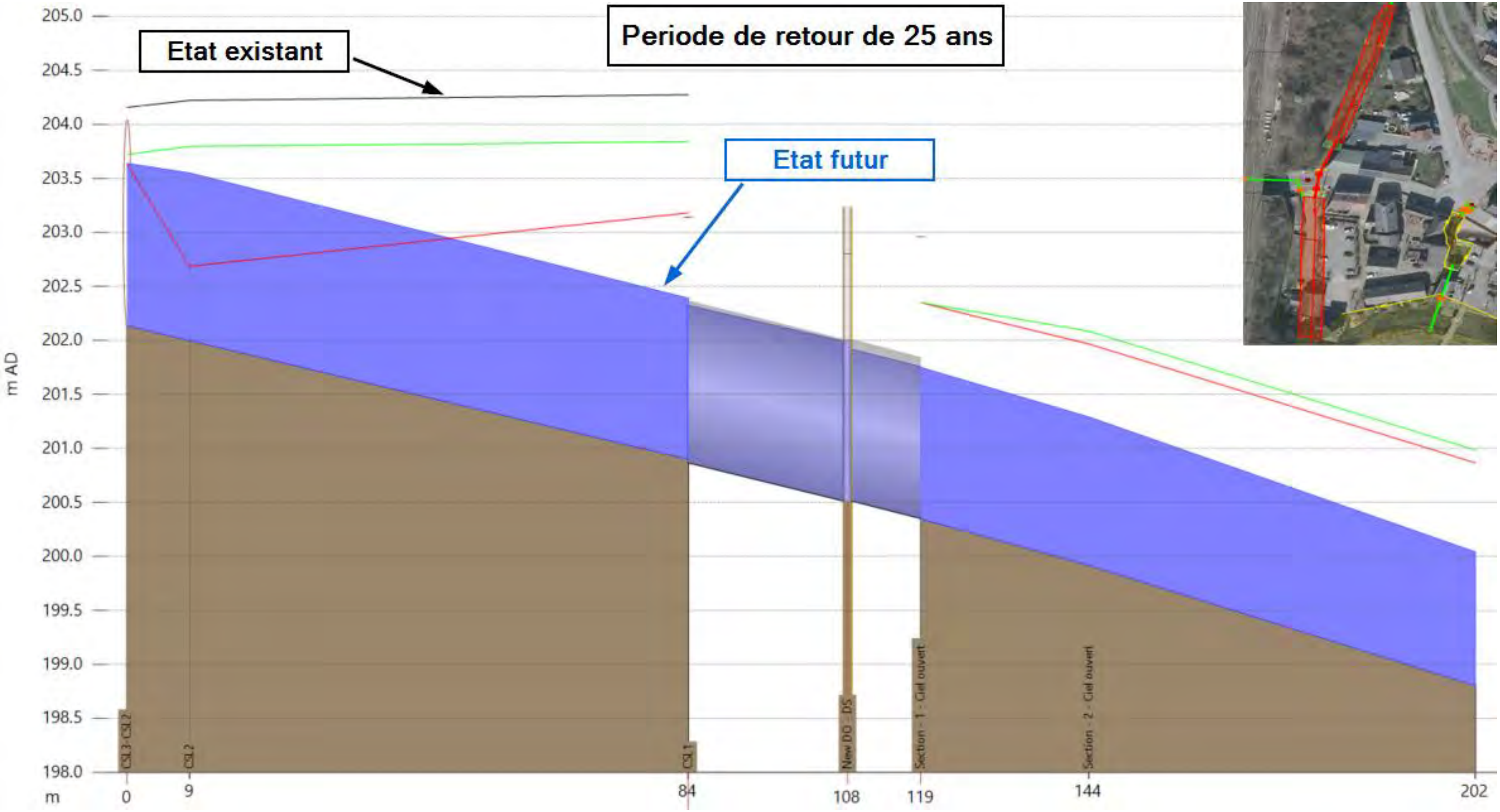


Figure 5.29; Longitudinal profile of the future situation (in blue) for the scenario of the opening of the watercourse for a return period of 25 years. A comparison with the existing one is shown (in black) for the forebay

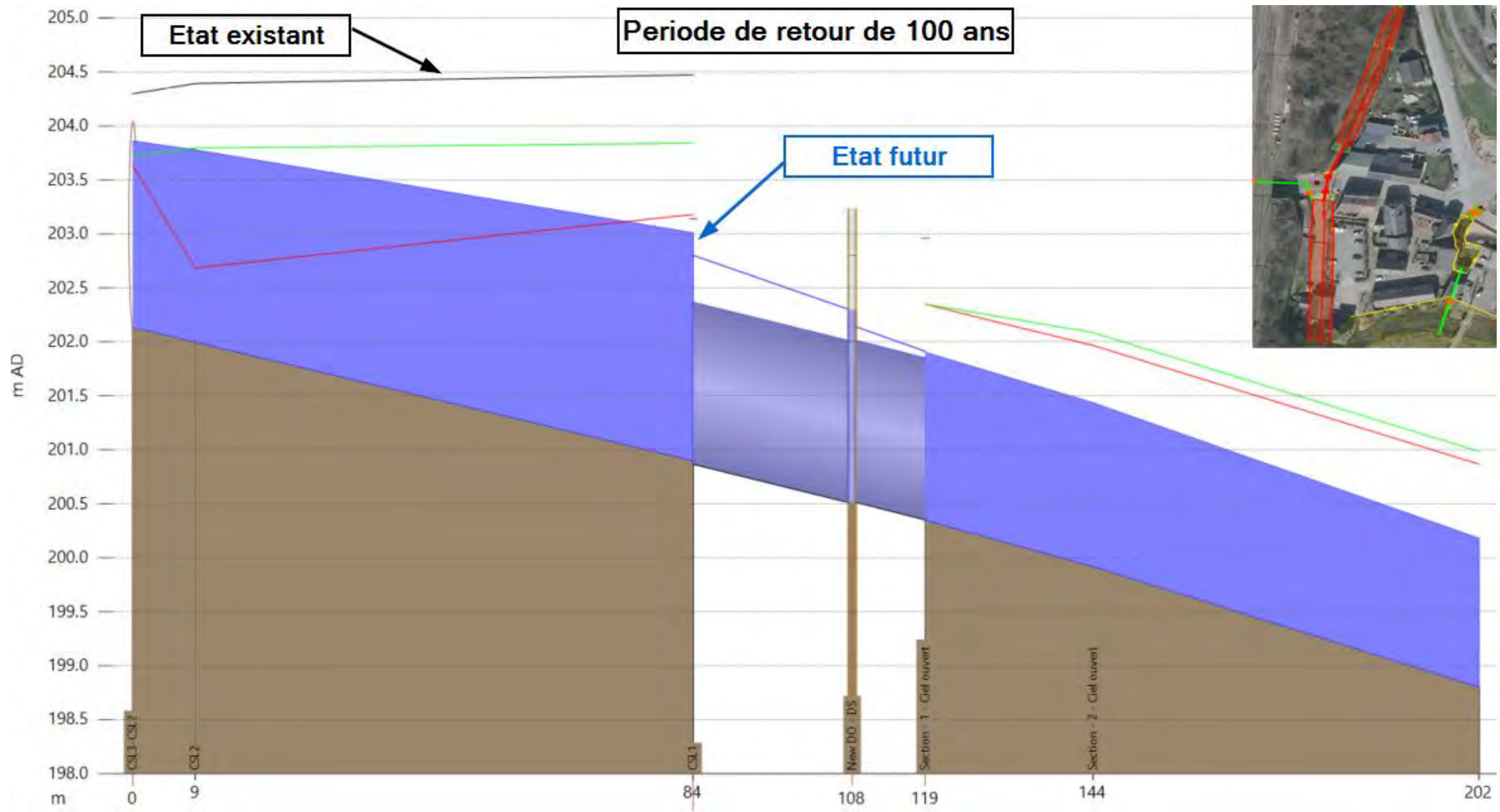


Figure 5.30: Longitudinal profile of the future situation (in blue) for the scenario of the opening of the watercourse for a return period of 100 years. A comparison with the existing one is shown (in black) for the forebay

Effect of the Vesdre

As for the existing situation, an analysis of the influence of the Vesdre on the opening of the Ruyff is carried out (always by injecting a flood flow of 90 m³/s upstream of the Vesdre reach).

Figure 5.31 shows the longitudinal profile from the future opening under the Rue du Moulin en Ruyff to the new confluence with the Vesdre (via the open-cut section) for simulation with a rainfall of 25 years return period. A difference in water level in the river downstream of the open section is observed, but there is no difference in water level upstream of this section. The local increase in water level downstream due to the Vesdre does not imply any overflow of the river and therefore no additional flooding. The same results are observed for a return period of 100 years. **The limited effect of the Vesdre on the flow on the downstream part of the Rhuyff in the future situation seems logical given the slopes involved**

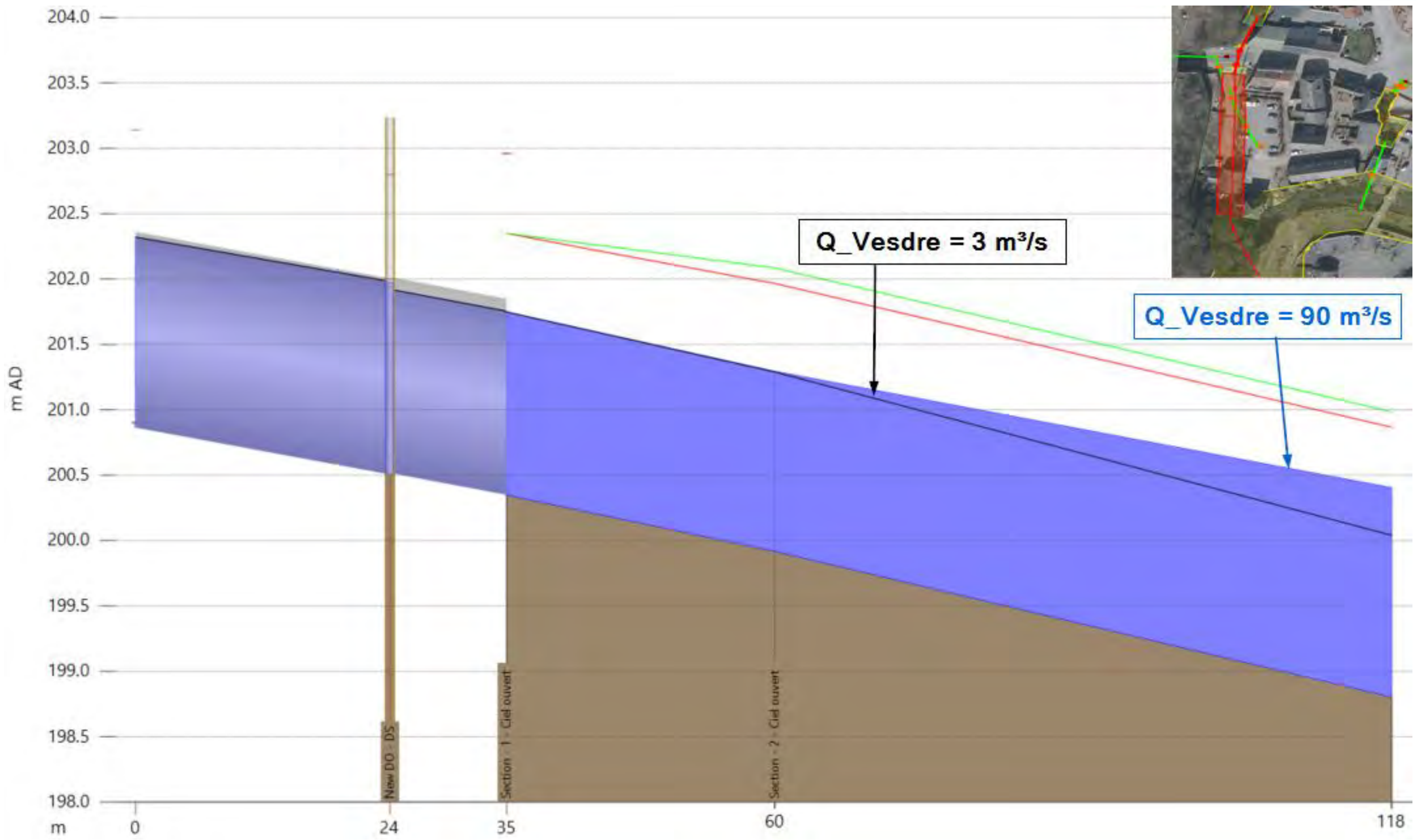


Figure 5.31: Comparison of the longitudinal profile of the open-cut section up to the Vesdre for a return period of 25 years for a flow of the Vesdre of 3 m³/s (in black) and 90 m³/s (in blue)

5.3 Conclusion on the effect of the development on flooding

A brief analysis of the number of houses affected by flooding according to the scenario and return period was carried out. A house is considered to be affected by flooding if the model indicates that the water level around the house is higher than an established threshold set in this study at 30 cm. The houses analysed are defined on the basis of the addresses available in the PICC.

Table 5.4 below shows the number of houses affected by flooding according to the return period for the three scenarios. These results are also illustrated in Figure 5.32.

Table 5.4: Number of houses affected by flooding by scenario and return period

	T2	T5	T10	T25	T50	T100
Etat existant	0	5	26	49	59	74
Scénario avec bassin d'orage	0	0	0	0	27	50
Scénario ouverture du cours d'eau	0	0	0	0	0	0

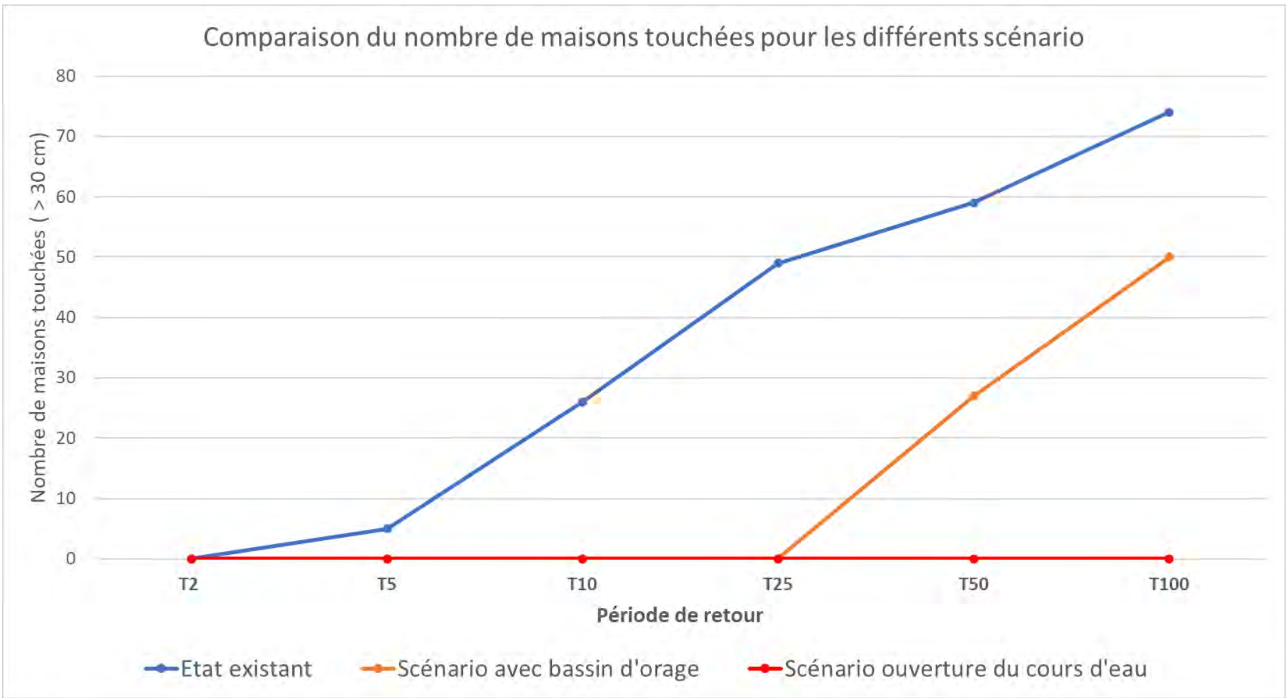


Figure 5.32: Comparison of the number of houses affected by flooding for the 3 scenarios.

The results presented above indicate that the future developments significantly reduce the number of houses affected by flooding. The scenario with the stormwater basin protects the critical area for a 25-year return period, as presented above. Nevertheless, the number of houses affected for the 50 and 100 years return periods remains high, despite a significant reduction compared to the existing situation. The scenario of opening the sluice fully protects the critical area and no houses are affected by flooding even for an extreme return period of 100 years. The latter scenario seems to be the most sustainable and effective flood protection solution.

6 Conclusions

The specific objectives of the study were multiple. Firstly, we built and validated a model of the Ruyff river with a detailed modelling of the hydraulic functioning on the downstream part including a potential implantation zone of a future temporary immersion zone and the critical zone of the Vieux-Moulin district located near the confluence with the Vesdre. In a second step, we studied development scenarios to reduce the risk of flooding in the critical sector downstream of the study area.

The hydraulic diagnosis of the existing situation shows the particularly critical character of the structures OA02 (opening made up of a rectangular upstream section and a downstream section made up of 4 rectangular openings in parallel) and OA2bis (circular pipe) passing under the rue du Moulin-en-Rhuyff. These 2 structures have a cumulative admissible capacity of 10 m³/s which is already reached for a return period of 2 years. The analysis of the model shows that they are the cause of the bottleneck in this area and of the overflows of the river in the sector. This is linked to their dimensions but also to the unfavourable configuration from a hydraulic point of view (bad alignment between the watercourse and the entrance to the structures, unfavourable junction within the opening with a 90° angle and changes in dimensions).

Several scenarios were considered to reduce flooding in the downstream critical area. Some scenarios had to be discarded due to the difficulty of implementation or their lack of effectiveness and impact on downstream flood reduction. The two most relevant scenarios were retained and studied in order to analyse downstream flood reduction.

A first development scenario consisting of the installation of a temporary immersion zone upstream of the critical sector of the Vieux-Moulin was evaluated to reduce the risk of flooding (under the assumption that there is no modification of the capacity of the limiting downstream structure). A dimensioning of the regulation structures was carried out on the basis of the critical downstream flow evaluated at 10 m³/s, the hydrographs to be buffered and a protection objective of 25 years. The volume to be buffered in the future area is estimated at around 25,000 m³, which requires the construction of a protective dam of around 50 m covering the minor and major river beds at a height of around 5 to 6 m above the level of the current river, in order to be able to find the volume available to buffer the 25-year return period flood without modifying the current natural terrain. The construction of such a structure significantly reduces the risk of flooding since the frequency of overflow is greatly reduced compared to the existing situation. The return period of overflow is thus reduced from 5 years in the existing situation to more than 25 years in the future situation. The TIA does not allow the neighbourhood to be protected for more significant floods and overflows are simulated for return periods of 50 years and 100 years, which is consistent with the protection objective for the implementation of the protection structure. The installation of the storm water basin seems to be effective in reducing the risk of flooding, but this must be weighed against the investment required to implement the protective dyke and the regulation structures on the Ruyff. There remains a residual risk of flooding for floods with return periods greater than 25 years.

A second development scenario consists of opening up the Ruyff and passing it in a straight line at the critical Vieux-Moulin sector. A dimensioning of the opening and the cross-sections on the section where the watercourse would be opened up was carried out. **The construction of such a facility would significantly reduce, if not completely eliminate, the risk of flooding for extreme return periods of up to 100 years. This scenario proves to be the most sustainable solution from a flooding point of view, given that the protection objective is higher than that of the storm water basin presented above**. In addition, this solution offers other advantages: easier maintenance due to easier access to the watercourse, added value in terms of biodiversity, etc. Despite the existing operational (impediment, adequate management of flow velocities, etc.) and financial constraints, the implementation of the opening of the watercourse is therefore the most effective solution for the reduction of the flood risk in the sector.

7 Annexes

7.1 Topographic survey work sheets

The structure sheets relating to the topographic surveys carried out are included in a separate file attached to this report.

7.2 Photos and videos of flooding in the Vieux-Moulin area

Historical elements for the validation of the model (photos and videos) collected from the residents of the Vieux-Moulin neighbourhood are included in a separate file attached to this report. Some of the elements particularly relevant to the validation are listed below.

Figure 7.1: Photograph from June 2016 - 3 rue du Vicinal

Figure 7.2: Photograph upstream of the ditch passing under the Rue du Moulin en Rhuyff taken in June 2016

Figure 7.3: Photograph taken in July 2021 - 70 rue du Moulin en Rhuyff

Figure 7.4: Photograph taken in July 2021 - 70 rue du Moulin en Rhuyff

7.3 Urban and rural sub-basin parameters

Table 7.1: Value of NC and time of concentration for each sub-basin

ID Sous bassins	CN	Tc (min)
1	60.38	24.5
2	63.34	11.6
3	56.24	4.7
4	62.89	110.7
5	61.8	72.9
6	62.89	7.3
7	57.99	49.7
9	62.77	89.9
10	58.56	22.1
11	69.41	35.5
12	60.15	31.5
13	68.26	46.6

Table 7.2: Main characteristics of the urban sub-basins.

ID Sous-bassin	Superficie (ha)	surfaces imperméables (rues, toitures)	prairies et champs	forêts	surfaces en eau	terre battues, chemins
URB01	11.9	35.1%	51.2%	12.2%	0.6%	0.9%
URB02	2.4	23.8%	37.4%	20.6%	15.1%	3.0%
URB03	36.6	48.9%	43.5%	6.0%	0.2%	1.4%
URB04	9.1	46.3%	43.4%	5.7%	0.0%	4.6%
URB05	4.6	40.4%	52.2%	6.0%	0.6%	0.8%
URB06	3.1	42.1%	47.7%	7.6%	0.3%	2.3%
URB07	1.0	54.6%	39.5%	4.8%	0.3%	0.8%
URB08	47.1	65.9%	24.9%	8.7%	0.1%	0.3%
URB09	21.1	39.9%	52.6%	6.2%	0.1%	1.2%
URB10	36.1	46.7%	46.0%	5.5%	0.2%	1.7%
URB11	5.3	32.6%	55.5%	8.2%	0.5%	3.2%
URB12	3.3	23.9%	67.7%	6.0%	0.5%	1.8%
URB13	21.9	49.6%	38.7%	7.7%	0.2%	3.8%
URB14	5.1	67.4%	21.8%	6.2%	0.0%	4.6%
URB15	57.3	61.2%	28.2%	9.4%	0.5%	0.6%
URB16	1.7	39.6%	51.4%	8.2%	0.0%	0.7%
URB17	3.5	42.7%	48.2%	7.3%	0.0%	1.8%
URB18	2.0	33.2%	60.5%	5.3%	0.0%	1.0%
URB19	8.4	31.9%	52.3%	13.3%	0.1%	2.4%
URB20	4.5	4.1%	79.7%	13.8%	0.0%	2.4%
URB21	12.2	31.0%	57.6%	8.6%	0.6%	2.2%
URB22	19.0	35.8%	49.8%	10.1%	0.2%	4.1%
URB23	5.1	45.1%	42.6%	10.1%	0.0%	2.1%
URB24	3.5	75.3%	15.0%	5.7%	2.8%	1.2%
URB25	0.9	73.4%	12.6%	11.0%	3.0%	0.0%
URB26	16.3	34.7%	53.1%	9.7%	0.1%	2.4%
URB09_2	0.015	100.0%	0.0%	0.0%	0.0%	0.0%

7.4 Flood maps for the existing situation

The flood maps for the 5-year, 10-year, 25-year, 50-year and 100-year return periods are included in a file attached to this report. These maps are based on the results of simulations without downstream effects of the Vesdre

7.5 Flood maps for the future situation - stormwater scenario

The flood maps for the 5-year, 10-year, 25-year, 50-year and 100-year return periods are included in a file attached to this report. These maps are based on the results of simulations without downstream effects of the Vesdre.

7.6 Analysis of the new storm overflow

A conceptual model of a new storm overflow is presented below. The working assumptions are as follows and are illustrated in Figure 7.5:

- A 300 mm diameter flow reducer
- A storm overflow connected to the Ruyff with a sill of about 30 cm above the base of the existing chamber. The water thus overflows to the Ruyff when the 300 pipe is under pressure.

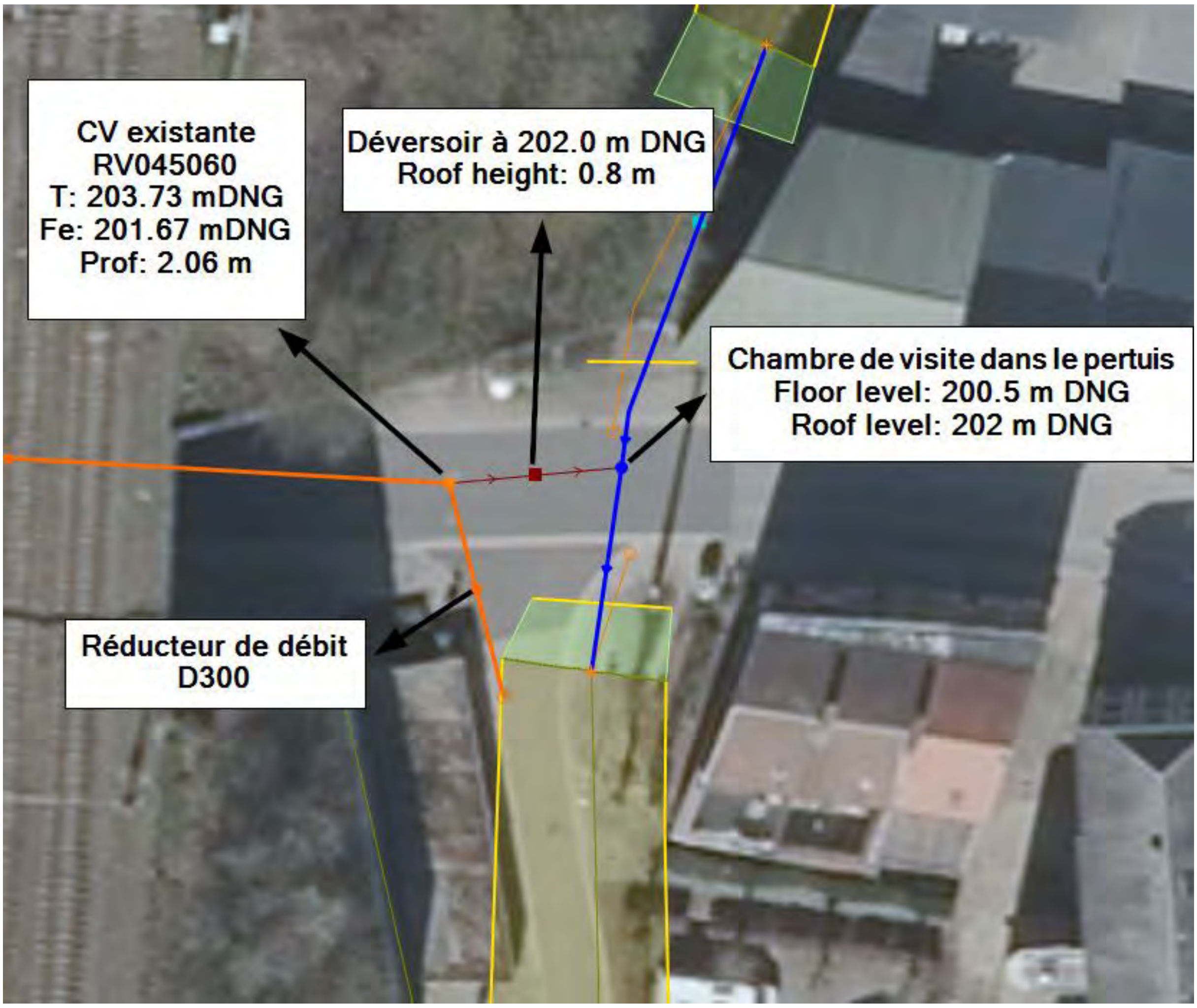
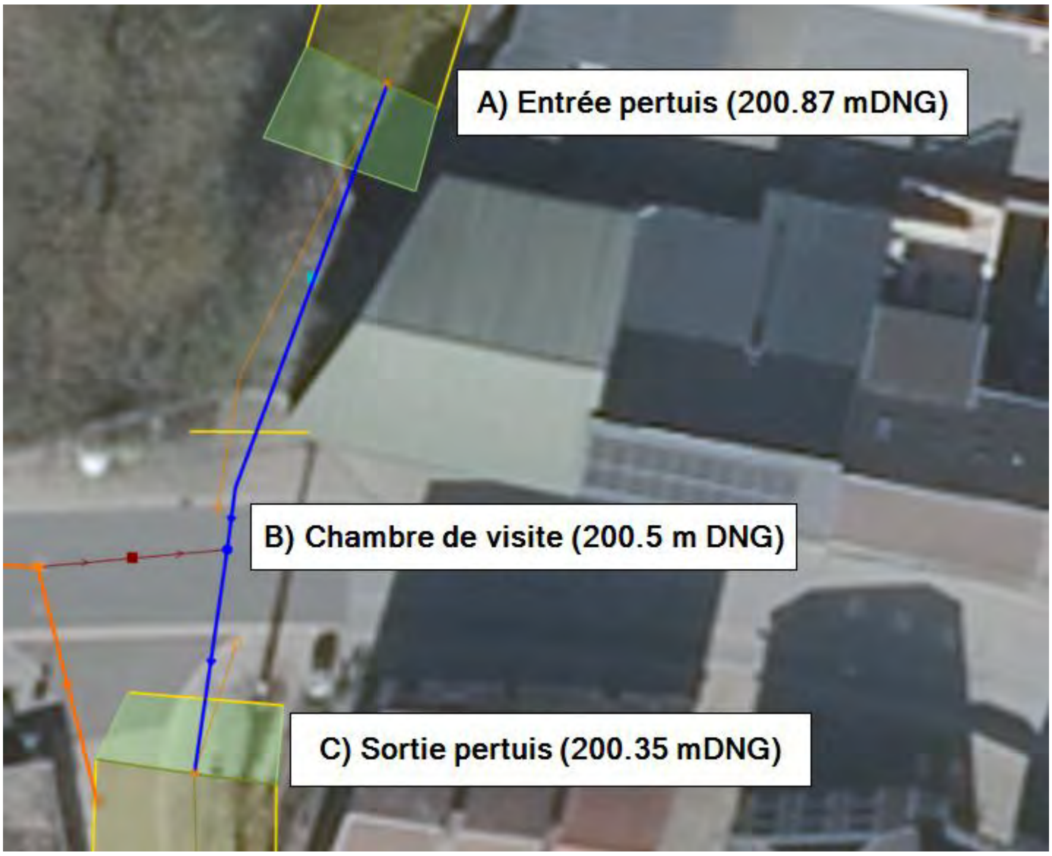


Figure 7.5: Conceptual modelling of the storm overflow

The simulation results are presented in the table below. They indicate that there is no interaction between the Ruyff and the storm overflow for return periods of 5, 10 and 25 years.

Table 7.3: Water level in the future floodgate according to the return period

	Niveau d'eau du pertuis sous la rue du Moulin en Ruyff			
	A) Entrée pertuis (m DNG)	B) Chambre de visite (m DNG)	C) Sortie pertuis (m DNG)	Hauteur d'eau (m)
T5	201.9	201.6	201.5	1.1
T10	202.1	201.8	201.6	1.2
T25	202.4	202	201.8	1.5



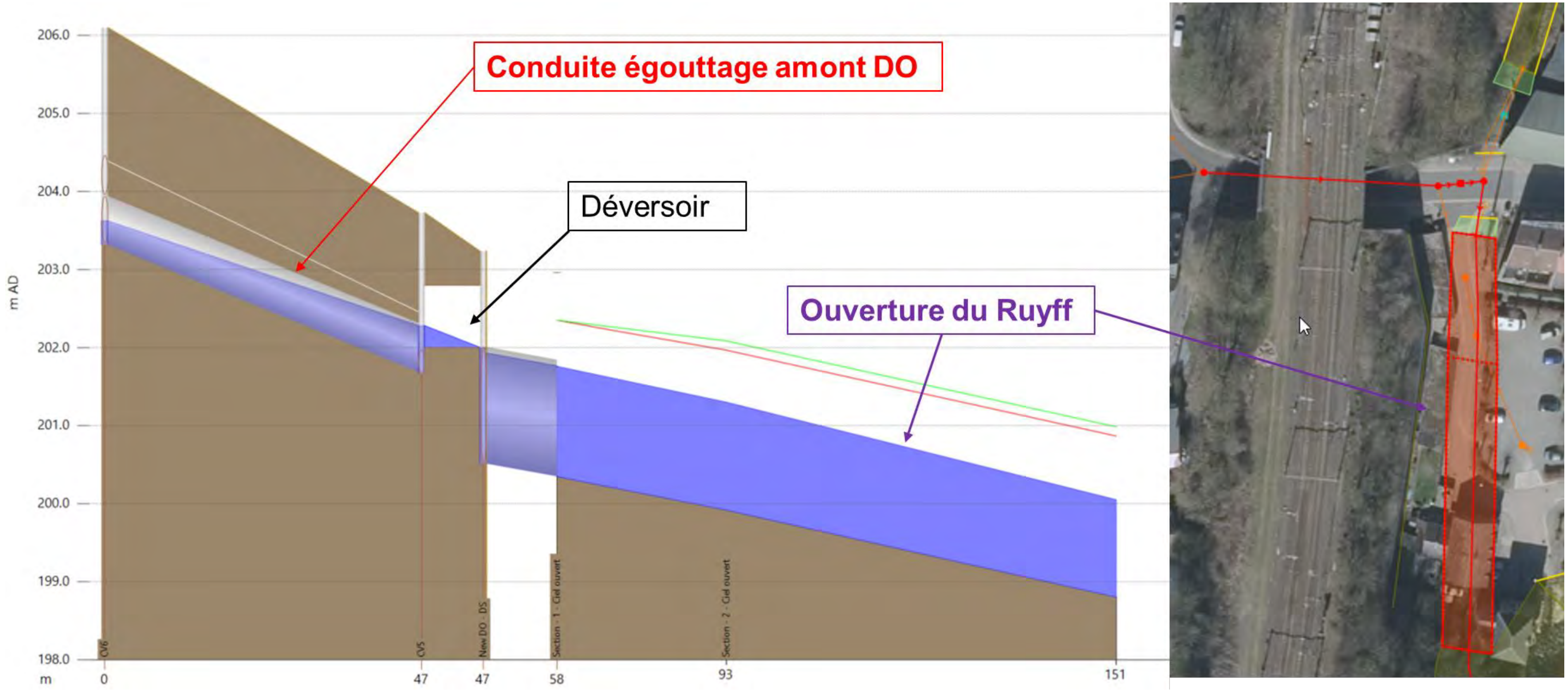


Figure 7.6: Longitudinal profile of the storm overflow