Integrated Management of Flood Risks and land Drainage Master Plan (LDMP), Mauritius

D4.1 - Feasibility Assessment of Risk Reduction Solutions

2022.05.09

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

ANNEXES

EXECUTIVE SUMMARY

Agence Française de Développement (AFD) launched Adapt'Action to support countries seeking technical assistance for the institutional, methodological and operational implementation of their commitments in relation with climate change.

Adapt'Action Facility ultimately aims at giving countries the technical and institutional tools they need to strengthen climate governance and mobilise international climate finance (such as the Green Climate Fund - GCF) and bi/multilateral donors (such as AFD) to scale up their action and therefore produce a leverage effect.

More specifically in Mauritius, AFD is engaged with the Republic of Mauritius (ROM) to support the implementation of its Nationally Determined Contribution (NDC), with a particular focus on climate change adaptation. To this effect, AFD and the ROM signed a memorandum of understanding on the 12th of December 2017, during the One Planet Summit, organised by the French Presidency, which sets the respective engagements under the Adapt'action initiative.

The main objective of the assignment is the elaboration of the **Integrated Land Drainage Master Plan** as a basis to the strategy, to be implemented by the Authorities and all stakeholders **to reduce vulnerability of the population and various activities to heavy rain and flood events, in the context of climate change and uncertainty.**

The Master Plan will include:

- An inventory and mapping of all the existing natural and manmade drainage infrastructures;
- An identification of vulnerable areas, including the impact of future developments on potential flood prone areas;
- The definition of a reference hydrology at the scale of each rainfall sub-catchment, based on new IDF curves,
- The elaboration of flood mapping and associated vulnerability assessment;
- The proposal for national rules in order to account for land drainage issues in territorial development with the objective of flood risk reduction, taking into account water quality and biodiversity preservation to improve the resilience of the country in the context of climate change.
- The definition of broad protection objectives at the scale of the most vulnerable catchments, based on detailed studies conducted at a more localised level;
- And finally, an action plan on the short and middle term.

1 INTRODUCTION

The SUEZ-MEGA DESIGN-ACTERRA-SCENE-RIES-DAY MARINE group was selected to conduct this study. The mission started contractually in January 2020 and was scheduled to last 15 months, until the end of March 2021. Due to various delays, the most significant being the lockdown due to Covid19 both in Mauritius and in France the Workplan had to be revised with completion of the study rescheduled for June 2020.

The formal start of the study was held on January 16, 2020 in Mauritius during an inception workshop bringing together a large number of stakeholders over an entire day.

This document constitutes deliverable **D4 – Feasibility Assessment report:**

- D4.1: for the priority sites
- D4.2: for the other

In this activity, the deliverables include the following items:

- List and justification of solutions studied
- D4b Summary description of the solution with location plan, sizing, cost benefit and multicriteria analysis
- D4c Impacts and Results assessment due to flood risks
- D4d – Raw data (Soft Annex)

List of Acronyms, Conventions and Abbreviations

AFD	Agence Française de Développement
CC	Climate Change
CCVA	Climate Change Vulnerability Assessment
CGDD	Commission Génerale pour le Developpement Durable
CORDEX	Coordinated Regional Climate Downscaling Experiment
C1	Component 1
C2	Component 2
C3	Component 3
CCVRA	Climate Change Vulnerability and Risk Assessments
DEM	Digital Elevation Model
DRR	Disaster Risk Reduction
DTM	Digital Terrain Model
D1	Deliverable 1
EIA	Environmental Impact Assessment
ER2C	Enhancing Resilience to Climate Change
GHG	GreenHouse Gas
GIS	Geographic Information System
ICZM	Integrated Coastal Zone Management
IDF	Intensity Duration Frequency
IPCC	Intergovernmental Panel on Climate Change
IOC	Indian Ocean Commission
LDMP	Land Drainage Master Plan
LIDAR	Light Detection And Ranging
MoH & LUP	Ministry of Housing and Land Use Planning
MMS	Mauritius Meteorological Services
MOI	Mauritius Oceanographic Institute
NDC	Nationally Determined Contribution
NDRRMC	National Disaster Risk Reduction and Management Centre
NDU	National Development Unit
NHDC	National Housing Development Company
O&M	Operation and Maintenance
ROM	Republic of Mauritius
PDS	Property Development Scheme
RP	Return Period
SCS	Soil Conservation Service
SLR	Sea Level Rise
TA	Technical Assistance
VAT	Value Added Tax
VRA	Vulnerability Risk Assessment
WMA	Wastewater Management Authority
WRCP	World Climate Research Programme
WRU	Water Resources Unit
RDA	Road Development Authority
ToR	Terms of Reference
USDA	United States Department of Agriculture
VRS	Voluntary Retirement Scheme
1D/2D	1 Dimensional / 2 Dimensional

2 OBJECTIVES AND GENERAL METHODOLOGY

The overall objectives of this phase of the study are to assess the feasibility of risk mitigation solutions by the conception, design, modelling and impact assessment of proposed hydraulic infrastructure, both structural and non-structural solutions for stormwater and flood risk management, including the effects of anticipated climate change.

2.1 Specific objectives

The specific objectives of Activity 4 are:

- to draw up design concepts (soft or hard) in order to guarantee a homogeneous and coherent level of protection at the scale of the catchment areas concerned.
- to propose works, hydraulic pre-dimensioning incorporating local constraints, as well as estimated costs
- to undertake an economic analysis thereon.

As discussed in the first part of the Land Drainage Masterplan, **Sustainable solutions** no longer seek the quickest way of channelling stormwater into a river or watercourse as these require a large infrastructure with the associated capital outlay and space requirement. They consist nowadays in finding ways and means of **controlling peak flows at source** and breaking the peak flows as much as possible through retardation basins, flood infiltration or flood expansion zones along or off watercourses, terracing, vegetable cover and the like prior to releasing them in a controlled manner into the drainage infrastructure. It is equivalent to releasing the same quantity of water but over a longer period of time.

In addition to the soft actions proposed above, the identification of the most appropriate alignments for the construction of the main primary drainage networks and cut off drains and their sizing are also addressed.

Finally, a **maintenance scheme for the main drainage system** is proposed in order to ensure its continuous efficiency, especially during flood events.

2.2 General methodology

The General Methodology revolves around the following stages:

- Detailed field investigations and diagnosis of the existing situation. This diagnosis is also based on the results of flood modelling in the current situation as established in Activity 3
    - Identification of solutions focusing on structural measures:
        - Storage: At source retention and delayed or partial evacuation
        - Slow down measures to reduce flow velocities and peak flow;
        - Cut off drain: Discharge into other works;
        - Total disposal – increased capacity of the infrastructure;
    - Siting and preliminary layout
- Sizing and optimisation using hydraulic modelling in designed conditions;
- Evaluation of the implementation costs;
- Evaluation of the impacts of the solutions (reduction of flood levels)
    - Comparative mapping before and after work implementation
    - Cost-benefit analysis

2.2.1 Detailed field investigations for diagnosis and solutions

2.2.1.1 Diagnosis of the existing situation

The preliminary step followed by the site teams is to complete the investigations initiated in the previous activities on the 5 priority + 11 complementary sites and comprising of:

- Visually inspecting and describing the areas prone to flooding
- Making an appreciation of the hydraulic functioning during flood events, in particular the drains, streams and rivers, the bridge crossings, as well as the areas without any drainage system but located within the natural runoff axes
- Completing the understanding of the hydraulic functioning through interviews with the residents

It is worth noting that following the flash floods of April 2021, particularly in the south-east and the central regions, localised investigations had been carried out, providing updated information on a number of sites (such as Nouvelle France and Henrietta Malakoff).

For each of these sites, the findings are summarised in the following chapters. Identification of solutions with a focus on structural measures

- **Location and preliminary setting out**

Following field investigations, the potential solutions to alleviate flooding in the main areas at stake are proposed on a GIS map.

The solutions are based on:

- Field observations coupled with real life experience from local residents and
- Topography, but also
- Covering sectors on which hydraulic studies are already in progress, the aim being to objectively review these studies for their feasibility, effectiveness, etc.

The following table indicates the study reports reviewed under previous activities for the sectors under study.

Table 1: Hydraulic studies reviewed for the 5 priority sites

Sector id	Sector name	Ref. hydraulic studies	Contractor
PS – Port Louis	Le Pouce stream, rivière des Créoles and Southwest Signal Mountain cut-off drain	ER2C - Assessment report on current situation for priority sites Short term priority measures Component 2 - Deliverables 3 and 4 – June 2019 (Tor)	DAI / SETEC / ACOA / Kairos consult / IREED
PS – Port Louis	Southwest Signal Mountain cut-off drain	ER2C - Assessment report on current situation for priority sites Short term priority measures Component 2 - Deliverables 3 and 4 – June 2019 (Tor)	DAI / SETEC / ACOA / Kairos consult / IREED
PS -Latanier	Rivière Lataniers at Cité La Cure	ER2C - Assessment report on current situation for priority sites Short term priority measures Component 2 - Deliverables 3 and 4 – June 2019 (Tor)	DAI / SETEC / ACOA / Kairos consult / IREED
PS – Canal Dayot	Saint Louis stream at Canal Dayot	ER2C - Assessment report on current situation for priority sites Short term priority measures Component 2 - Deliverables 3 and 4 – June 2019 (Tor)	DAI / SETEC / ACOA / Kairos consult / IREED
PS - Nouvelle France	Nouvelle France	ER2C - Assessment report on current situation for priority sites Short term priority measures Component 2 - Deliverables 3 and 4 – June 2019 (Tor)	DAI / SETEC / ACOA / Kairos consult / IREED
PS – Bel Ombre	Bel Ombre	ER2C - Assessment report on current situation for priority sites Short term priority measures Component 2 - Deliverables 3 and 4 – June 2019 (Tor)	DAI / SETEC / ACOA / Kairos consult / IREED
PS - Grand Baie Perebeyre	Grand Baie and Pereybere	*ER2C - Assessment report on current situation for priority sites* *Short term priority measures* *Component 2 - Deliverables 3 and 4 – June 2019 (Tor)* *No solution studied*	DAI / SETEC / ACOA / Kairos consult / IREED
PS - Flic en Flac	Flic en Flac	*ER2C - Assessment report on current situation for priority sites* *Short term priority measures* *Component 2 - Deliverables 3 and 4 – June 2019 (Tor)* *No solution studied*	DAI / SETEC / ACOA / Kairos consult / IREED

- **Sizing and its optimisation using hydraulic modelling in designed conditions.**

Once the location and characteristics of the works have been defined by the field teams, an initial sizing is carried out by the modelling teams in order to confirm and refine the sizing of the proposed works, in particular the volume of the detention basins and the acceptable controlled discharge onto the area downstream. The objective of the design is to find the best efficiency within the feasibility limits of the structures

2.2.2 Methodology implemented in the framework of design calculation

2.2.2.1 Basis for hydraulic modelling

In order to be able to assess runoff in drains, but also runoff on slopes and streets, as well as overflows from watercourses, it is necessary to know the flow regime in advance.

There are two types of hydraulic flow regimes:

1. The uniform flow regime

For the uniform flow regime, the flow characteristics (velocity, height, flow rate) are independent of time and position.

In the case of uniform free surface flow, the slope of the water line, as well as the slope of the energy line (J) are strictly parallel to that of the bottom (I). The pressure distribution is hydrostatic, all liquid threads are assumed to be parallel to each other and to the bottom.

The equality between the slope of the bottom and the slope of the headline (J=I) means that the relations between the different hydraulic quantities (water height, average speed and flow) are univocal. For example, the Manning Strickler can be used to calculate the velocity for a particular value of water head. This calculation is only done for steady state (i.e. constant flow only).

Uniform flows are only found in water supply and irrigation channels. They are exceptional in drainage systems, especially during rainy periods when temporal variations in flow are added to spatial variations in the network structure. However, this type of flow is the only one taken into account for the design of drains in the absence of hydraulic modelling, e.g. by using Manning's formula. **The hydraulic design on this basis then represents a purely theoretical operation compared to the reality of flood flows.**

2. The gradually varied flow regime

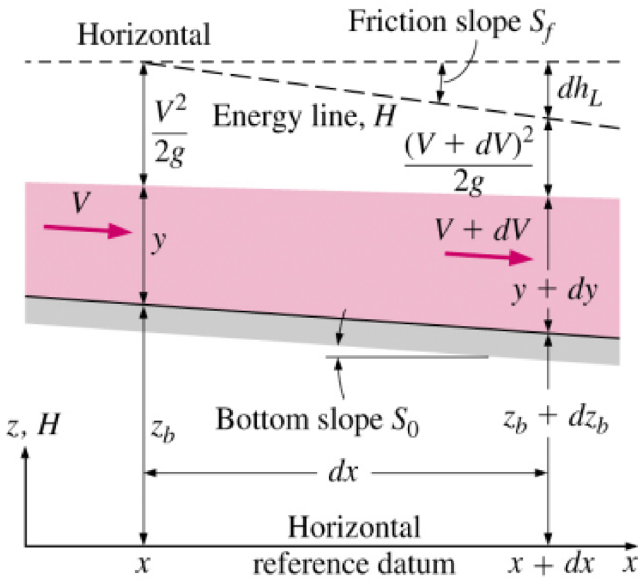
The gradually varied regime is the regime to consider flows in drainage systems or rivers in flood. It may correspond to:

- a permanent regime (Flow regime such that the different hydraulic quantities (height, speed and flow) are independent of time)
- or a transitory regime (Flow regime characterised by the fact that the different hydraulic quantities (head, velocity and flow) change with time).

Depending on the flow rate and the operating conditions (slope, roughness, flow energy), the water height (y) can be higher, lower or equal to the critical height (yc). We can thus distinguish between the following regimes

- fluvial if $y>y_c$ (Froude number lower than 1)
- torrential if $y<y_c$ (Froude number greater than 1)
- critical if $y=y_c$ (Froude number equal to 1)

The knowledge of the flow regime is important for the hydraulic simulation of the operation of drains and rivers in flood.



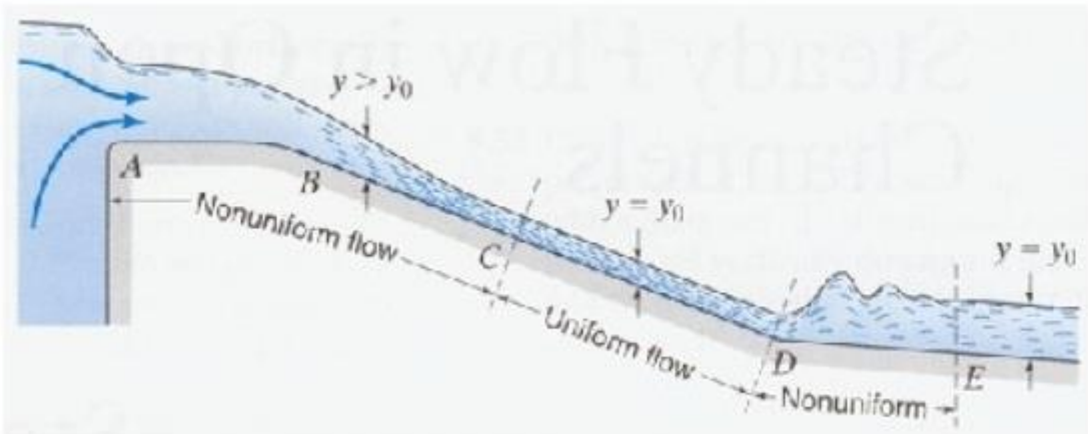


Figure 1 : Flow regime

3. Use of transitory hydraulic modelling

The hydraulic modelling study therefore consists of a diagnosis of the study area for any hydraulic event in order to:

- calculate the elevation of the water line and the energy line for gradually varied flows, in transient regime (by flood hydrograph)
- design drains and structures under varied flow regimes
- identify overflow areas.

The hydraulic analysis for the establishment of the flood areas, the search for solutions (design) and the mapping of the flood areas after the implementation of the drains, is therefore based on the hydraulic modelling by using the HEC-RAS software (HYDROLOGIC ENGINEERING CENTER, RIVER ANALYSIS SYSTEM), which is a computer software capable of modelling free surface flows in natural and artificial channels with the consideration of crossing structures in a gradually varied transitory regime: **The calculations of water lines in dynamic regime simulating the various obstacles along the drain are then more precise than calculations in uniform regime type simple application of the formula of Manning.**

2.2.2.2 Design parameters

For design purposes, the methodology used is based on the Hydrological method described in report D5.1 - Land drainage Master plan - First Part (see Chapter 4). Reference is made in particular to the IDFs, the determination of times of concentration, and the choice of runoff coefficients according to the rational method and SCS.

For the study areas covered by deliverables D4.1 and D4.2, the details of the hydrological calculations relating to the catchment areas and the definition of peak flows and flood hydrographs are detailed in deliverable D2.3. The main results are included in this document.

With regard to the analysis of carrying capacity and the sizing of measures, the design is based on :

- On the HEC RAS models implemented on each of the 5 + 11 sectors. The design and results for the main structures, in particular the watercourses and natural drains
- On additional capacity calculations for drains in urban areas, based on Manning's equations.

For diagnosis and design purposes, the following guide values are considered for drains.

Table 2: Mains Manning coefficient

Type of drains	Manning coefficient
Masonry drain	0.013 – 0.017
Earthen drain	0.05 – 0.035

In detail, the following table presents the ranges of Manning coefficients to be applied in the calculations according to the nature of the drains.

Table 3: Manning coefficient

Nature of the surfaces	Surface conditions			
	Perfect	Good	Fairly good	Poor
A) Artificial channels				
Smooth cement	0.01	0.011	0.012	0.013
Cement mortar	0.011	0.012	0.013	0.015
Planed wooden aqueducts	0.012	0.014	0.016	0.018
Unplaned wooden aqueducts	0.017	0.02	0.025	0.03
Concrete lined channels	0.025	0.03	0.033	0.035
Rough mortar	0.013	0.014	0.015	0.017
Dry stone	0.011	0.012	0.013	0.015
Upright moelons	0.0225	0.025	0.0275	0.03
Metal aqueducts with smooth semicircular section	0.017	0.02	0.0225	0.025
Metal aqueducts with pleated semicircular section	0.025	0.03	0.033	0.035
Straight and uniform earthen canals	0.035	0.04	0.045	-
Smooth and uniform stone channels	0.0225	0.025	0.0275	0.03
Rough and irregular stone channels	0.025	0.0275	0.03	0.033

Nature of the surfaces	Surface conditions			
	Perfect	Good	Fairly good	Poor
Earthen channels with wide meanders	0.028	0.03	0.033	0.035
B) Natural watercourses				
1) Clean, straight banks	0.025	0.0275	0.03	0.033
2) Same as 1 with some grass and stones	0.03	0.033	0.035	0.04
3) Meandering, with some ponds and shallow areas, clean	0.035	0.04	0.045	0.05
4) Same as 3, water at low water level, lower gradient and sections	0.04	0.045	0.05	0.055
5) Same as 3, with some grass and stones	0.033	0.035	0.04	0.045
6) Same as 4, with stones	0.045	0.05	0.055	0.06
7) Areas with slow flowing water with weeds or very deep pools	0.05	0.06	0.07	0.08
8) Areas with many weeds	0.075	0.1	0.125	0.15

Source - University of Louvain

For more details on roughness in flood plains, refer to the following document: Guide for Selecting Manning's - Roughness Coefficients for Natural Channels and Flood Plains - Paper 2339 (By GEORGE J. ARCEMENT, JR., and VERNE R. SCHNEIDER) - https://pubs.usgs.gov/wsp/2339/report.pdf

The hydraulic design of the drains is carried out in an iterative way in order to determine the efficiency of the drains while taking into consideration the available space and thus the feasibility of implementing the measures.

Thus, as these are curative structural measures for the existing situation, the aim is not only to determine the occurrence of protection, but also to design the measures allowing a global reduction of the flow depths for floods between 10 and 100 years return period.

The hydraulic capacities are given including the freeboard. Concerning the freeboard for the design, they are defined in accordance with the D5.1 guide:

Table 4: Minimum free board required

S.N	Infrastructure	Minimum Rainfall Return Period1 (years)	Minimum free-board required in m
1	Drains (urban area)	25	0.3 m
2	Discharge into watercourses (including Feeders, Rivulets, Rivers)	100	0.5 m
3	Culverts	50	0.5 m
4	Bridges	100	1.0 m **

***: by ensuring that the free board is greater than $V^2/2g$ (v = flow velocity under the bridge)*[2]

The Drawings standards are attached as an annex to deliverable D5.2

1 Higher return periods are recommended for regions with known vulnerabilities to flooding.

2 «Wasser Energie Luft» – 105. Jahrgang, 2013, Heft 2, CH-5401 Baden - https://www.swv.ch/fr/wp-content/uploads/sites/2/2018/03/Recommendation-sur-la-revanche_CIPC-2013-1.pdf

2.2.3 Cost estimates of proposed works

Once the sizing has been finalised, and on the basis of a unit cost breakdown of the works, each infrastructure work item or group of work items has been calculated in order to estimate the cost of the whole works based on the feasibility stage study. These cost estimates form the basis for the economic analysis during the next step: the cost-benefit analysis.

The following table summarises the main unit costs used in building the cost estimates for the works.

Table 5: Mains unit costs

Item no	Description	Unit	Rates (MUR) Range	
1	**Excavation**			
1.1	Bulk excavation in any material, including rock not exceeding 40%	m^3	300	350
1.2	Trench excavation in any material, including rock not exceeding 40%	m^3	500	600
2	**Concrete**			
2.1	Blinding C15 concrete	m^3	5,000	6,000
2.2	Structural C25 concrete	m^3	6,000	6,500
2.3	Structural C30 concrete	m^3	6,500	7,000
3	**Formwork**			
3.1	Formwork F1 Finish	m^2	600	650
3.2	Formwork F2 Finish	m^2	700	800
4	**Reinforcement**			
4.1	High tensile reinforcement, including cutting, bending and placing	kg	80	90
5	**Roadworks**			
5.1	Kerb K1 and K3	m	650	1,000
5.2	Drainage kerb with inlet (K2 + K4)	m	1,200	1,500
5.3	Grouted stone pitching	m^2	5,100	5,600
5.4	Spalls 200 - 300 mm	m^3	1,500	2,000
5.5	Road base crusher run (150 mm thick)	m^3	2,400	2,700
5.6	Prime coat (0.6 L/m^2)	m^2	50	70
5.7	Tack coat (0.6 L/m^2)	m^2	40	60
5.8	Asphalt concrete base course (50 mm thick)	m^2	495	600
5.9	Asphalt wearing course (50 mm thick)	m^2	530	600
6	**Build-up Rates Complete**			
6.1	Stone masonry drain, sloping face 1H:3V, internal bed width 2.0 m, height 1.0 m	m	27,000	
6.2	Stone masonry drain, sloping face 1H:3V, internal bed width 3.0 m, height 1.5 m	m	32,000	
6.3	Reinforced concrete drain, sloping face 1H:3V, internal bed width 3.0 m, height 2.0 m	m	98,500	
6.4	Floodwall, height 1.0 m	m	16,500	
6.5	Floodwall, height 1.5 m	m	22,500	
6.6	Floodwall, height 2.0 m	m	45,000	

This data will then be used to carry out the economic analysis of the next step: the cost-benefit analysis.

Remark:

“The rates prices and cost estimates based on recent and current market rates, coupled with assumptions on projected fluctuations and experience on various factors that are considered to be reasonable under the present circumstances.

Given the unknown future impact that Covid 19, with respect to its severity and duration, might have on market fluctuations, the cost estimates are being reported on the basis of plant, labour material and transport costing uncertainty and a higher degree of caution should therefore be attached to the estimates than would normally be the case. It is recommended that the cost estimates be kept under frequent review.”

2.2.4 Solution impact analysis

2.2.4.1 Current and protected water depth maps and comparative mapping

In order to compare the situations before and after implementation of the project, the most effective visualisation tool consists in calculating the differences in water level for a specific flood subject to these two situations.

Thus, the maximum flood levels (water depth) in the projected situation (P) for a flood frequency Ty (from T10 to T100) are compared to the maximum level at the current / actual situation (A) for the same frequency Ty.

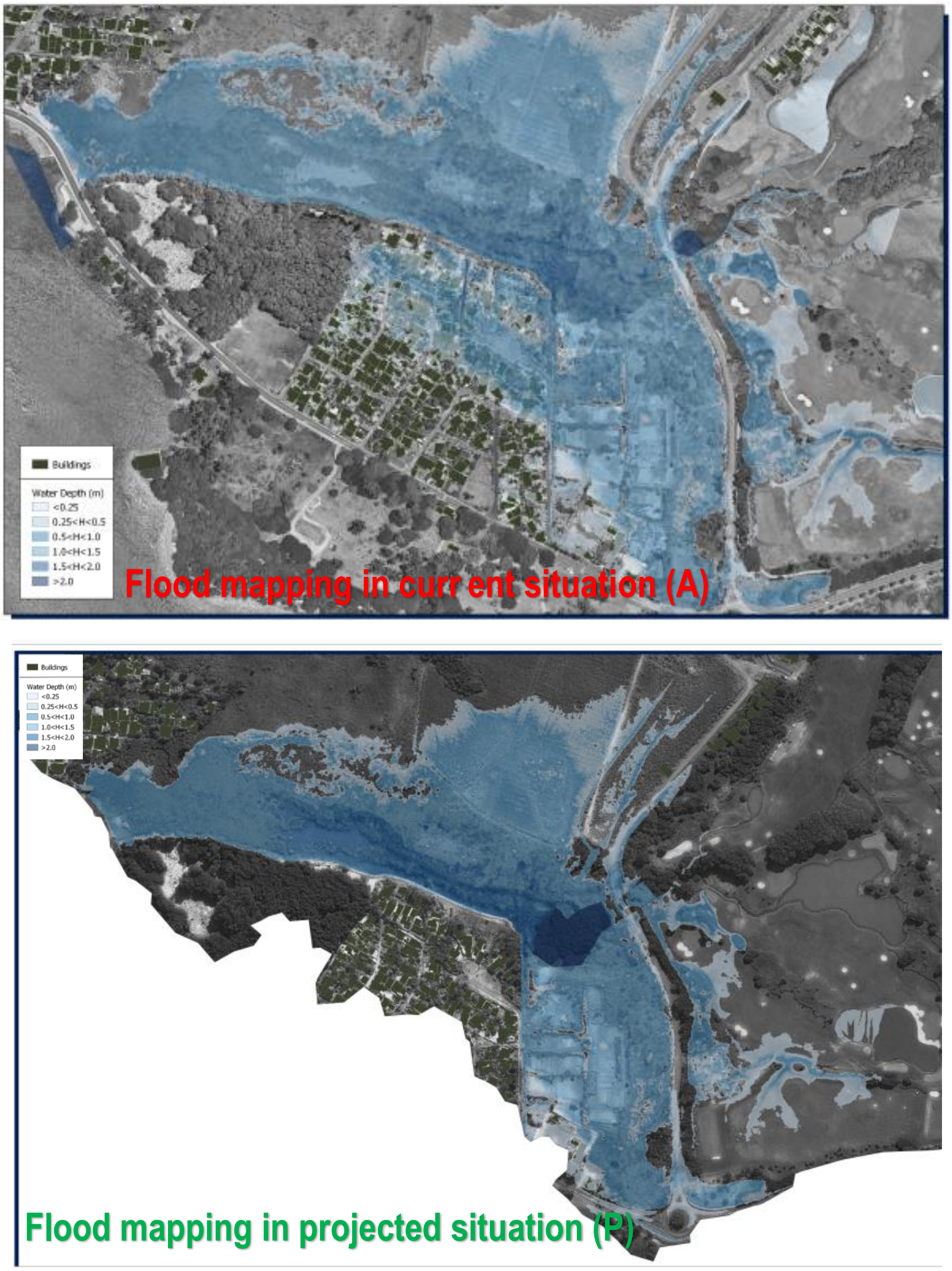


Figure 2: Flood mapping in current situation (A) -"Do Nothing Scenario" / in projected situation (P) – situation with solutions

Then, we compare the water depth maps in the projected and actual situation: (P) - (A) water depth mapping.

The map thus obtained as a result of the implementation of the project, identify zones :

- with attenued water depth (P water depths are less than A water depth)
- with increased water level (P water depths are greater than A water depth)
- with liberated from inundation (Out of flood with project, where (P water depths are nul).

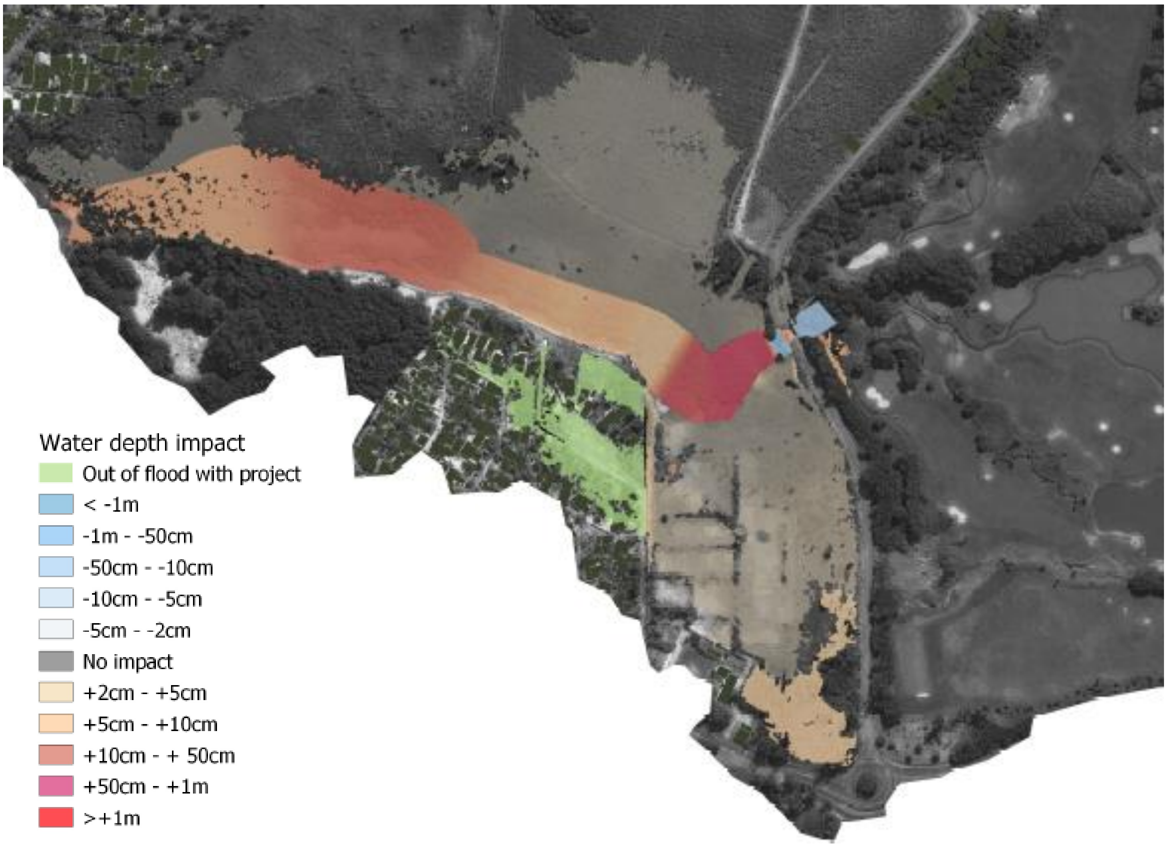


Figure 3: Comparative flood mapping legend – example

Presentation of results and impacts:

- Maps for current situation (A) are deliverable of the *Activity 3 - Report on existing flood maps*, (D3.2.1 for Priority Sites, D3.2.2 for Other Sites). These maps are also attached in Annex 0.

- Locations of Measures are attached in Annex 1

- Projected maps (P) are attached in Annex 2

- Comparative flood maps - (P)-(A) water depth - are attached in annex 2.

2.2.4.2 Analysis of the impacts of the solutions - Cost Benefit Analysis (CBA) and multicriteria analysis

2.2.4.2.1 Introduction - CBA

The benefits derived from a project are measured through the extent of damage avoided by its implementation. The indicators are thus calculated before and after the project implementation measures for a single flood scenario corresponding to the level of protection chosen for the project.

Flood CBA assesses the costs and benefits of a project. It is based on the concept of "damage avoided": the benefits correspond to the total damage that is avoided by such measures.

The indicators of damage avoided to buildings are added to the damage avoided to road networks

2.2.4.2.2 Average costs used to estimate the value of fixed assets

The following average costs have been used to estimate the value of different types of construction:

Table 6: Average costs to estimate the value of different types of constructions

	Average cost (inflation adjusted 2018)
Buildings (MUR/m2)	
Residential buildings	13 270
Transport facilities (MUR/line meters)	
Motorways	33 174
Major roads	27 645
Municipal Roads	22 116
Forest tracks and roads	8 846

Source: ER2C – C1D2a+b - DRR report (Republic of Mauritius, Ministry of environment and sustainable Development, DRR Strategic Framework and Action Plan, Final Report, August 2012), *inflation adjusted using the input cost index for construction 2012-2018 (source: Statistics Mauritius).*

The damage cost calculations allow sensitivity tests to be carried out for different scenarios according to the depth of flooding (less than 50 cm, greater than 50 cm). An average cost is presented above, irrespective of the flood depth. Different costs benefit figures for different scenarios are given in the annex.

2.2.4.2.3 Damage calculation

Once damage costs are plotted against flood frequency the overall cost over the entire country can be calculated for different flood events.

The construction of the annual average damage curve consists in drawing the relationship between the cost of the damage and the return frequency of the floods studied (between 0 and 1). An example is provided below. We extract from it the annual average damage and the annual average avoided damage.

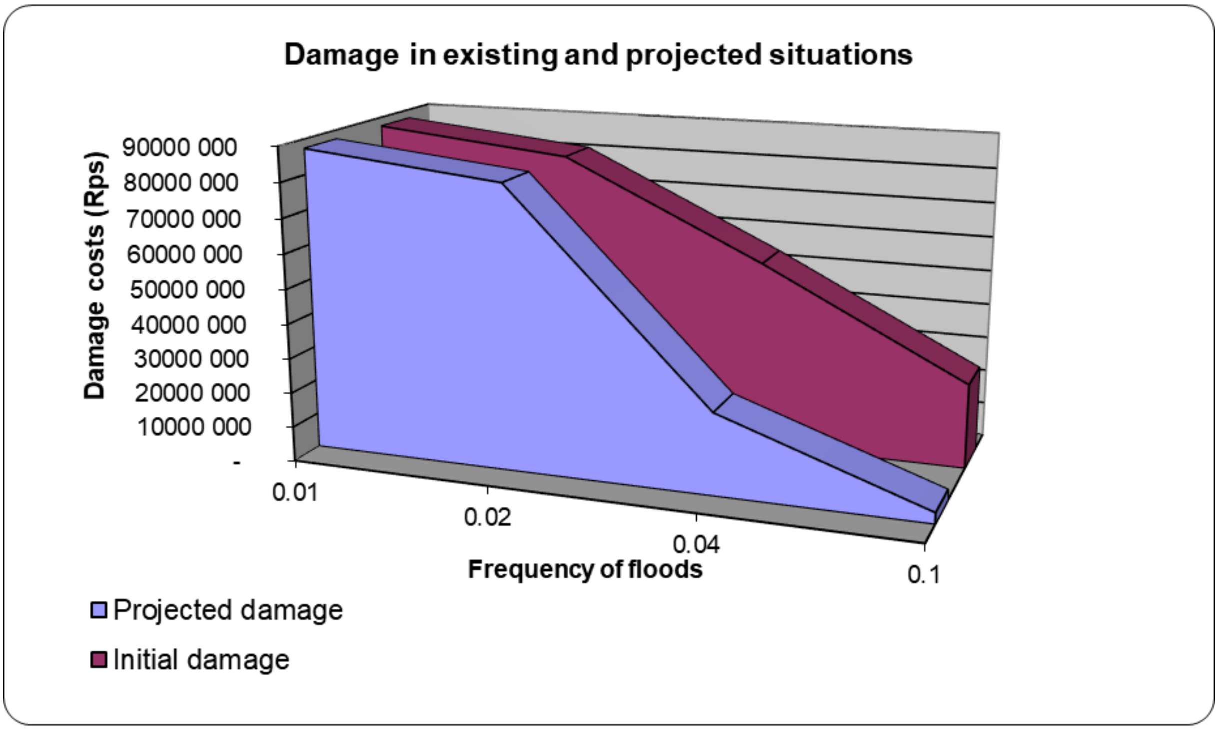


Figure 4: Typical of damage curves (initial and projected situations) – Bel Ombre

2.2.4.2.3.1 Net Present Value (NPV) 3

The Net Present Value (NPV) can be interpreted as the amount of damage avoided, representing a saving by the organisation, less the costs, as a result of the implemented investments.

The computation of the Net Present Value (NPV) consists in comparing the total **cost** of the project (investment and operating costs during its lifetime), calculated using the average estimated unit cost, with the average annual damage avoided- the **benefits**. The year at which break even occurs (when the costs equal the discounted benefits) is noted as the time after which the benefits outweigh the costs and the project is considered profitable.

The NPV is normally calculated by adding the present value of future benefits and the residual value at the ruling interest rate for any particular year, and subtracting therefrom the investment costs and the discounted operational costs and future expenses. The NPV depends on the discounted rate used to calculate these values.

NPV is calculated using the following formula:

$$NPV = -C0 + \sum_{i=0}^{n} \frac{1}{(1+ri)} \times (AAAD - Ci)$$

Where :

- C0 is the sum of the initial costs of the works,
- Ci is the sum of the maintenance costs in year i,
- AAAD is the average annual avoided damage,
- n is the time horizon for the profitability of the works, and
- ri is the discounted rate for any particular year i

[3] L'ACB (analyse coût/bénéfice) : une aide à la décision au service de la gestion des inondations – Centre Européen de Prévention du Risque inondation - 2011

The following parameters have been included in the cost benefit analysis:

- **Maintenance cost: 1% per year** (see detail below)
- **n: The time horizon for which profitability is required: 50 years**
- ri: Discount rate = 4% for the first 30 years then asymptotically decreases to 2.5% at 100 years and 2% at infinity

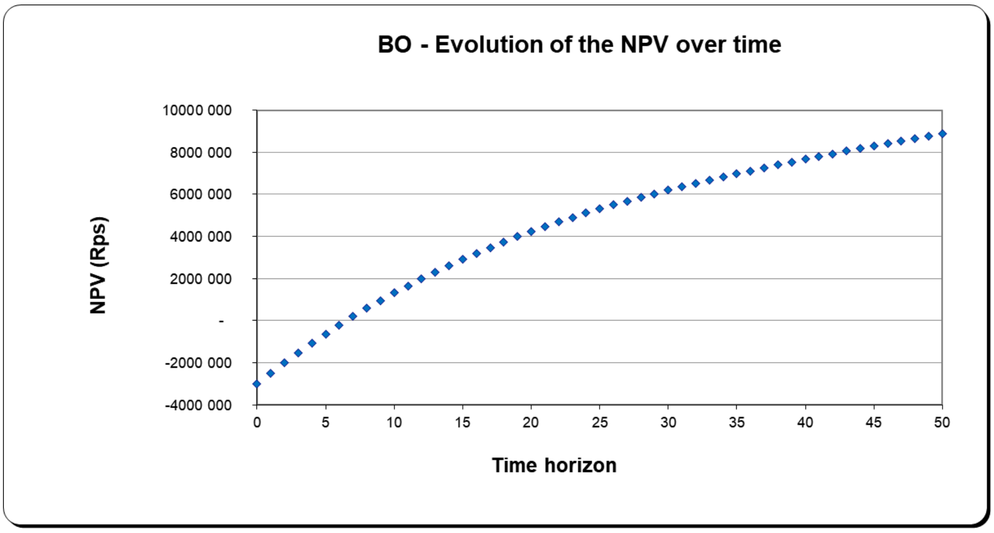


Figure 5 : Example of the evolution of an NPV

The net present value (NPV) allows the costs to be subtracted from the benefits (avoided damage) of the predicted measures.

If NPV is positive, the measure studied, on ground of the geographical perimeter selected and according to the issues and damage taken into account, then the measure studied is relevant from an economic point of view.

2.2.4.2.4 Maintenance Cost

Most components in a drainage system can be prone to erosion, blockage or subsidence. Minimal maintenance is required to preserve the expected hydraulic capacity within the system. Activities can be categorised either as preventive or remedial.

2.2.4.2.4.1 Preventive maintenance

Preventive maintenance includes periodic inspection of the system, monitoring, regular maintenance and analysis of data on reported complaints and problems.

Regular inspection activities should normally include the following:

- Street cleaning
- Removal of debris and sediment from catch basins
- Supervision of connections
- Cleaning of outfalls and culverts
- Inspection of physical conditions of pipes and manholes (visually or by camera if necessary)
- Repair or replacement of damaged pipes,

- manholes, catch basins and other components

The implementation and follow-up of a maintenance register is essential to keep records of maintenance activities. Data for each component of the system should be kept up to date, ideally including:

- Date of construction of the systems (possibly including the name of the designer and contractor)
- Type, size and shape of pipes
- Area served and land use
- Manholes and catch basins (location, type and invert (for manholes)
- Inspections (date, methods, location and results)
- Complaints reported (location, nature, date, time, rainfall characteristics leading to complaints)
- Repairs and replacements made.

Several options for managing this information are possible, ranging from codified access to printed plans or a computerised management system to complete geographic information systems (GIS) that integrate system data and spatial representation of this information. A GIS can typically include several other types of data (sanitary sewer system, water supply, roads, etc.) and is the most advanced and effective approach. **We recommend integrating the use of GIS systems to track informations and maintenance.**

2.2.4.2.4.2 Corrective maintenance

Corrective maintenance is not usually predictable and becomes necessary in emergency situations. These are actions that require immediate attention, such as a broken pipe or blocked culvert inlets. These actions must be taken to reduce the potential for flooding and limit damage, to prevent injury or to protect receiving environments.

Some risk factors can however be identified and minimised. For example, in the case of blocked culverts, there are physical factors that increase the risk of a culvert being blocked by debris jams during a high flood: banks in a more or less advanced state of erosion, presence of shrubs and trees in a precarious position, debris and objects littering the floodplain and likely to be moved during a flood.

Details of maintenance operations and their frequency are provided in the second part of the Land Drainage Master Plan.

2.2.4.2.5 Multicriteria analysis

The multi-criteria analysis, which also quantifies intangible benefits such as time savings, project sustainability as well as social and environmental positive impacts, is based on the following steps:

- Defining the problem and identifying the different solutions,
- Identifying the scope of study,
- Characterising the hazards required for study,
- Characterising the occupation of the country,
- Computing the costs and benefits of the project,
- Analysing the final results.

A sensitivity analysis on the damage costs to buildings according to the water level (50 cm threshold) can also be done in the attached CBA calculation sheets (Annex3).

3 SOLUTIONS STUDIES ON PRIORITY SITES

3.1 Nouvelle France

3.1.1 INTRODUCTION

The village of Nouvelle France forms part of the priority sector 47 and within the main rainfall catchment which extends over an area of 4.95 km^2. The catchment which is further sub-divided into 10 sub-catchments, extends from uphill from Curepipe point (681 m amsl) to Nouvelle France B95 Link road (407m amsl) and it is drained primarily by Rivière La Chaux and its two tributaries, as shown below.

Figure 6: Catchment area of Nouvelle France

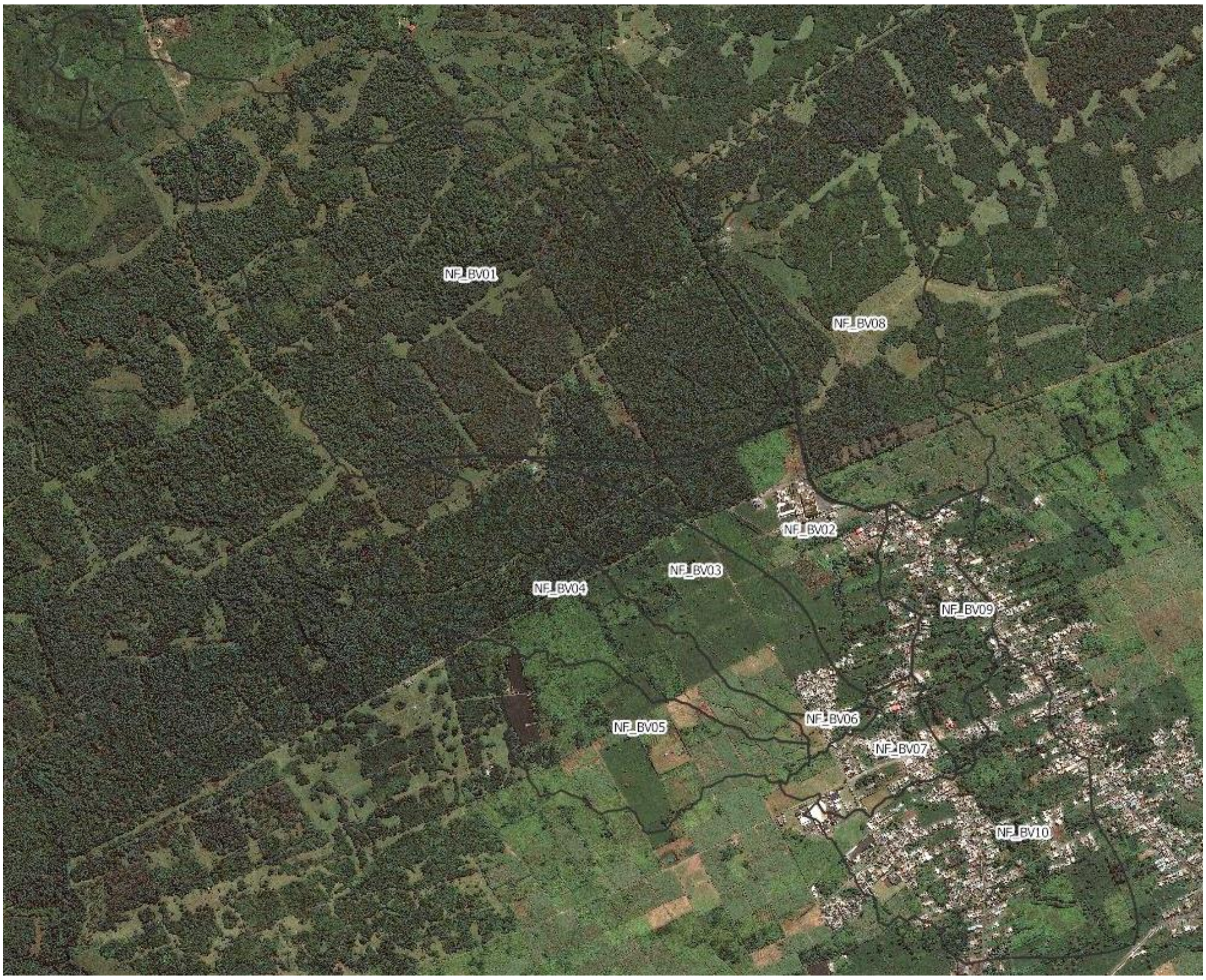


Figure 7: Sub-division of catchment area of Nouvelle France into sub-catchments (PLEIADE 2013)

Table 7: Nouvelle France – Physical Characteristics of individual Sub-catchments

Name	Area (ha)	Area (km²)	Low level (m)	High level (m)	Length	Slope (m/m)	Slope (%)
NF_BV01	171.35	1.71	497	681	3020	0.06	6.09
NF_BV02	38.77	0.39	447	557	2215	0.05	4.98
NF_BV03	40.85	0.41	448	576	1884	0.07	6.79
NF_BV04	45.71	0.46	451	590	1835	0.08	7.57
NF_BV05	35.30	0.35	445	560	1349	0.09	8.53
NF_BV06	2.27	0.02	438	453	419	0.04	3.60
NF_BV07	13.08	0.13	433	453	824	0.02	2.45
NF_BV08	70.10	0.70	468	551	1912	0.04	4.36
NF_BV09	20.29	0.20	433	472	1223	0.03	3.21
NF_BV10	57.89	0.58	407	453	1576	0.03	2.91
NF_Global	**495.61**	**4.96**	**407**	**681**	**5918**	**0.05**	**4.63**

The flows obtained for Nouvelle France for return periods of 10, 25, 50 and 100 years are shown in the table below.

Table 8: Nouvelle France – Flows for sub-catchments and at outlet of catchment for return periods of 10, 25, 50 and 100 years

BVs	Q10 (m3/s)	Q25 (m3/s)	Q50 (m3/s)	Q100 (m3/s)
NF_BV01	0.79	3.68	6.39	9.77
NF_BV02	14.03	19.80	23.74	29.30
NF_BV03	8.50	13.12	16.66	21.62
NF_BV04	3.56	6.91	9.59	13.25
NF_BV05	9.69	14.51	18.03	23.25
NF_BV06	1.38	1.61	2.05	2.33
NF_BV07	8.50	10.82	12.55	15.11
NF_BV08	1.78	4.14	5.93	8.37
NF_BV09	13.05	16.58	19.17	22.79
NF_BV10	35.38	45.59	52.49	62.78
Outlet of Nouvelle France	**76.3**	**108**	**129.4**	**159.5**
Q /A ($m^3/s/km^2$)	***15.4***	***21.8***	***26.1***	***32.2***

3.1.2 BACKGROUND

The NDRRMC identified in its report of 2018 several flooded areas following the recurrent flooding problems in Nouvelle France. Reportedly, the village has in the past been affected by four main flooding events namely in May 2016, May 2017, February and April 2018. More recently, during the rainy spells of 27th April 2021 and 28th April 2021, several houses and roads bordering Rivière La Chaux and its tributaries as well as Nouvelle France roundabout which is sited within a depression (400 m amsl) along Motorway M2 got inundated.

Records from the Mauritius Meteorological Services show that 87.4 mm rainfall was recorded on 27th April 2021 over a 12 hour period at Rose Belle gauging station and which rainfall is equivalent to an intensity 7.28 mm/hr corresponding to a recurrence interval of less than 10 years. A torrential rain warning bulletin was also issued on 27th April 2021 at 16hr30 which was effective up to 28th April 2021 at 20hr00.

Site visits were undertaken on 3rd May 2021 at the following vulnerable locations shown below.

A	-	Culvert crossing B95 Nouvelle France link road	R	-	Comlone road
B&C	-	Allee Jacques Lane	S	-	Indira Gandhi Street
D	-	Footbridge across Riviere La Chaux	T	-	Low lying house at A9 road
E & F	-	Culverts across Aubeeluck Lane	U	-	Bheeroo Lane
G & I	-	Bridge crossing A9 Nouvelle France road	V	-	Nav Hind Lane
J/K/L/M	-	Kanpur road	W	-	Grand Port A10 Road
N	-	NHDC Housing Estate	X	-	Mr. Bissoonee's House
O	-	Mr Roudard's House	Y	-	Nouvelle France Roundabout
P	-	Bus stop near Savemart	Z	-	Drive parallel to Aubeeluck Lane
Q	-	Deamprice Supermarket			

3.1.3 SITE DESCRIPTION

In general, it could be observed that the regions vulnerable to flooding and which had in fact been affected by the recent floods are those located either on the banks of Rivière La Chaux and its two tributaries or those whereby there have been encroachment by building construction over the natural drainage axes.

3.1.3.1 Kanpur Road

A residential agglomeration has been built immediately downstream of the tea plantations in between Rivière La Chaux and its western tributary crossing Kanpur road.

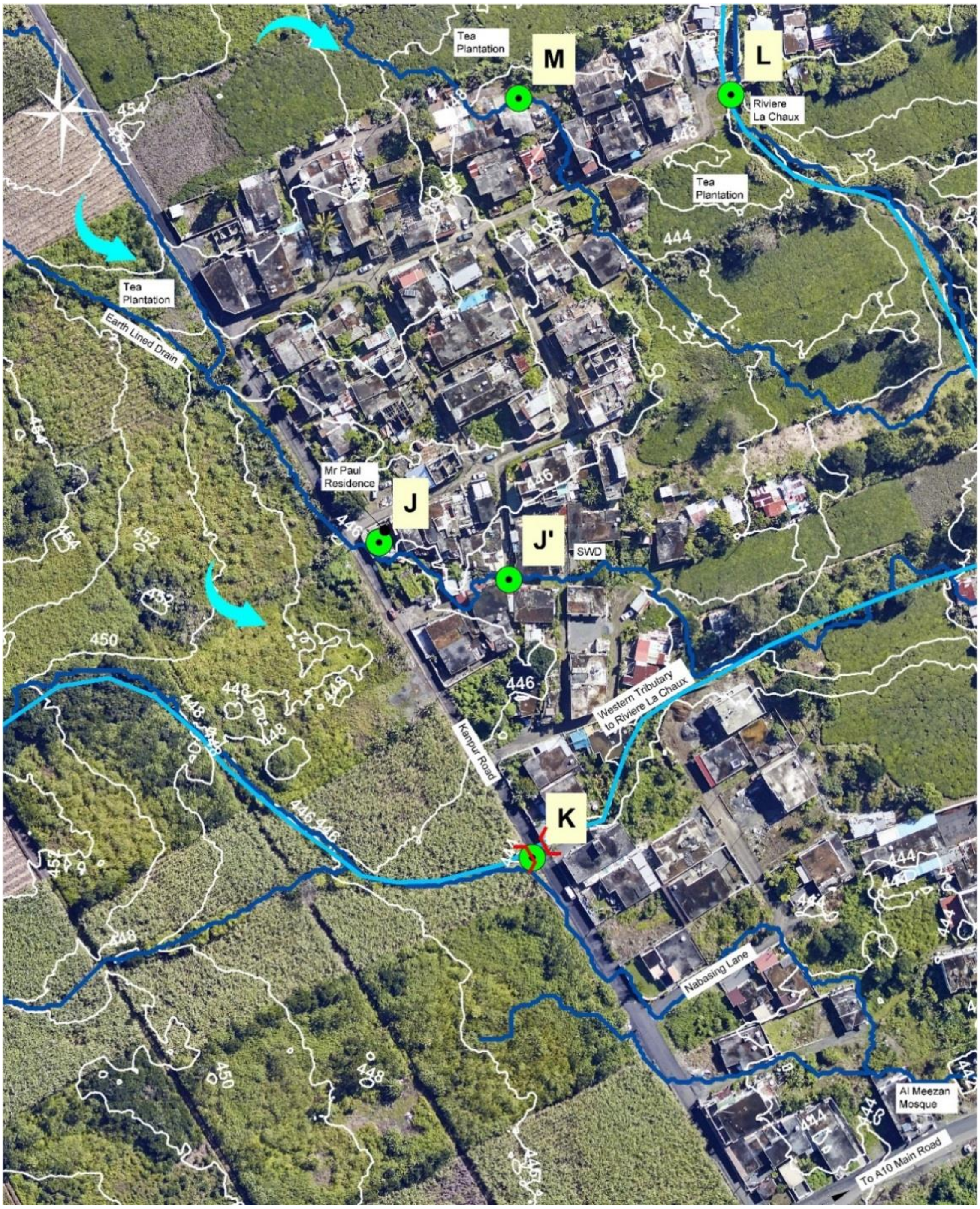


Figure 8: Layout showing different drainage paths at Kanpur

At point J, a natural earth-lined drain emanating from the western side of the tea plantations crosses Kanpur road through a narrow cross drain (600 mm wide by 750 mm deep) and continues its route as a buried drain underneath a few houses which have been built squarely over the drain. Therefrom, the drain continues as a RC U-drain, covered at its upstream section, until it discharges to the tributary of Rivière La Chaux.

Earth lined drain upstream of crossing at Kanpur

House built squarely over drain at Kanpur

The constricted drain size severely impedes evacuation of surface runoff emanating from the tea plantations along its natural drainage axis. The flooding problem is accentuated by contributing overland flows converging from other surface water drains at point J'.

At point K, a tributary to Rivière La Chaux crosses Kanpur road through a 2.4 m wide by 1.5 m deep culvert. The hydraulic cross sectional area of the culvert is further constricted by the presence of two masonry steps and two CWA services pipes across the culvert. A roadside drain constructed apparently with the aim to evacuate water from the localised depression along Kanpur road serves no purpose since it discharges at the upstream section of the tributary with its invert level at the same as that of the culvert.

Upstream Section of Culvert K

Downstream Section of Culvert K

During the last rainfall event, the watercourse overflowed its banks to flood Kanpur road and Nababsing Lane, with overland flows flooding the area behind Al Meezan Mosque and the junction of Kanpur road with Nouvelle France A9 main road in front of Aubeeluck restaurant.

The photographic records below illustrate the extent of flooding at Kanpur road during the recent rainfall events of April 2021.

River stage upstream of Culvert K

River stage downstream of Culvert K

Overland flow along Aubeeluck Lane

Overland flow along Nabasing Lane

Further North of the residential agglomeration at Kanpur road, more precisely at point M (Mr. Itchia's residence), a few houses have been built across a natural drainage axis (540amsl) and draining surface runoff from the tea plantations uphill into Rivière La Chaux some 1600m downstream. Only a narrow passage of width 1.5 m in between houses exists for water evacuation through overland flow, which is inadequate to channel runoff emanating from the vast expanse of tea plantations uphill.

Narrow passage between houses

3.1.3.2 Rivière La Chaux

Rivière La Chaux emanates from its source at *Seizieme Mille (elevation 560m* amsl) and flows over a length of 4.35km across Nouvelle France to the B95 Link road and the M2 Motorway south of the roundabout in the direction of Rose Belle village.

Flooding problems exist as far as upstream at point L which is characterised by a culvert of dimensions 3.6 m wide by 1.4 m deep. The river banks downstream of the culvert had been lined by a RC wall on either side for an approximate length of 15m with the west end section of the wall abruptly changing direction, resulting in backflow at that location and inundation of the properties to the east of the river.

Downstream end of RC wall

Figure 9: Layout at point L

Some 190 m downstream of point L, the western tributary of Rivière La Chaux of typical cross section of 2.0 m wide by 1.2 m deep joins the main river course. At that location, marked H on the drawing, the houses of several squatters got inundated during the recent rainfall event. Reportedly, the intervention of the Fire Services is required at every heavy rainfall event to evacuate the squatters from the floodplain. This location also serves as the discharge outlet of the existing surface water drain built along Bakery Lane.

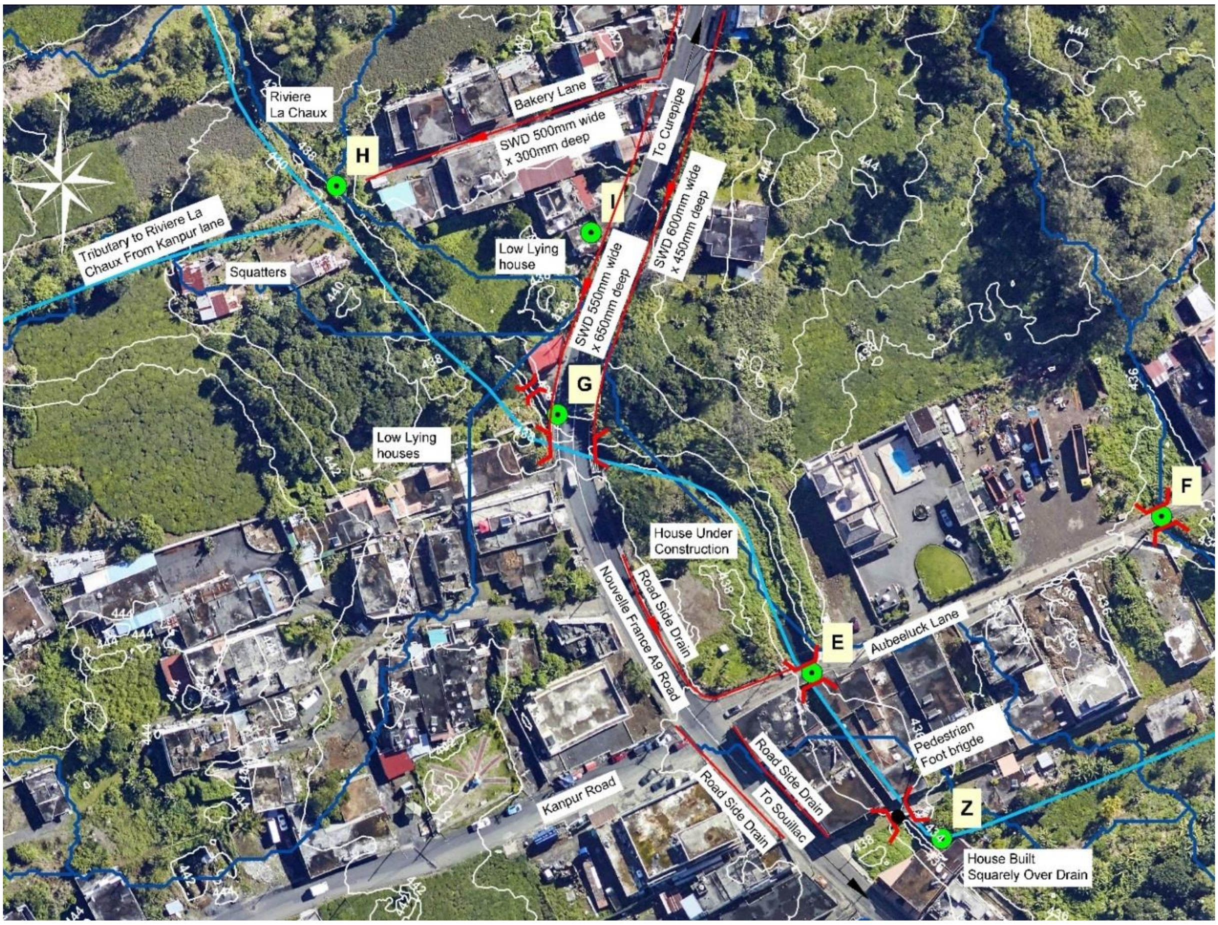


Figure 10: Layout at point H, I, G, E, F & Z

Houses of Squatter bordering Rivière La Chaux

At point G, Rivière La Chaux crosses Nouvelle France A9 road through a bridge of span 6.6 m and depth 2.8 m. The bridge itself has insufficient hydraulic capacity to accommodate flows upstream from Rivière La Chaux. One house (I) (belonging Mrs. Mohitop) located right over the floodplain upstream of the bridge and a few makeshift tin houses right next to the river bank some 100 m upstream (H) had sustained inundation during the April 2021 rainfall event.

Bridge crossing A9 road

Likewise, a house constructed right on the western bank of Rivière La Chaux (G), with its boundary wall encroaching into the water course and a few other low-lying houses in its proximity are subject to recurrent flooding.

A recently constructed covered drain of length 90 m and size 2.0 m wide x 1.3 m deep along Aubeeluck Lane transfers a substantial amount of stormwater from upstream of the stone masonry culvert E of size 2.8 m wide and 3.2 m deep across Rivière La Chaux to downstream of culvert F of size 3.2m wide x 1.2 m deep across its tributary.

Culvert at E (note water pipe crossing)

Culvert at F

The downstream section E to Z of the Rivière La Chaux had been severely encroached into by houses constructed on either bank, disregarding any reserve whatsoever. Some 65 m downstream (marked Z) the house had been extended squarely over the river.

Downstream Section of Culvert E

House encroaching into river course

Flood level reaching soffit of house

River stage downstream of Culvert E

River stage upstream of Culvert E

Transfer drain from culvert E to F

Overland flow over transfer drain

Figure 11: Location of Footbridge D

River stage at footbridge-Normal weather

A steel footbridge at point D of total span 14 m x 1.65 m high founded on solid bed rock across a shallow section of the river serves a single house on the east bank of the river. Very high velocity flows were experienced during the recent flood.

Figure 12: Layout at Allée Jacques Lane (Points C & B)

An undulating landform along Allée Jacques creates localised depression resulting in water accumulation (section B to C) across the road. An absorption trench had been built at the eastern end B.

At the upstream section of culvert marked C of size 2.8 m wide by 3.55 m deep constructed across Rivière La Chaux at Allée Jacques Lane, an industrial building has part of its foundation into the minor river bed, hindering free flow The presence of a water pipe across the culvert causes further obstruction to the flow. Severe scouring to the foundation of the building could be observed. During heavy rainfall events, backflow upstream of this culvert finds its way behind and around the building to flood Allée Jacques Lane.

Industrial building partly founded on river bed

Figure 13: Layout at Culvert across B95 Nouvelle France Link Road (Point A)

Nouvelle France B95 link road is sited at the outlet of the catchment area of Rivière La Chaux at Nouvelle France at elevation 407m amsl. A RC box culvert across the B95 road of 3 spans, each 2.5 m wide x 2.0 m deep overflowed during the recent rainfall event, partly due to extensive blockage of the culvert by transported rock and sediment. The backflow flooded KNaaz factory premises, resulting in overland flows along B95 road flooding the M2 roundabout.

Culvert Blocked by transported rocks/sediment

3.1.3.3 Eastern tributary of Riviere La Chaux

The eastern tributary of Riviére La Chaux starts at elevation 518m amsl and joins the main river some 2.3km downhill at the level of Aubeeluck Lane. Its flow is ephemeral.

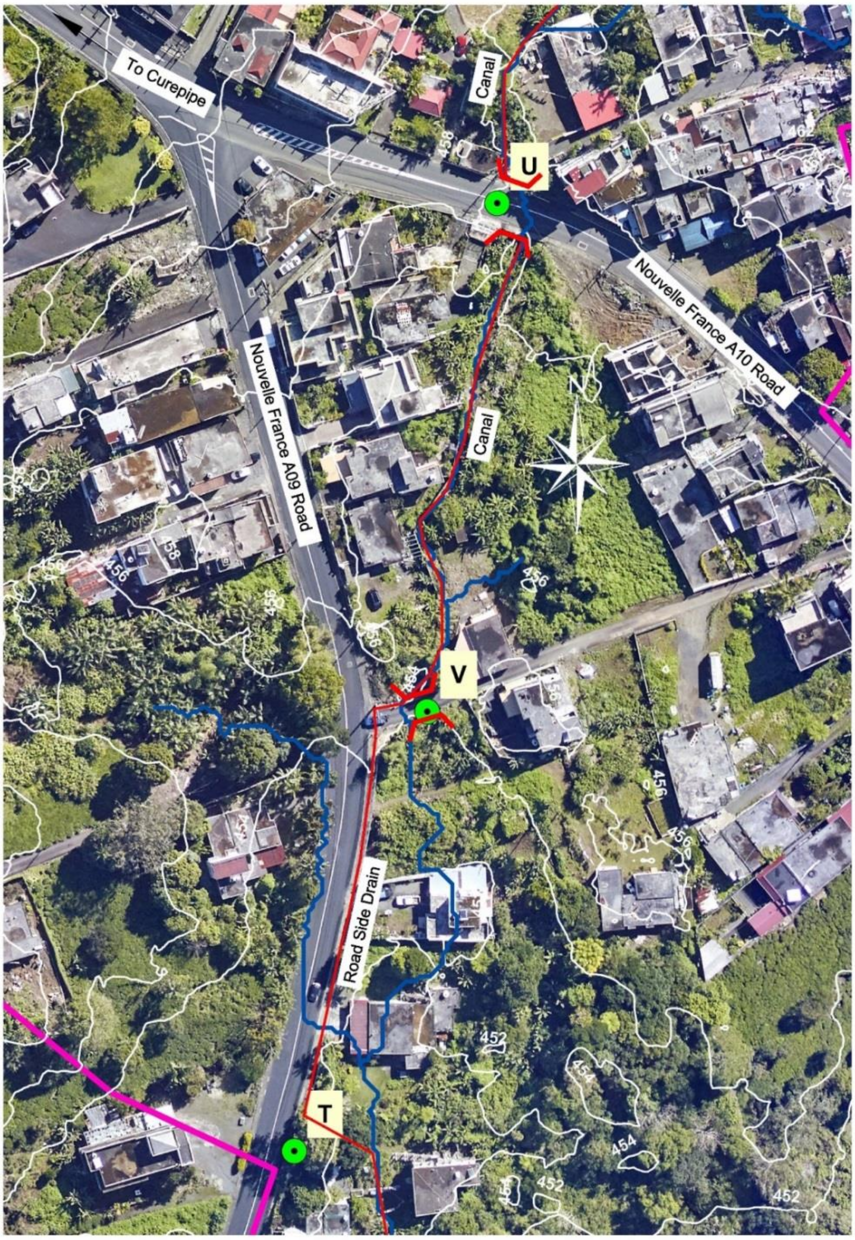


Figure 14: Nouvelle France North Bus Stop – Point P

Substantial flow from a natural drainage path intercepted by the A10 road flows across and inundates the road.

Points U and V – Bheeroo and Nav Hind Lanes

At Bheeroo Lane, the first house to the left has its boundary wall encroaching into a natural drain, causing backflow into properties upstream.

The watercourse crosses A10 road through a 2.5m wide by 3.0m deep (stone masonry) culvert. A200mm ø DI pipe located approximately mid height of the culvert cross-section poses severe restriction to free flow. The watercourse continues its route as an earth lined drain to Nav Hind Lane where it crosses the lane at point V through a twin 750 mm diameter concrete culvert. For unexplained reasons, the culvert had been routed into the A9 roadside drain instead of discharging into a natural drain, across the Nav Hind road. Two houses built over the natural drain still get inundated by overland flow across the A9 road.

Twin 750 mm diameter concrete culvert across Nav Hind Lane

3.1.3.4 Grand Port A10 Road

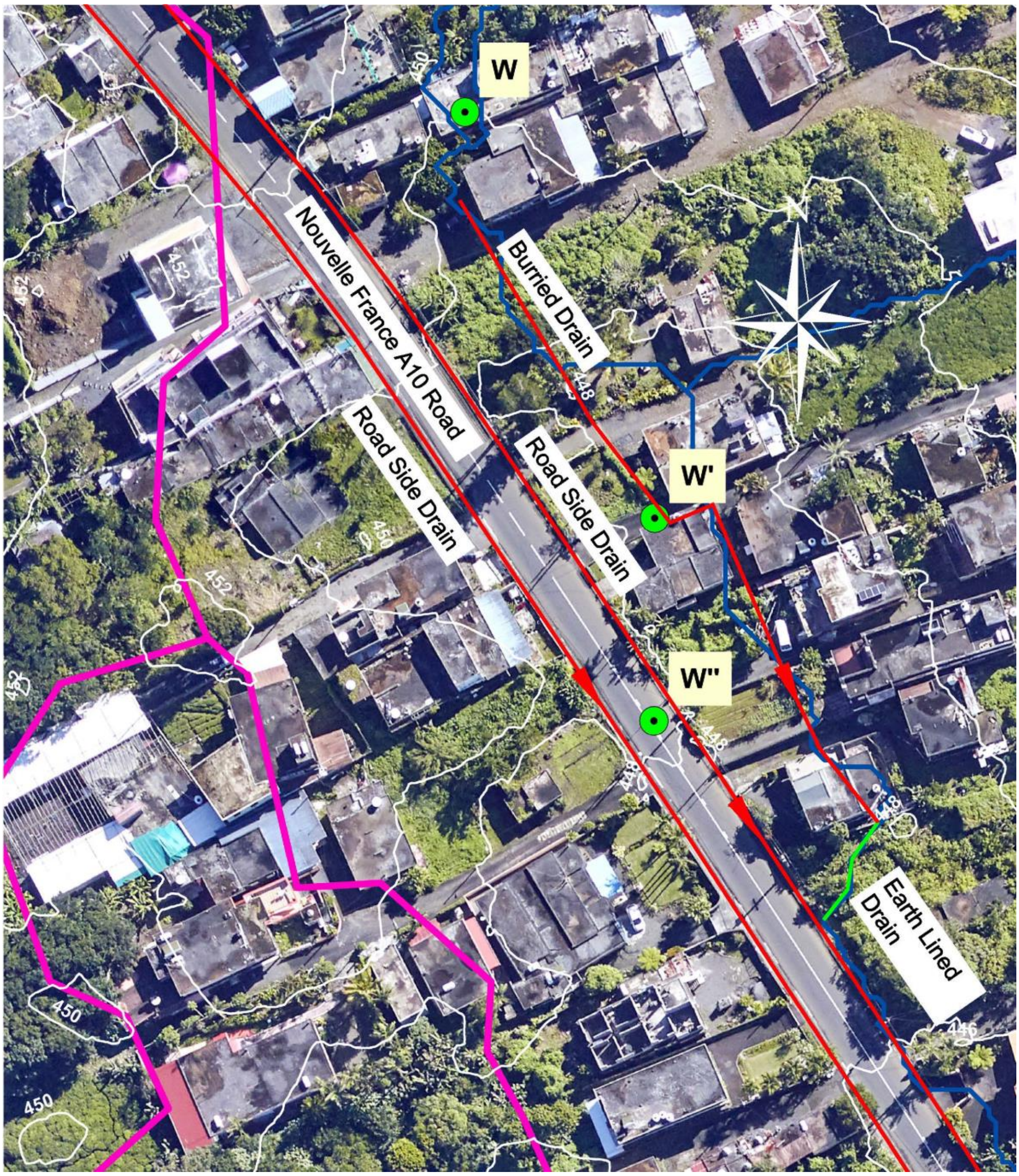


Figure 15: Layout at point W, W', W"

The A10 road crossing Nouvelle France is mostly located within a valley with a major drainage axis passing very near or next to its alignment. This densely populated area receives surface runoff from a contributing catchment of area 0.2 km², different from that of Rivière La Chaux.

Building encroachment onto this drainage axis and undersized covered drains at point W constrain natural flow and cause inundation to adjacent properties.

Building encroachment at point W **Natural drain and covered drain at point W**

Building encroachment, natural drain, covered drain and marshland in succession at point W

At W’, the natural drain had been diverted by means of a 550 x 330 mm covered drain to suit housing construction.

Drain diversion to suit construction at point W’

At W”, building encroachment and outlet of diverted drain into the already undersized A10 roadside drain floods the entire area.

Building encroachment and diverted drain into A10 roadside drain at point W”

3.1.3.5 Point X- Bissonee's Residence

Figure 16: Layout at X-Bissonee's Residence

Buildings constructed over the natural drainage axis west of the Grand Port A10 have severely impacted on the evacuation of surface runoff in this region some 200m upstream of Nouvelle France roundabout and more dramatically where overland flow during the recent heavy rains deflected by a boundary wall (with only a small aperture 500 mm wide × 600 mm deep for normal flow) impounded as much as 1900mm against Mr. Bissonee's house flooding in its wake his residence and the A10 main road.

Flooding at Bissonee's residence

Floodmark at Bissonee's Residence

Point Y Nouvelle France Roundabout

Nouvelle France Roundabout is situated within a depression at 400m amsl and was inundated by flood flows emanating from all the M2, A10 and B95 roads. The culvert across Riviere La Chaux crossing the M2 motorway south of the roundabout reportedly did not overflow onto the roads, the culvert of size having sufficient capacity to accommodate the flow.

Point P – Nouvelle France Bus Stop

The bus stop at the northern of Nouvelle France village has been constructed in a depression and gets inundated by surface runoff emanating both from the higher elevations of the Nouvelle France A10 road and from the large expanse of forest uphill. A 500mm wide by 500mm deep absorption drain constructed in front of the bus stop is heavily silted (350mm). Even if properly cleared, it would not evacuate the water accumulations due to the absence of a proper outlet.

Point N – NHDC site

The NHDC site is outside the priority site of Nouvelle France, but the flooding there is worth noting. The leaching field at the NHDC Housing Estate had been constructed over the natural drainage axis. Reportedly, a borehole dug for water extraction which once served to evacuate some of the surface runoff had been decommissioned and backfilled over.

Figure 17: Overland flows at NHDC site

During rainy spells, overland flows follow the access roads within the estate to ultimately join the natural drainage path on the other side of the leaching field.

Points Q, Q', O, R & S.

Flood prone areas south of the priority site are described briefly:

Access roads serving residential properties in this area cross a multitude of drainage axes and are undulating with numerous valleys and ridges. Boundary walls across these drainage axes concentrate and deflect overland flows onto roads and these flows, together with surface flow from either side of the depression, flood houses located downstream.

Point Q: An isolated house built in lowland, shown to us by a counsellor of the Grand Port District Council and located next to Dream Price Supermarket sustained flooding during the recent rainfall.

Figure 18: Layout showing Q,Q' & 0

Point Q^1: A localised depression along the side road crossing a valley and over which houses had been built is the cause of water accumulation.

Point O: Flood flows leaving the banks of the water course inundate a few houses.

Point R (Comlone): Comlone is an assembly hall built across a drainage axis with a boundary wall along the road concentrating and diverting overland flows across the road onto residential properties located downstream.

Figure 19: Layout showing R

Point S: Similar to Point Q^1: The absorption drain recently constructed reportedly was not effective and part of the asphalt surface had been broken in front of Mr. Gooblall residence to evacuate floods into its natural drainage axis running across a bare land adjacent.

Figure 20: Layout showing Point S

3.1.4 PROPOSED SOLUTIONS

An overview of the proposed solutions is given in the figure below. A detailed description of individual solutions follow

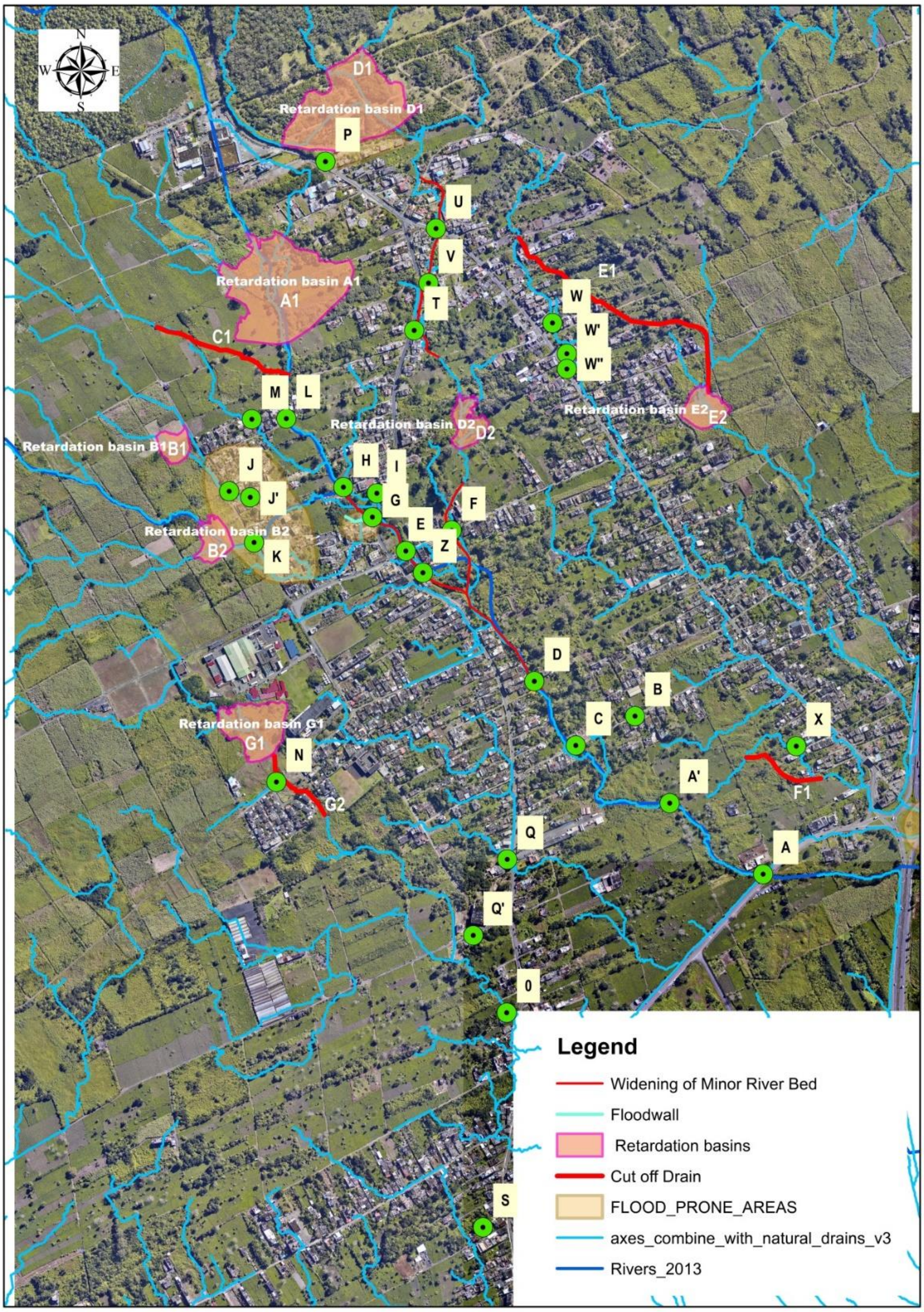


Figure 21: Overview of proposed solutions on background of flood prone areas

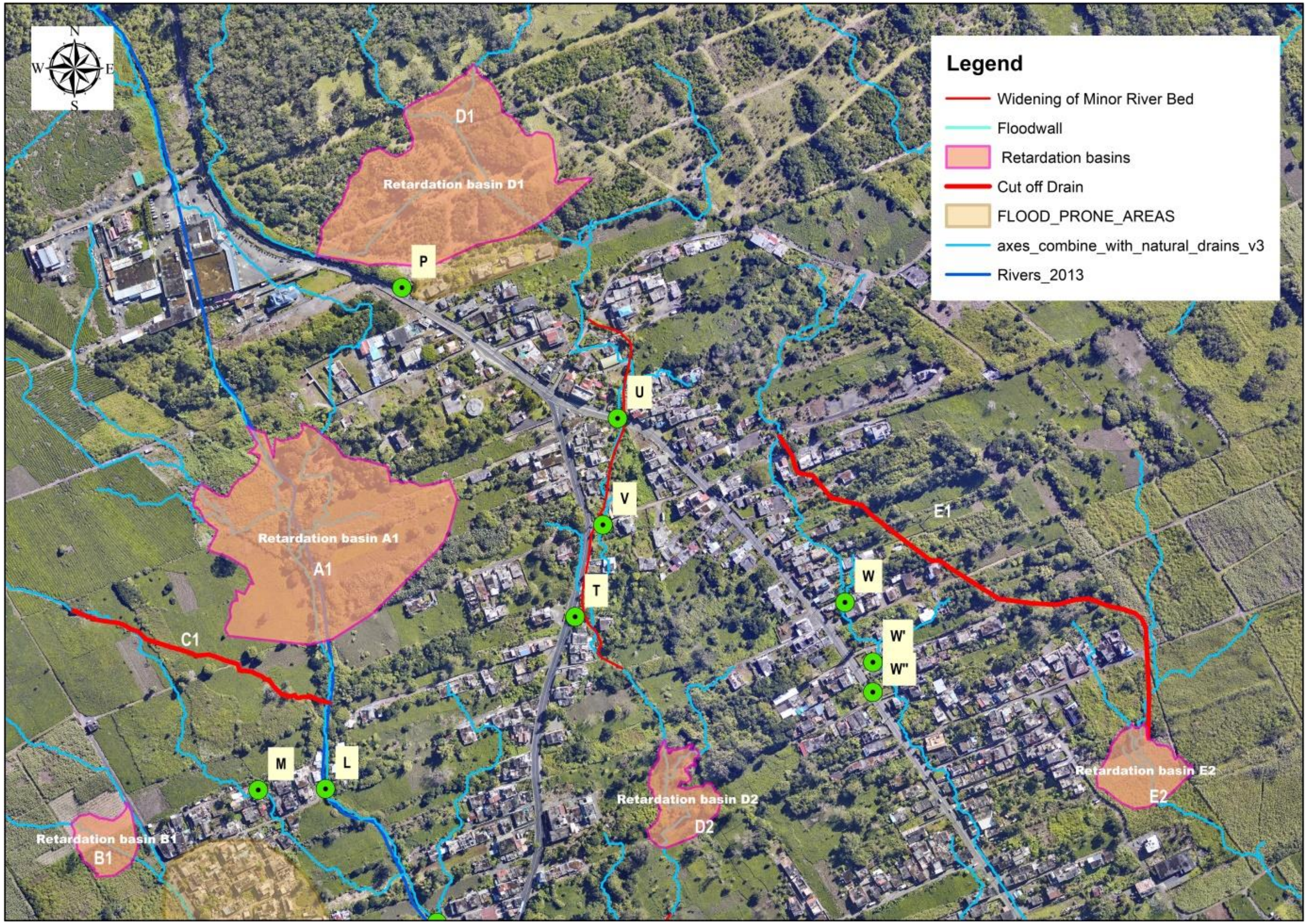


Figure 22: Overview of proposed solutions on background of flood prone areas – Zoom 1

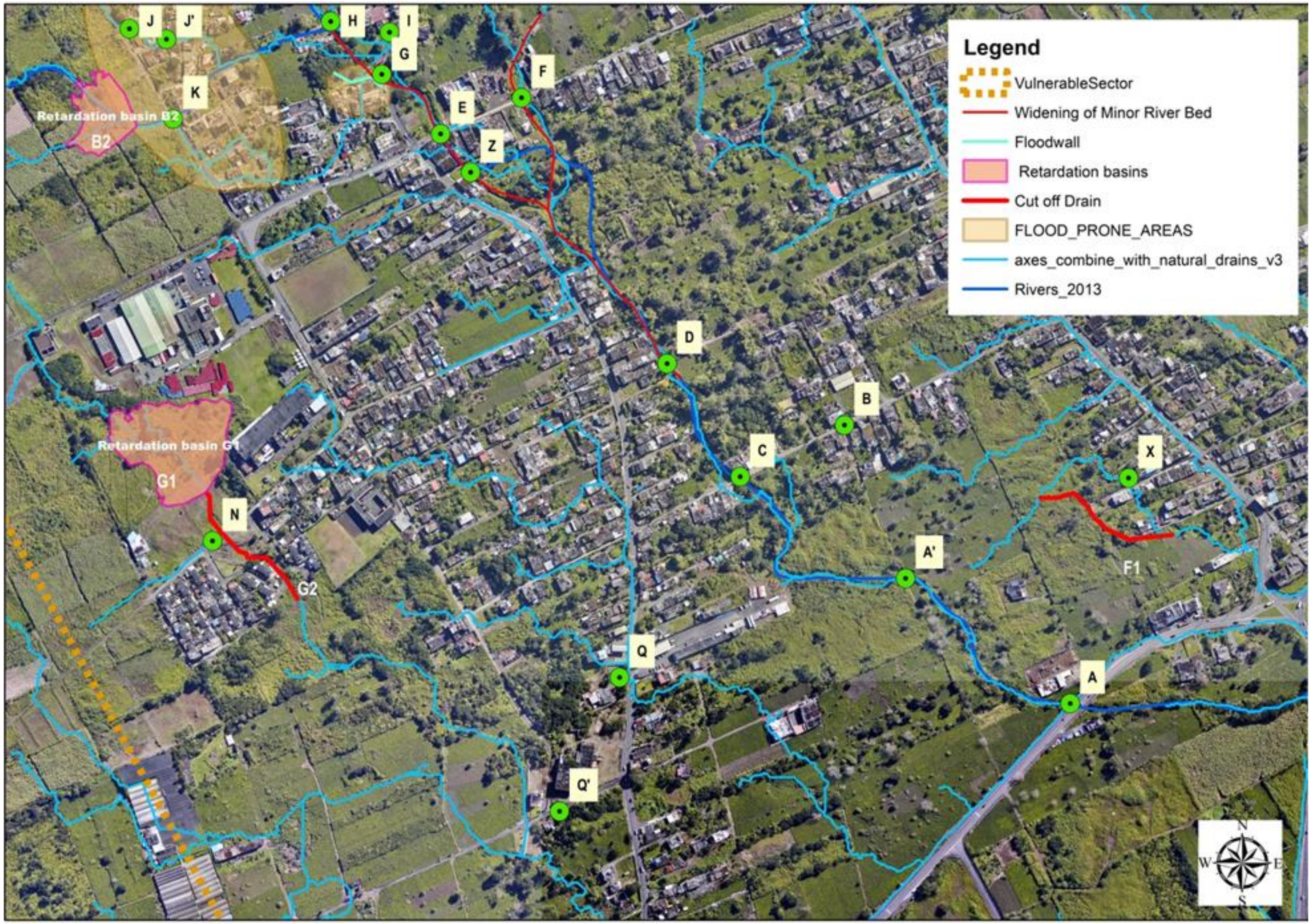


Figure 23: Overview of proposed solutions on background of flood prone areas – Zoom 2

3.1.4.1 ER2C recommendations

The ER2C report concentrates on one section of the sector, whose description follows. Following the preliminary analyses carried out as part of the ER2C study, the measures illustrated below have been included in the present modelling study (see figure below for location):

- Main tributary around the tea factory: inspection of the underground structure under the tea factory should make it possible to estimate its capacity: the actual dimensions of the structure over its entire length, and the possible presence of jams. According to these conclusions, a complementary diversion structure needs to be created. Given the complexity of freeing up the rights-of-way, most of this structure will be underground; downstream of the existing car parks, an open channel should reach the bottom of the thalweg.

- Riviere la Chaux around A9 road crossing: On this section where the slope is steeply reduced, the minor bed and associated bridges will have to be resized to allow the passage of high water levels. The removal of part of the buildings that severely restrict the current bed (4 metres wide) will free up the space needed for this work. A flood wall will have to be installed upstream of the A9 road to protect the houses located downstream the watercourse, on its right bank.

- Secondary tributary: The minor bed and crossing structures will have to be resized to allow for the passage of high water levels. Backfills and walls that prevent the free flow of water will have been removed beforehand, as indicated above.

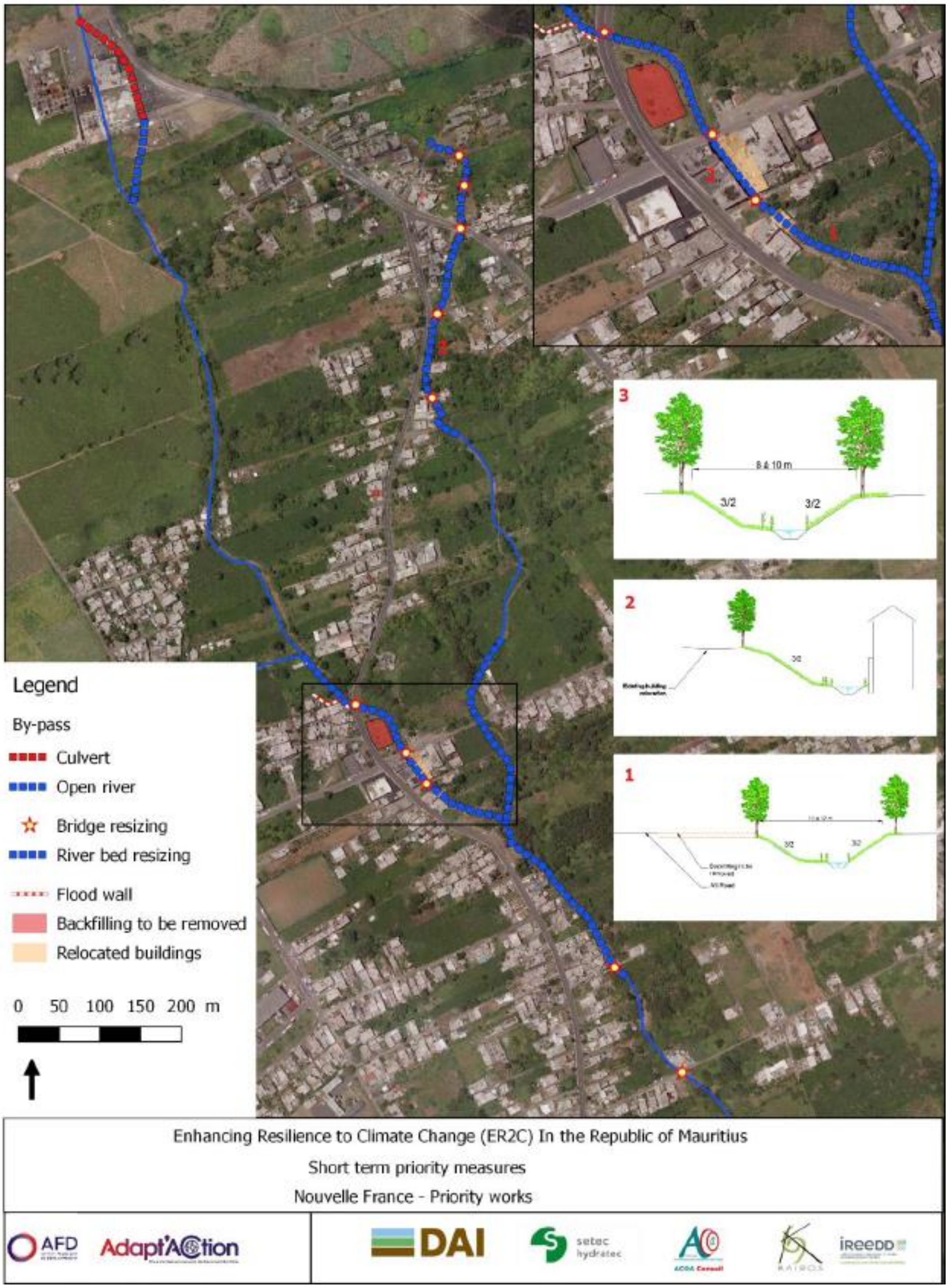


Figure 24: Nouvelle France - ER2C recommendations

WEST DRAIN

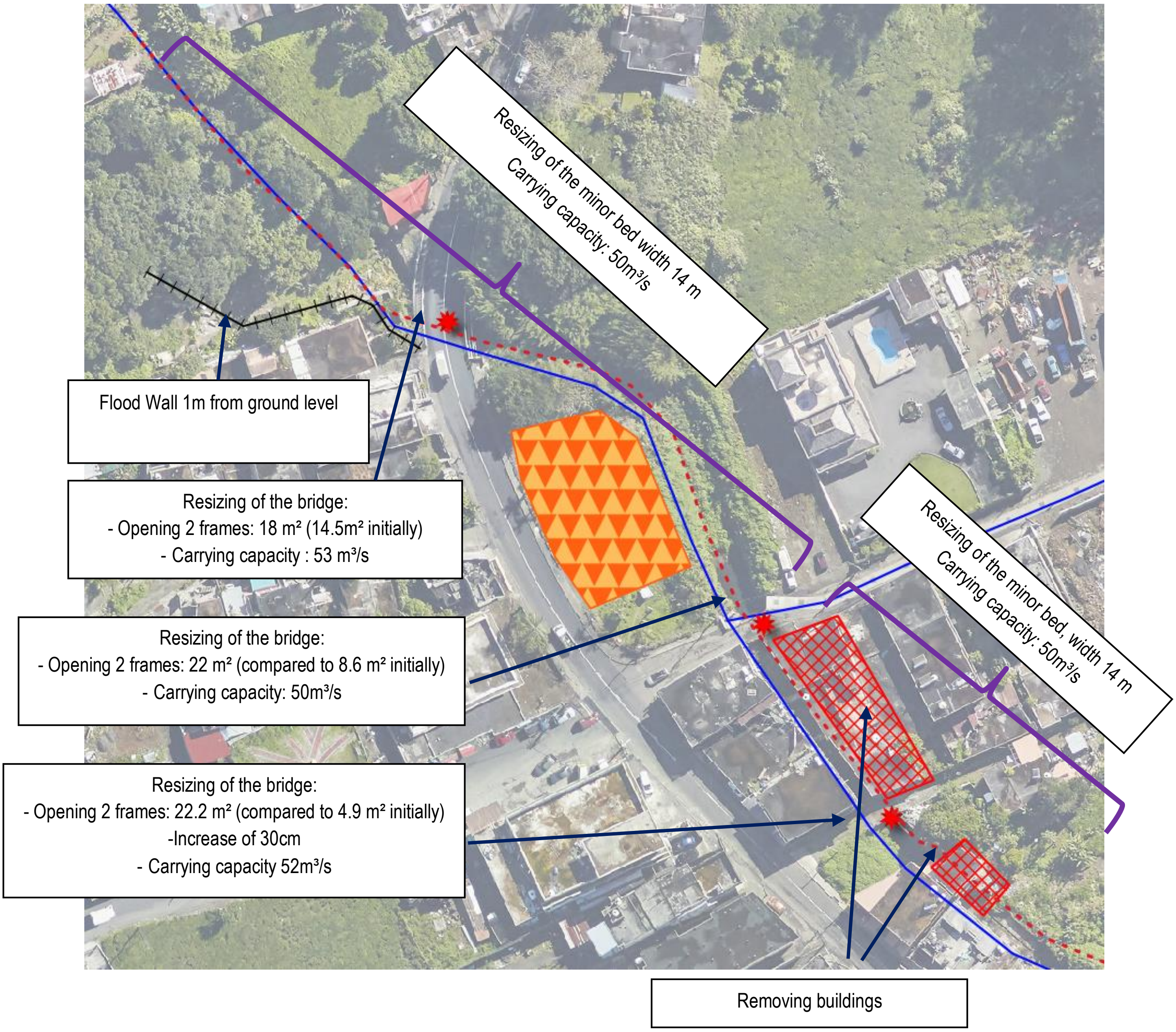


Figure 25: Nouvelle France - ER2C recommendations – West Drain

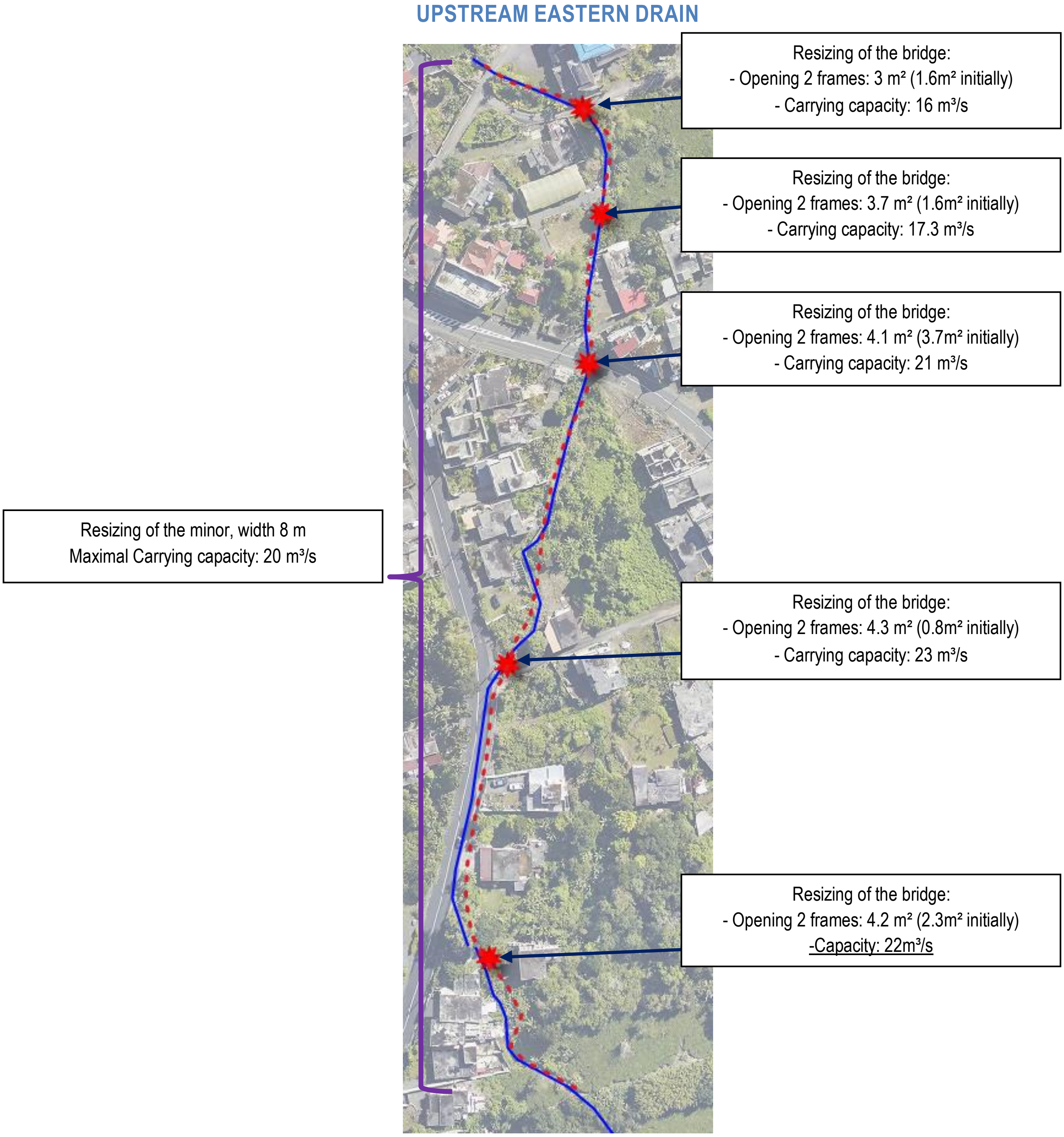


Figure 26: Nouvelle France - ER2C recommendations – Eastern Drain

RECOVERY OF MINOR BED CONFLUENCE AND DOWNSTREAM

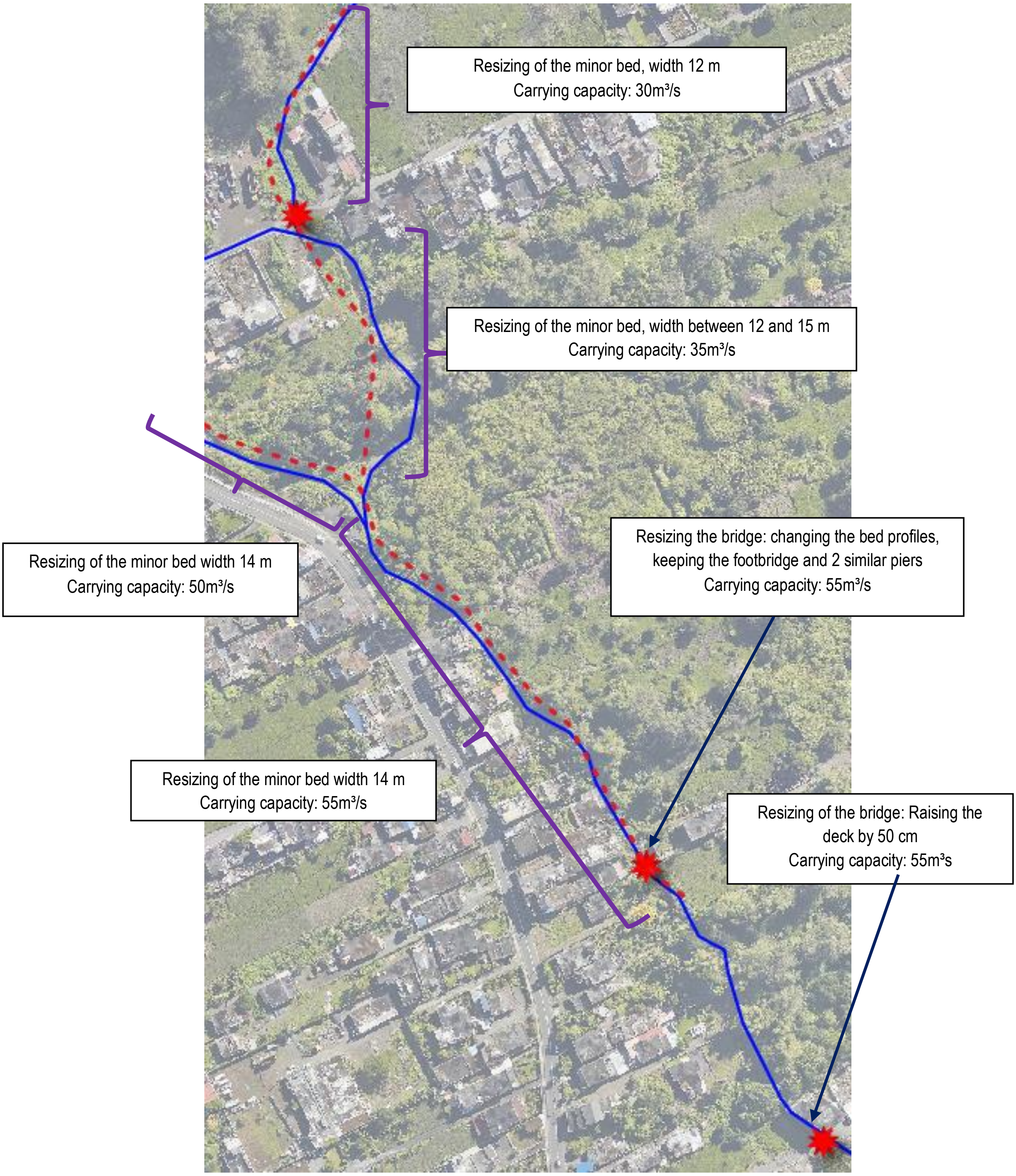


Figure 27: Nouvelle France - ER2C recommendations – Bed confluence and downstream

The ER2C dwells within a section of this sector. Modelling study of measures recommended within this section shows no overflow.

3.1.4.2 Additional recommendations

Proposals for the whole sector follows.

As discussed in D5.1, the first part of the Land Drainage Masterplan, Sustainable solutions no longer seek the quickest way of channelling stormwater into a river or watercourse as these require a large infrastructure. They consist nowadays in finding ways and means of controlling peak flows at source and breaking the peak flows as much as possible through retardation basins, flood infiltration or flood expansion zones along or off watercourses, terracing, vegetable cover and the like prior to releasing them in a controlled manner into the drainage infrastructure. It is equivalent to releasing the same quantity of water but over a longer period of time.

At source control stormwater management for mitigating the impacts of urbanisation on baseflow should become a basic design principle.

The various proposed options to attenuate peak flows upstream are summarized below and described more illustratively on the master drawing, with extracts reproduced herein where relevant. Inconvenience and damage due to floods occur in this sector even during rainfalls of higher frequencies (as low as return period of 2 to 5 years) mostly because of encroachment into the river section and construction across major drainage axes, but also because of the large expanse of undeveloped and agricultural land upstream of the urbanised area, producing high peak flows even during low rainfall intensity.

A. Flow attenuation along Rivière La Chaux

A1: Flood detention basin at location (558538.24, 7747338.77) across the upper catchment downstream of the Chartreuse tea factory of net capacity 187,000m³ involving construction of a dyke across the river of length 330 m and maximum height 5 m.

The physical characteristics of the basin are as follows:

Table 9: Physical characteristics A1 – La Chaux

	A1
Catchment Area (ha)	31.8
Foundation level (m amsl)	453
Top water level (TWL)	457
Water depth (m)	4
Area to TWL (m^2)	37,418
Capacity (m^3)	187,000
Crest level (m amsl)	458
Dyke height (m)	5
Dyke length (m)	330

This will break the peak flow as follows:

Table 10: Peak flow along Rivière La Chaux

Return period (years)	**10**	**25**	**50**	**100**
Peak flow (m^3/s) (without structure)	17.4	23.9	28.4	34.5
Control outflow (m^3/s)		10.6	13	15.7
Resultant Peak Outflow (m^3/s)	8.5	10.6	13.0	15.7
Percentage Peak Flow Reduction (%)	51	55	54	54

The discharge structure has a capacity of 22 m³/s which corresponds to the capacity of the river bed downstream. It is a 4m wide x 1.3m high culvert.

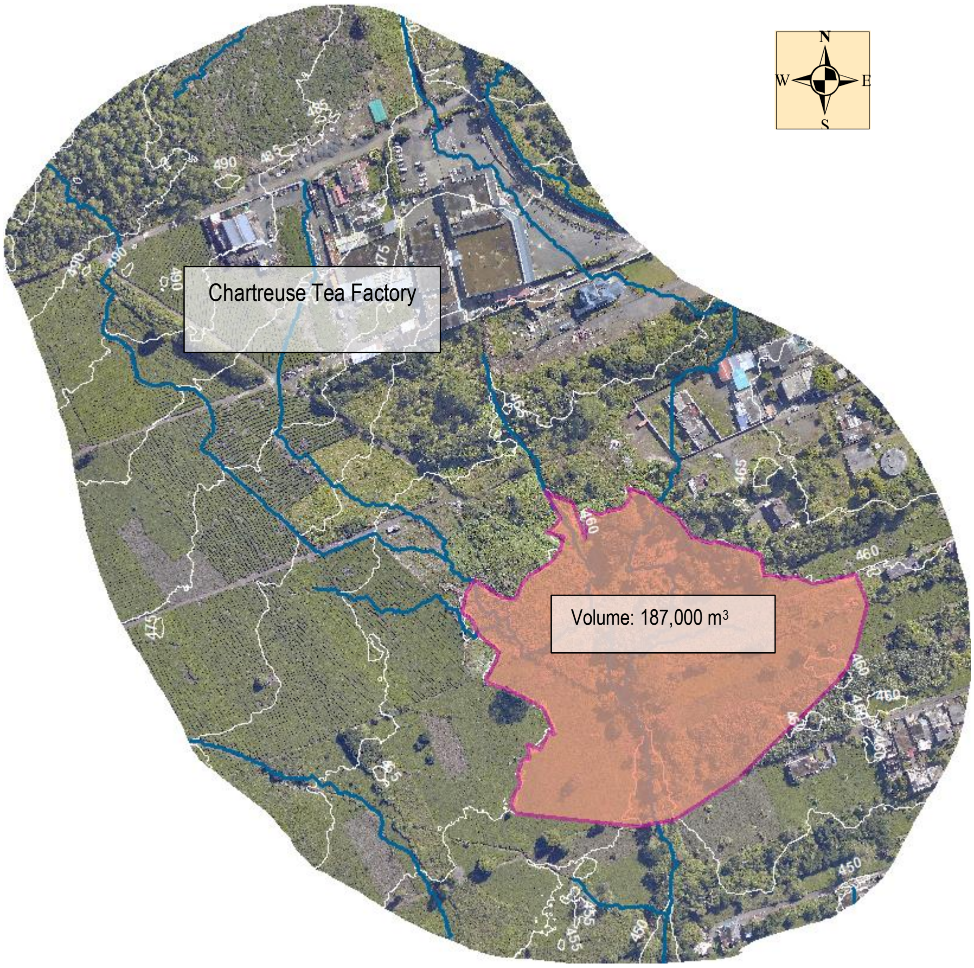


Figure 28: Flood detention basin downstream of the Chartreuse tea factory

B. Flow attenuation along the western tributary of Rivière La Chaux

Two flood attenuation basins upstream of Kanpur Lane:

B1: Dyke at location (558347.98, 7747116.61) of length 95m, maximum height of 3 m, and of net capacity 10,000m³. The basin intercepts a catchment area of 16 ha.

The basin discharge is a 0.75m wide x 0.4m high culvert which has a capacity of 0.5m³/s

Table 11: Peak flow along the western tributary of Rivière La Chaux

Return period (years)	10	25	50	100
Peak flow (m³/s) (without structure)	0.6	1.5	2.3	3.3
Control outflow (m³/s)	0.1	0.1	0.3	0.5
Resultant Peak Outflow (m³/s)	0.1	0.1	0.3	0.5
Percentage Peak Flow Reduction (%)	83	93	87	84

B2: Dyke at location (5587436.88, 7746930.48) of length 120m, maximum height of 3 m and of net capacity 13,500 m³.

The basin discharge is a 2m wide x 1.2m high culvert with a capacity of 17m³/s. For the 100- year flood and the 50-year flood, the resultant peak flow is higher than the control outflow because the water overflows the top of the dyke.

Table 12: Peak flow along the western tributary of Rivière La Chaux

Return period (years)	10	25	50	100
Peak flow (m^3/s) (without structure)	11.7	19.8	25.0	33.1
Control outflow (m^3/s)	10.5	17	17.1	17.1
Resultant Peak Outflow (m^3/s)	10.5	16.8	21.5	27.8
Percentage Peak Flow Reduction (%)	10	15	14	16

Figure 29: Flood detention basin upstream of Kanpur Road

The physical characteristics of the basins are as follows:

Table 13: Physical characteristics B1 and B2 – La Chaux

	B1	B2
Catchment Area (ha)	16	81
Foundation level (m amsl)	450.5	445
Top water level (TWL)	452.5	447
Water depth (m)	2	2
Area to TWL (m^2)	3,338	4,497
Capacity (m^3)	10,000	13,500
Crest level (m amsl)	453.5	443
Dyke height (m)	3	3
Dyke length (m)	95	120

C. Cut off drain upstream of Kanpur Lane into Rivière La Chaux to divert flow from a major drainage axis which presently cuts into a built-up area.

Dimension: Stone masonry cut off drain, average hydraulic slope 1%, sloping face 1H:3V, 3.0 m x 1.2 m deep

The carrying capacity of the cut off drain is 10m³/s (obtained through modelling), which represents 99% of the incoming peak flow that the drain is able to intercept.

The modelling in the projected situation shows that this measure has no negative impact on the urbanised areas near the main natural drain (Carrying capacity 30 m³/s with adequate maintenance).

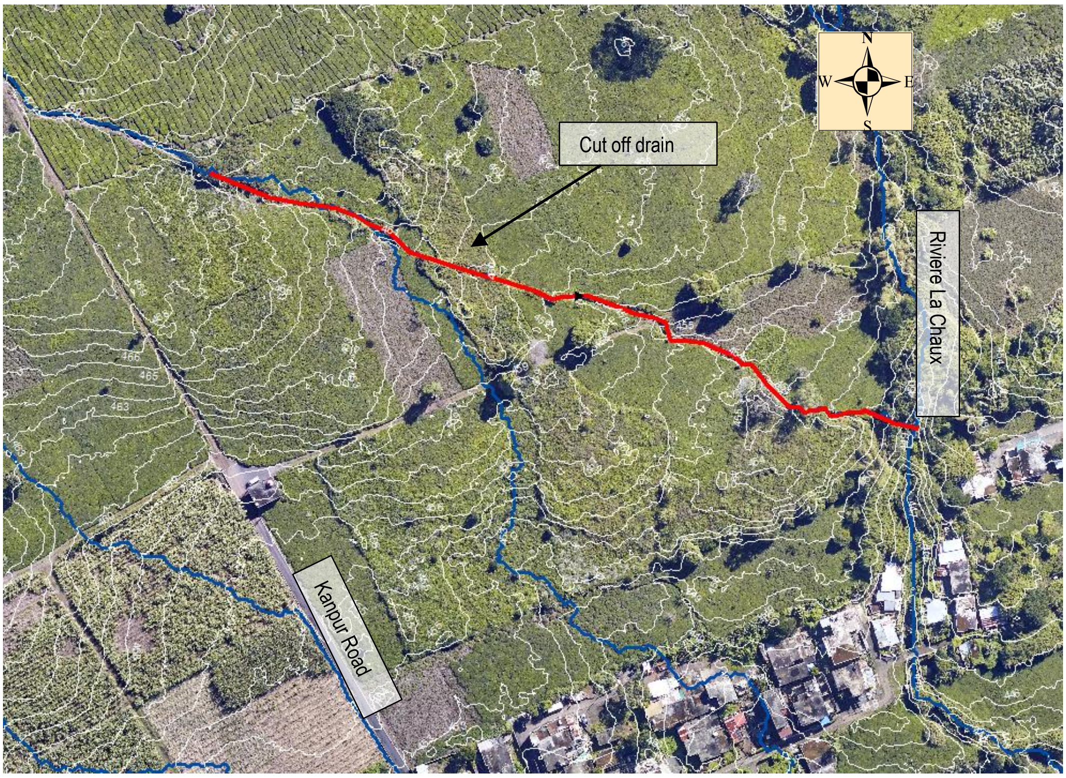


Figure 30: Cut off drain upstream of Kanpur Road

D. Flood attenuation along eastern tributary of Rivière La Chaux

D1: A flood attenuation basin at location (558762.90, 7747772.91), upstream of the junction A10/A9 roads of net capacity 155,000m³, involving the construction of a dyke of length 310m and maximum height of 5 m.

The basin discharge is a 1.5m wide and 0.7m high culvert with a theorical capacity of 2.7m³/s.

Table 14: Peak flow along the eastern tributary of Rivière La Chaux at D1

Return period (years)	**10**	**25**	**50**	**100**
Peak flow (m3/s) (without structure)	1.8	4.2	6	8.4
Control outflow (m3/s)	0.6	1.8	2.4	2.8
Resultant Peak Outflow (m3/s)	0.6	1.8	2.4	2.8
Percentage Peak Flow Reduction (%)	66	57	60	66

D2: A flood attenuation basin at location (558891.22,7747126.01), upstream of Aubeeluck Lane of net capacity 14,800m³, involving the construction of a dyke of length 90m and a maximum height of 3 m.

The basin discharge is a 2.5m wide and 1.5m high culvert with a theorical capacity of 21 m³/s

Table 15: Peak flow along the eastern tributary of Rivière La Chaux at D2

Return period (years)	**10**	**25**	**50**	**100**
Peak flow (m³/s) (without structure)	15.2	17.4	20.5	25.5
Control outflow (m³/s)	11.1	14.5	16.2	20.1
Resultant Peak Outflow (m³/s)	11.1	14.6	16.4	20.3
Percentage Peak Flow Reduction (%)	27	16	20	20

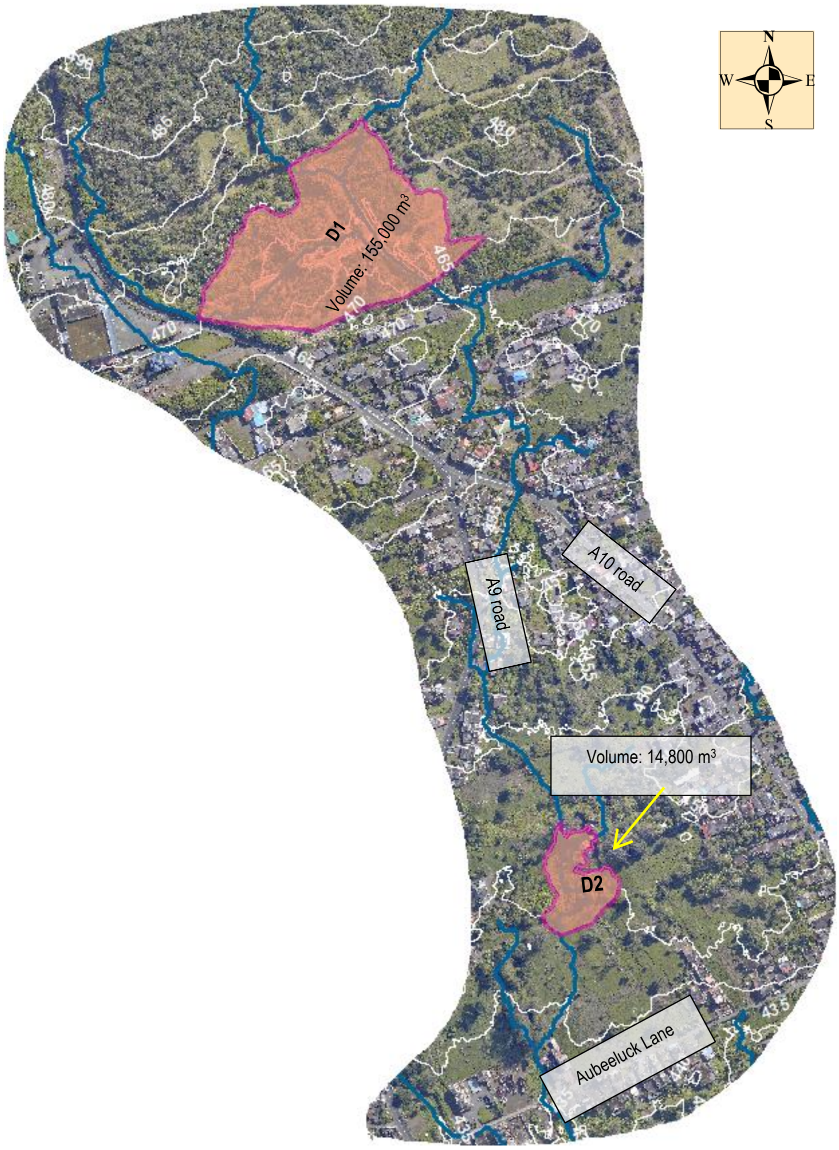


Figure 31: Flood attenuation along eastern tributary of Rivière La Chaux

The physical characteristics of the basins are as follows:

Table 16: Physical characteristics D1 and D2 – La Chaux

	D1	D2
Catchment Area (ha)	63	86
Foundation level (m amsl)	465	442
Top water level (TWL)	469	444
Water depth (m)	4	2
Area to TWL (m^2)	30,924	4,931
Capacity (m^3)	355,000	14,800
Crest level (m amsl)	470	445
Dyke height (m)	5	3
Dyke length (m)	310	90

E. East side of A10 road

E1: A diversion drain on the east side of A10 road to re-direct flow from the water course along A10 road into another drainage axis flowing mostly across undeveloped land. Stone masonry cut off drain, sloping face 1H:3V, 3.0 m x 1.8 m deep. The carrying capacity of this diversion drain is 20 m^3/s and and its slope is on average 0.8%.

E2: A small flood attenuation basin at location (559405.84,7747174.96) of net capacity 15,500 m^3 involving the construction of a dyke of length 110 m and a maximum height of 3 m in order not to increase the present flow across the built up area downstream as a resulting the cut- off drain.

The physical characteristics of the basins are as follows:

Table 17: Physical characteristics E2 – Nouvelle France

	E2
Catchment Area (ha)	18
Foundation level (m amsl)	440.5
Top water level (TWL)	442.5
Water depth (m)	2
Area to TWL (m^2)	5,198
Capacity (m^3)	15,500
Crest level (m amsl)	443.5
Dyke height (m)	3
Dyke length (m)	110

The basin discharge is a 0.9m wide and 0.4m high culvert with a theorical capacity of 0.6m^3/s.

Table 18: Peak flow along the east side of A10 road

Return period (years)	**10**	**25**	**50**	**100**
Peak flow (m^3/s) (without cut-off drain)	0	0	0	0
Peak flow (m^3/s) (with cut-off drain)	2.3	2.5	2.6	2.7
Basin Control outflow (m^3/s)	0.1	0.3	0.6	0.7
Resultant outflow (m^3/s)	0.1	0.3	1.1	1.2
Percentage Peak Flow Reduction (%)	96	88	58	56

For the 100- year flood and the 50-year flood, the resultant peak flow is higher than the control outflow because the water overflows via the spillway of the dyke.

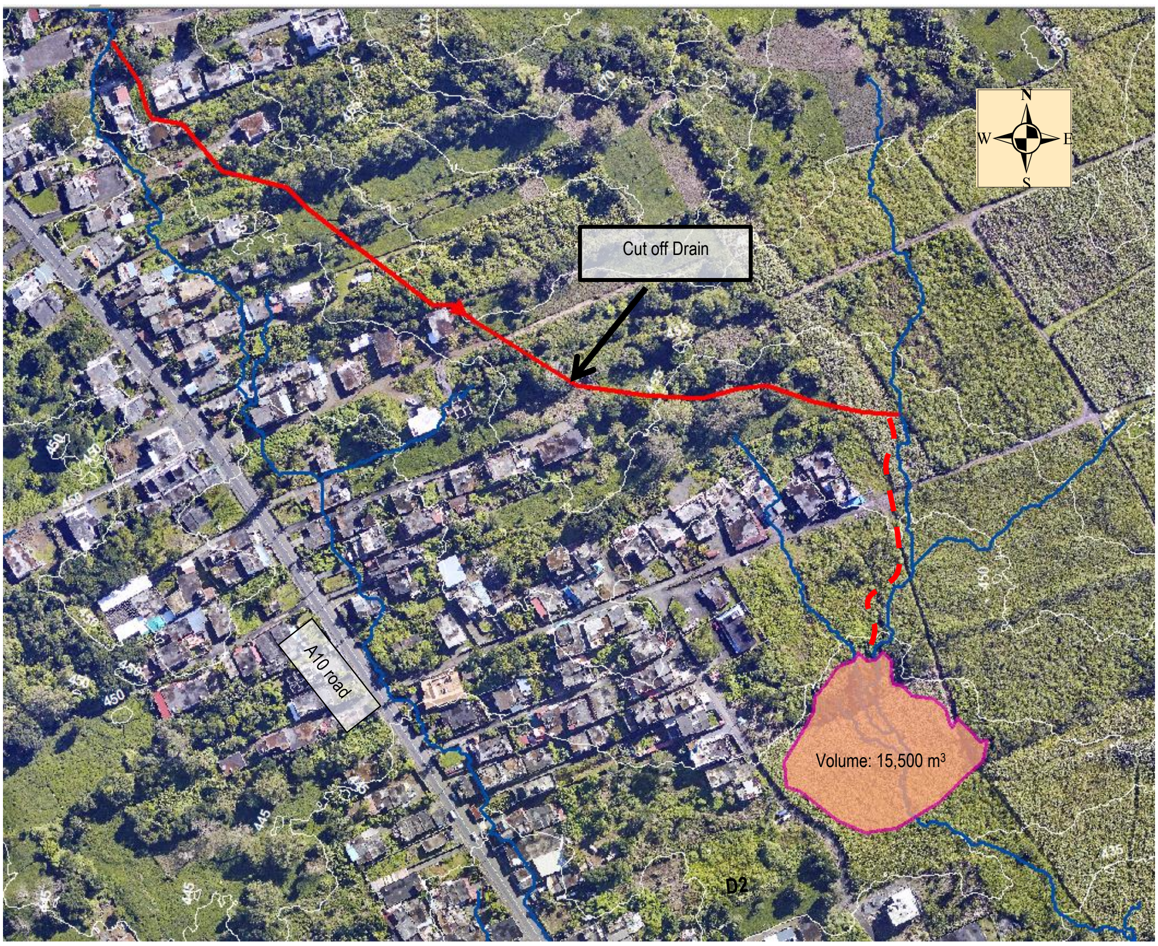


Figure 32: Cut off drain & flood attenuation on the east side of A10 road

F. **Diversion drain** at location (559467.62, 7746508.83) upstream of Mr. Bissonee's house to re-direct flow away from residential properties back into the **natural** drainage axis, length 175m.

Stone masonry cut off drain, sloping face 1H:3V, 1.5 m x 1.0 m deep. The carrying capacity of this diversion drain is 10m³/s and and its slope is on average 3 %.

Figure 33: Cut off drain upstream of Mr. Bissonee's house

G. **NHDC**

G1: A flood attenuation basin, upstream of NHDC site at location (558519.18, 7746515.58) of net capacity 34,460 m³ involving the construction of a dyke of length 160 m and a maximum height of 3 m.

The physical characteristics of the basins are as follows:

Table 19: Physical characteristics G1 – Nouvelle France

	G1
Catchment Area (ha)	27
Foundation level (m amsl)	441
Top water level (TWL)	443
Water depth (m)	2
Area to TWL (m^2)	11,487
Capacity (m^3)	34,460
Crest level (m amsl)	444
Dyke height (m)	3
Dyke length (m)	160

The basin discharge is a 1m wide x 0.7m high culvert with a theorical capacity of 1.6m³/s.

Table 20: Peak flow along the east side of A10 road

Return period (years)	10	25	50	100
Peak flow (m³/s) (without basin)	8.3	16.3	20.7	25.2
Basin Control outflow (m³/s)	1.6	1.6	1.6	1.6
Resultant outflow (m³/s)	1.6	8.3	12.2	15.5
Percentage Peak Flow Reduction (%)	81	49	41	38

For the 100- year flood, the 50-year flood and the 25 - year flood, the resultant peak outflow is higher than the control outflow because the water overflows via the spillway of the dyke.

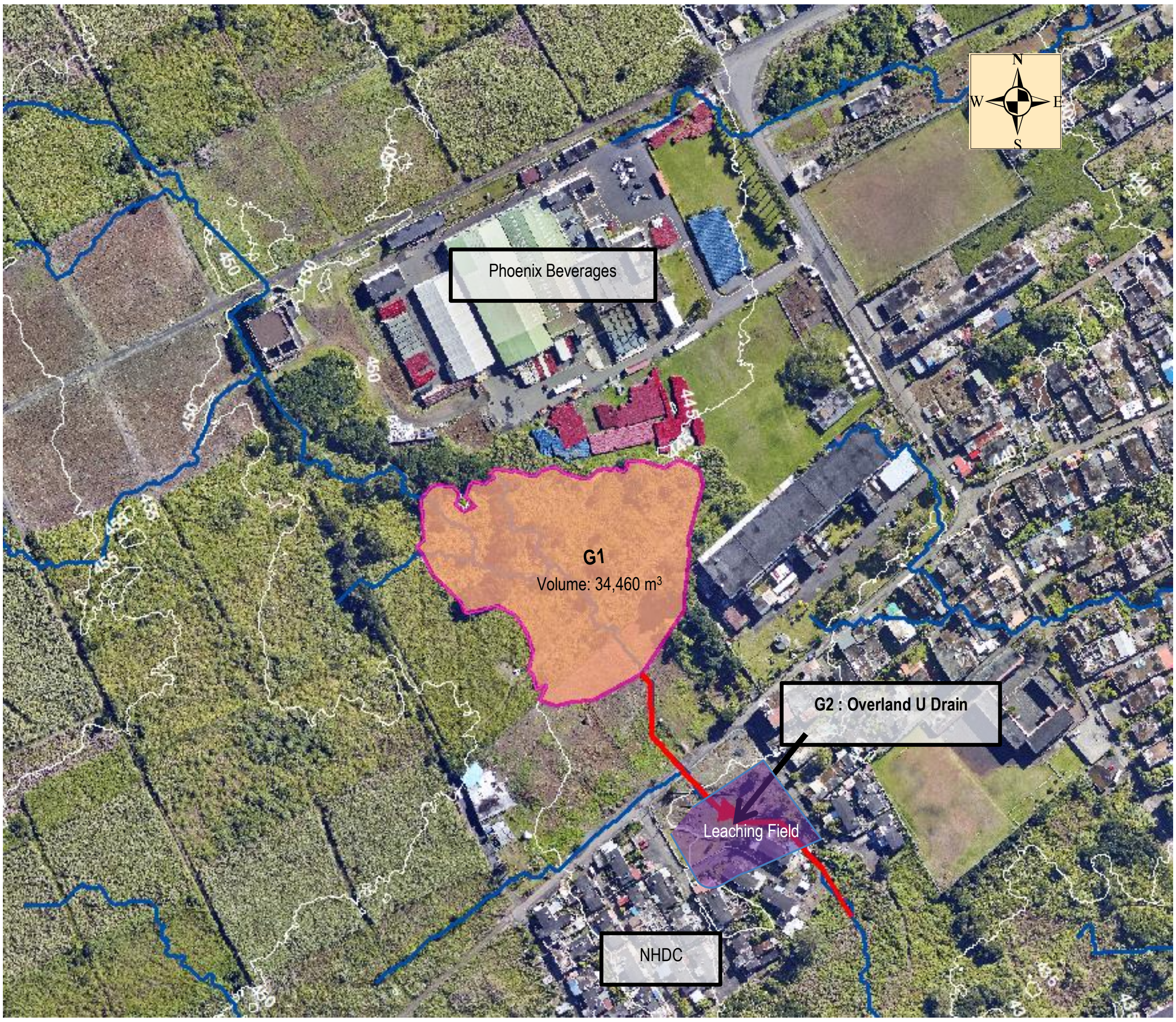


Figure 34: Flood attenuation upstream of NHDC Site & Overland U Drain

G2: An overland U Drain built over the existing leaching field to divert overland flow away from the leaching field back into the natural drainage axis (sloping face 1H:3V, 3 m x 2.0 m deep). The carrying capacity is 16 m^3/s with a slope of 3%.

H. Nav Hind Lane (V)

Gibb's proposal to divert the drain along Nav Hind Lane at point V into the roadside drain on the A9 road. Endorsed in the ER2C report should be reviwed if not already implemented. The drain should be allowed to continue its course through the twin 750mm diameter pipe culvert and along the earthen drain behind the two houses which had been built over the natural drainage path. The carrying capacity of this diversion drain is $10m^3/s$ and the slope is 4% on average.

I. Water accumulation at specific isolated low-lying locations to be addressed locally.

3.1.5 MAIN IMPACTS, COST AND COST-BENEFIT

For ER2C recommendations:

- Main tributary around the tea factory: The capacity of the existing watercourse is sufficient, which does not require the need for a diversion structure. However, in order to mitigate the risk of debris and overflow, it will be necessary to install a set of vertical combs.
- Positive impacts:
  - A reduction of around 30 cm at Q100 and Q10 in water levels in the area under study which is in the confluence zone.
  - The improvements will attenuate peak flow in the secondary drains downstream but will not completely check overflows. Constrictions to drains imposed by buildings and other encroachments have to be eliminated.;

The following table summarises the measures retained and their associated costs. The Icoalisation of the proposed measures is included in Annex 1.

The comparative flood maps and the cost-benefit analyses together with cost details are given in Annexes 2 and 3.

The flood maps for the current situation (ie " Do Nothing Scenario") are attached in Annex 0: they are also attached to Deliverable D3.

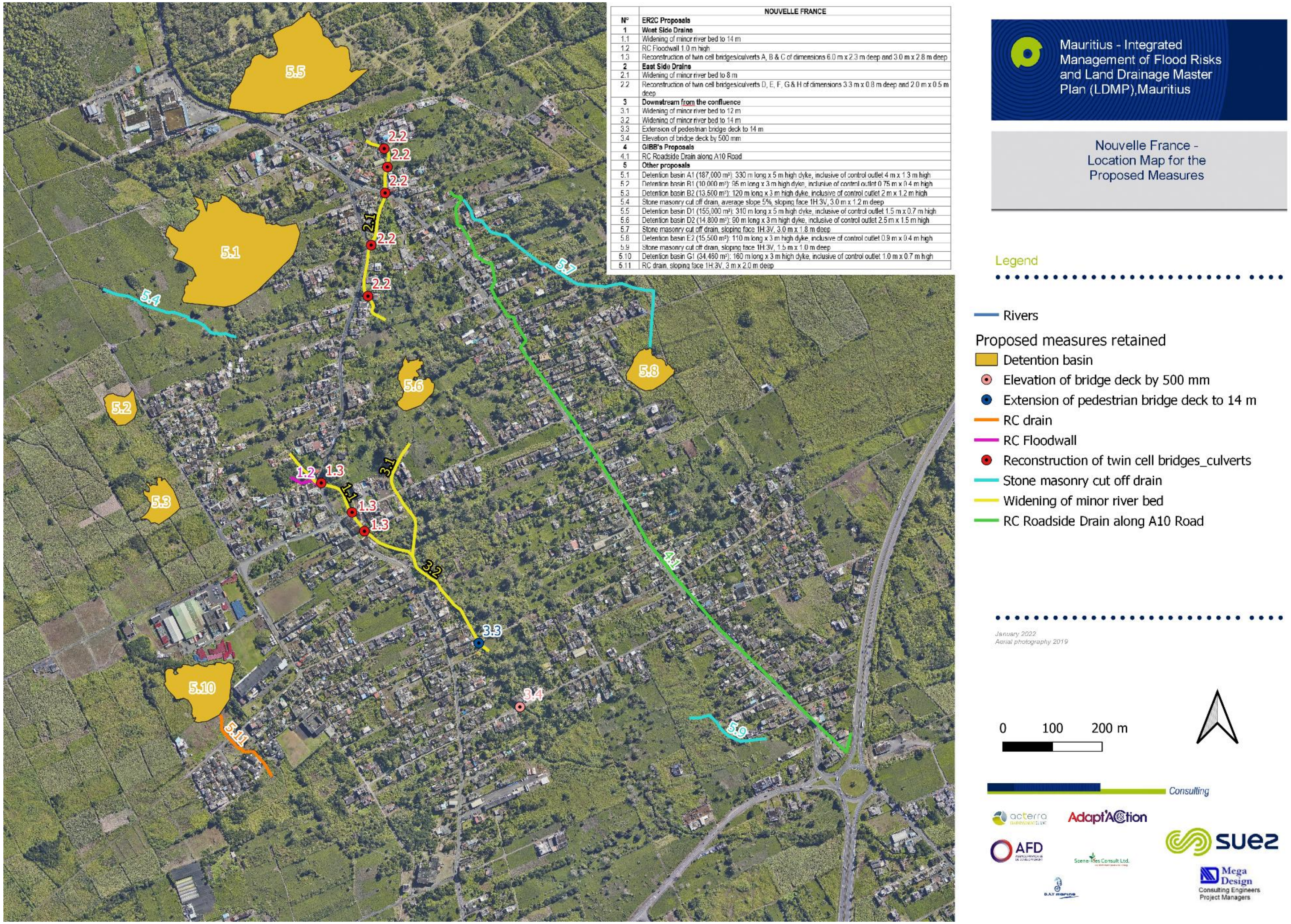


Figure 35: Nouvelle France – Location map for proposed measures

Table 21: Nouvelle France – Measures and costs

	NOUVELLE FRANCE	Unit	Quantity	Rate	Amount
N°	**ER2C Proposals**				
1	**West Side Drains**				
1.1	Widening of minor river bed to 14 m	m	330	25 000	8 250 000
1.2	RC Floodwall 1.0 m high	m	60	16 500	990 000
1.3	Reconstruction of twin cell bridges/culverts A, B & C of dimensions 6.0 m x 2.3 m deep and 3.0 m x 2.8 m deep	No	3	26 000 000	78 000 000
	Sub Total				**87 240 000**
2	**East Side Drains**				-
2.1	Widening of minor river bed to 8 m	m	400	8 000	3 200 000
2.2	Reconstruction of twin cell bridges/culverts D, E, F, G & H of dimensions 3.3 m x 0.8 m deep and 2.0 m x 0.5 m deep	No	5	15 000 000	75 000 000
	Sub Total				**78 200 000**
3	**Downstream from the confluence**				-
3.1	Widening of minor river bed to 12 m	m	85	25 000	2 125 000
3.2	Widening of minor river bed to 14 m	m	400	25 000	10 000 000
3.3	Extension of pedestrian bridge deck to 14 m	No	1	5 000 000	5 000 000
3.4	Elevation of bridge deck by 500 mm	No	1	3 400 000	3 400 000
	Sub Total				**20 525 000**
4	**GIBB's Proposals**				
4.1	RC Roadside Drain along A10 Road	m	1450	25 000	**36 250 000**
5	**Other proposals**				
5.1	Detention basin A1 (187,000 m^3): 330 m long x 5 m high dyke, inclusive of control outlet 4 m x 1.3 m high	Sum			31 600 000
5.2	Detention basin B1 (10,000 m^3): 95 m long x 3 m high dyke, inclusive of control outlet 0.75 m x 0.4 m high	Sum			9 000 000
5.3	Detention basin B2 (13,500 m^3): 120 m long x 3 m high dyke, inclusive of control outlet 2 m x 1.2 m high	Sum			11 500 000
5.4	Stone masonry cut off drain, average slope 5%, sloping face 1H:3V, 3.0 m x 1.2 m deep	m	295	32 000	9 440 000
5.5	Detention basin D1 (155,000 m^3): 310 m long x 5 m high dyke, inclusive of control outlet 1.5 m x 0.7 m high	Sum			29 000 000
5.6	Detention basin D2 (14,800 m^3): 90 m long x 3 m high dyke, inclusive of control outlet 2.5 m x 1.5 m high	Sum			9 000 000
5.7	Stone masonry cut off drain, sloping face 1H:3V, 3.0 m x 1.8 m deep	m	450	35 000	15 750 000
5.8	Detention basin E2 (15,500 m^3): 110 m long x 3 m high dyke, inclusive of control outlet 0.9 m x 0.4 m high	Sum			10 000 000
5.9	Stone masonry cut off drain, sloping face 1H:3V, 1.5 m x 1.0 m deep	m	175	25 000	4 375 000
5.10	Detention basin G1 (34,460 m^3): 160 m long x 3 m high dyke, inclusive of control outlet 1.0 m x 0.7 m high	Sum			15 000 000
5.11	RC drain, sloping face 1H:3V, 3 m x 2.0 m deep	m	175	98 500	17 237 500
	Sub Total				**161 902 500**
	Total				**384 117 500**
	ADD:				
	Provision for wayleave and Land Acquisition				50 000 000
	Relocation of Houses				40 000 000
	Relocation of services & traffic diversion				10 000 000
	Contingencies 15%				57 617 625
	Project Management 5%				19 205 875
	Grand Total				**560 941 000**

3.2 Bel Ombre

3.2.1 OVERVIEW

Bel Ombre is situated in the south of the island within a catchment of area 6 km^2, drained by River St. Martin and Rivière Citronniers. The catchment is sited between amsl 387 m and 1 m, with urbanisation located on a low lying area between the river and the coastal dune at elevations varying between 3.0 m and 1.0 m amsl. The coastal road is aligned over the crest of the dune. The catchment divided into 10 sub-catchments for the purpose of hydrological study has a long drainage path of approximately 5.6 km and a sharp topography on the upstream part (in excess of 10%) and a very mild one (1.74%) on its downstream coastal strip, over which coastal strip Bel Ombre village is sited.

Figure 36: Catchment area of Bel Ombre

Table 22: Bel Ombre – Physical Characteristics of individual Sub-catchments

Name	Area (ha)	Area (km²)	Low level (m)	High level (m)	Length	Slope (m/m)	Slope (%)
BO_BV01	119.02	1.19	16	345	2865	0.11	11.50
BO_BV02	71.02	0.71	16	384	3658	0.10	10.07
BO_BV03	37.20	0.37	1	118	1836	0.06	6.37
BO_BV04	63.82	0.64	1	69	1739	0.04	3.90
BO_BV05	7.99	0.08	0	6	547	0.01	1.01
BO_BV06	179.36	1.79	1	376	3693	0.10	10.14
BO_BV07	4.05	0.04	0	5	332	0.01	1.32
BO_BV08	4.02	0.04	1	3	133	0.02	1.74
BO_BV09	82.46	0.82	2	193	2102	0.09	9.08
BO_BV10	28.87	0.29	1	108	1017	0.11	10.55
BO_Global	**597.80**	**5.98**	**1**	**384**	**5565**	**0.07**	**6.90**

The flows obtained for Bel Ombre for return periods of 10, 25, 50 and 100 years are shown in the table below.

Table 23: Bel Ombre – Flows for sub-catchments and at outlet of catchment for return periods of 10, 25, 50 and 100 years

BVs	Q10 (m3/s)	Q25 (m3/s)	Q50 (m3/s)	Q100 (m3/s)
BO_BV01	17.39	24.21	28.64	33.38
BO_BV02	7.25	10.53	12.88	15.34
BO_BV03	7.07	9.45	10.92	12.42
BO_BV04	14.06	18.40	21.02	23.88
BO_BV05	1.53	2.07	2.37	2.70
BO_BV06	21.82	30.90	37.09	43.76
BO_BV07	0.94	1.28	1.44	1.62
BO_BV08	0.85	1.18	1.34	1.51
BO_BV09	14.15	19.48	22.98	26.79
BO_BV10	5.28	7.18	8.45	9.72
Outlet of Bel Ombre	**76.7**	**106.3**	**125.9**	**146.9**
Q /A (m³/s/km²)	***12.8***	***17.8***	***21.1***	***24.6***

3.2.2 DESCRIPTING OF EXISTING SITUATION

Bel Ombre is affected by flooding from Rivière St. Martin whose downstream part is colonised by mangroves.

The urban area comprising Cité EDC to the north and Cité Longtill to the south, both aggregating 350 housing units, is bounded by a flood wall on the north and east sides, to mitigate flooding. Stormwater is drained through a small ditch within the urban area, ending its course into the river. Another concrete drain is presently under construction. The flood wall is made of two sections, with the east side made of stone masonry and the north side, bordering the river, a rock revetment with a crest level of 600 mm above the existing ground level.

Stone Masonry

Transition From Stone Masonry to Rock revetment

Rock Revetment

Floodwall bordering urban area

The recently rehabilitated drain to the north eastern side of the site terminates into a twin box outlet, each of size 1000 mm wide by 1000 mm deep and with an invert level of 0.48 m. The flap valve made of robust steel frame and steel sheet weighs some 100 kg and is too heavy to operate, the differential head between accumulated water level within the estate and the river stage being too small to open it. No doubt it was dismantled and placed aside.

Existing Drain at start

Existing drain at exit into river

Twin box outlet

Dismantled Flap valve

The new drain under construction on the east side starts with two branches of internal widths 500 mm, with one having a starting depth of 80 mm and the other a depth of 350 mm, both culminating to 700 mm wide x 1500 mm deep at their junction and at the outlet into the river.

Drain 500mm wide x 80 mm deep

Drain 500mm wide x 350 mm deep

Drain Outlet 700 mm wide x 1500 mm deep into river

Drain under construction within Cite EDC

3.2.3 ANALYSIS

There is no reported incidence of the floodwall having been overtopped by the river stage, except for penetration via one breach into the stone masonry wall and oozing through the joints.

Model analysis, however, shows that for a 10 year flood recurrence period, the urbanized area of the estate gets flooded both by back flows from the St. Martin river through the drain and through over-topping of the flood wall.

Breach into stone masonry floodwall

Flooding within the estate occurs more frequently through direct rainfall accumulation. The river flow is tidal and flows to the sea at a mere 2 m per minute during low tide. Dredging of the river had recently been undertaken, with the dredged material left on its bank. Dredging below the river bed is equivalent to increasing only the dead storage of the river and will not lower the river stage on this tidal segment of the river. Stacking of dredged material on the river bank will rather decrease the river width and therefore the flood expansion volume.

Stacking of dredged material on river bank

3.2.4 *PROPOSED SOLUTIONS*

3.2.4.1 ER2C recommendations

Following the preliminary analyses carried out as part of the ER2C study, the measures described below were included to be investigated in the present project modelling study:

- Installation of a non-return valve at the junction between river St Martin and the urban drain outlet(s).
- Rehabilitation/raising of the existing wall (breach) in view of providing sufficient freeboard for a 100-year flood event.

Figure 37: Bel Ombre - ER2C recommendations

3.2.4.2 Additional options investigated and recommendations

Two other optins were investigated in view of attenuating peak discharage into the river segment bordering the estate and of lowering the river stage, at least during low tide, namely:

(i) Formalising the low land on the north bank of the river into a flood expansion zone by lowering the 2.0 m contour to 1.0 m.

(ii) Creation of a secondary outlet across the marshland bordering the access road to the golf estate and across the coastal road into the sea.

River crossing access road to golf estate

Wetland along access road

These options analysed through modelling were, however, deemed not to be that effective.

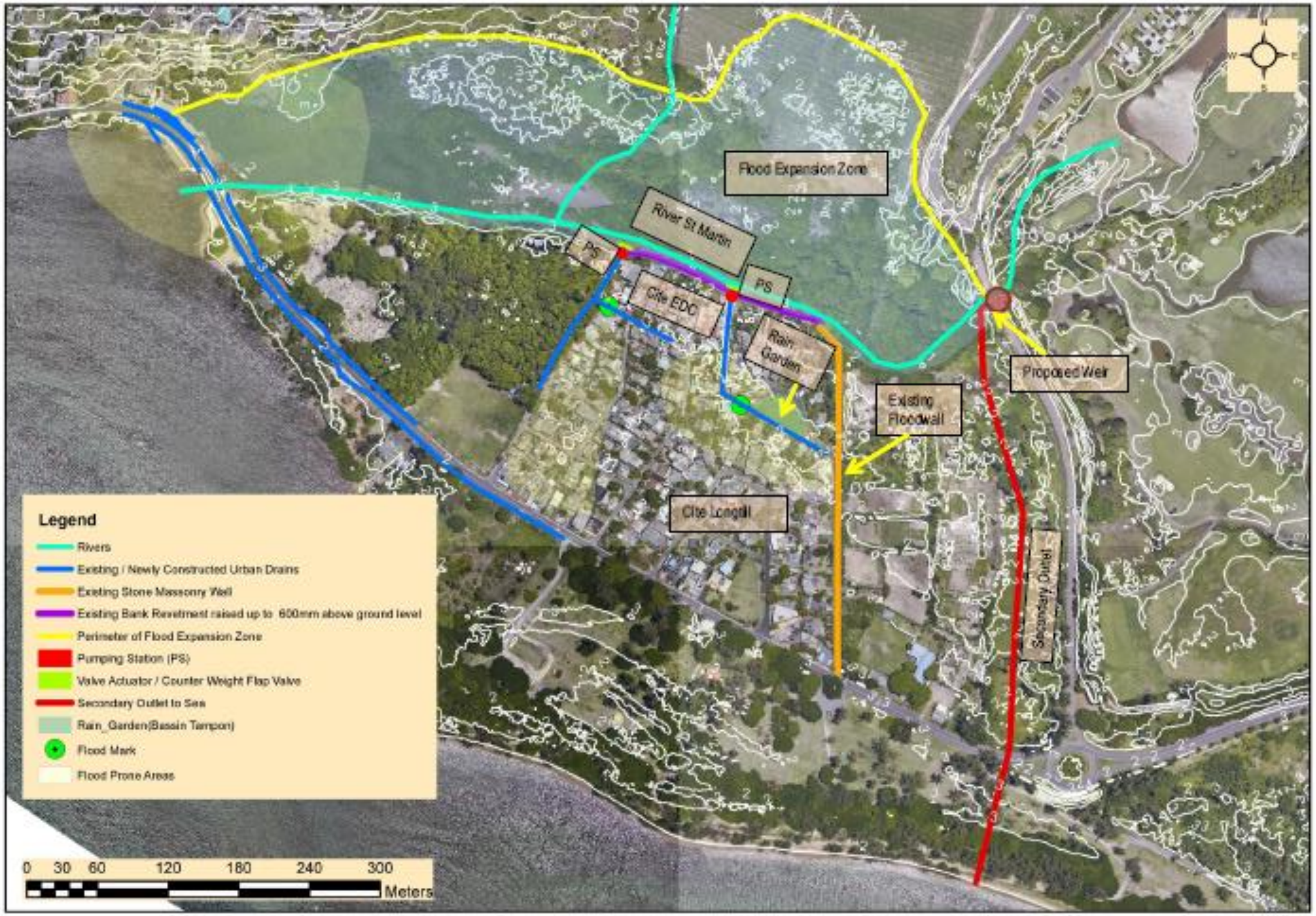


Figure 38:Investigated options and proposed works

The only other alternative to evacuate water accumulation within the estate is to provide a pumping system at both outlets, to be put into operation mode either manually or remotely during intense rainy spells when the river stage is too high to permit drainage by gravity.

Flap valves are not usually recommended for stormwater discharge because of the small differential hydraulic head and the risk of blockage by debris.

However, in circumstances where the stormwater outlet is submerged during floods, like at Bel Ombre, one of the following equipment may be considered:

- Lightweight flap valves with counter balancing weight
- Hand operated or actuated sluice gate equipped with sensors as a final recourse. The sensors are able to detect differential heads between the upstream and downstream faces of the gate and promt the gate to open or close accordingly.

Flap Valve

The bio-pond (rain garden) is located at the lowest section of the estate and no overflow into the river is possible without the risk of backflow. The biopond will rely on infiltration and evapotranspiration to dissipate retained water. In the event of significant local rainfall, the rain garden will overflow into the existing drain which terminates at the outlet into the river where a pumping station will be installed.

The recently constructed drains (Berry Lane) will serve the purpose of conveying residual surface water to the river when the river stage recedes.

3.2.5 MAIN IMPACTS, COST AND COST-BENEFIT

The flood wall raised by 50 cm, inclusive of a freeboard of 20 cm, will be sufficient to protect against a 100-year flood.

The following table summarises the measures retained and their associated costs. The works item number are cross referenced in the plans provided in Annex 1.

The comparative flood maps and the cost-benefit analyses together with cost details are given in Annexes 2 and 3.

The flood maps for the current situation (ie “ Do Nothing Scenario”) are attached in Annex 0: they are also attached to Deliverable D3.

The proposed measures will keep Bel Ombre FPAs area out of flood, regardless of the occurrencyof floods.

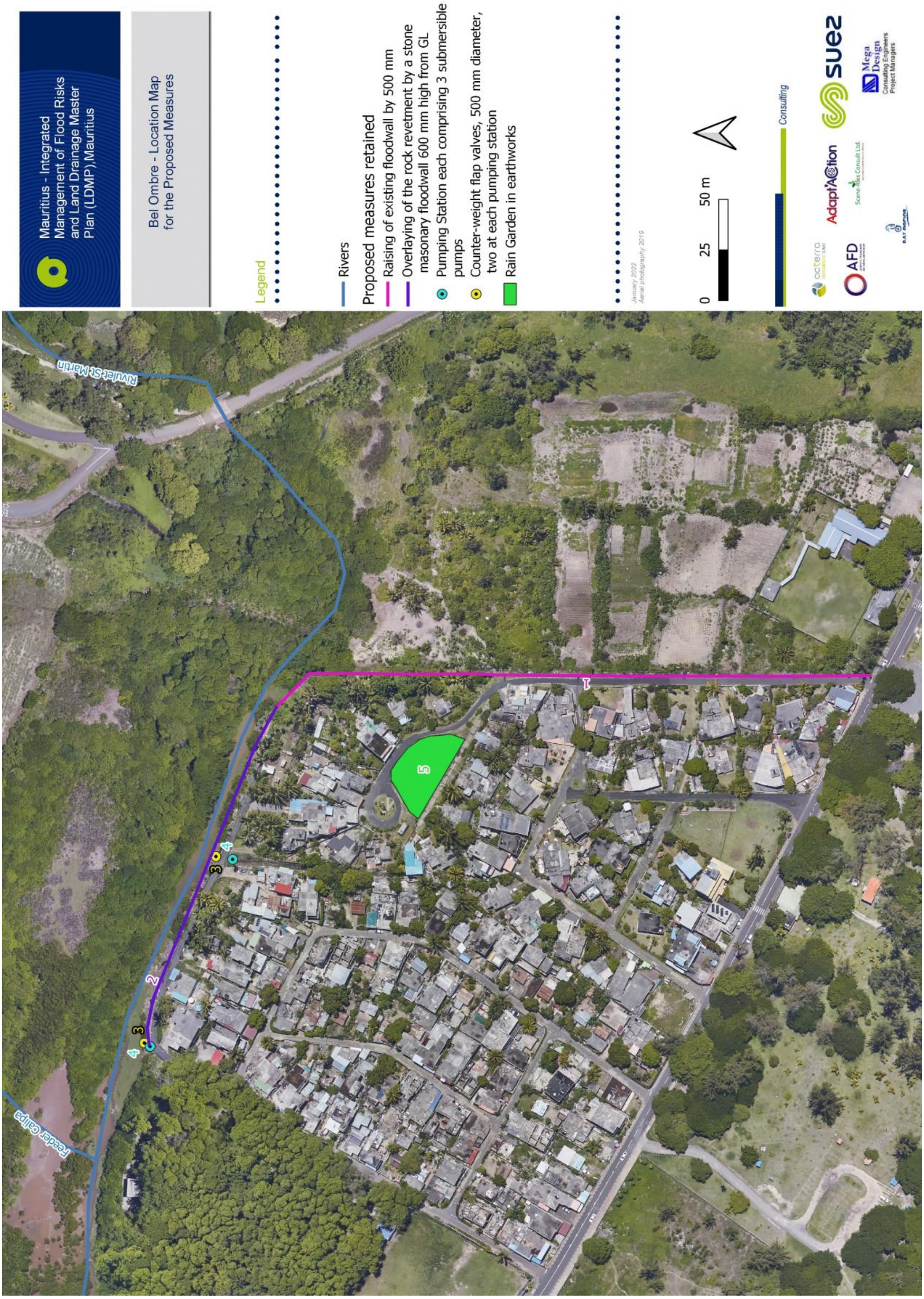


Figure 39: Belle Ombre – Location map for proposed measures

Table 24: Bel Ombre – Measures and costs

N°	BEL OMBRE	Unit	Quantity	Rate	Amount
1	Raising of existing floodwall by 500 mm **	m	300	6 000	**1 800 000**
2	Overlaying of the rock revetment by a stone masonry floodwall 600 mm high from GL	m	180	15 000	**2 700 000**
3	Installation of counter-weight flap valves, 500 mm diameter, two at each pumping station (Alternative; 2 no actuated sluice valves)	No	4	300 000	**1 200 000**
4	Pumping station, each comprising 3 submersible pumps @ 25 m^3/min inclusive of civil and electrical works	No	2	4 000 000	**8 000 000**
5	Construction of a rain garden * (Basin Tampon) in earthworks	m^3	800	1 000	**800 000**
6	Secondary outlet to sea	m	500		Not retained
7	Creation of flood expansion zone, approximate area 108,000 m^2	Sum			Not retained
	Total				**14 500 000**
	ADD:				
	Provision for wayleave and Land Acquisition				2 000 000
	Relocation of Houses and/or Services				2 000 000
	Contingencies 15%				2 175 000
	Project Management 10%				1 450 000
	Grand Total				**22 125 000**

* A buffer pond (basin tampon) contrary to a detention basin has no separate outlet (similar to a flood expansion zone or an air vessel in a pump assembly). A biopond has the same meaning as rain garden (see definitions in the main Report D5.1).

** including 200 mm freeboard

3.3 Grand Baie – Pereybere

3.3.1 OVERVIEW

Grand Baie/Pereybere catchment covers an area of 30 km^2 and because of its low lying coastal areas, it has been divided into 14 sub-catchments, with the smaller sub-catchments located on the coastal fringe.

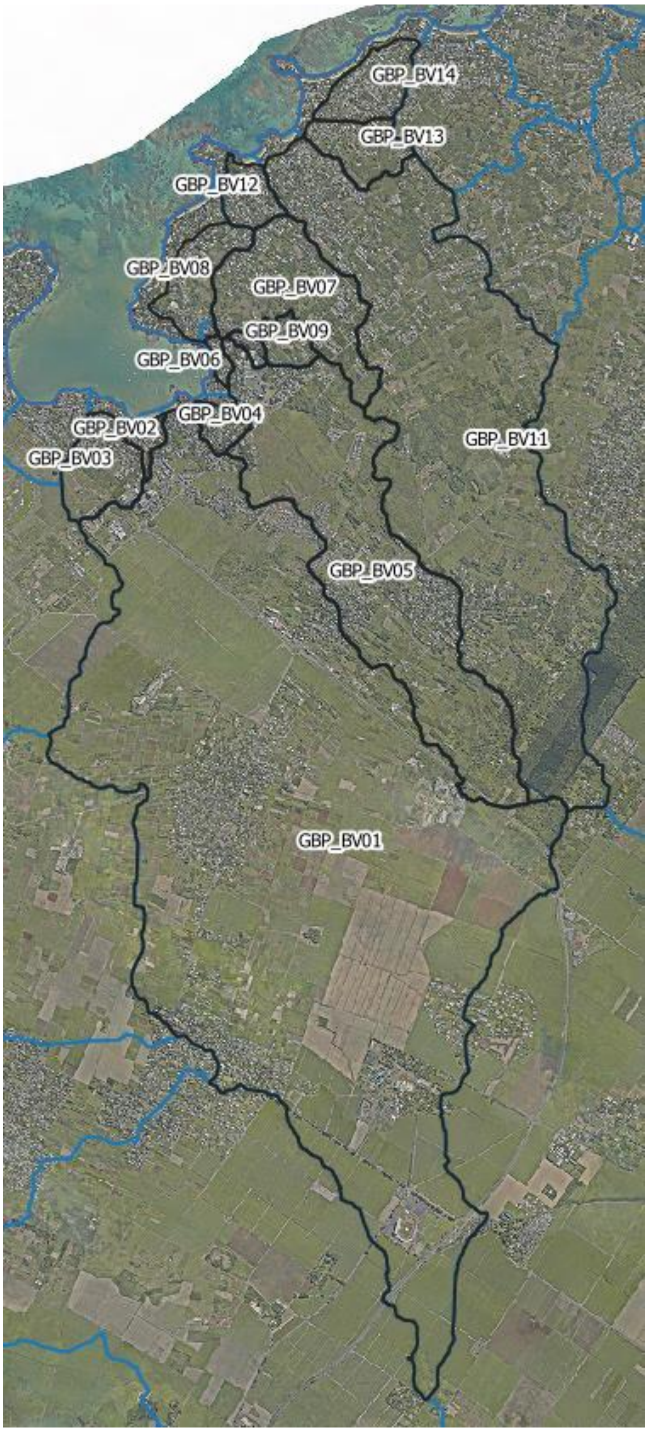


Figure 40: Sub-division of catchment area of Grand Baie / Pereybere into sub-catchments (Orthophoto 2019)

Except for Fond du Sac, urban development is concentrated over the coastal lowlands and around the wetlands, with higher grounds occupied by a vast expanse of agricultural lands interspersed with two small villages (Vale and Sottise).

There is no flowing watercourse within this catchment and wetlands and small waterlogged areas play an essential role in the storage and infiltration of run-off and the preservation of water quality in the lagoon. Stormwater runoff flows through mostly unmarked drains and via overland flow into the lagoon.

Table 25: Grand Baie / Pereybere – Physical Characteristics of individual Sub-catchments

Name	Area (ha)	Area (km²)	Low level (m)	High level (m)	Length	Slope (m/m)	Slope (%)
GBP_BV01	1628.11	16.28	2	140	10096	0.01	1.37
GBP_BV02	9.51	0.10	2	17	580	0.03	2.54
GBP_BV03	46.64	0.47	3	24	1110	0.02	1.85
GBP_BV04	14.75	0.15	2	16	576	0.03	2.54
GBP_BV05	346.14	3.46	0	77	5479	0.01	1.41
GBP_BV06	4.95	0.05	2	6	164	0.03	2.83
GBP_BV07	108.48	1.08	1	22	1923	0.01	1.07
GBP_BV08	42.24	0.42	1	9	492	0.02	1.51
GBP_BV09	17.05	0.17	1	13	325	0.04	3.56
GBP_BV10	4.96	0.05	0	10	232	0.04	4.23
GBP_BV11	675.99	6.76	1	81	9547	0.01	0.84
GBP_BV12	22.78	0.23	1	6	753	0.01	0.72
GBP_BV13	34.38	0.34	1	14	592	0.02	2.08
GBP_BV14	38.13	0.38	1	9	624	0.01	1.24

The flows obtained for Grand Baie/Pereybere for return periods of 10, 25, 50 and 100 years are shown in the table below.

Table 26: Grand Baie / Pereybere – Flows for sub-catchments and at outlet of catchment for return periods of 10, 25, 50 and 100 years

BVs	Q10 (m3/s)	Q25 (m3/s)	Q50 (m3/s)	Q100 (m3/s)
GBP_BV01	128.60	172.35	202.16	233.23
Q /A ($m^3/s/km^2$) BV01	*7.9*	*10.6*	*12.4*	*14.3*
GBP_BV02	1.55	1.93	2.22	2.41
GBP_BV03	6.84	8.76	9.92	11.08
GBP_BV04	2.28	2.95	3.27	3.73
GBP_BV05	31.63	42.25	49.28	56.72
Q /A ($m^3/s/km^2$) BV05	*11.2*	*14.5*	*16.8*	*19.2*
GBP_BV06	0.82	1.02	1.16	1.21
GBP_BV07	12.40	16.19	18.68	21.39
GBP_BV08	6.20	7.94	8.97	10.09
GBP_BV09	1.91	2.55	2.95	3.40
GBP_BV10	0.73	1.02	1.06	1.21
GBP_BV11	45.02	61.18	72.38	84.14
Q /A ($m^3/s/km^2$) BV011	*7.2*	*9.7*	*11.4*	*13.2*
GBP_BV12	3.37	4.28	4.85	5.49
GBP_BV13	4.74	6.11	6.96	7.90
GBP_BV14	5.74	7.23	8.23	9.22

For this sector, the (HECRAS) modelling does not work using the conventional rational formula, but simulates a rainfall of a specified intensity and generates a peak flow value by a distributed modelling (Q10 or Q100 etc), from which the sizing of the infrastructure is derived.

3.3.2 REGIONAL HYDROGEOMORPHOLOGY

Altitude ranges from sea level to 120m amsl at the upper end of the catchment. The landform is almost flat to gently undulating with slopes mostly less than 2 %.

The substrate is made of a thin and discontinuous layer of weathered and clayish facies sandwiched between an intermediate and recent lava flows. The recent lava layer is gently sloping and very permeable, which explains the quasi absence of rivers and the resurgence of ground water at different locations.

The coastal fringe is made of regosols, dark brown sand and loamy sand on light grey to very pale brown coral sand. The soil is highly permeable but the water table is high, which creates overland flows into the lagoon.

The hinterland is made of latosolic reddish prairie soils, brown on reddish brown silty clay loam with frequent gravels and stones and few boulders, well drained. The superficial soil is fairly shallow overlying the recent lava which accounts for a lower infiltration rate than at the coastal areas.

3.3.3 DESCRIPTING OF EXISTING SITUATION

The NDRRMC identified in its report of 2018 several flooded areas following the recurrent flooding problems in the following regions within the catchment area:

- Fond du Sac
- Grand Baie – Kapukaye, Camp Carol and La Croisette Roundabout
- Pereybere

Site inspections were undertaken by the Consultant's team on the catchment at the above mentioned vulnerable locations in particular.

3.3.3.1 Fond du Sac

Fond du Sac Village is devoid of any natural watercourse which could serve as a sink to any proposed drainage system. The only terminal outlet is the Grand Bay shoreline located some 3.5 km from the village.

Surface run-off emanates from the south east, mainly from:

- Bois Mangues and
- Butte aux Papayes

Additionally, numerous underground intrusions within the shallow basaltic formation conduct ground water from higher pervious ground into and across the village, adding to overland flow.

3.3.3.1.1 Problem areas

Problem areas identified in a design report by Mega Design and from the Consultant's own data collection are depicted in the figure below.

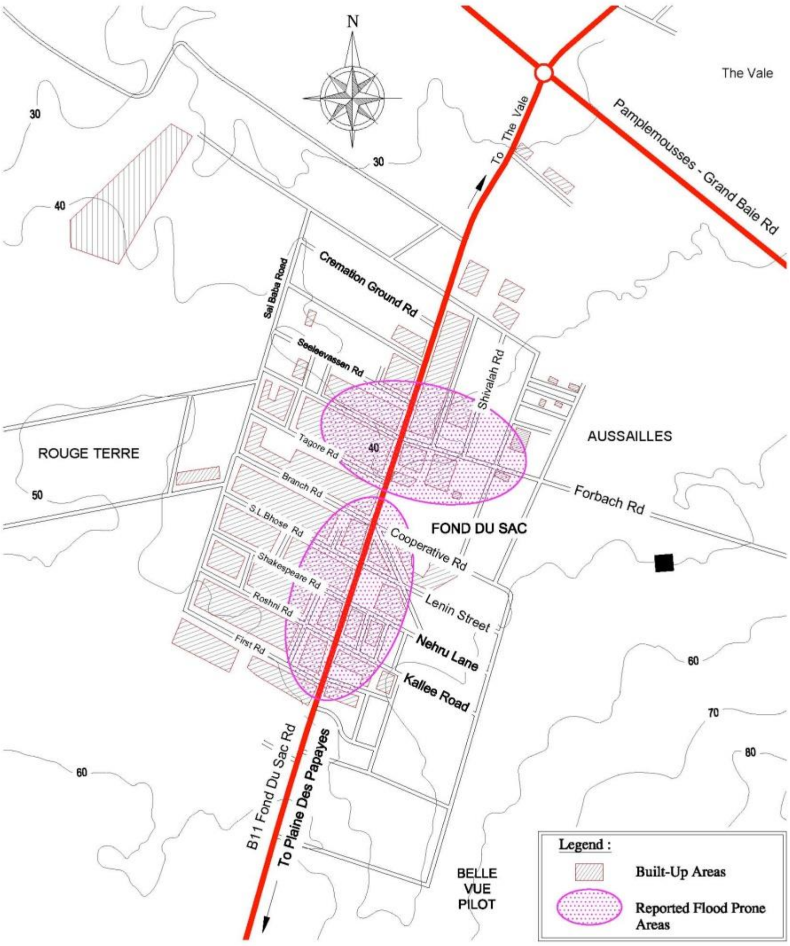


Figure 41: Problem areas at Fond du Sac

The most flood prone areas are located at the following spots:

(i) The football ground with floods emanating mostly from uphill of Lenin Street and which eventually inundates Subhash Chandra Bose and Shakespeare Roads

Football playground inundated

(ii) In front of the State Bank of Mauritius Branch along the B11 Fond du Sac Road

Road in front of Citizen Advice Bureau and SBM building

(iii) Kallee Road whereby houses had been built within a depression and across a natural drainage path as shown in the figure below.

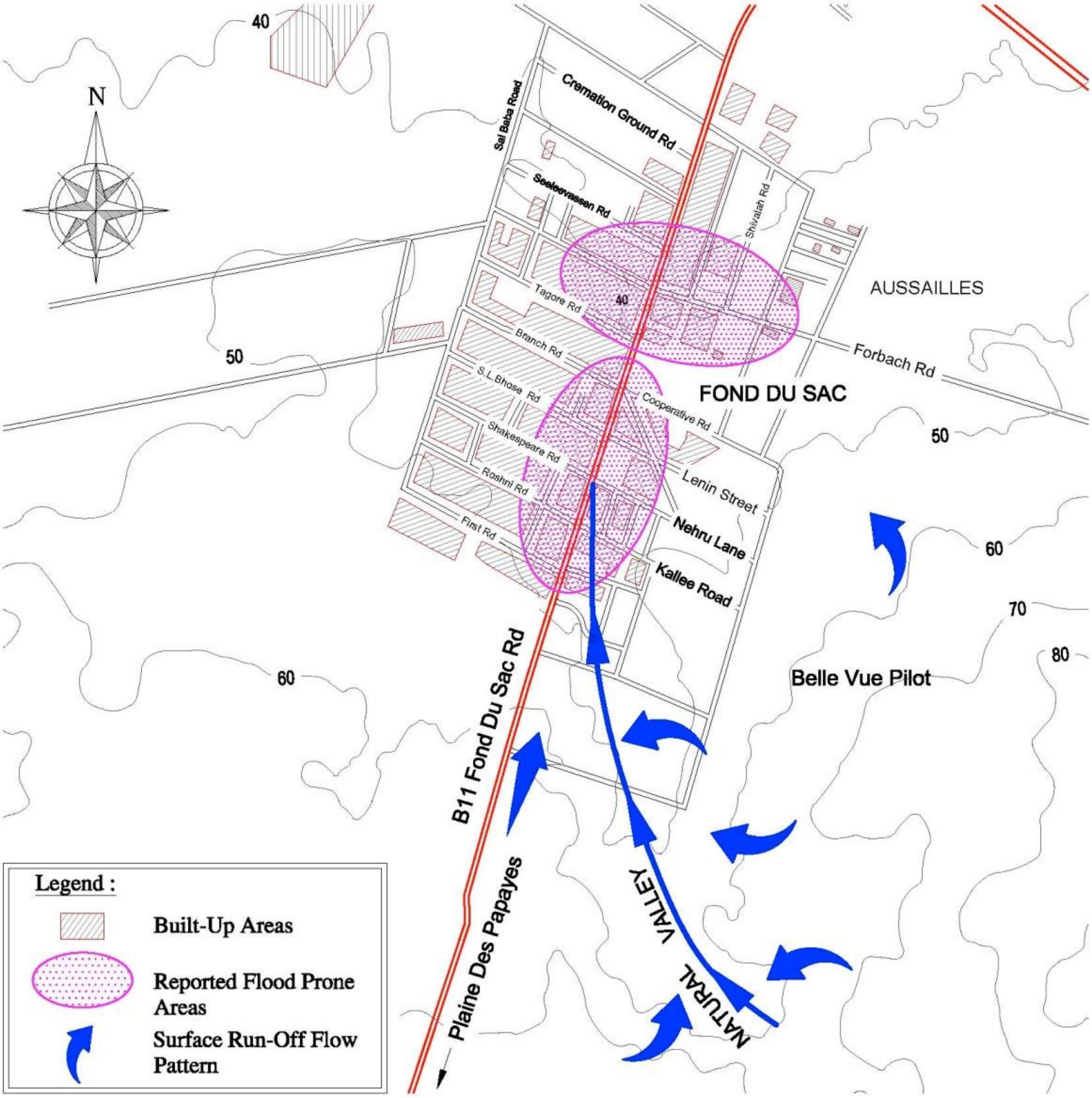


Figure 42: Natural Drainage Path

(iv) The Forbach and B11 Fond du Sac Road junction

The main B11 roadside drains had been built with undulating invert following the road surface profile, incorporating numerous troughs along its invert, the consequence of which is backflow into the drain itself and overflow into depressed areas.

3.3.3.1.2 Flood Attenuation Works

Flood alleviation works on the Fond du Sac sector is nearing completion and include:

- A flood retardation basin constructed out of a natural depression almost midway between Fond du Sac and Terra Mauricia Sugar Mill, serving as a temporary detention of flash floods, to be released as a controlled flow thereafter. The basin is maintained in a dry condition between storm events.

Flood Retardation Basin

- A flood wall cum cut-off drain on the upstream side along the eastern boundary of the village to intercept overland flows.

Floodwall

- A length of 1150m of swales (shallow, broad and vegetated channels) to promote infiltration and convey the remaining flow to the north, away from the urbanised area.

Swale

3.3.3.2 Grand Baie

Given the very gentle slopes in this area, the capacity of drains is very limited and will only allow very frequent events to be controlled within the drain geometry.

3.3.3.2.1 Kapukay

This area is located within the catchment GBP_ BV_06 of area 0.05 km^2. It has a drainage path of 164 m with a gentle slope of 2.8 %. Stormwater within the urbanised area, aggregating some 205 housing and commercial units, is drained through a combination of partly open and partly covered drains ending their course through two twin pipe culverts, each of diameter 700 mm into the sea.

Covered RC Drain 500 mm wide x 350 mm deep

Open RC drain 470 mm wide x 450 mm deep

Twin pipe culverts

Drainage system within Kapukay

A few houses had been built squarely over the open drain at a few locations.

Overland flow emanates from a wetland located upstream behind Grand Baie Bazaar and gets channeled through the open drain. The drains provide relief only during frequent rainfall events, otherwise overflowing onto properties.

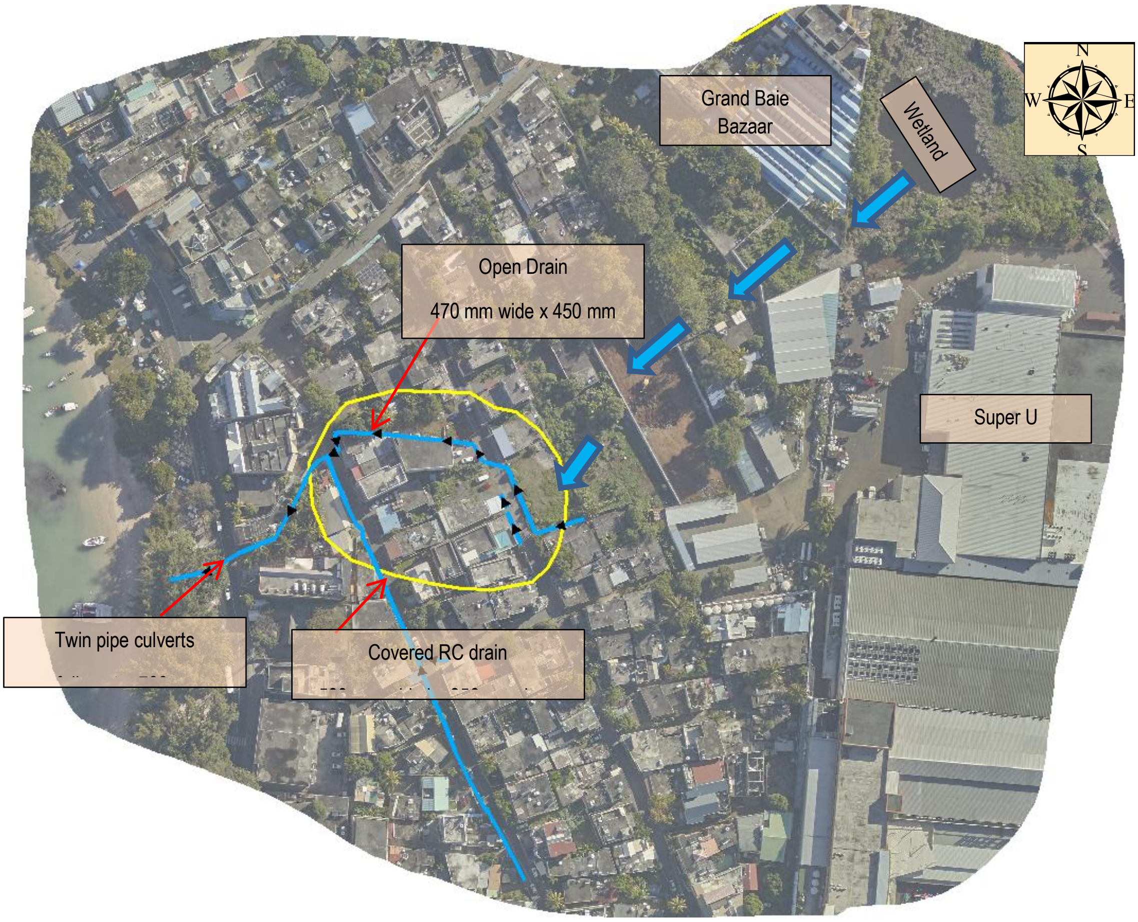


Figure 43: Overland flow emanating from wetland

3.3.3.2.2 Racket Road

This area is located within the catchment GBP_ BV_07 of area 1.08 km^2. It has a drainage path of 1923 m with a mild slope of 1.07 %.

The area around Racket road leading to Grand Baie Bazaar reportedly does not get flooded during frequent rainfall events because of its higher elevation relative to areas nearer the coastline; any reported inundation is presumably due to drainage paths being encroached by construction.

3.3.3.2.3 Camp Carol

This area is located within the catchment GBP_ BV_08 of area 0.42 km^2. It has a drainage path of 492 m with a gentle slope of 1.51 %.

Camp Carol had recently been provided with a partly constructed network of drains. Implementation of the whole design did not get completed and completion of the works through a different Works Contract is scheduled during the next 12 months.

Figure 44: Existing & Proposed Drainage network at Camp Carol

3.3.3.2.4 La Croisette Roundabout

The roundabout at La Croisette gets flooded during heavy rainfall events. The main cause of flooding is due to a villa development across the main drainage axes, obstructing flow and resulting into water accumulation at the roundabout.

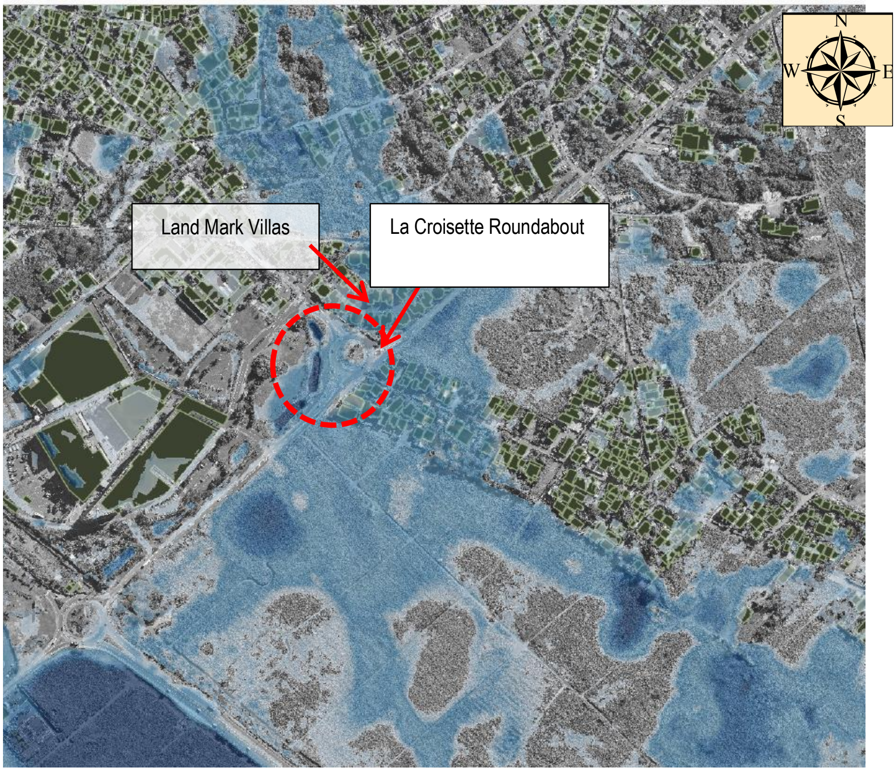


Figure 45: Flood modelling during a 10 year rainfall event

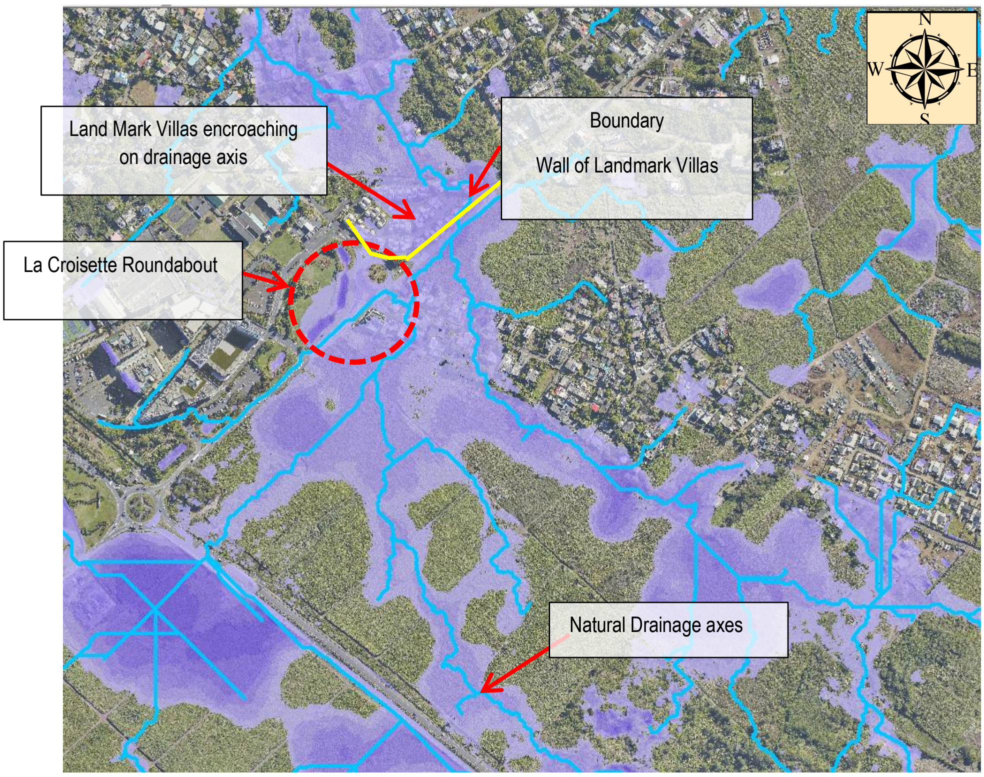


Figure 46: Flood modelling during a 100 year rainfall event

Flooding at La Croisette Roundabout

3.3.3.2.5 Near Police Station

The main cause of flooding at this location is due to buildings encroaching over the natural drainage axes, obstructing overland flow.

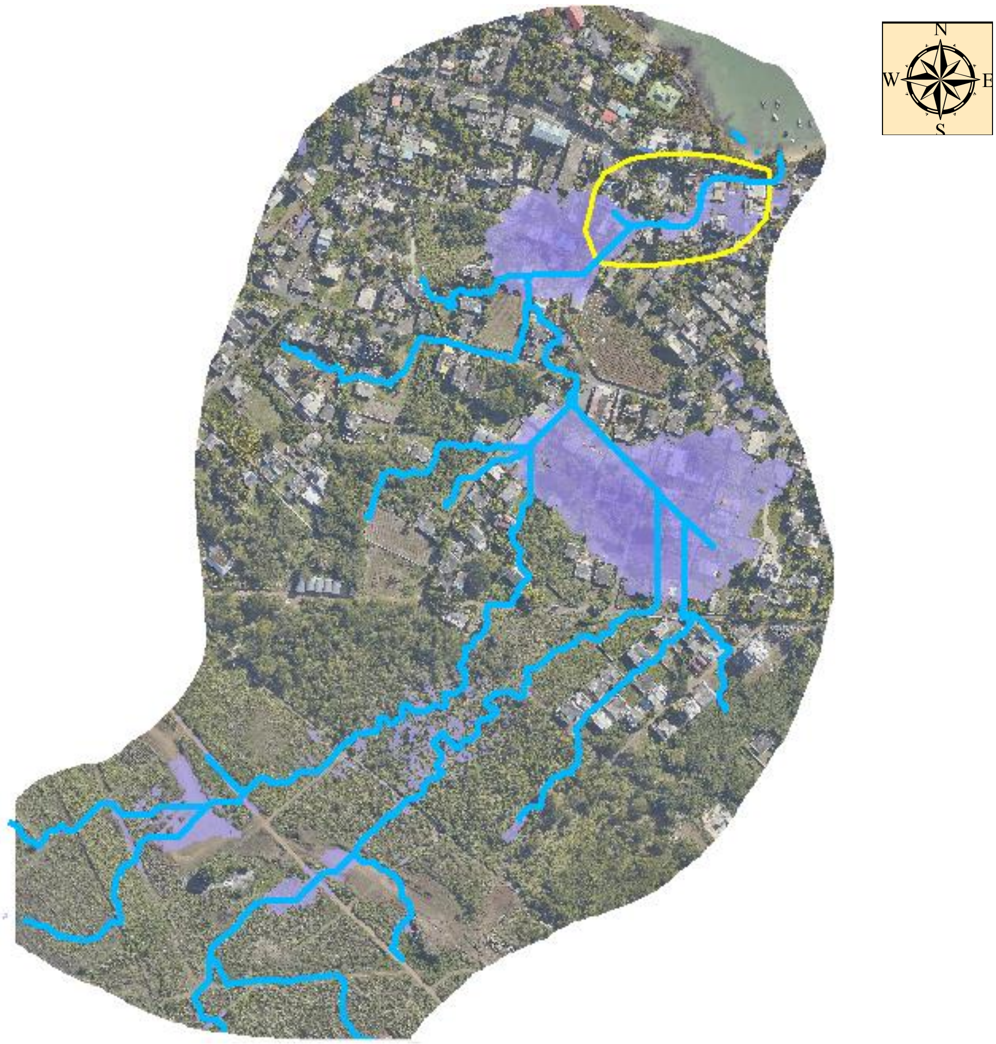


Figure 47: Flood modelling during heavy rainfall events

3.3.3.3 Pereybere

3.3.3.3.1 Pyndia Lane

A complex named "KI residences" is presently under construction over the lowland. Stormwater is diverted via a rehabilitated drain of size 700 mm wide by 600mm deep. Part of the unfilled wetland is evident south of the development.

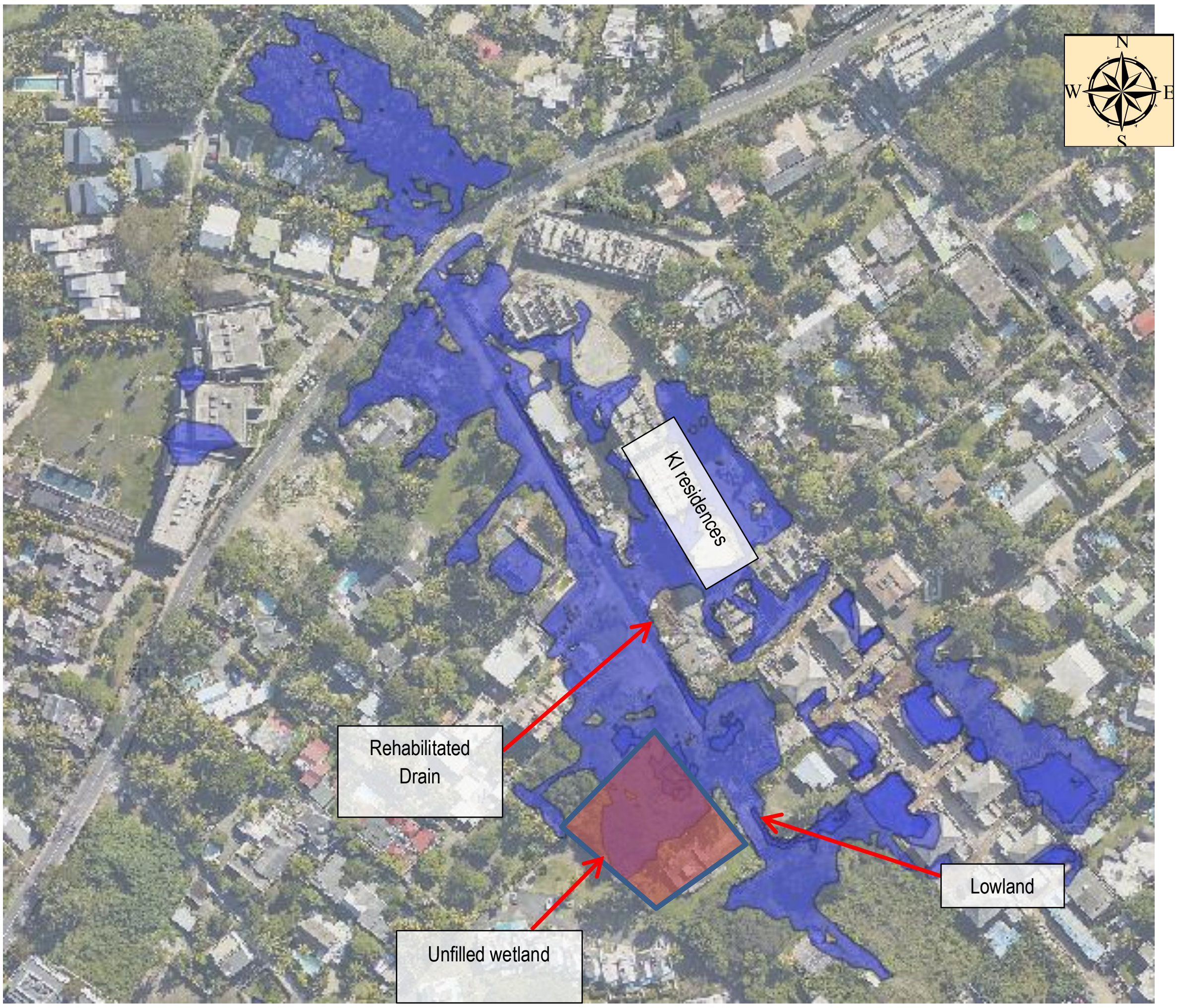


Figure 48: Layout at Pyndia Lane

Rehabilitated drain 700 mm wide by 600 mm deep

3.3.3.3.2 Beach Lane

The village of Pereybere has a relatively flat topography with several localised depressions along both coastal and access roads.

Given the very gentle slopes in this area, the capacity of drains is very limited and will only allow very frequent events to be controlled within the drain geometry.

Pereybere had recently been provided with a network of shallow drains, the layout of which is shown below.

Figure 49: Network of drains at Pereybere

3.3.4 ANALYSIS

3.3.4.1 Fond du Sac

As- made details for the Fond du Sac Project are given below.

Flood Retardation Basin

Basin Capacity	:	118,000 m^3
Embankment Height	:	6.5 m
Freeboard	:	500 mm

A. Floodwall/Cut-off Drain

Chainage (m)	Length (m)	Width of drain (mm)		Height of wall above GL (mm)	Depth of drain below GL/mm
		Top	Bottom		
-17 to 33	50	-	-	750 to 1000	-
30 to 100	70	-	-	230 to 1500	-
100 to 117	17	2300	1250	1500 to 1100	600 to 2100
117 to 427	310	4800	2500	1500 to 1100	600 to 2100
800 to 940	140	4800	2500	0 to 2500	4000 to 2300
940 to 960	20	5500	2500	2500 to 2300	2300 to 2000
960 to 986	26	5500	2500	2300 to 1100	2120 to 3230

B. Underground Conduit

Chainage	Length (m)	Internal Size (mm)	Backfill height from G.L to top of slab (mm)	Thickness of slab (mm)
427 to 700	273	2100 x 2100	430 to 5000	350
700 to 800	100	2100 x 2100	5000 to 1550	350

C. Earthen Swale

Chainage	Length (m)	Top Width (mm)	Depth (mm)	Soils characteristics
990 to 1100	110	20,000	4000 to 2750	Brown silty Clay loam
1100 to 1517	417	20,000	3000 to 1850	Brown silty Clay loam
1517 to 1760	243	20,000	1500 to 1850	Brown silty Clay loam
1770 to 1870	100	15,000	2200	Brown silty Clay loam
1880 to 1950	70	20,000	2000 to 2900	Brown silty Clay loam
1950 to 2040	90	20,000	3600 to 2750	Brown silty Clay loam
2040 to 2145	105	11,000	2750 to 2000	Brown silty Clay loam

D. Open Masonry Drain

Chainage	Length (m)	Top Width (mm)	Bottom Width (mm)	Depth (mm)
2145 to 2180	35	4000	2000	2000 to 3000
2190 to 2210	20	4000	2000	3000
2210 to 2250	40	4000	2000	2700 to 2000
2250 to 2294	44	4000	2000	2000 to 1850

The capacity of the drains at Fond du Sac is summarised in the following table.

Location	Remarks	Description	Slope (%)	Capacity (m^3/s)
Cut off drain	Maximum capacity	Top width 5.5m / bottom width 2.5m / 3.2 m depth	1.0	85.5
Underground conduit		2100 x 2100	1.0	19
Earthen swale	Minimum capacity	11m width and 2m deep	1.2	62
	Maximum capacity	20m width and 4m deep	1.2	352
Open masonry drain	Minimum capacity	Top width 4m / bottom width 2m / 1.85 m depth	1.0	28
	Maximum capacity	Top width 4m / bottom width 2m / 3 m depth	1.0	52

3.3.4.2 Grand Baie

The capacity of the existing drains in Grand Baie is summarize in the following table.

Location		Description	Slope (%)	Capacity (m^3/s)
Kapukaye		Covered RC Drain 500 mm wide x 350 mm deep	0.8	0.24
		Open RC drain 470 mm wide x 450 mm deep	2.5	0.53
		twin pipe culverts, each of diameter 700 mm	2.8	2.20
Camp Carol	Minimum capacity	drain 300 mm wide x 350 mm deep	1.9	0.18
	Maximum capacity	drain 1200 mm wide x 1200 mm deep	5.7	10.9

3.3.4.3 Pereybere

The capacity of the existing drains in Pereybere is summarize in the following table.

Location		Description	Slope (%)	Capacity (m^3/s)
Pindya Lane		Drain 500 mm wide by 500mm deep	0.4	2.7
Beach Lane	Maximum capacity	Drain 2000 mm wide by 1000 mm deep	3.2	20.9

3.3.5 PROPOSED SOLUTIONS

An overview of the proposed solutions is given in the figure below. A detailed description of individual solutions follow.

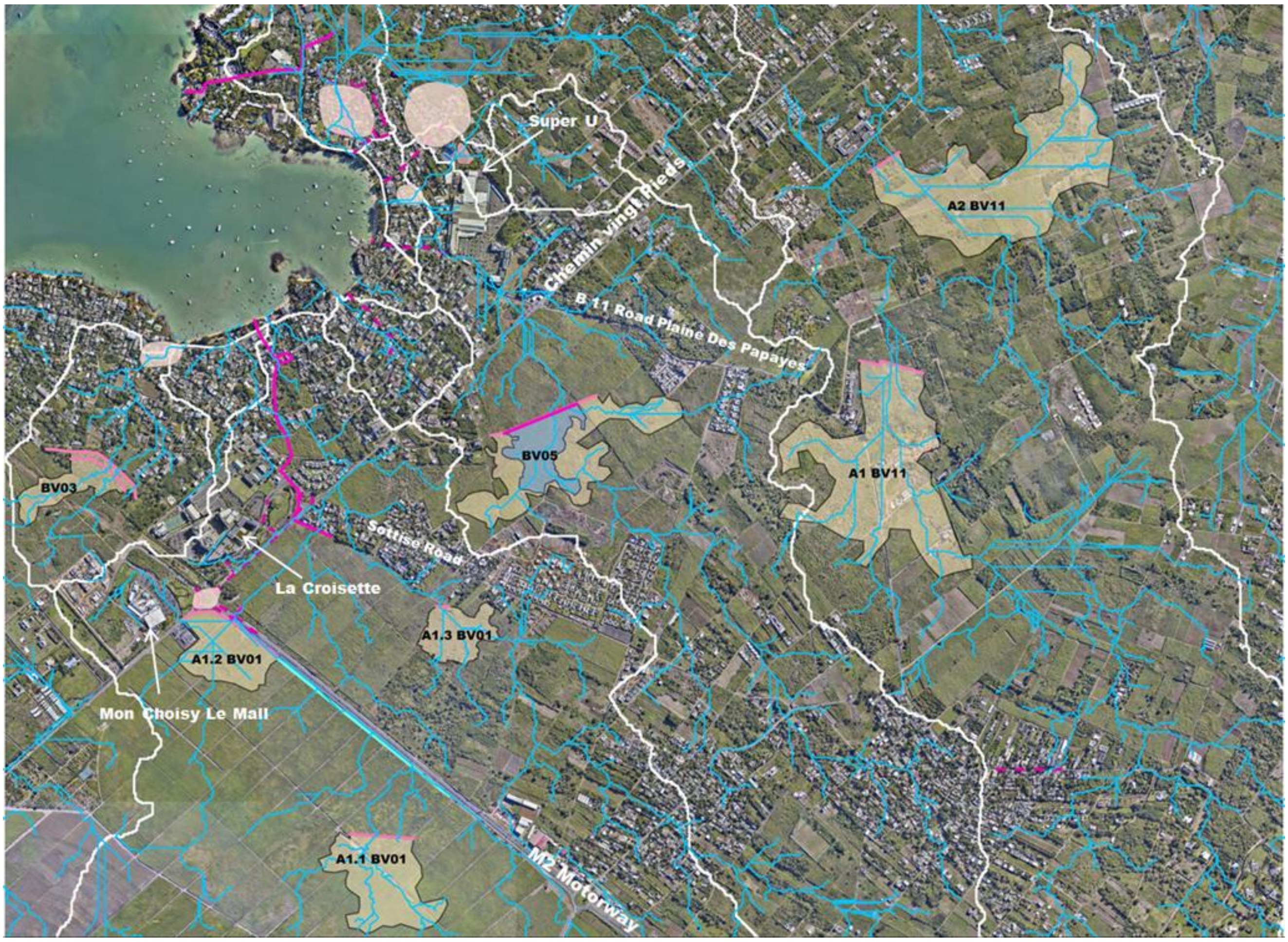


Figure 50: Overview of proposed solutions on background of catchment boundaries and flood prone areas

3.3.5.1 Fond du Sac/Grand Baie

As discussed in D5.1, the first part of the Land Drainage Masterplan, Sustainable solutions no longer seek the quickest way of channelling stormwater into a river or watercourse as these require a large infrastructure. They consist nowadays in finding ways and means of controlling peak flows and reducing flow volumes at source and breaking the peak flows as much as possible prior to releasing them in a controlled manner into the drainage infrastructure.

Flood infiltration through swales and flood expansion zones along or off watercourses help in reducing the flow volumes.

Flood retardation basins, terracing and vegetative cover assist in breaking peak flows as much as possible prior to releasing them in a controlled manner into the drainage infrastructure.

It is equivalent to releasing the same quantity of water but over a longer period of time.

At source stormwater management for mitigating the impacts of urbanisation on baseflow should become a basic design principle.

The various proposed options to attenuate peak flows upstream are summarized below and described more illustratively on the layout drawing, with extracts reproduced herein where relevant. Inconvenience and damage due to floods occur in this sector even during frequent rainfall events, mostly because of the large expanse of agriculture in the hinterland and upstream of the urbanised area, producing high peak flows even during low rainfall intensities.

The following proposals are made:

A. Flow attenuation downstream of Fond du Sac GBP_ BV01

- Three flood attenuation basins are proposed in the catchment of GBP_ BV01. Hydraulic modelling provides the following outputs:
  - **A1.1:** Dyke at location (561115.82, 7784879.88), south of M2 motorway, length 265 m, height at its maximum point 3 m (2m to top water level) and net capacity 188,000 m³.

    The basin discharge is a 2.1m wide and 1m high culvert which has a theoretical capacity of 6.5 m³/s.

Table 27: Peak flow downstream of Fond du Sac – A1.1

Return period (years)	10	25	50	100
Peak flow (m³/s) (without structure)	67.8	82.6	93.8	104.4
Control outflow (m³/s)	6.6	6.7	6.7	6.8
Resultant Peak Flow (m³/s)	52.9	63.9	72.3	80.2

Percentage Peak Flow Reduction (%)	22	22	23	23

- **A1.2**: Dyke in RC wall at location (59053.51, 7785602.19) near M2 motorway at Mon Choisy Le Mall roundabout, length 400 m, height at its maximum point 3 m, (2m to top water level) and net capacity 175,000 m³, coupled with a flood wall cum drain of length 760m along the M2 motorway.
-

  The basin discharge is a 2m wide and 1m high culvert which has a theoretical capacity of 6.3 m³/s.

Table 28: Peak flow downstream of Fond du Sac – A1.2

Return period (years)	10	25	50	100
Peak flow (m³/s) (without structure)	66.5	71.5	75.9	83.9
Control outflow (m³/s)	6.5	6.6	6.6	6.6
Resultant Peak Flow (m³/s)	54	59.2	64.4	67.8
Percentage Peak Flow Reduction (%)	19	17	15	19

- **A1.3:** Dyke at location (561338.07, 7785689.50), north of M2 motorway, length 50 m, height at its maximum point ~~of~~ 2 m (1m to top water level) and net capacity 30,000 m³.

  The basin discharge is a 1.1m wide and 0.7m high culvert which have a theoretical capacity of 6.3 m³/s.

Table 29: Peak flow downstream of Fond du Sac – A1.3

Return period (years)	10	25	50	100
Peak flow (m³/s) (without structure)	54.3	73.8	87.4	99.1
Control outflow (m³/s)	1.9	1.9	1.9	1.9
Resultant Peak Flow (m³/s)	45.9	67.3	81.3	92.3
Percentage Peak Flow Reduction (%)	15	9	7	7

For these three flood attenuation basins, the resultant peak flow is higher than the control outflow because part of the inflows spill over the dike and part flows around it. These attenuation basins will be very effective for frequent floods but less so for floods of frequency higher than 10 years.

- Floodwall cum drain to channel flow from flood retardation basin A1.1 to A1.2 without flooding the M2 motorway.

- Drain to further divert overland flow away from the La Croisette Roundabout to the sea. Carrying capacity of the drain is 5 m³/s (Stone masonry drain average slope 0.5%, sloping face 1H:3V, 2.5 m x 0.75 m depth)

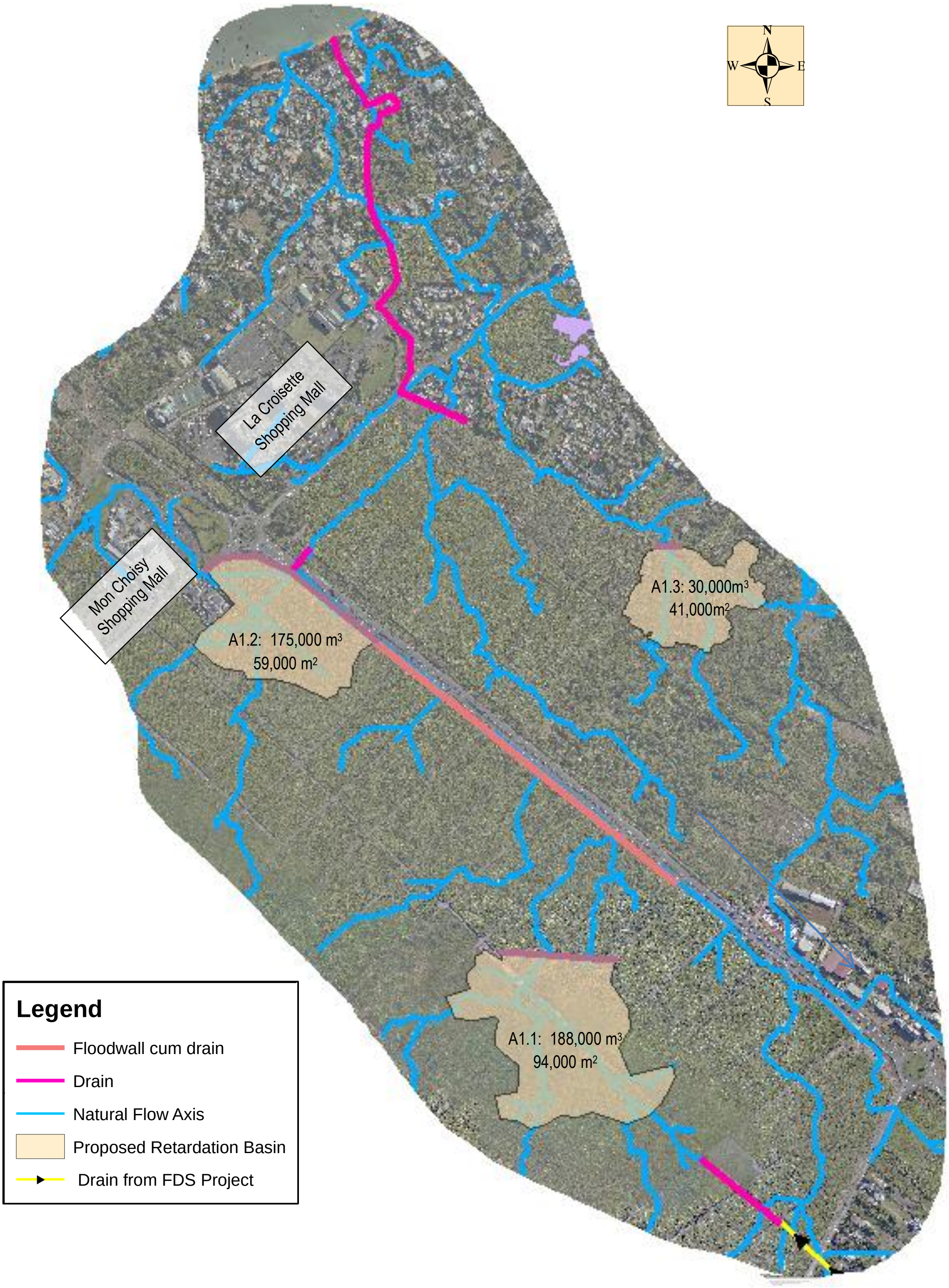


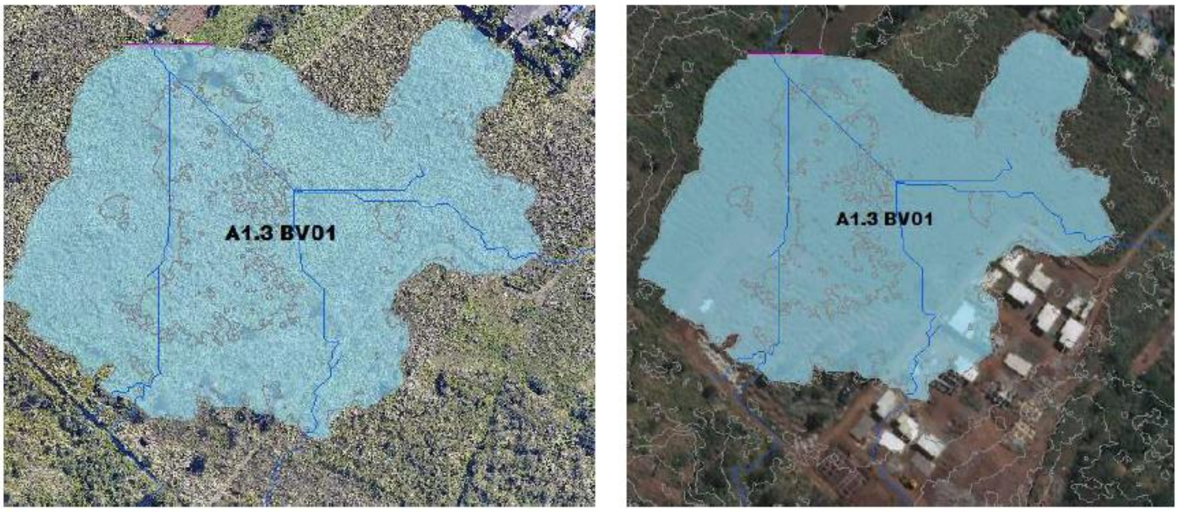
Figure 51: Proposed drainage system within GBP_ BV01

The physical characteristics of the basins are as follows:

Table 30: Physical characteristics A1.1, A1.2 and A1.3 – Grand Baie

	A1.1	A1.2	A1.3
Catchment Area (ha)	99	52	214
Foundation level (m amsl)	22.4	15.4	12.3
Top water level (TWL)	24.4	17.4	13.3
Water depth (m)	2	2	1
Area to TWL (m^2)	93,392	59,053	41,043
Capacity (m^3)	188,000	175,000	30,000
Crest level (m amsl)	25.4	18.4	14.3
Dyke height (m)	3	3	2
Dyke length (m)	265	775	50

Related to A1.3-BV01: Proposed detention pond A1.3 has been located in a partly built area.



Basin A1.3 overlaid on orthophoto   Basin A1.3 overlaid on World Imagery Basemap

Figure 52: Orthophoto and World imagery Basemap Basin A1.3

It is unfortunate that such a new construction has cropped up along the high water flood mark of basin A1.3 BV01, more so in view of a low dyke making it possible to have such a significant detention capacity. Short of having to forsake this basin, a 1.0m high floodwall as shown below can be built to protect the few houses against a 50 year flood event.

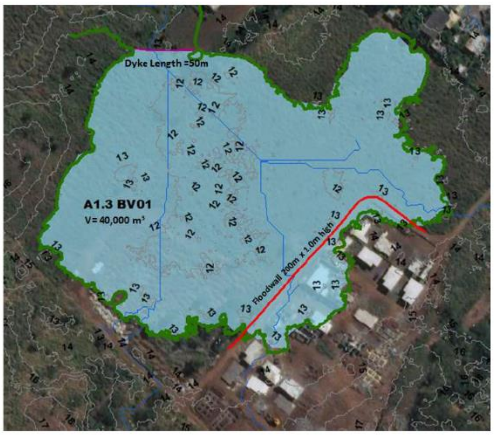


Figure 53: Basin A1.3 and additional floodwall

B. Flow attenuation GBP_BV05

Two options are proposed:

- A flood attenuation basin at location (561729.78, 7786371.07), South East of Chemin Vingt Pieds of net capacity 550,000 m^3, involving the construction of a dyke of length 400m and a maximum height of 4 m.

- A dyke of length 275 m, approximate height 2 m and net capacity of 91,000 m^3 was also investigated but not selected because of its low storage capacity.

The basin discharge is a 2.1m wide and 1m high outlet with a theoretical capacity of 6.5m^3/s.

Table 31: Dyke length 400 m, approximate height 4 m

Return period (years)	**10**	**25**	**50**	**100**
Peak flow (m^3/s) (without structure)	7.3	11.3	14.8	18.4
Control outflow (m^3/s)	4.5	5.4	5.7	6.0
Resultant Peak Flow (m^3/s)	4.5	5.4	5.7	6.0
Percentage Peak Flow Reduction (%)	38	52	61	67

- Free flow as flood expansion zone from the outlet of the basin to Chemin Vingt Pieds
- Existing culvert to channel flow across Chemin Vingt Pieds to the existing drain along Route de la Salette and ultimately to the sea.

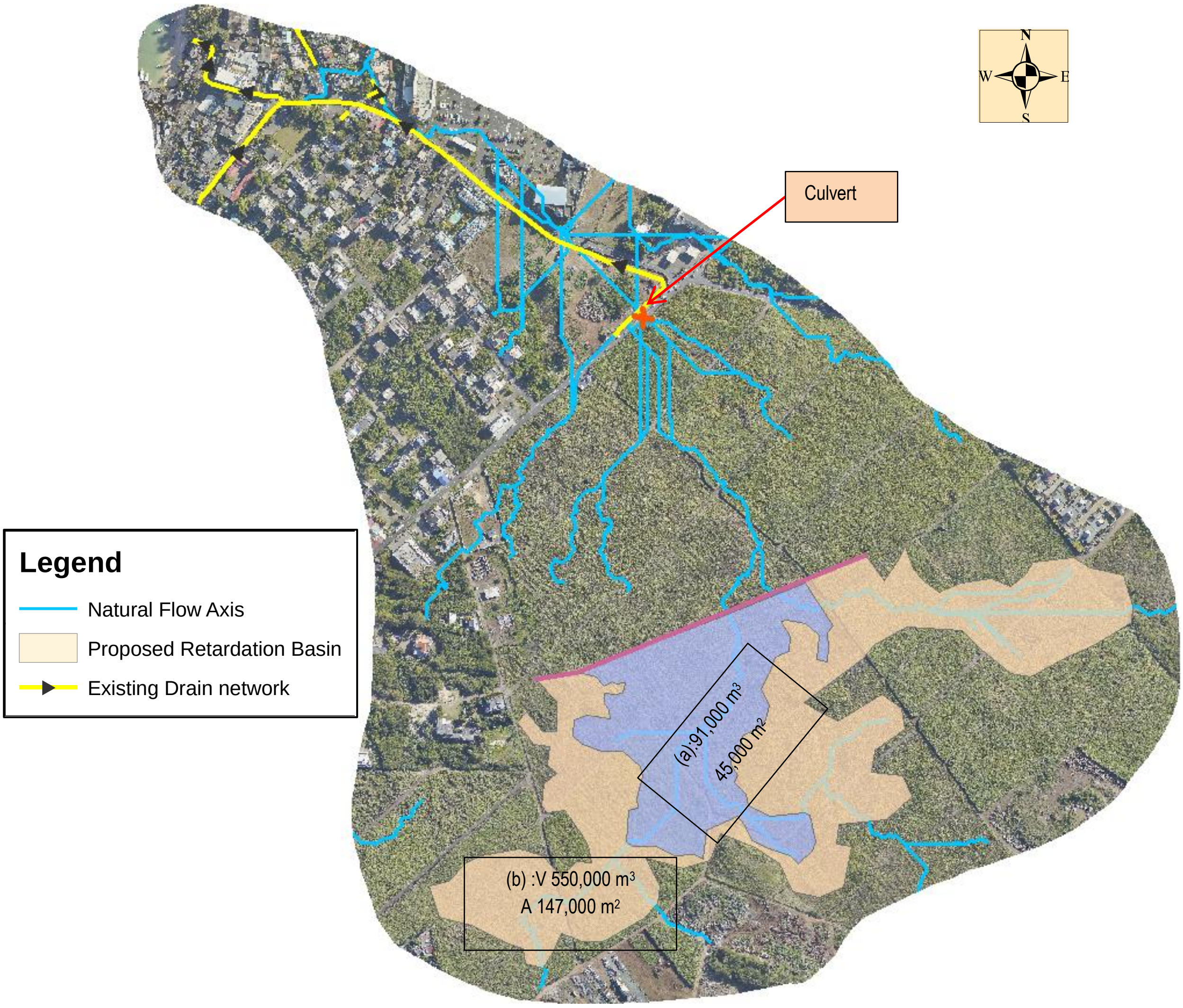


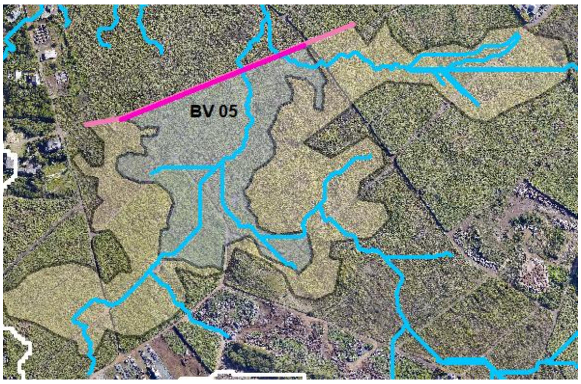
Figure 54: Proposed drainage system within GBP_ BV05

The physical characteristics of the basins are as follows:

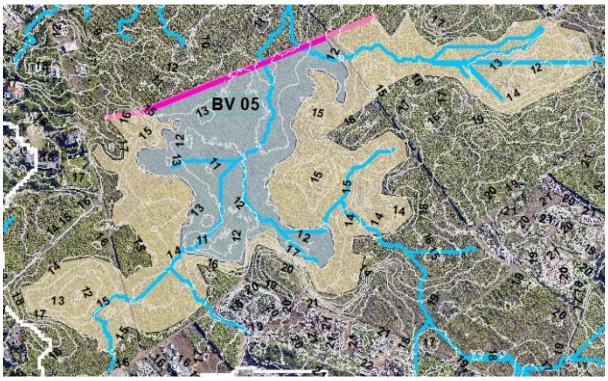
Figure 55: Physical characteristics B1 and B2 – Garnd Baie

	B1	B2
Catchment Area (ha)	156	156
Foundation level (m amsl)	10.6	10.6
Top water level (TWL)	13.6	11.6
Water depth (m)	3	1
Area to TWL (m^2)	147,000	45,000
Capacity (m^3)	550,000	91,000
Crest level (m amsl)	14.6	12.6
Dyke height (m)	4	2
Dyke length (m)	400	275

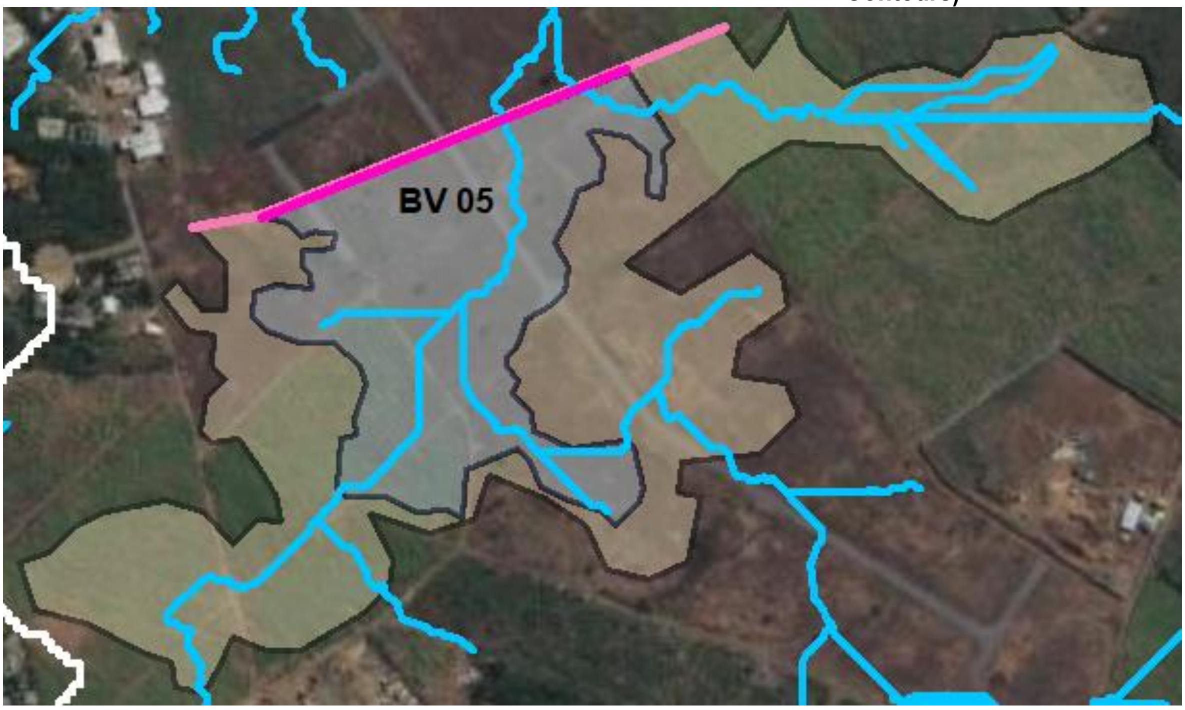
Related to BV05: Proposed detention pond BV05 is situated within ongoing built-up areas (Several houses and commercial buildings already put in place)



Basin BV 05 Overlaid on Orthophoto



Basin BV 05 Overlaid on Orthophoto (With Contours)



Basin BV 05 Overlaid on Google Map

Figure 56: Orthophoto and Google Map Basin BV 05

It is again unfortunate that such a recent development had been permitted across three major natural drainage paths. A decision has therefore to be made by LDA on whether to forego such a promising flood attenuation infrastructure (detention volume of 550,000 m3) and allow the morcellement to proceed at the expense of yet another flood prone built-up area to cope with.

C. Flood attenuation at Kapukay

- Expansion of the existing wetland at Grand Baie Bazaar.

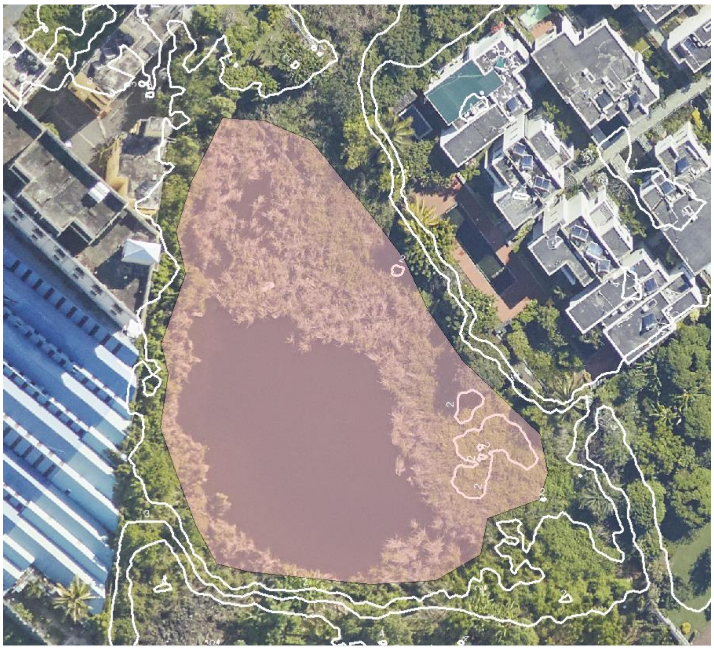


Figure 57: Expansion of existing wetland at Grand Baie Bazaar

Expansion of the existing wetland will provide additional storage to attenuate spill over to built-up areas downstream.

D. Flood attenuation at Camp Carol

- Expansion of the existing wetland
- Drain to divert overflow from the wetland to the sea, away from Camp Carol.
  Carrying capacity of the drain is near 5 m³/s (Stone masonry drain, average slope 0.3% , included downstream boundaries effect , sloping face 1H:3V, 3.0 m x 2.0 m deep)

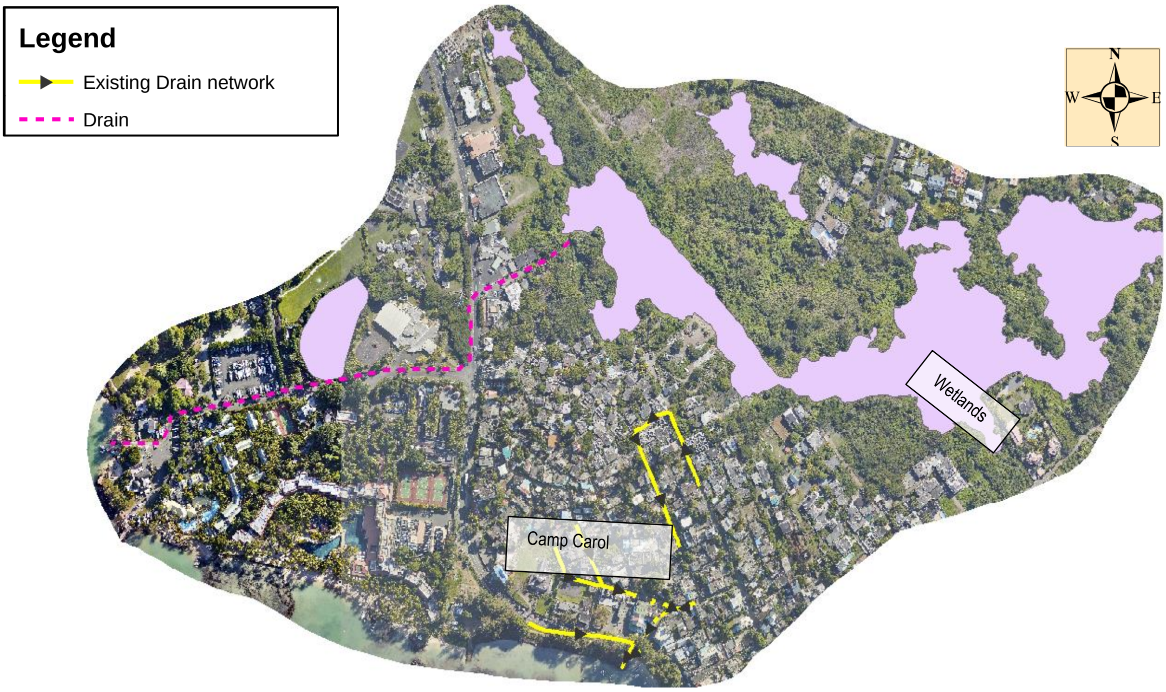


Figure 58: Proposed solution at Camp Carol

E. Flood attenuation GBP_ BV03

- RC floodwall at location (560162.65, 7786168.20) of length 245 m, 2 m high at its maximum, (inclusive of a spillway on to the natural flow axes).

  The physical characteristics of the basins are as follows:

Figure 59: Physical characteristics E1 – Grand Baie

	E1
Catchment Area (ha)	19
Foundation level (m amsl)	12.5
Top water level (TWL)	13.5
Water depth (m)	1
Area to TWL (m^2)	36,042
Capacity (m^3)	36,000
Crest level (m amsl)	14.5
Dyke height (m)	2
Dyke length (m)	245

The basin discharge is a 1.5m wide and 0.8m high culvert with a theorical capacity of $3.2m^3/s$.

Table 32: Peak flow downstream of GBP_ BV03

Return period (years)	**10**	**25**	**50**	**100**
Peak flow (m^3/s) (without structure)	3.9	4.7	5.2	5.7
Control outflow (m^3/s)	2.8	3.0	3.1	3.5
Resultant Peak Flow (m^3/s)	2.8	3.0	3.1	3.7
Percentage Peak Flow Reduction (%)	28	36	40	35

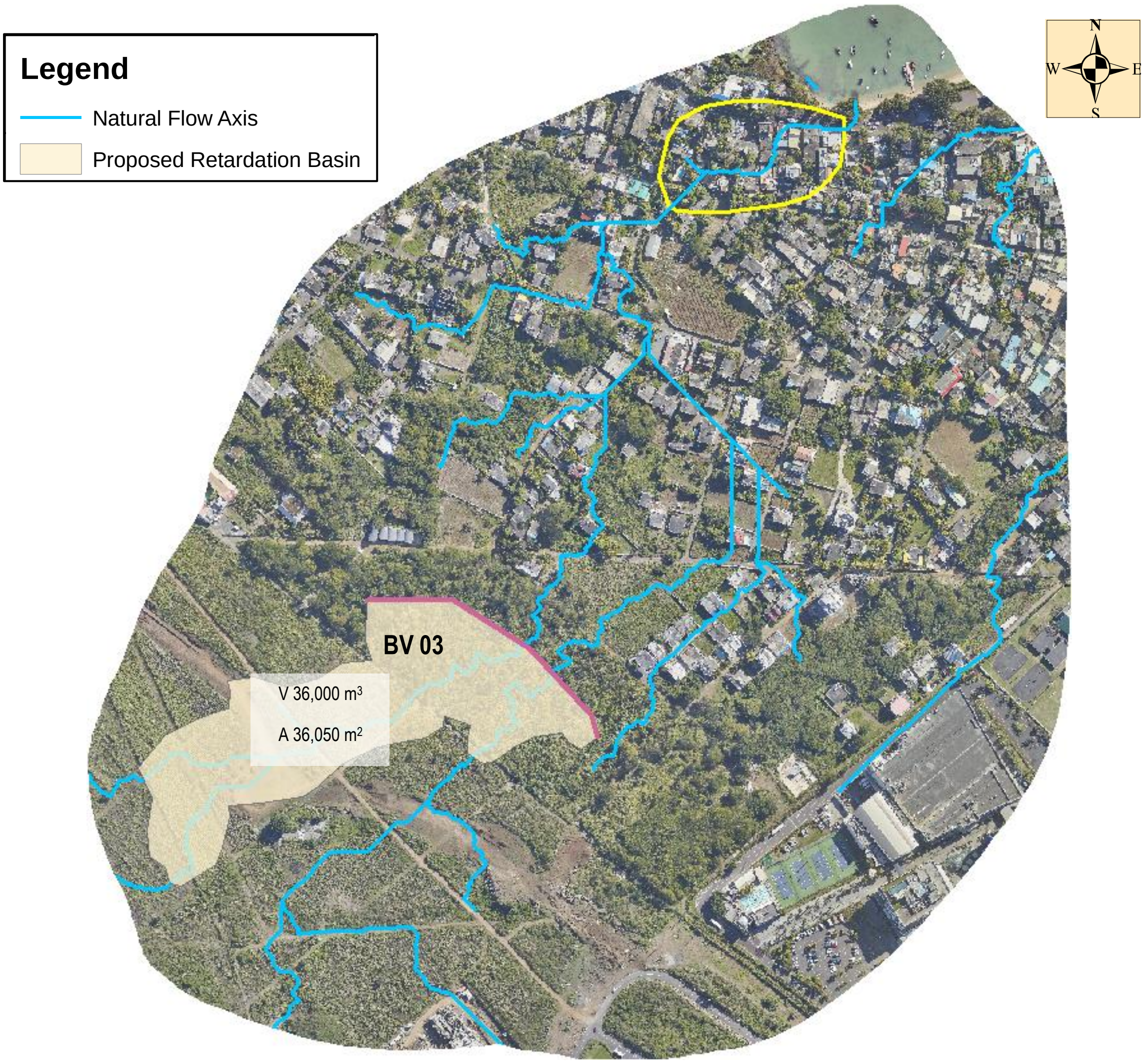


Figure 60: Proposed solution at GPB_ BV03

The floodwall will prevent widespread inundation downstream and concentrate flow through the two drainage axes via the control outlet(s). For any storm event of less than 10year recurrence period there will be minimal impoundment. Impoundment will start incrementally, with spillover occurring during a storm event of more than 50 years, when backflow will reach the area surrounding the Grand Bay –La Croisette link road up to 500mm depth. The road itself will be inundated to a lessor extent depending on its height above the surrounding ground.

The proposed detention pond BV03 is cutting across the Grand-Baie-La Croisette Link Road which is already operational.

Below is a picture of the roundabout built over the natural flow axis and a drawing of water impoundment of BV 03 with the roundabout superimposed thereon. This measure is fundamentally a floodwall to prevent widespread inundation downstream and concentrate the flow through the two drainage axes.

Figure 61: Roundabout Grand-Baie-La Croisette Link Road anddetention BV03

The maximum impounded water level corresponding to a 1 in 50 year event is 14m amsl and the road had been built some 500mm above the original ground level (ie at 14.5m amsl) which means that it is in itself a dyke impounding flow from upstream to a level of 14.5m, with excess flow spilling over to the other side in the absence of a cross drain. The proposed floodwall will impound residual flow and will therefore have no adverse impact on the road. Had the road not being existent or had been constructed to a higher level, we would have proposed a higher floodwall with more retention capacity.

Given the challenge of having to protect areas downstream, we recommend that this development be maintained and the road embankment monitored over time. It is highly recommended that the RDA considers building a cross drain structure to respect the existing and mapped natural path, as it should have been the case in a proper hydraulic design of the road.

3.3.5.2 Pereybere

A. Flow attenuation GBP_BV11

- Two flood attenuation basins are proposed in catchment GBP_ BV11, namely:
  - **A1:** Dyke at location (562933.64, 7786549.67), south east of Chemin Vingt Pieds length 228 m, approximate height 1 m, and net capacity 245,000 m³.
  - The basin discharge is a 2m wide and 0.85m high culvert with a theorical capacity of 5.1m³/s.

Table 33: Peak flow downstream of GBP_ BV11

Return period (years)	10	25	50	100
Peak flow (m³/s) (without structure)	0.7	2.2	4.9	8.5
Control outflow (m³/s)	0.1	0.1	1.1	5.0
Resultant Peak Flow (m³/s)	0.1	0.1	1.1	5.0
Percentage Peak Flow Reduction (%)	86	95	77	41

  - **A2:** Dyke at location (562930.33, 7787280.58), east of Chemin Vingt Pieds length 125 m, approximate height of 1 m, and net capacity 245,000 m³.

    The basin discharge is a 0.75m wide and 0.4m high culvert with a theorical capacity of 0.5 m³/s.

Table 34: Peak flow downstream of GBP_ BV11

Return period (years)	10	25	50	100
Peak flow (m³/s) (without structure)	0.4	0.5	0.55	0.6
Control outflow (m³/s)	0.1	0.1	0.1	0.2
Resultant Peak Flow (m³/s)	0.1	0.1	0.1	0.2
Percentage Peak Flow Reduction (%)	75	80	82	67

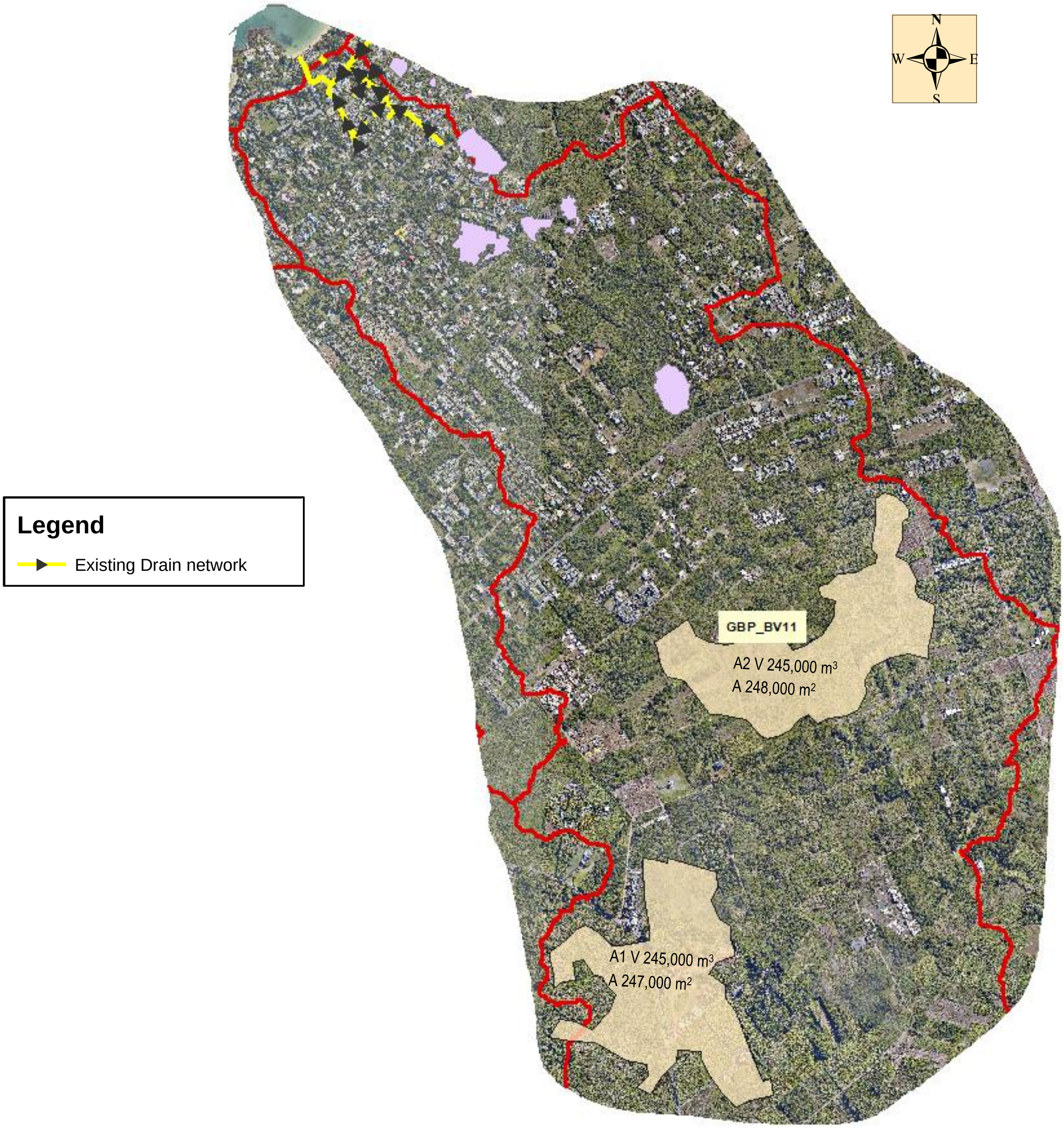


Figure 62: Proposed solutions within GBP_BV 11

The physical characteristics of the basins are as follows:

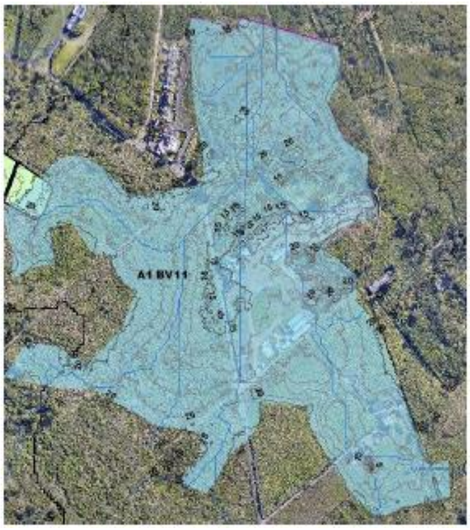
Figure 63: Physical characteristics A1 ad A2 - Grand Baie

	A1	A2
Catchment Area (ha)	65	107
Foundation level (m amsl)	15.2	7.1
Top water level (TWL)	0.5	0.5
Water depth (m)	16	7.6
Area to TWL (m^2)	247,063	248,933
Capacity (m^3)	245,000	245,000
Crest level (m amsl)	16.5	8.1
Dyke height (m)	1	1
Dyke length (m)	245	125

Related to A1-BV11: Proposed detention basin BV11 cuts across Plaine Des Papayes Road, and several built-up villas

It is a fact that the B11 Road cuts across detention basin BV11. However, B11 Road is sited within a low land and always gets flooded in its present state, as illustrated below.

B11 road and development within lowland



Footprint of basin A1 BV 11

Flood Prone Area depicted in the model output

Flood Prone Area depicted in Exzeco

Raising the road across the middle part of the impounded area with cross drains (culverts) will resolve the problem of the road being inundated.

Notwithstanding the advent of the detention basin, many of the already built-up villas will get inundated anyway and a decision has to be made on whether to allow this development across a deep natural valley to progress without mitigation measures, with the risks of having to have recourse to the intervention of the fire department at every high rainfall event.

In conclusion:
For all similar situations, it is recommend that priority be given to include in the planning guidelines, reserved areas for building flood control facilities. In this respect, and in order to protect the territory, it is recommend that strict application of the No Go zones and hence a halt to all new construction across natural flow axes and flood expansion areas be enforced, more so in such low lying areas already identified since many years. Indeed, these sectors are potential candidates for hydraulic infrastructure developments to mitigate flooding downstream.

With regard to road infrastructure, it should include a rigorous hydraulic design, taking into consideration flood risks and in particular the reinstatement of natural flow paths as defined in the Land Drainage Master Plan (D5.1 - first part).

3.3.6 MAIN IMPACTS, COST AND COST-BENEFIT

The following table summarises the measures retained and their associated costs.

The comparative flood maps and the cost-benefit analyses together with cost details are given in Annexes 2 and 3.

The flood maps for the current situation (ie “ Do Nothing Scenario”) are attached in Annex 0: they are also attached to Deliverable D3.

The proposed measures will reduce the water level in the proposed situation by about 10 to 20 cm for the following FPAs in the village area:

- 181 Grand bay
- 182 Camp carol
- 183 Kapukay
- 184 Racket road
- 185 Pyndia Lane
- 254 La Croisette

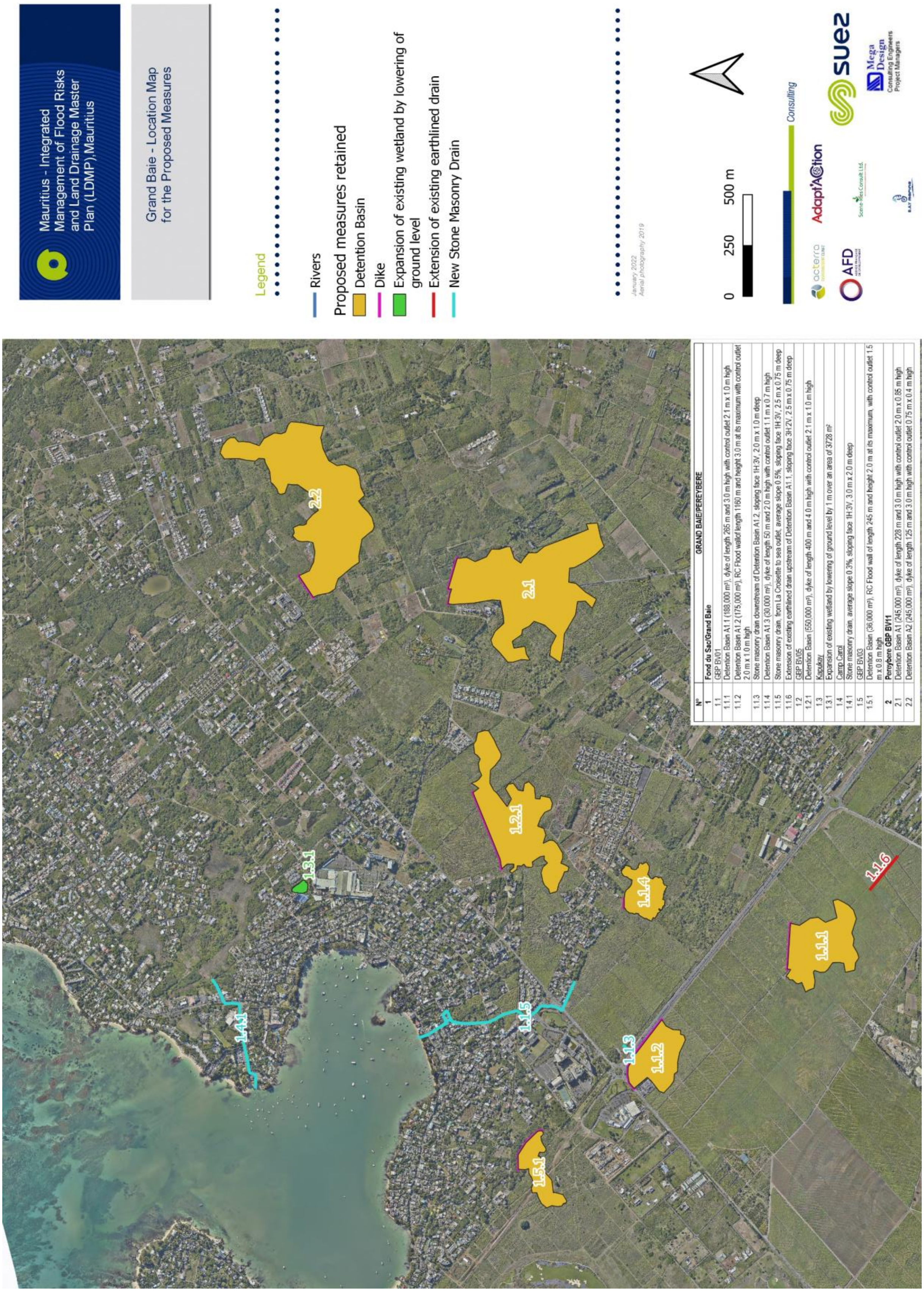


Figure 64: Grand Baie – Pereybere – Location map for proposed measures

Table 35: Grand Baie, Pereybere – Measures and costs

N°	GRAND BAIE/PEREYBERE	Unit	Quantity	Rate	Amount
1	**Fond du Sac/Grand Baie**				
1.1	GBP BV01				
1.1.1	Detention Basin A1.1 (188,000 m^3), dyke of length 265 m and 3.0 m high with control outlet 2.1 m x 1.0 m high	Sum			25 000 000
1.1.2	Detention Basin A1.2 (175,000 m^3), RC Flood wallof length 1160 m and height 3.0 m at its maximum with control outlet 2.0 m x 1.0 m high	Sum			45 000 000
1.1.3	Stone masonry drain downstream of Detention Basin A1.2, sloping face 1H:3V, 2.0 m x 1.0 m deep	m	66	27 000	1 782 000
1.1.4	Detention Basin A1.3 (30,000 m^3), dyke of length 50 m and 2.0 m high with control outlet 1.1 m x 0.7 m high	Sum			5 000 000
1.1.5	Stone masonry drain, from La Croisette to sea outlet, average slope 0.5%, sloping face 1H:3V, 2.5 m x 0.75 m deep	m	960	32 000	30 720 000
1.1.6	Extension of existing earthlined drain upstream of Detention Basin A1.1, sloping face 3H:2V, 2.5 m x 0.75 m deep	m	200	5 300	1 060 000
	Sub Total				**108 562 000**
1.2	GBP BV05				
1.2.1	Detention Basin (550,000 m^3), dyke of length 400 m and 4.0 m high with control outlet 2.1 m x 1.0 m high	Sum			**30 000 000**
1.3	Kapukay				
1.3.1	Expansion of existing wetland by lowering of ground level by 1 m over an area of 3728 m^2	m^3	3728	950	**3 541 600**
1.4	Camp Carol				
1.4.1	Stone masonry drain, average slope 0.3%, sloping face 1H:3V, 3.0 m x 2.0 m deep	m	630	42 000	**26 460 000**
1.5	GBP BV03				
1.5.1	Detention Basin (36,000 m^3), RC Flood wall of length 245 m and height 2.0 m at its maximum, with control outlet 1.5 m x 0.8 m high	Sum			**24 000 000**
2	**Pereybere**				
2.1	Detention Basin A1 (245,000 m^3), dyke of length 228 m and 3.0 m high with control outlet 2.0 m x 0.85 m high	Sum			22 000 000
2.2	Detention Basin A2 (245,000 m^3), dyke of length 125 m and 3.0 m high with control outlet 0.75 m x 0.4 m high	Sum			11 000 000
	Sub Total				**33 000 000**
	Total				**225 563 600**
	ADD:				
	Provision for wayleave and Land Acquisition				50 000 000
	Relocation of Houses				30 000 000
	Contingencies 15%				33 834 540
	Project Management 7.5%				16 917 270
	Grand Total				**356 315 410**

3.4 Flic-en-Flac

3.4.1 OVERVIEW

The catchment area of Flic en Flac is situated in the west of the island covering an area of 21.1 km^2. It is an elongated catchment drained by several streams and drainage ditches. The catchment is sited between amsl 355 m and 2 m, with most urbanisation located on low lying area between a natural watercourse and the coastal dune, where the water table almost reaches ground level. One particular development, Morcellement de Chazal built over a disused sand quarry, is lower with water omni-present in the road side drains.

The catchment which is divided into 13 sub-catchments for the purpose of hydrological study has a long drainage path of approximately 12.9 km and an average slope of 2.7 % on the upstream part and a very mild slope (0.5% to 0.6%) on its downstream coastal strip bordering Morcellment de Chazal.

Figure 65: Catchment area of Flic en Flac

Table 36: Flic-en-Flac – Physical Characteristics of individual Sub-catchments

Name	Area (ha)	Area (km²)	Low level (m)	High level (m)	Length	Slope (m/m)	Slope (%)
FeF_BV01	817.47	8.17	23	352	10015	0.03	3.28
FeF_BV02	39.13	0.39	4	66	1528	0.04	4.07
FeF_BV03	73.40	0.73	3	95	2538	0.04	3.62
FeF_BV04	158.75	1.59	2	169	4654	0.04	3.59
FeF_BV05	5.13	0.05	2	5	545	0.01	0.60
FeF_BV06	82.30	0.82	2	69	2041	0.03	3.26
FeF_BV07	10.94	0.11	2	5	517	0.01	0.64
FeF_BV08	195.04	1.95	2	138	4581	0.03	2.96
FeF_BV09	58.67	0.59	2	8	1650	0.00	0.35
FeF_BV10	37.70	0.38	0	8	1491	0.01	0.52
FeF_BV11	228.05	2.28	2	142	4771	0.03	2.93
FeF_BV12	242.37	2.42	1	111	4654	0.02	2.36
FeF_BV13	158.64	1.59	1	59	3195	0.02	1.83
FeF_Global	**2107.59**	**21.08**	**0**	**352**	**12870**	**0.03**	**2.73**

The flows obtained for Flic en Flac for return periods of 10, 25, 50 and 100 years are shown in the table below.

Table 37: Flic-en-Flac – Flows for sub-catchments and at outlet of catchment for return periods of 10, 25, 50 and 100 years

BVs	Q10 (m3/s)	Q25 (m3/s)	Q50 (m3/s)	Q100 (m3/s)
FeF_BV01	127.63	163.75	187.12	212.19
FeF_BV02	9.37	11.74	13.11	14.62
FeF_BV03	14.60	18.49	21.05	23.74
FeF_BV04	33.34	41.71	46.85	52.69
FeF_BV05	0.97	1.35	1.53	1.74
FeF_BV06	9.25	12.82	15.34	18.09
FeF_BV07	1.70	2.29	2.65	3.18
FeF_BV08	21.54	29.56	35.14	41.25
FeF_BV09	12.65	15.66	17.57	19.68
FeF_BV10	6.69	8.64	9.90	11.29
FeF_BV11	18.01	25.92	31.79	38.07
FeF_BV12	19.71	28.21	34.58	41.25
FeF_BV13	13.26	18.90	23.01	27.50
Outlet of Flic-en-Flac	**190.4**	**249.9**	**290.0**	**332.3**
Q /A (m³/s/km²)	***9.0***	***11.9***	***13.8***	***15.8***

For this sector, the (HECRAS) modelling does not work everywhere using the conventional rational formula, but also partially simulates a rainfall of a specified intensity and generates a peak flow value by a distributed modelling (Q10 or Q100 etc), from which the sizing of the infrastructure is derived.

Urbanization, which started along the coastline next to the dune, subsequently extended far inland with some construction encroaching into the wetlands which play an essential role in the storage and infiltration of run-off and the preservation of water quality in the lagoon. Its expansion continues to progress, in particular, along the upstream side of the water course. The proposed smart city by Medine Ltd projected to be built over an extent of 227 ha span over four sub-catchments, with some 50 % draining into the watercourse at Flic en Flac.

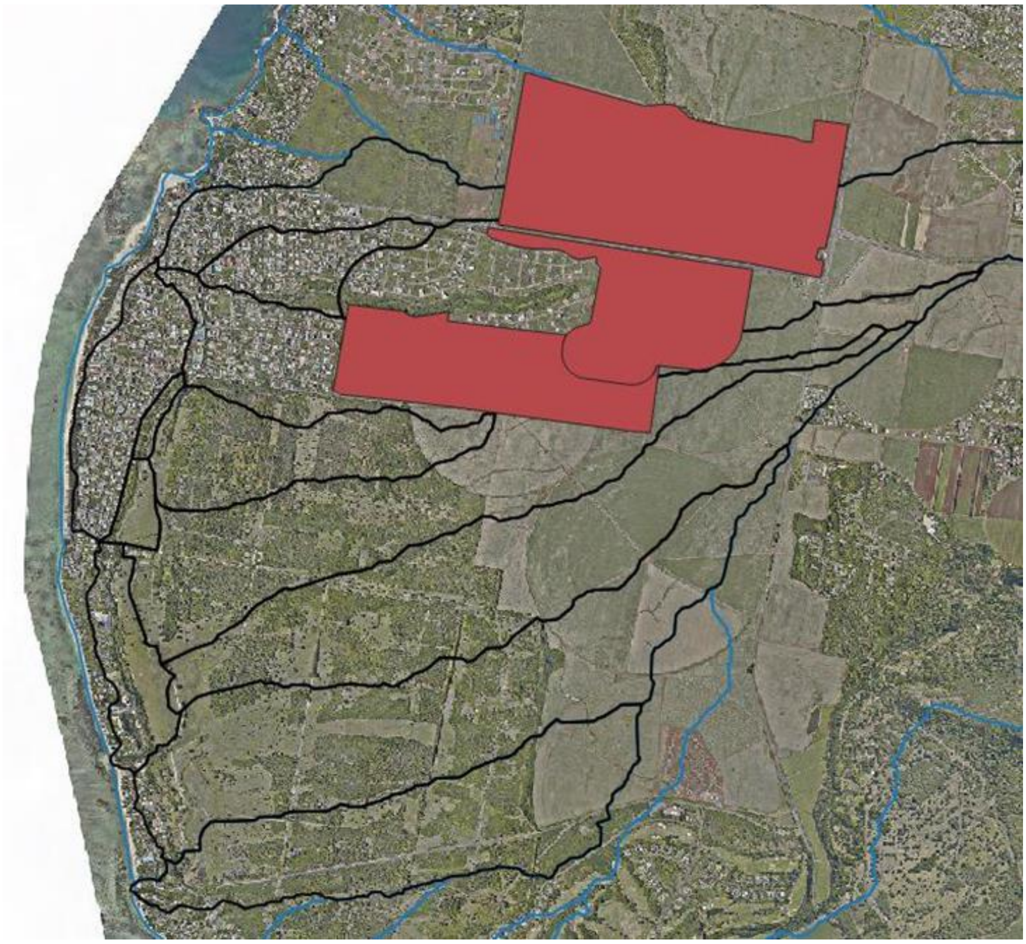


Figure 66: Proposed smart city by Medine

The watercourse has retained its natural width on its upstream part where urbanisation has not yet crept in, but gets progressively backfilled into reclaimed land with construction material, narrowing into concrete lined drains as it passes through built-up area. This drain gradually widens at the downstream end of the urbanized area to regain its natural profile across the low lying floodplains till its terminal outlet into the sea.

River in pristine state | **River being backfilled**

River converted into Narrow Drain and into narrower drain

Wider Waterway at Morc Safeland and across Floodplain

3.4.2 REGIONAL HYDROGEOMORPHOLOGY

The altitude of the catchment area ranges from sea level to 355 m amsl at its upper end. The landform is mildly sloping at the hinterland, slightly undulating as it approaches the coast, getting flatter at the coast line.

The coastal fringe is made of regosols, dark brown sand and loamy sand on light grey to very pale brown coral sand. The soil is highly permeable but the water table is high, which limits infiltration.

The hinterland is made of grey plastic and reddish brown silty clay with frequent gravels, boulders and stones. The superficial soil is shallow to very shallow overlying the recent lava which also accounts for a low infiltration rate.

3.4.3 DESCRIPTING OF EXISTING SITUATION

The NDRRMC in its report of 2018 and LDA identified several flood prone areas following the recurrent flooding problems within the catchment area. These areas are:

- Flic en Flac village (near Villa Caroline)
- Morcellement Bismik
- Morcellement Palmyre (RiverWalk)
- Morcellement de Chazal

In addition, two floodmarks at Morcellement Bismik not located within the identified flood prone areas were identified.

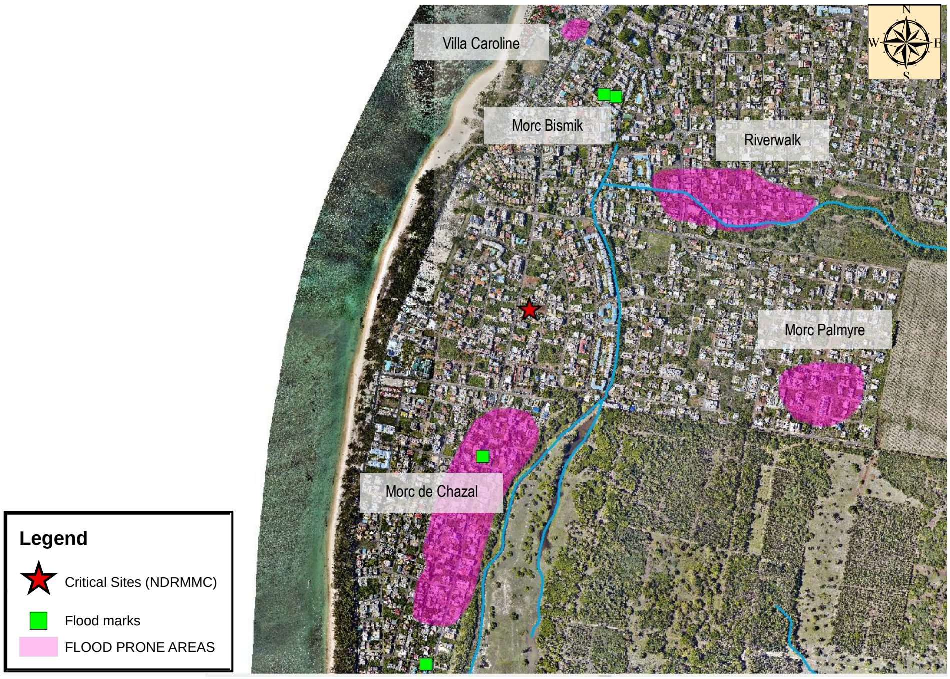


Figure 67: Critical Sites, Flood Marks and Flood Prone Areas

The coastal area of Flic en Flac comprises in the main five residential areas namely:

- Morcellement Anna
- Flic en Flac village (near Villa Caroline)
- Morcellement Bismick
- Morcellement Palmyre
- Morcellement Safeland
- Morcellement De Chazal

Morcellement Anna is built on higher and rocky ground.

The village in the vicinity of Villa Caroline is on a flat coastal but permeable sandy terrain. Except for one localised spot this area does not sustain any major flooding problem.

Much of Morcellement Bismick had been built around and over wetlands with original watercourse alignment diverted and converted into narrow drains built to convey flow from a steep ravine upstream of the development to the slow flowing river across the floodplain downstream.

Morcellement Palmyre is built on fairly higher ground. However, stormwater accumulates at a few parcelled plots located within localized depressions.

Morcellement Safeland had been built on either side of the river course. The eastern part is on higher ground. The west side had been built over an old sand quarry, backfilled for the purpose of the development using imported fill from the eastern side. This development is not prone to flooding.

Morcellement De Chazal had been built squarely over the old sand quarry, graded to level for the purpose of the development without prior reinstatement to its original ground level. It had been developed over low land and is subject to high water table and widespread flooding.

Flooding following cyclonic weather at Morcellement De Chazal

The whole river stretch with Morcellement De Chazal on one side and the floodplain occupied by Societé de Marco on the other side is silted and overtaken by prolific aquatic plant, severely restricting flow into the sea.

Site inspections were undertaken by the Consultant’s team on the catchment at the above mentioned vulnerable locations in particular.

3.4.4 REVIEW OF PREVIOUS STUDIES AND IMPLEMENTATION WORKS

3.4.4.1 Feasibility Report 2005

Gibb in its 2005 report reiterated the recommendations made in the 2003 “Land Drainage Study” to intercept the water course upstream of the developed area behind the cemetery and to divert part of the flow across the north end of Flic en Flac public beach into the sea by means of a reinforced concrete drain / culvert varying in size from 3 to 4 m wide and 0.9 to 3.4 m deep.

This recommendation was not retained. Besides, the construction of such drain would have had the effect of transporting and depositing sediment directly into a pristine lagoon with a public beach as frontage. Presently sediment laden water gets clarified through its slow passage down the river course and across the floodplain before discharging into the sea.

3.4.4.2 Feasibility Study, Design and Works Implementation 2010

Mega Design Consulting Engineers in their December 2010 report made the following proposals:

(i) Clearing the watercourse of accumulated silt and overgrowth and widening its bed.

(ii) Dredging a secondary relief watercourse parallel to the existing one within the floodplain to intercept surface run-off from the hinterland to the east, thus reducing overland flow into the existing water course.

(iii) Collection of surface run-off from direct precipitation at the morcellement into sumps and disposal into the river by pumping during river floods, and by gravity in normal weather conditions.

(iv) Construction of a levee along the seaward bank of the river course to prevent the river flooding the morcellement.

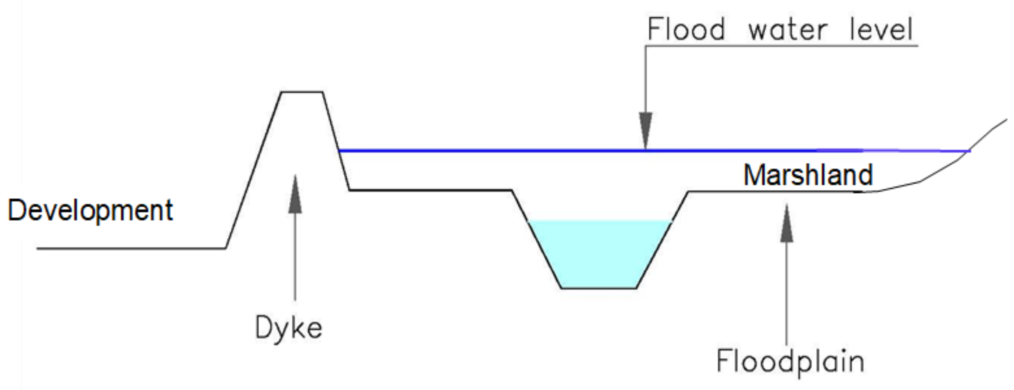


Figure 68: Levee construction

(v) Shifting of the fence bordering the seaward (right) bank of the river to its landward (left) bank to give unfetted access to the river at all times for regular clearing and maintenance without the need to pass through Societé de Marco's premises.

Following revision of the proposals by the Authorities, the following works got implemented:

(i) The river was cleaned of its overgrowth and its width enlarged to enhance flow and reduce the stage.

Cleaning of watercourse

(ii) A floodwall was built along the seaward bank of the river and against the existing fence to a height of 2.0m above mean sea level.

(iii) 6 no flap valves at each road dead-end to the wall to drain surface water from the morcellement into the river when the river stage is low.

(iv) A sump at each toe of the flap valves to make provision to house a submersible pump in the future to pump water accumulation away from the morcellement during flood events.

Floodwall with flap valve and pump sump

Discharge apron into river

No submersible pumps had been installed. Neither was the secondary intercepting relief watercourse constructed.

3.4.4.3 Supplementary Report 2015

In a supplementary report in May 2015, Mega Design recommended the following measures:

(i) A vehicular access across the river within Société de Marco premises constructed with Rill damming the river should be removed forthwith.

(ii) The flap valves removed by the Fire Services in an attempt to allow greater flow of standing water out of the morcellement, should be reinstated and any blockage to its proper functioning by dumped material material inside Societé de Marco cleared.

(iii) The construction of a secondary relief water course should be reconsidered and marshy spots opened up to join the relief water course as flood expansion zones up to its discharge point at sea.

River course

Marshy spots

3.4.4.4 Preliminary Design Report (PDR) 2020

Luxconsult in their Preliminary Design Report of February 2020 for the project "Drainworks at Morcellement De Chazal, Flic en Flac, in constituency no. 14" made certain observations, commented on previous works undertaken and made certain recommendations for Morcellement De Chazal as described below. These are followed by comments from LDA and a review from the Consultant.

Observations:

(a) The area had been built over an abandoned sand quarry without the ground levels being raised to their original state, resulting into some areas being almost at sea-level.
(b) The existing drainage infrastructure comprising road side drains and sumps do not have any gravity assisted outlet resulting into water retention into the same. The provided drainage infrastructure would only have been functional if pumps were implemented.
(c) Provisions for installation of pump outlets into the water course were found during the site visit. However, the sizing of the existing road side drains and sumps were found to be inadequate.
(d) Overland flows from Societé de Marco swells the river and "since the Morcellement de Chazal is about 700 mm higher to the river, the water flows from the river into the Morcellement".
(e) "It was hoped that this wall, which is 2.0 m above mean sea level (amsl) would prevent the water from flowing into the Morcellement. However, this proved to be an ineffective solution according to some of the residents of the vicinity since the opposite occurs during heavy downpours."
(f) The existing pipe culverts found about 2 km downstream of the Morcellement is of inadequate carrying capacity, resulting into backflow during heavy downpours.

Recommendations:

(a) Construct a new stormwater network in RC drains of dimensions varying from 500 to 1200 mm wide by 300 to 700 mm deep.
(b) Connect the existing drains to the proposed new stormwater network.
(c) Channel the storm run-off into four different sumps with a retaining capacity of 6 m^3 each.
(d) Equips sumps with 2 pumps to discharge collected run off into the adjacent water course.
(e) Wall up the 6 opening in the floodwall to prevent overflowing water from the watercourse into the Morcellement during heavy downpours.

(f) Increase the carrying capacity of the watercourse by cleaning and desilting works.
(g) Landscape on extent of 16,850 m^2 into a pond of 35 000 m^3 in order to contain, depollute, desilt, prevent swelling of the watercourse and lower its discharge rate into the sea.
(h) Reconstruct the pipe culverts into a RC box culvert (3 box culverts of 3 m wide x 2 m deep).

Comments of LDA on Lux Consult's report:

In a correspondence dated 10th August 2020 LDA made the following comments and proposals on Luxconsult's report, following a presentation on 2nd June 2020 and a joint site visit on 18th June 2020.

(a) According to DRR study the region is subject to riverine flooding for a return period of 100 years.
(b) There are two flood areas besides Morcellement De Chazal, at River Walk and at Flic en Flac.
(c) A catchment based approach including upstream development such as Medine Smart City should be considered in the design.
(d) Maintain one of the existing sumps to be used as pumping station.
(e) A water accumulation up to 100 mm was observed in the existing drains, indicating a high water table. The effective depth of the existing drains should therefore be considered in the design.
(f) A retention basin upstream of the flood prone area with a regulated outlet to be considered.
(g) The pipe culverts to be replaced by a RC bridge of single span for ease of maintenance and free flow of water.

Consultant's review of PDR report:

(a) It is to prevent overflow from the river that the floodwall was built.
(b) The purpose of the existing road side drains is to transfer stormwater from frequent rainfall events to the sumps. Larger capacity drains or sumps will serve little purpose during less frequent (higher) rainfall events since the whole area including the drains and the roadway is inundated, the pumps serving to keep standing water to a safe level.
(c) Walling up the openings into the floodwalls would prevent gravity flow from the morcellement into the river during frequent rainfall events when the river is not overflowing. The flap valves prevent reverse flow when the river stage is higher than the morcellement ground.
(d) The "pond" supposedly intended as a retention pond will serve no purpose, being located within an already flooded plain.
(e) The invert level of a new culvert crossing cannot be made lower than the existing invert level of the pipe culvert as this will create a trough along the river profile.
(f) There is a limit on the height to which a culvert or a bridge, can be built with a raised road profile at this location. It is dictated by the maximum stage of the river, beyond which the river will overflow its banks into adjoining land and the road stretches on either side. It is also unlikely that blockage of a box culvert will occur at this location as to warrant a single span, given that this is a slow flowing river with no big trees around as to give rise to floating branches during cyclonic weather.
(g) It is also noted that Luxconsult did not take into consideration the whole catchment in its analysis and computation, having used a total surface area of <u>only</u> 8.2 km^2 instead of 21.1 km^2.

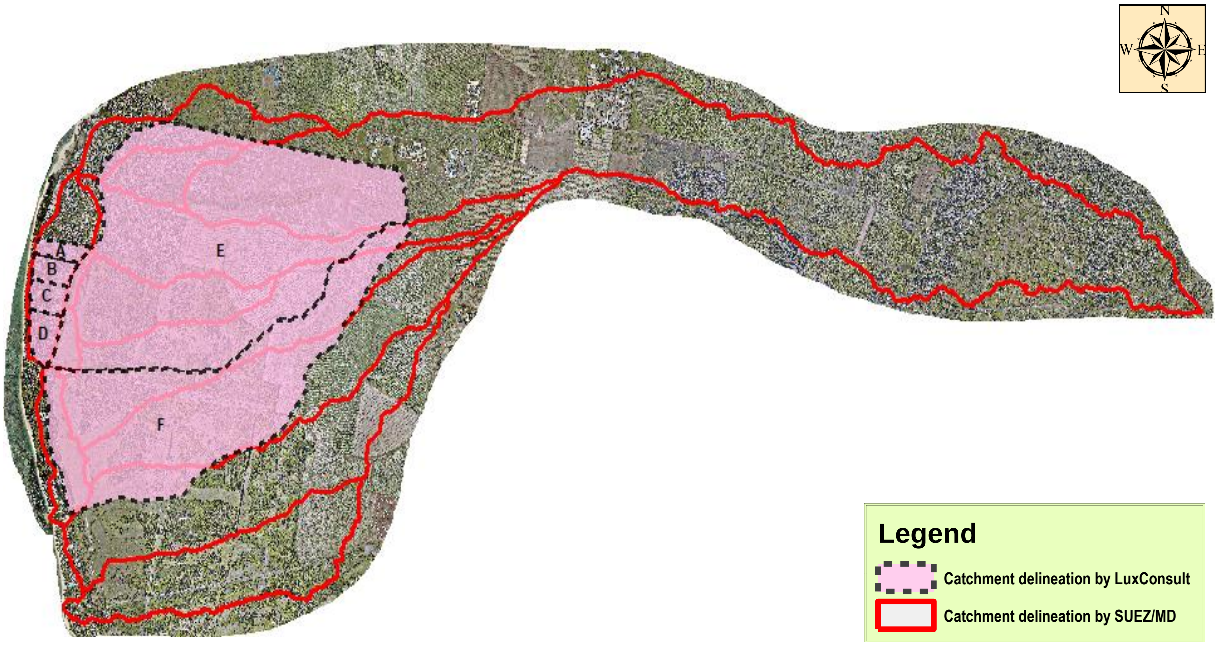


Figure 69: Catchment delineation by Lux Consult

3.4.5 PROPOSED SOLUTIONS

Except for Morcellement de Chazal which is highly vulnerable to storm water management within the whole catchment, the problems within the other flood prone areas are localised, necessitating localised solutions.

Recommendations to alleviate the flooding problems on those isolated areas are described together with an analysis of the problems.

3.4.5.1 Flic en Flac Village near Villa Caroline

A localized depression along a road stretch (Loday Lane) has been provided with a cross drain at each end and an absorption drain in between. However, it has also been provided with a speed ramp on each end which prevents surface runoff from the higher areas to be channeled into this drain resulting in water accumulation at the junction of Loday Lane and Villa Caroline road.

A simple solution is to channel surface runoff into the absorption drain by cutting a drainage path across each extremity of the speed ramps.

The work proposed is simply to clear the obstruction posed by the hump across the road and the feasibility study does not go to such an extent of detailing.

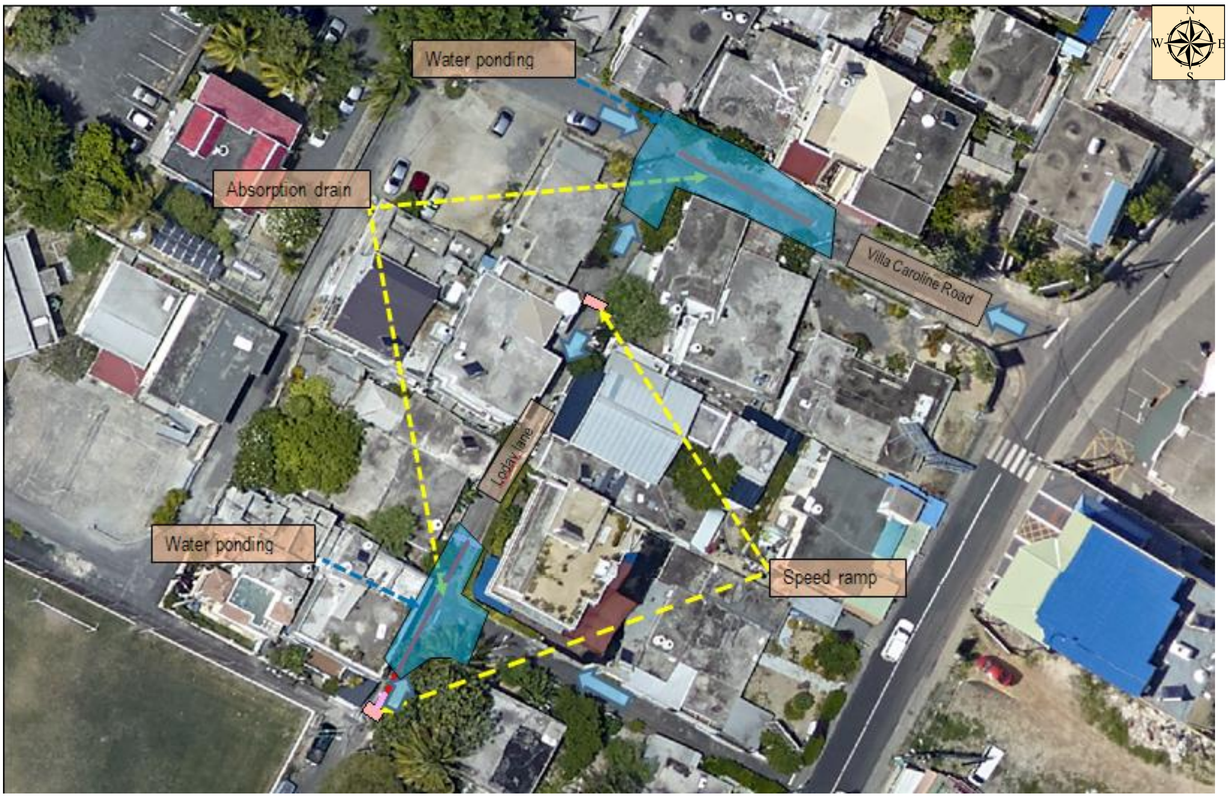


Figure 70: Coastal Road near Villa Caroline

3.4.5.2 Morcellement Bismisk

Like at numerous places at Morcellement Bismick, the roadside drains at Little Mermaid kindergarten near Nenuphar road culminate into a dead end with no exit to a major drain, a drainage path or a watercourse, resulting in water accumulation during frequent rainfall events before it percolates into the sandy soil.

This area is some 100m from an environmentally sensitive area (ESA) and a drain extension to this water body will resolve the water accumulation problem.

Boundary wall construction over the inner wall of roadside drains should also be discouraged and the ESA within the development under construction preserved.

Boundary wall encroachment onto roadside drain

Construction adjoining ESA

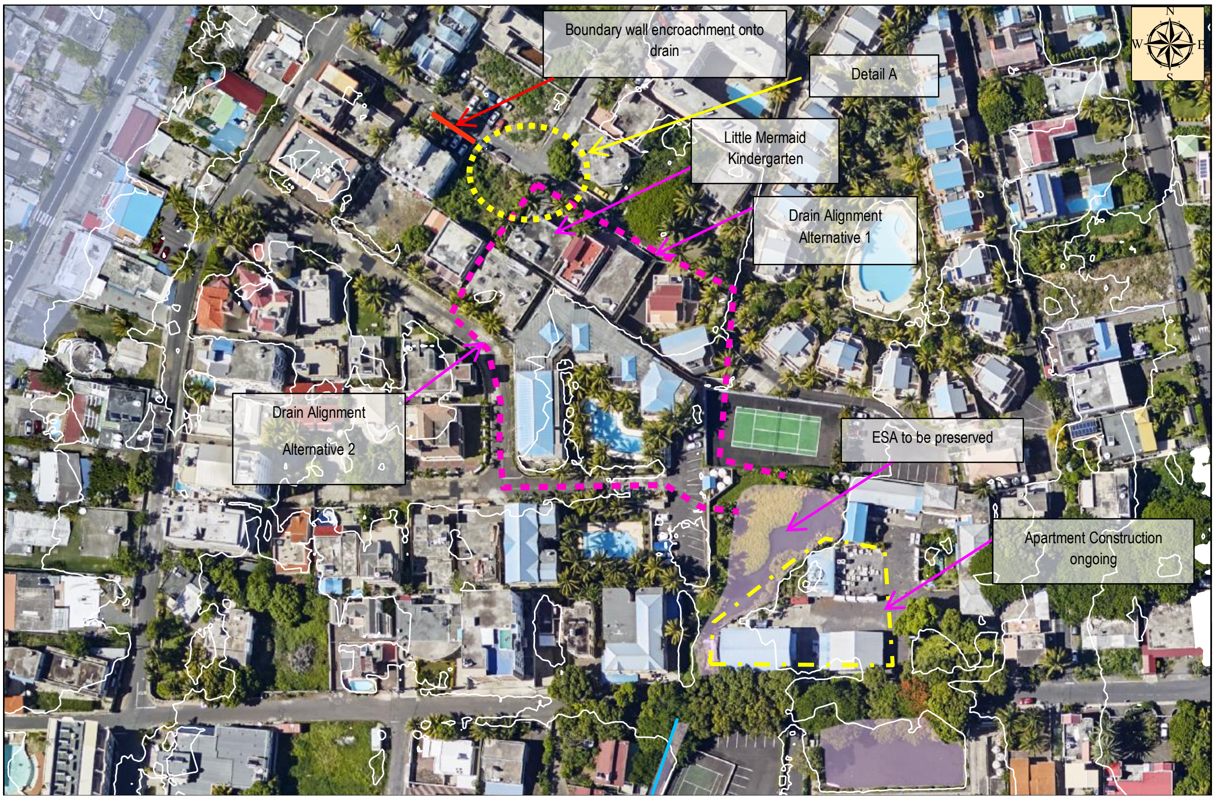
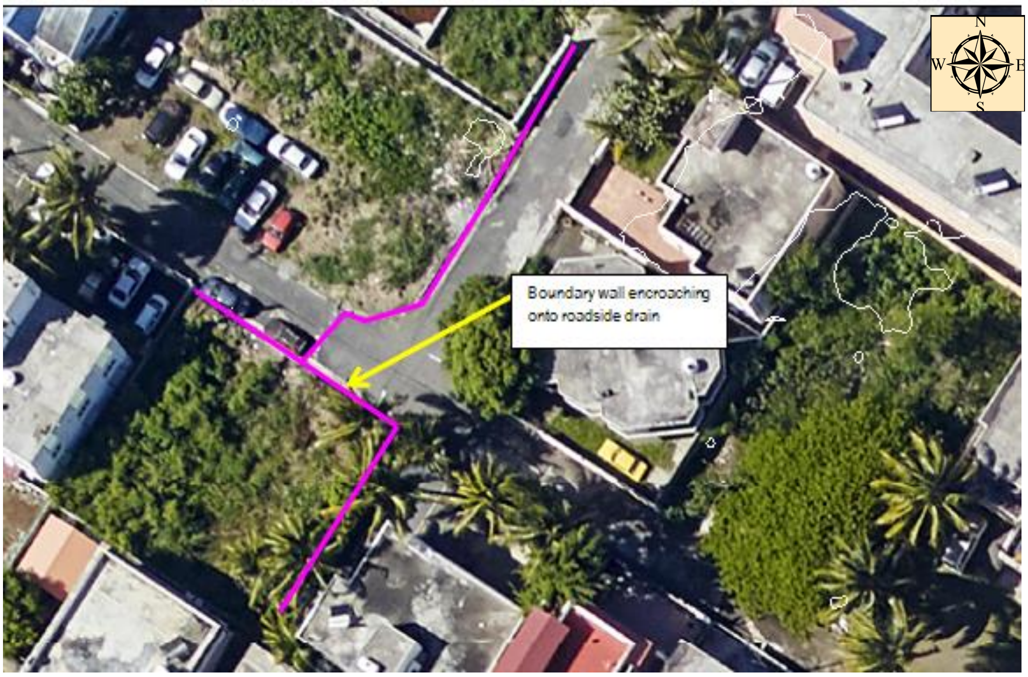


Figure 71: Proposed solutions at Morc Bismik



Detail A

Figure 72: Proposed solutions at Morc Bismik - Detail A

The proposed drain dimensions are 0.6m wide x 0.6m depth @ 1% slope. Carrying capacity is $1m^3/s$

The proposed measure will resolve the flooding problem in as much as the water body can accommodate the flow, the whole area being coastal and having a high water table.

3.4.5.3 Morcellement Palmyre

Two roads with adjoining plots constructed across a fairly deep valley receives flows from dead ended roadside drain as well as overland flow into their trough, inundating the roadway and the adjoining plots.

To resolve this localised problem, the depressions should be provided with a drain routed against the gradient and across the adjacent plot to join the invert of the existing roadside drains to the south.

The proposed drain dimensions are 0.6m wide x 0.6m depth @ 1% slope. Carrying capacity is $1 m^3/s$ (of the order of $2\ m^3/s$.)

Until and unless these drains are constructed, the adjoining plots should be left undeveloped and potentially converted into bio retention ponds.

The proposal made is to simply drain a low spot

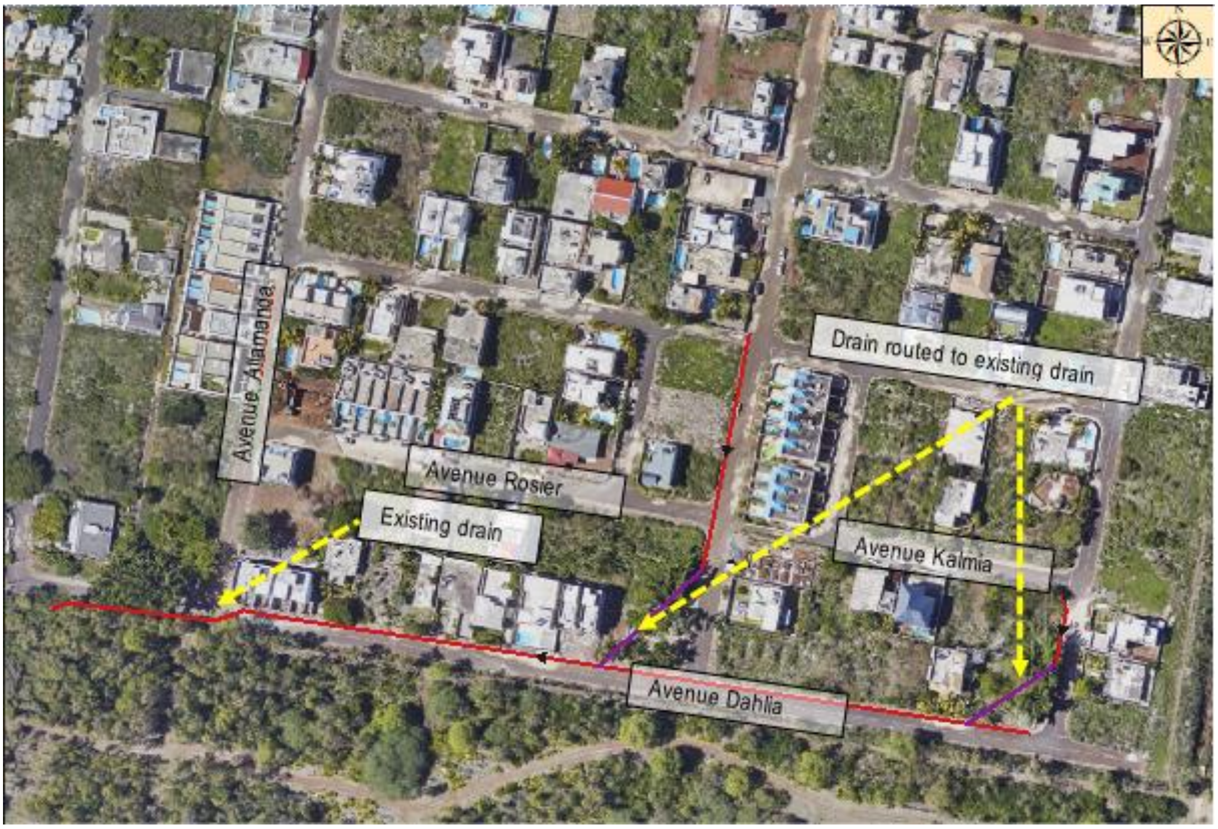


Figure 73: Proposed solutions at Morc Palmyre

On the figure, the red line is an existing drain to which the low points at morcellement Palmyre will be connected.

The proposed cut-off/ diversion drain shown in "Figure 74: Proposed solutions at Morc de Chazal" is a different drain altogether, not shown on the above drawing. It will be routed on the opposite side of the road.

3.4.5.4 Morcellement De Chazal

The following proposals have been investigated and are illustrated and described below:

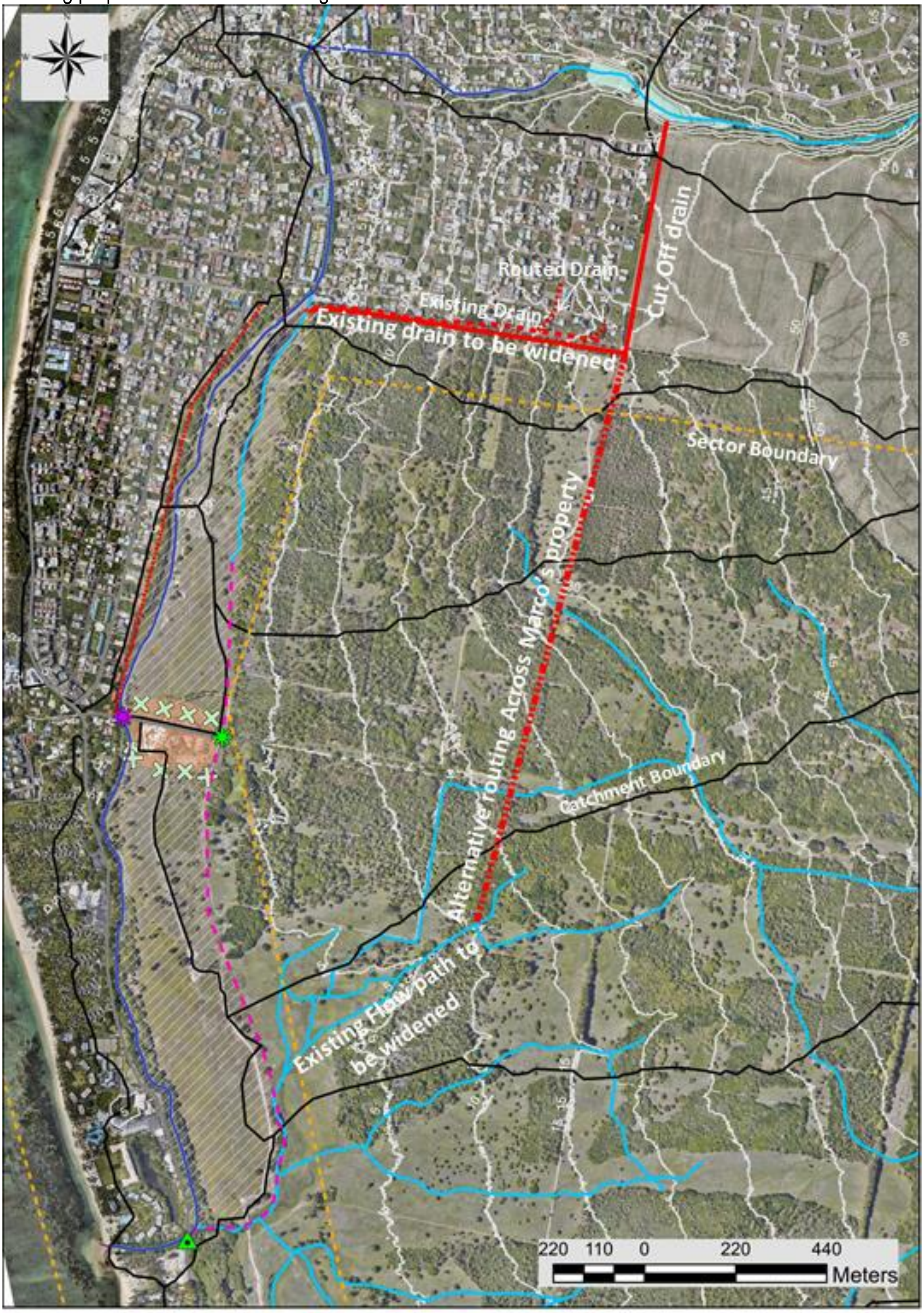


Figure 74: Proposed solutions at Morc de Chazal

(i) Construction of a dyke across the river to provide a retardation basin to break peak storm flow. Details on different options are given below.

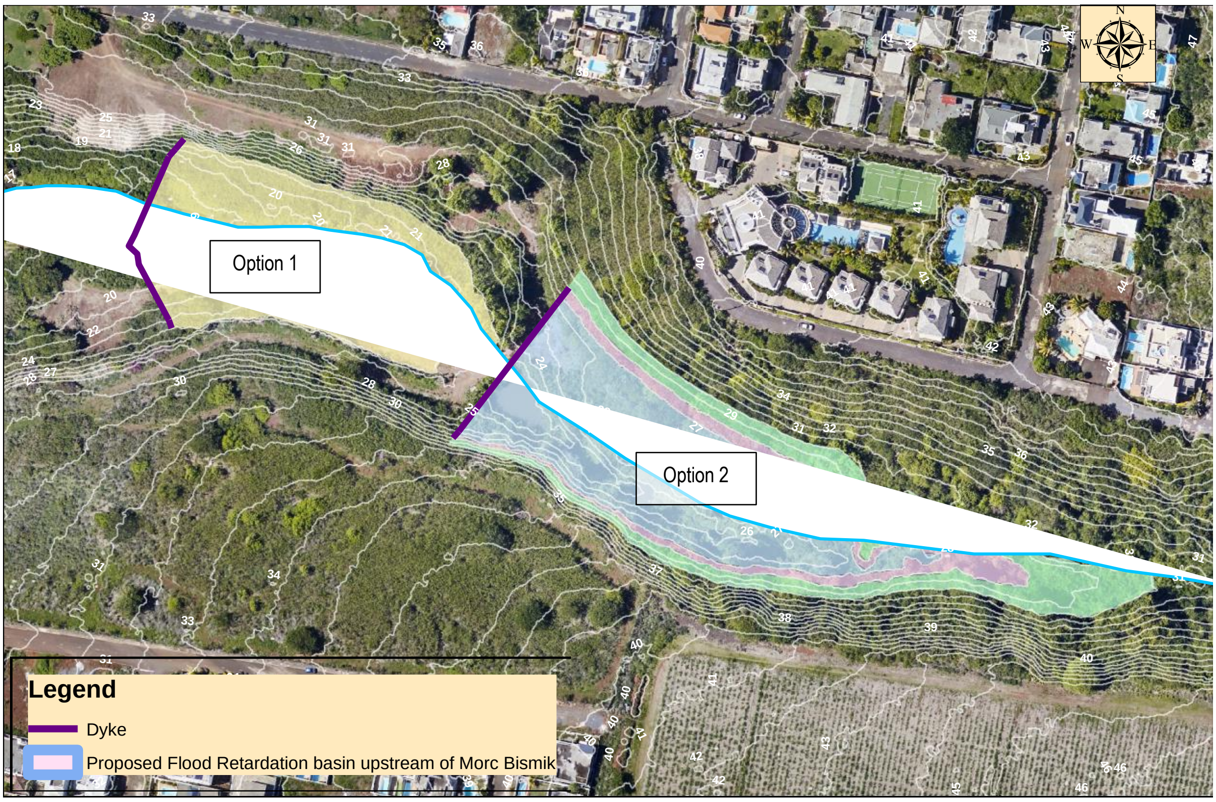


Figure 75: Options 1 & 2

Option 1:

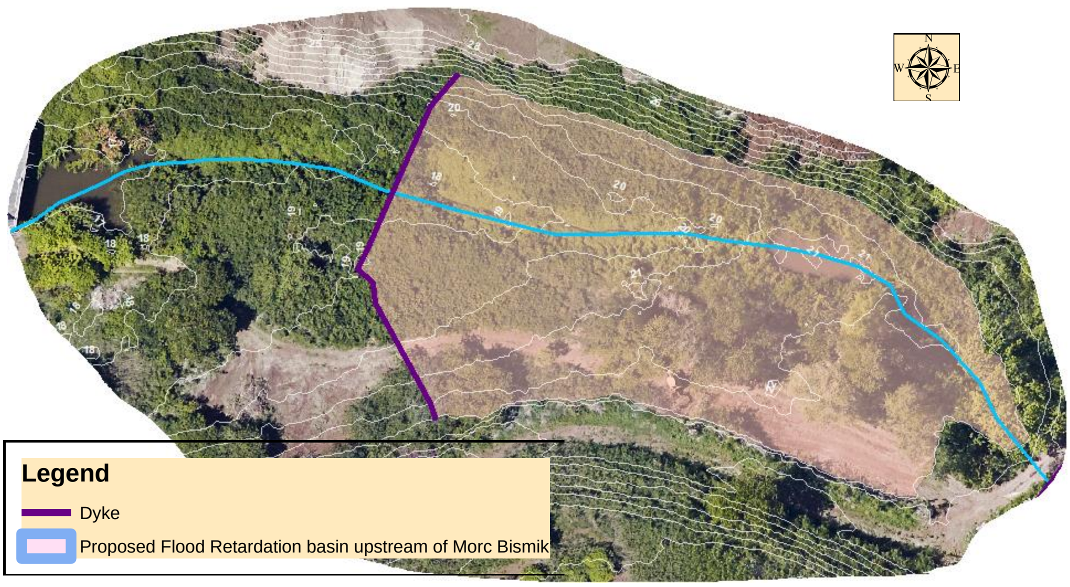


Figure 76: Blowup of Option 1

Height (m)	Surface area (m^2)	Net Capacity (m^3)
4	6000	12,000

Option 2:

Figure 77: Blowup of Option 2

Height (m)	Surface area (m²)	Net Capacity (m³)
3	2712	4000
4	5316	11000
5	7000	18000

The physical characteristics of the basins are as follows:

Figure 78: Physical characteristics Option 1 and option 2 – Flic-en-Flac

	Option 1	Option 2
Catchment Area (ha)	759	750
Foundation level (m amsl)	18	24
Top water level (TWL)	21	28
Water depth (m)	3	4
Area to TWL (m^2)	6,000	7,000
Capacity (m^3)	12,000	18,000
Crest level (m amsl)	22	29
Dyke height (m)	4	5
Dyke length (m)	76	78

Option 2 has been tested in the model as it provides a bigger storage capacity and its location ensures discharge into the cut-off drain (COD) in the recommended configuration as shown below.

Table 38: Peak flow downstream of dyke

Return period (years)	10	25	50	100
Peak flow (m^3/s) (without structure)	127.4	163.6	186.7	212.0
Control outflow (m^3/s)	10	10	10	10
Resultant Peak Flow (m^3/s)	127.0	162.4	184.6	208.4

The primary role of this particular basin is not to attenuate floods but to elevate the hydraulic grade line to enable discharge into the cut off drain (not feasible otherwise because of the steepness of the gorge).

The basin outlet is a 2.2m wide and 1m high culvert having a capacity of $10m^3/s$.

The dyke should be equipped with a safety spillway at 28.92 m amsl to allow water to spill over during intense rainfall events.

(ii) A secondary relief drain at the level of Morcellement de Chazal running parallel to the existing watercourse and discharging directly into the existing sea outlet. The recommended dimensions for this drain are 10m wide x 2.5 m deep with a slope of 0.1%. Carrying capacity is 26 m^3/s
This relief drain could ultimately be made to merge with the existing watercourse of width 15-20m to form a flood expansion strip of minimum width of 150 m, with all obstructions (river damming, dumped construction material and plant overgrowth) removed and the Societe de Marco offices relocated if possible.

(iii) A cut-off drain to intercept most of the flow at the upper end of Morcellement Palmyre into the relief drain. Recommended dimensions are the same as the secondary relief drain.
An alternative cut-off drain alignment would be directly across Societé de Marco from uphill of Morcellement Palmyre to the existing sea outlet. This alignment would however be outside the direct control of public authorities and may hinder the operation of Societé de Marco.

The flow in m3/s intercepted by both watercourse for a return period of 100y is summarize in the table bellow

	Existing state	Project state
Existing watercourse	80	70
Secondary relief drain	5.6	26

the flows are outside the minor bed - the flows within the two watercourses will merge.

The existing water course varies between 15m and 20m with depths not exceeding 2.5m, much of which being dead storage filled by transported sediments

About alternatives of Cut of drain upstream of Morcellement De Palmyre :

Our recommendations in terms of feasibility concerns the layout of the annexed figure, i.e. the perpendicular cut-off drain to technically ensure the water intake in the river. There will be no problem with head loss if the bends are hydrologically well designed. The earth lined cut of drain intercepts 13m³/s.

(iv) In order to permit regular monitoring and maintenance of the watercourse cum flood expansion strip by Government Authorities, shifting of the fence along the landside of the flood expansion zone and construction of an access track between the floodwall and the watercourse. This will revert some 47 ha of land including the watercourse and reserve to the Government.

(v) Construction of a new culvert (10 x 2.5m deep) at Societé de Marco premises, if its offices are not relocated. Carrying capacity of the culvert will be 90m³/s.

To summarise, the whole stretch within Societé Marco will be expanded, except for the island housing Marco's offices where a culvert additional to that existing is proposed in the event that the offices cannot be relocated.

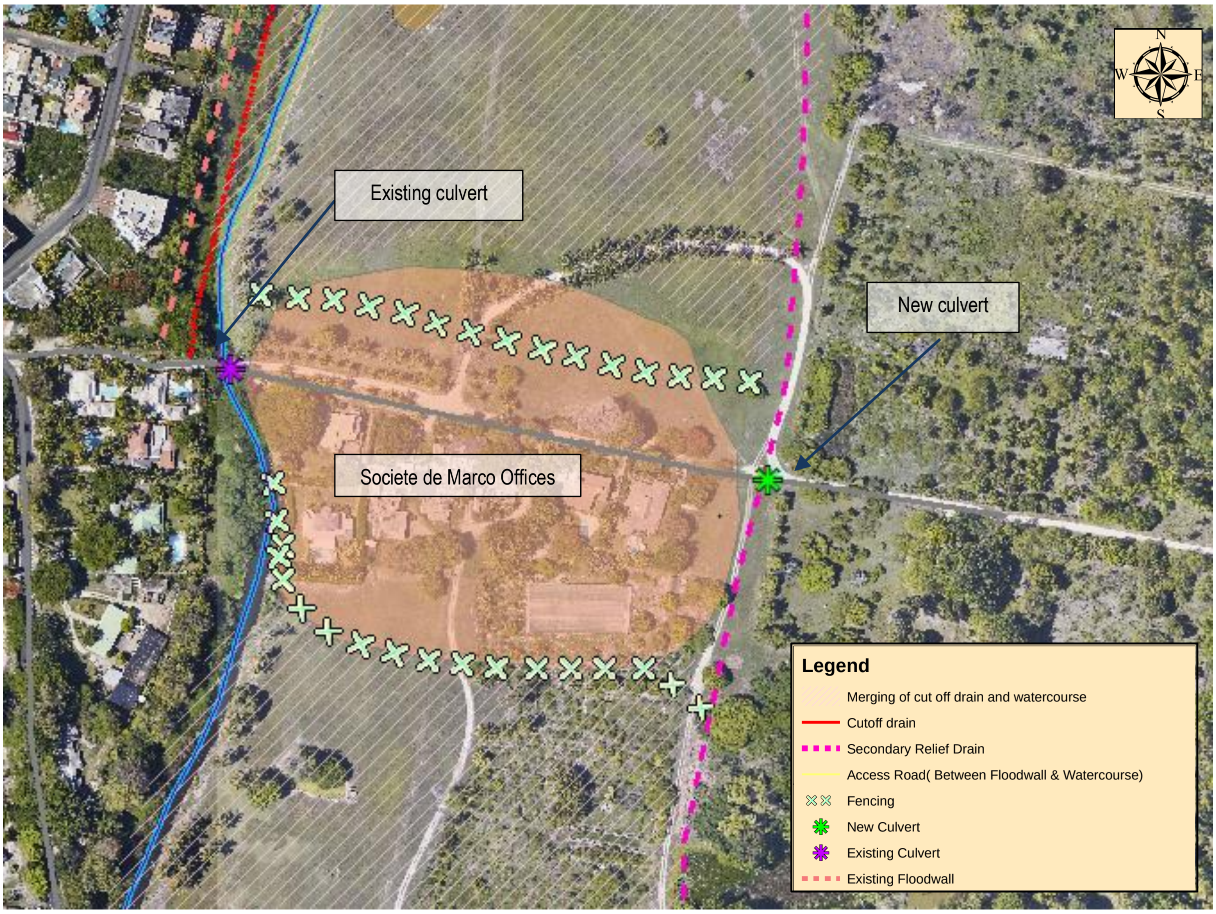


Figure 79: New culvert at Societé de Marco premises

(vi) Reinstatement of the flap valves at the 6 outlets within the floodwall to drain run-off into the water course during frequent (small) rainfall events when the river stage is lower than the ground level at the morcellement.

It is generally not common practice to have flap valves in open channel flow as the differential head is too low to promote operation of the valve and developers should design their project to ensure free discharge flow above the receiving water level.

Should this however not always be feasible like at Flic en Flac:

- hand operated or actuated sluice gate equipped with sensors may be a final recourse. The sensors are able to detect differential heads between the upstream and downstream faces of the gate and prompt the gate to open or close accordingly.
- Lightweight flap valves with counter balancing weight can be acceptable as an alternative.

Flap Valve

In the case of Flic en Flac a valve (sluice gate or flap valve) is mandatory to prevent rising water from the watercourse during high flows from infiltrating into the residential area.

(vii) Provide pumps into the 6 sumps at the outlets to discharge run-off resulting from direct precipitation at Morcellement De Chazal during very heavy rainfall events when the river stage is higher than the ground level at the morcellement. Submersible pumps have the drawback of getting easily clogged when used to pump foul or stormwater. Archimedes screw pumps which are low head/high lifting capacity pumps, although involving a higer capital cost, carry a much lower operation and maintenance cost and run almost clog free.

Archimedes pumps

(viii) Construction of a wide multiple cell culvert of aggregate width 22 m x 1.0 m deep in replacement of the pipe culvert outfall into the sea to pass a 50 year return peak flow of 290m^3/s against a high tide level of 0.4m amsl.

Downstream of culvert

Sea Outfall

Further details:

- The report does not recommend the proposal made in Gibb's report of 2005 to create a second sea outlet at the level of the north end of the public beach on ground that this would cause severe environmental damage (Ref 3.4.4.1)

- Creation of a secondary relief drain parallel to the existing watercourse within the property of Marco is equivalent to increasing the capacity of the watercourse (Ref 3.4.5.4 (ii))

- The cut-off drain actually reduces the flow into the watercourse downstream, which permits flood routing by the retardation basin.

- An alternative routing of the cut-off drain would be squarely inland across Societé de Marco directly to the sea as suggested by one of the stakeholders during the steering committee meeting

- Flood wall along Morcellement De Chazal: The existing floodwall prevents overflow from the watercourse during flood events from inundating Morcellement de Chazal. The residual problem is how to evacuate direct rainfall at the morcellement with the stage of the river higher than standing water within the morcellement, and the only solution is pumping.
  The extent of land between the existing floodwall bordering Morcellement de Chazal/ fencing further south and the right bank of the secondary relief drain is approximately 53 ha and this includes the existing watercourse and its reserve. A cadastral survey will be required to identify the actual extent of land belonging to or leased to Societé de Marco. It is to be noted that a proposed fence (deer fence made of 8/10g galvanised steel wire @150mm along the right bank of the secondary drain to demarcate Societé de Marco's property had been discounted by the LDA on ground that it will retain floating debris emanating from the upland agricultural lands.

- Confirmation has been obtained that the Flic-en Flac bypass will be routed along the east-west leg of the cut-off drain and a culvert will be required at the crossing of the upstream leg.

- Reconstruction of the pipe culvert at the sea outlet will require widening of the river bed on its left bank bordering the public beach by 10m and raising of the deck level to permit a minimum active depth of 1.0m which will also involve reprofiling the longitudinal section of the road over a stretch of some 50m on either side of the new bridge.

- Hydraulic modelling against sea level: Hydraulic modeling in projected situations has been performed with the sea level 400mm above mean sea level

- Measure and backflow analysis:

The figure below shows the streamlines generated by the modelling.

Figure 80 : Flic en Flac Outlet: Modelled streamlines around seaoutlet

The secondary relief drain has a capacity of 26m³/s and the existing water course has a capacity of 30 m³/s. The enlarged bridge and widened section of the watercourse downstream of the bridge (sea outfall) will have a capacity between 50m³/s and 60m³/s. The enlarged bridge and the sea outfall will thereby be able to transit the flows from the existing water course and the proposed secondary relief drain as shown in the figure above.

With respect to sand accretion combined with high tides, only a regular maintenance programme can render the recommended measures effective. Maintenance checks and maintenance works should be carried out before the advent of such predictive occurences. With regard to the tidal level, especially in relation to climate change (rising sea level), the only practical solution is to strategically retreat or relocate infrastructure in the medium and long term (relocation).

3.4.6 MAIN IMPACTS, COST AND COST-BENEFIT

The following table summarises the measures retained and their associated costs. The works item number are cross referenced in the plans provided in Annex 1.

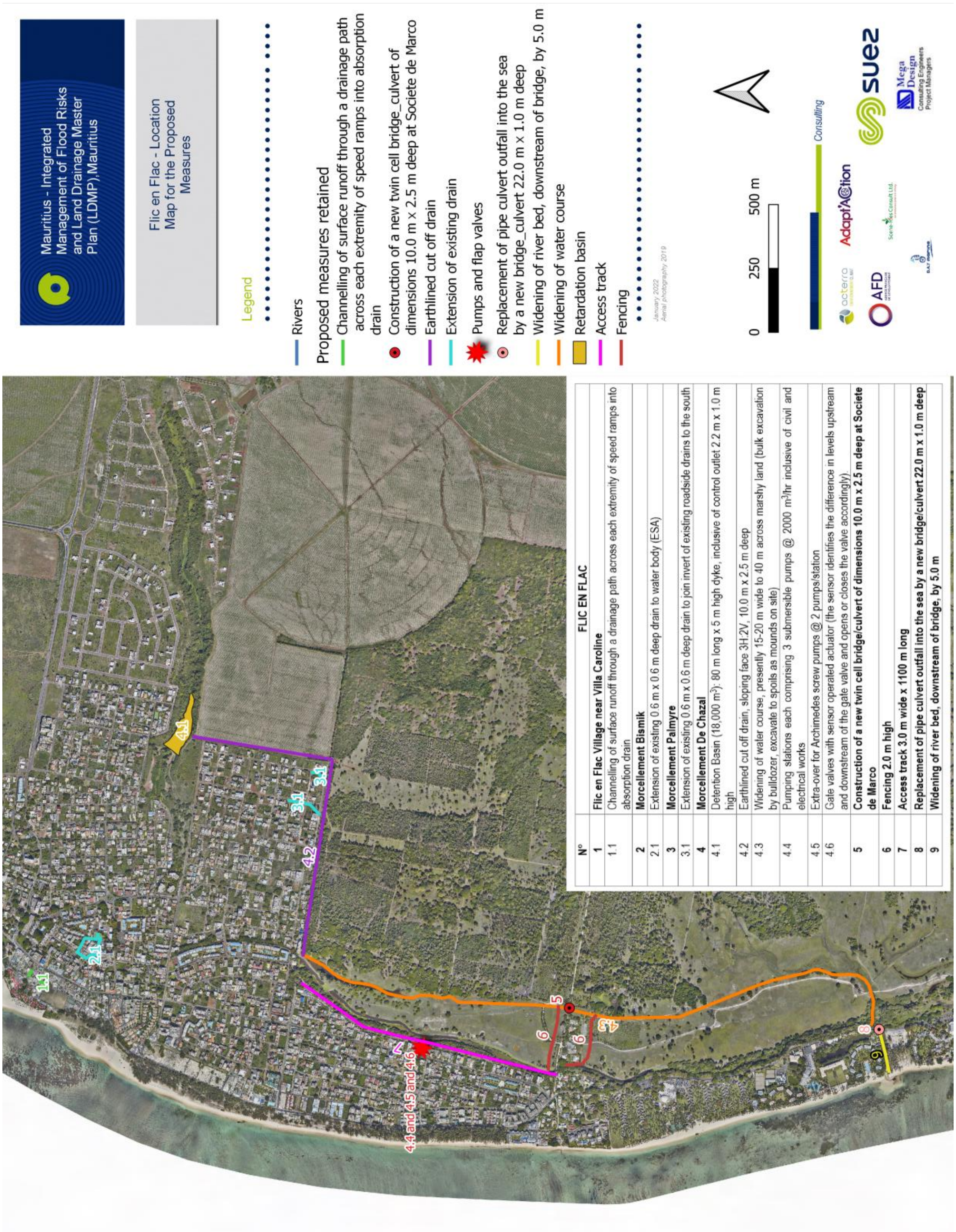
The comparative flood maps and the cost-benefit analyses together with cost details are given in Annexes 2 and 3.

The flood maps for the current situation (ie “ Do Nothing Scenario”) are attached in Annex 0: they are also attached to Deliverable D3.

The proposed measures will reduce the water level in the proposed situation by about 5 to 15 cm for the following FPAs in the village area:

- 243 and 234 - Flic en Flac
- 242 - Morcellement Chazal

The intention of the study is to reduce the depth of water accumulation within a coastal environment, not to provide a “dry feet” solution, in particular for a development constructed in an old sand quarry without consideration of water table.



N°	FLIC EN FLAC
1	**Flic en Flac Village near Villa Caroline**
1.1	Channelling of surface runoff through a drainage path across each extremity of speed ramps into absorption drain
2	**Morcellement Bismik**
2.1	Extension of existing 0.6 m x 0.6 m deep drain to water body (ESA)
3	**Morcellement Palmyre**
3.1	Extension of existing 0.6 m x 0.6 m deep drain to join invert of existing roadside drains to the south
4	**Morcellement De Chazal**
4.1	Detention Basin (18,000 m^3): 80 m long x 5 m high dyke, inclusive of control outlet 2.2 m x 1.0 m high
4.2	Earthlined cut off drain, sloping face 3H:2V, 10.0 m x 2.5 m deep
4.3	Widening of water course, presently 15-20 m wide to 40 m across marshy land (bulk excavation by bulldozer, excavate to spoils as mounds on site)
4.4	Pumping stations each comprising 3 submersible pumps @ 2000 m^3/hr inclusive of civil and electrical works
4.5	Extra-over for Archimedes screw pumps @ 2 pumps/station
4.6	Gate valves with sensor operated actuator (the sensor identifies the difference in levels upstream and downstream of the gate valve and opens or closes the valve accordingly)
5	**Construction of a new twin cell bridge/culvert of dimensions 10.0 m x 2.5 m deep at Societe de Marco**
6	**Fencing 2.0 m high**
7	**Access track 3.0 m wide x 1100 m long**
8	**Replacement of pipe culvert outfall into the sea by a new bridge/culvert 22.0 m x 1.0 m deep**
9	**Widening of river bed, downstream of bridge, by 5.0 m**

Figure 81: Flic en Flac – Location map for proposed measures

Table 39: Flic en Flac – Measures and costs

N°	FLIC EN FLAC	Unit	Quantity	Rate	Amount
1	**Flic en Flac Village near Villa Caroline**				
1.1	Channelling of surface runoff through a drainage path across each extremity of speed ramps into absorption drain	Sum			**50 000**
2	**Morcellement Bismik**				
2.1	Extension of existing 0.6 m x 0.6 m deep drain to water body (ESA)	m	170	9 000	**1 530 000**
3	**Morcellement Palmyre**				
3.1	Extension of existing 0.6 m x 0.6 m deep drain to join invert of existing roadside drains to the south	m	75	9 000	**675 000**
4	**Morcellement De Chazal**				
4.1	Detention Basin (18,000 m^3): 80 m long x 5 m high dyke, inclusive of control outlet 2.2 m x 1.0 m high	Sum			12 000 000
4.2	Earthlined cut off drain, sloping face 3H:2V, 10.0 m x 2.5 m deep	m	1398	32 000	44 736 000
4.3	Widening of water course, presently 15-20 m wide to 40 m across marshy land (bulk excavation by bulldozer, excavate to spoils as mounds on site)	m^3	73860	200	14 772 000
4.4	Pumping stations each comprising 3 submersible pumps @ 2000 m^3/hr inclusive of civil and electrical works	No	6	4 000 000	24 000 000
4.5	Extra-over for Archimedes screw pumps @ 2 pumps/station	No	6	13 000 000	78 000 000
4.6	Gate valves with sensor operated actuator (the sensor identifies the difference in levels upstream and downstream of the gate valve and opens or closes the valve accordingly).	No	6	800 000	4 800 000
	Sub Total				**178 308 000**
5	**Construction of a new twin cell bridge/culvert of dimensions 10.0 m x 2.5 m deep at Societe de Marco**	Sum			**25 000 000**
6	**Fencing 2.0 m high**	m	500	2500	**1 250 000**
7	**Access track 3.0 m wide x 1100 m long**	m	1100	3800	**4 180 000**
8	**Replacement of pipe culvert outfall into the sea by a new bridge/culvert 22.0 m x 1.0 m deep**	Sum			**50 000 000**
9	**Widening of river bed, downstream of bridge, by 5.0 m**	m	200	10000	**2 000 000**
	Total				**262 993 000**
	ADD:				
	Provision for wayleave and Land Acquisition				50 000 000
	Contingencies 15%				39 448 950
	Project Management 7.5%				19 724 475
	Grand Total				**372 166 425**

3.5 Port Louis – Rivière du Pouce, La Poudrière Stream, Ruisseau des Créoles & Cut-off drain

3.5.1 OVERVIEW

The whole catchment area drained by Rivière du Pouce, La Poudrière Stream, Ruisseau des Créoles and Cut-off drain have an area of approximately 8.6 km² and has been sub-divided into 20 sub-catchments.

The catchment area drained by Ruisseau des Créoles (excluding Cut-off drain) has an area of approximately 1.1 km^2, a drainage path of approximately 2.6 km and an average slope of 2.5%.

The catchment area drained by the upstream part of Rivière du Pouce has an area of approximately 5.45 km^2, a longer drainage path of approximately 6.35 km and an average slope of 12.75%.

The catchment area drained by La Poudrière Stream has an area of approximately 0.95 km^2, a drainage path of approximately 2.2 km and an average slope of 7 %.

Figure 82: Sub-division of catchment area drained by Rivière du Pouce, La Poudrière Stream, Ruisseau des Créoles and Cut-off drain into sub-catchments (Orthophoto 2019)

Table 40: Rivière du Pouce, La Poudrière Stream, Ruisseau des Créoles and Cut-off drain – Physical Characteristics of individual Sub-catchments

Name	Area (ha)	Area (km²)	Low level (m)	High level (m)	Length	Slope (m/m)	Slope (%)
SMCoD_BV01	48.90	0.49	45	373	806	0.41	40.79
SMCoD_BV02	43.89	0.44	32	394	1041	0.35	34.82
SMCoD_BV03	18.12	0.18	26	314	756	0.38	38.09
RC_BV01	29.81	0.30	13	65	657	0.08	7.86
RC_BV02	26.12	0.26	7	45	941	0.04	4.04
RC_BV03	11.77	0.12	3	16	982	0.01	1.35
RC_BV04	14.78	0.15	6	38	233	0.14	13.69
RC_BV05	22.05	0.22	0	9	829	0.01	1.04
RC_BV06	5.27	0.05	0	4	393	0.01	1.31
RP_BV01	260.31	2.60	46	811	3266	0.23	23.44
RP_BV02	23.49	0.23	19	130	1245	0.09	8.88
RP_BV03	130.51	1.31	20	629	2362	0.26	25.79
RP_BV04	13.83	0.14	10	65	796	0.07	6.94
RP_BV05	42.64	0.43	39	358	1400	0.23	22.79
RP_BV06	30.61	0.31	10	154	1128	0.13	12.78
RP_BV07	20.04	0.20	1	29	1239	0.02	2.21
RP_BV08	18.38	0.18	0	14	813	0.02	1.72
RP_BV09	5.42	0.05	0	4	248	0.02	1.77
PS_BV01	68.08	0.68	4	154	1367	0.11	10.97
PS_BV02	27.40	0.27	0	78	1321	0.06	5.90
RC_Global (Excluding cut-off drain)	**109.81**	**1.10**	**0**	**65**	**2629**	**0.03**	**2.52**
RP_Global	**545.24**	**5.45**	**0**	**811**	**6357**	**0.13**	**12.76**
PS_Global	**95.48**	**0.95**	**0**	**154**	**2199**	**0.07**	**6.99**

Table 41: Rivière du Pouce, La Poudrière Stream, Ruisseau des Créoles and Cut-off drain – Flows for sub-catchments and at outlet of catchment for return periods of 10, 25, 50 and 100 years

BVs	Q10 (m3/s)	Q25 (m3/s)	Q50 (m3/s)	Q100 (m3/s)
PS_BV01	20.19	25.07	28.02	31.32
PS_BV02	7.87	9.71	10.88	12.15
Outlet of Poudrière Stream	**26.8**	**31**	**34.2**	**37.3**
Q /A (m³/s/km²)	***28.1***	***32.5***	***35.8***	***39.1***
RP_BV01	42.65	57.47	67.63	78.60
RP_BV02	6.73	8.36	9.38	10.48
RP_BV03	25.05	33.20	38.68	44.54
RP_BV04	4.35	5.31	6.02	6.67
RP_BV05	8.80	11.52	13.43	15.48
RP_BV06	9.32	11.63	12.97	14.53
RP_BV07	5.59	6.89	7.76	8.57
RP_BV08	5.18	6.44	7.18	8.10
RP_BV09	1.55	1.92	2.20	2.50
Outlet of Rivière du Pouce	**76.3**	**104.5**	**123.4**	**143.5**
Q /A (m³/s/km²)	***14.0***	***19.2***	***22.6***	***26.3***

BVs	Q10 (m3/s)	Q25 (m3/s)	Q50 (m3/s)	Q100 (m3/s)
RC_BV01	4.95	5.50	5.91	6.34
RC_BV02	4.17	4.58	4.95	5.24
RC_BV03	1.78	1.94	2.13	2.25
RC_BV04	2.67	2.91	3.14	3.35
RC_BV05	3.28	3.61	3.89	4.19
RC_BV06	0.83	0.92	0.96	1.05
SMCoD_BV01	5.06	6.04	6.82	7.60
SMCoD_BV02	4.56	5.39	6.13	6.76
SMCoD_BV03	1.84	2.21	2.50	2.78
Outlet of Ruisseau des Créoles	**24.6**	**28.2**	**31**	**33.7**
Q /A (m³/s/km²)	***22.4***	***25.7***	***28.2***	***30.7***

3.5.2 REVIEW OF PREVIOUS STUDIES

To homogenize the flow capacity on Le Pouce Stream and Ruisseau des Creoles hydrographic network to frequent events, corresponding to rainfall intensities of about 65mm over 1 hour, or 95m, the Port Louis drainage study conducted in 2015 recommended the following measures:

- **Le Pouce Stream and main tributaries:**
  - Resizing of the bridge over Gravier street on Tranquebar Stream (WxH=5x1.2m),
  - New drain (205 meter long) under the D'Estaing street (WxH=2x1.2 m) and parallel to the D'Estaing Stream to limit its flow, and flood protection walls with 0.5m height for about 35 meter long,
  - Access ramp demolition on Le Pouce Stream (works already engaged),
  - Flood protection walls on Le Pouce Stream right bank, with 0.5 m height for about 75 meter long and 1 m height for about 190 meter long,
  - Stream invert restoration on about 150 m (same level as existing)

- **Signal Mountain cut-off drain:**
  - Works have already been done

- **Ruisseau des Creoles:**
  - Flood protection walls with 0.5m height for about 125 meter long

Priority works recommended in the ER2C report comprise:

- M1 bridge resizing: width of 30 m (25 m projected on the axis perpendicular to the watercourse) and height under the deck of 2.60 m,
- Downstream channel toward Caudan basin desilting,
- Suppression of covered sections on Le Pouce Stream (350 m), La Poudrière Stream (85 m) and Ruisseau des Créoles (100 m),
- In order to facilitate overflow flows for events greater than the design of these structures, concrete traffic separators on the M1 between Le Pouce Stream and place d'Armes should be removed.

In the longer term, to

(i) Increase the flow capacity of Le Pouce Stream to allow the flow of rare events, corresponding to rainfall intensities of about 140 mm over 1 hour, or 200 mm over 2 hours, and

(ii) Limit overflows on the roadway in the upstream watershed of Ruisseau des Créoles, between the Signal Mountain cut-off drain and Volcy Pougnet Stream.

The following additional proposals were made:

- **On Ruisseau des Creoles**
    - New stormwater network along Raoul Rivet street and d'Artois street

- **On Le Pouce Stream**
    - Enlargement of Le Pouce Stream section about 18m downstream of Labourdonnais street (currently between 10 and 12m on this line) in order to increase its capacity and maintain the same width as the existing one upstream. The widening will be done first on the left bank, then on the right bank at the right of the current roof and again on the left bank downstream of Chaussée street (condemning of Reverend Lebrun street).
    - Downstream of Jardin des Compagnies (rue de la Chaussée), the slope of the Pouce Stream decreases sharply, from 0.54% to 0.18%. As the existing buildings do not allow the bed to be widened to compensate for the decrease in the slope, part of the flow of Le Pouce Stream must be discharged towards the topographic low point upstream of the Brown Sequard Street bridge (in the Jardin des Companies) to join the La Poudrière Stream.
    - La Poudriere Stream, whose current capacity is very low given its reduced cross section, its low slope (0.2%) and the small opening of the structure crossing the Chaussée street and the downstream building, will be widened to 10m to allow the flow discharged from the Pouce Stream to flow. This enlargement requires the demolition of KFC built on the watercourse immediately downstream of la Chaussée street.
    - The flow towards the port of Caudan will be guaranteed by the prior widening of the M1 bridge and the dredging of the downstream channel.

These proposed works are illustrated in the figures below.

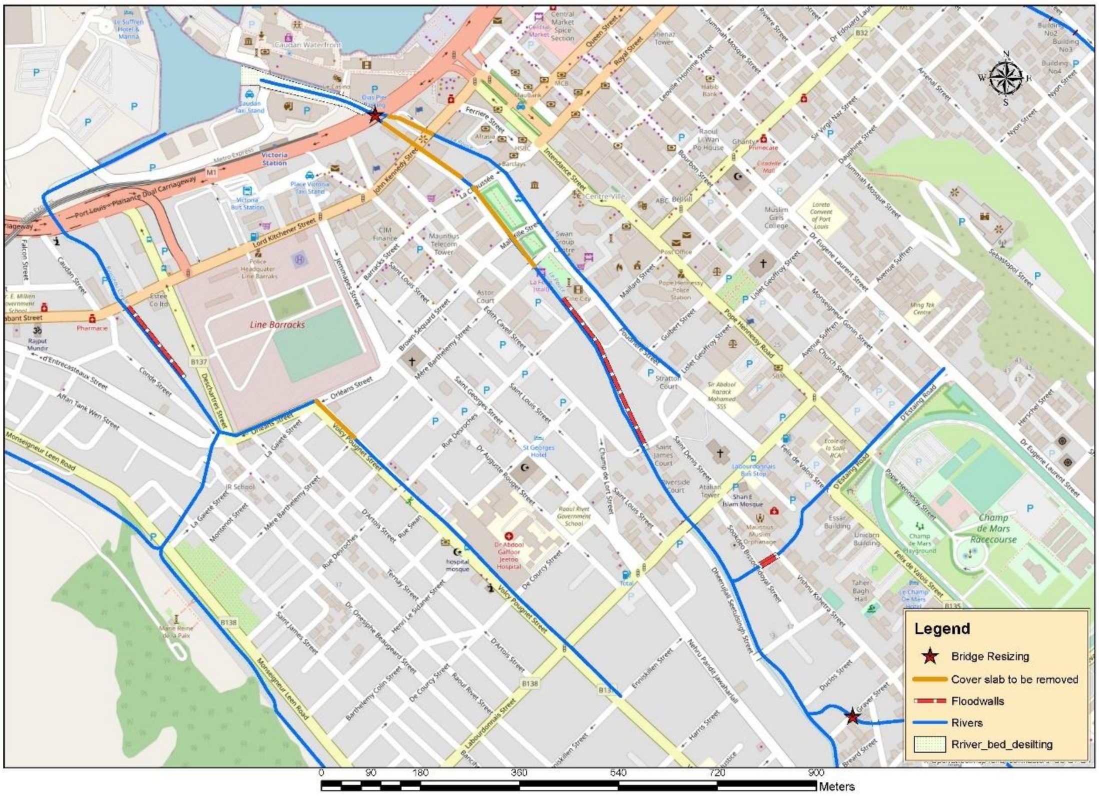


Figure 83 : Pouce Stream - Short Term Priority Measures

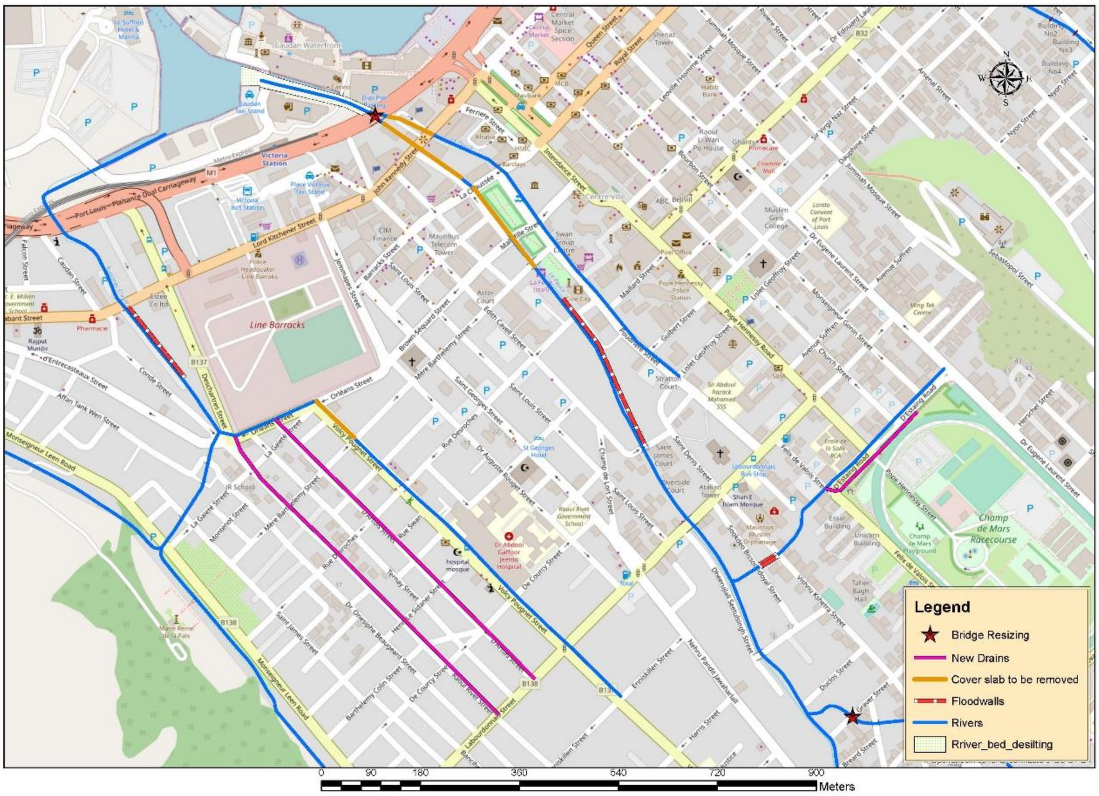


Figure 84 : Pouce Stream - Short and Long Term Priority Measures

Gibb's proposal on the Drainage Report for Port Louis 2021 recommends the following measures which are mostly those recommended in the ER2C report, except that no mention is made of the covered parking of the Pouce Stream at Rogers in the proposal.

- **Volcy Pougnet Stream**

A new stormwater drain along Dr Raoul Rivet St. from its junction with Labourdonnais St. up to its junction with Condé St. and the reconstruction of the bridge at Labrillane and Barthelemy St.

- **Ruisseau des Creoles Stream**

Raising of the banks on both sides of the stream over a length of 50 m by 400 mm and realignment of the CWA pipelines and thrust block obstructing flow at approximately 50 m upstream of M1 motorway.

- **La Poudrière Canal**

Removal of KFC building constructed over Poudrière Stream and upgrading of the canal from John Kennedy St. up to M1 motorway to cater for the new drain along Pope Hennessy St.

- **Le Pouce Stream:**

(i) Construction of a new drain along Frère Felix de Valois St., d'Estaing St. and Pope Hennessy St to discharge into the upgraded Poudrère canal after John Kennedy St.
(ii) Reconstruction of several bridges across Le Pouce Stream, including Gayasingh bridge, Lord Baden Powel bridge and two bridges within the Caudan premises.
(iii) Raising of the stream walls at several locations to prevent overtopping.
(iv) Removal of the cover slab on Le Pouce Stream from Majestic Cinema to Chaussée St. (Hawker's Area)
(v) Raising the M1 motorway and increasing the hydraulic capacity of the existing culvert.
(vi) Relocation of services encroaching into the waterway.
(vii) Possibility of an overflow drain to divert flow from Le Pouce Stream after the motorway crossing to another outlet at the Caudan Taxi stand.

- **La Paix Stream**

(i) Reconstruction of bridge at Boulevard Victoria and relocation of CWA pipelines.
(ii) Reconstruction of footbridge near Nyon Street.
(iii) Reconstruction of bridges at Renganaden Seeneevassen St. and Farquar St. and upgrading of the covered channel near the temple.
(iv) Raising or widening the culvert across M1 motorway.

- **Victoria Urban Terminal**

A new drain along Lord Kitchener Street discharging into Ruisseau Creoles and Le Pouce Stream.

3.5.3 HYDRAULIC MODELLING OF PROPOSED WORKS

A summary of the impacts of the proposed infrastructure is shown in the Table below. Details on the impacts of the works are also given.

3.5.3.1 Synthesis

Table 42: Proposed infrastructures Port Louis - Synthesis

	Infrastructure	Comments
Ruisseau Créole	**Removal of cover slabs**	Lowering of water depth by 200 mm, without preventing the zone being flooded
	Raising of banks	Prevents overtopping
Pouce Upstream	**Resizing of bridge at Gravier street**	Resizing this bridge reduces water depth by 250 mm at the confluence without preventing the zone being flooded
	New drains in addition to d'Estaing canal (near Champ de Mars)	Increase capacity
	Raising of banks of D'Estaing canal	Raising of the banks of the existing drain will reduce water depth without preventing the zone from being flooding.
Pouce-Poudière	**Raising of wall on right bank**	Raising of the banks prevents overtopping without preventing the zone from flooding due to backflow upstream of the raised banks and at Poudrière Stream
	Removal of cover slabs on Poudrière stream	Lowering of water depth by 700 mm over the whole stretch of Pouce and Poudrière without eliminating flooding in this sector.
	Removal of cover slabs on Pouce stream	
	Widening of Pouce stream to 18 m	This produces a rise in water level by 1.0 m at the confluence between Pouce and Poudrière as a result of the absence of overflows with removal of the cover slabs
	Widening of Poudrière stream to 10 m	
	By-pass between Pouce and Poudrière Streams	
	Resizing of bridge at M1	
	De-silting of sea outlet of Pouce	No impact because of the downstream level at 0.4 m

In addition to the infrastructure proposed above, the following infrastructure has been tested:

Ruisseau des Créoles : Duplication of the canal in Zone 1

Increasing the gradient at the bridges in Zone 2

Raising both banks in Zone 3 by 1.20 m

Results obtained are:

- Duplication of the drain in zone 1 enables this zone to be completely drained
- The velocity ramp in zone 2 decreases the standing water depth, but does not completely drain the zone.
- Widening of the drains in Zones 1 and 2 induces more flooding downstream.
- Raising of both banks in zone 3 enables this zone to be completely drained.

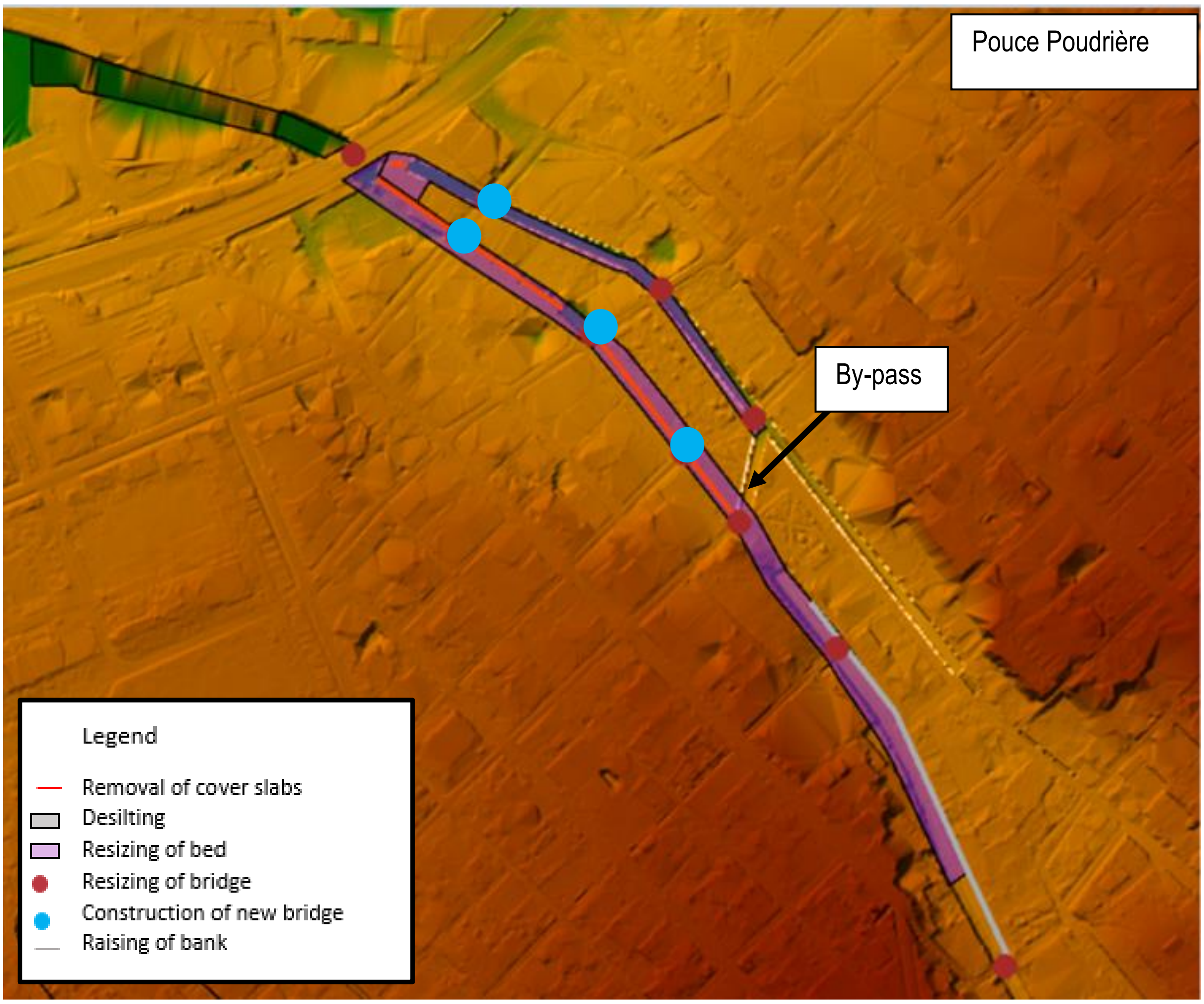
3.5.3.2 Hydraulic modelling of Pouce, Poudrière and Ruisseau Créole

3.5.3.2.1 Description

Infrastructure proposed comprises:

Table 43: Proposed infrastructures Port Louis – Dimensions and remarks

	Proposed Infrastructure Works	**Dimensions/Remarks**
Ruisseau Créole	Removal of cover slabs	
	Raising of walls	500 mm over 125 m
Pouce Upstream	Resizing of bridge at Gravier street	5 m x 1.2 m (Carrying capacity 15m³/s)
	New drains in addition to d'Estaing canal (near Champ de Mmar)	2x1.2m – 200m – 2.5% Carrying capacity 18m³/s)
	Raising the banks of D'Estaing canal	Test on raising the wall by 500 mm
Pouce-Poudrière	Raising of wall on right bank	0.5 m over 75 m and 1 m over 190 m (ER2C)
	Removal of cover slabs on Poudrière stream	Requires construction of a bridge
	Removal of cover slabs on Pouce stream	Requires 3 new bridges (Carrying capacity 90m³/s)
	Widening of Pouce stream to 18 m	Requires resizing of 2 bridges (Carrying capacity 90m³/s)
	Widening of Poudrière stream to 10 m	Requires resizing of 2 bridges (Carrying capacity 30m³/s)
	By-pass between Pouce and Poudrière Streams	Same size as Poudrière Stream
	Resizing of bridge at M1	30 m x 2.6 m, including removal of middle piles/wall (Carrying capacity 100m³/s)
	De-silting of sea outlet of Pouce	- 2m (maximum)



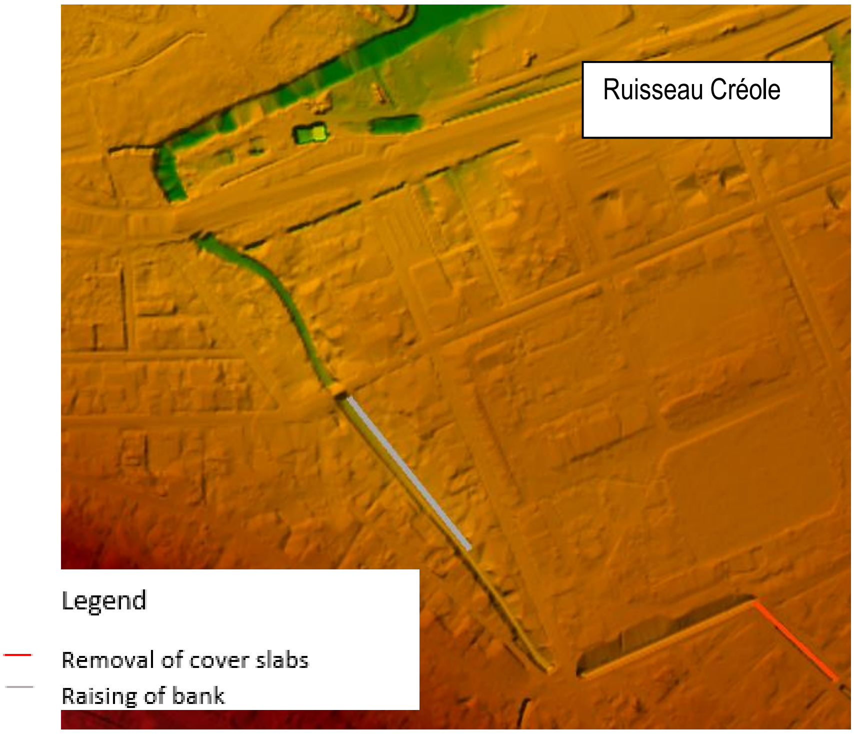


Figure 85 : Pouce Stream, Poudrière and Ruisseau Créole – basic infrastructure proposed (1/2)

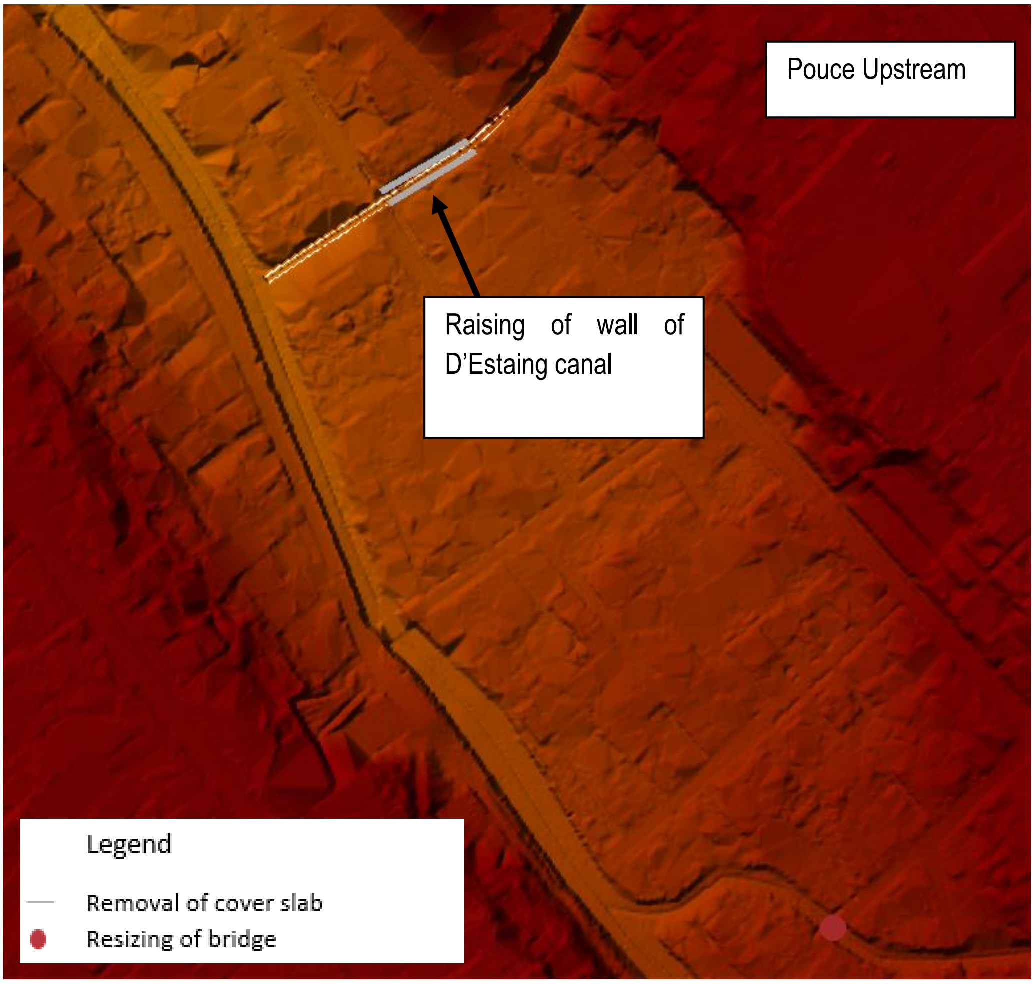


Figure 86 : Pouce Stream, Poudrière and Ruisseau Créole – basic infrastructure proposed (2/2)

3.5.3.2.2 Results

A screenshot of trial run showing the results for a Q_{100} flood, considering the infrastructure proposed above.

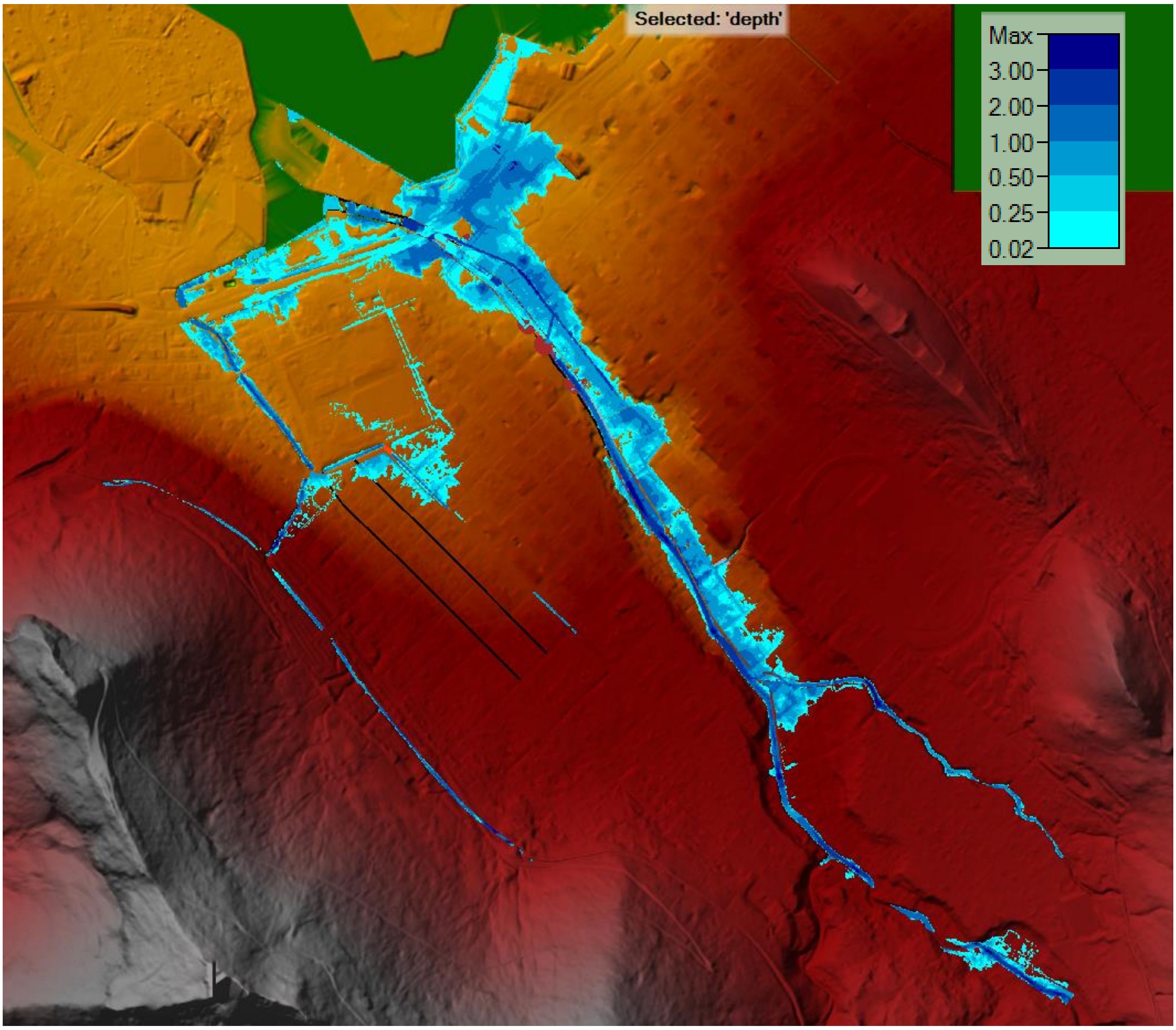


Figure 87 : Pouce Stream, Poudrière and Ruisseau Créole – Water depth (100 y event)

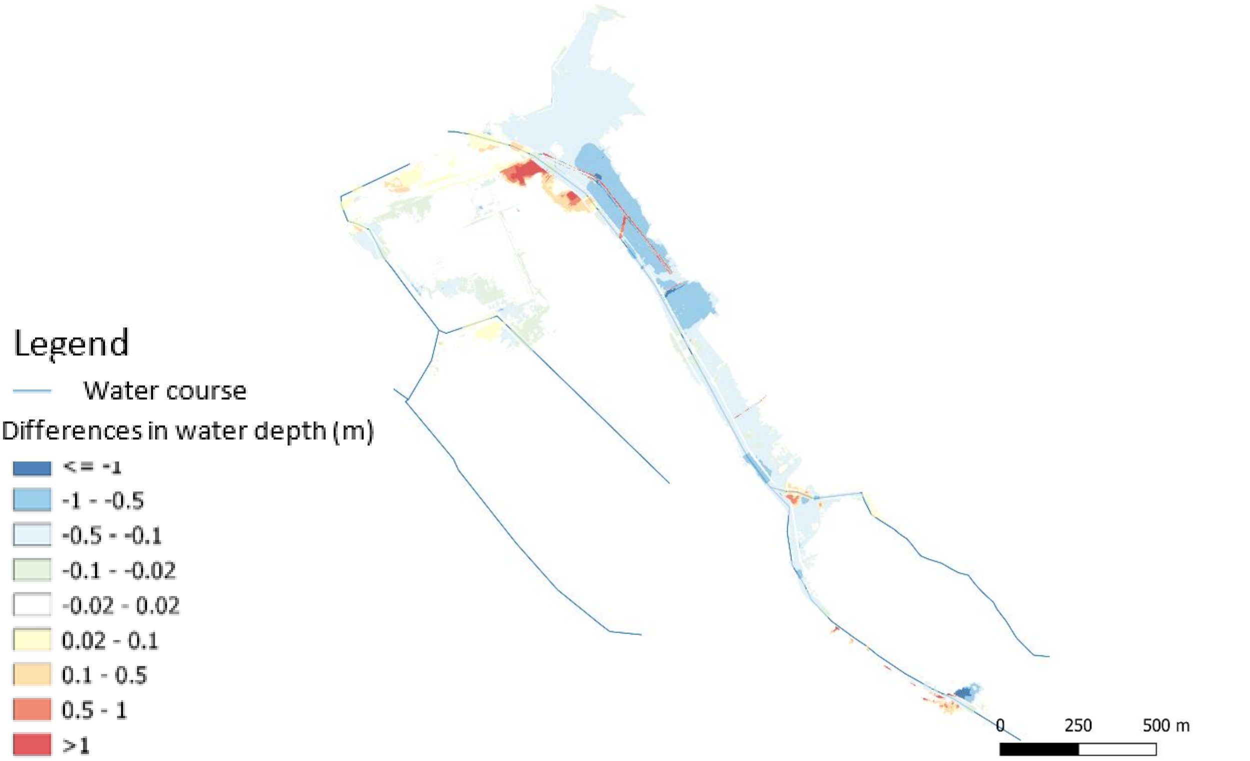


Figure 88 : Pouce Stream, Poudrière and Ruisseau Créole – Water depth impact (100 y event) with basic infrastructure

A summary of the impacts of the proposed infrastructure is shown in tabular form below.

Table 44: Summary of the impacts of the proposed infrastructure – Ruisseau Créole, Pouce upstream, Pouce Poudrière

	Infrastructure	Comments
Ruisseau Créole	Removal of cover slabs	Lowering of water depth by 200 mm, without preventing the zone being flooded
	Raising of banks	Prevents overtopping
Pouce Upstream	Resizing of bridge at Gravier street	Resizing this bridge reduces water depth by 250 mm at the confluence without preventing the zone being flooded
	Raising of banks of D'Estaing canal	Raising of the banks of the existing drain will reduce water depth without preventing the zone from being flooding.
Pouce-Poudrière	Raising of wall on right bank	Raising of the banks prevents overtopping without preventing the zone from flooding due to backflow upstream of the raised banks and at Poudrière Stream
	Removal of cover slabs on Poudrière stream	Lowering of water depth by 700 mm over the whole stretch of Pouce and Poudrière without eliminating flooding in this sector.
	Removal of cover slabs on Pouce stream	
	Widening of Pouce stream to 18 m	This produces a rise in water level by 1.0 m at the confluence between Pouce and Poudrière as a result of the absence of overflows with removal of the cover slabs
	Widening of Poudrière stream to 10 m	
	By-pass between Pouce and Poudrière Streams	
	Resizing of bridge at M1	
	De-silting of sea outlet of Pouce	No impact because of the downstream level at 0.4 m

3.5.3.2.3 Additional Infrastructure

In addition to the infrastructure proposed in the ER2C report, the following infrastructure has been tested:

Ruisseau Créole:

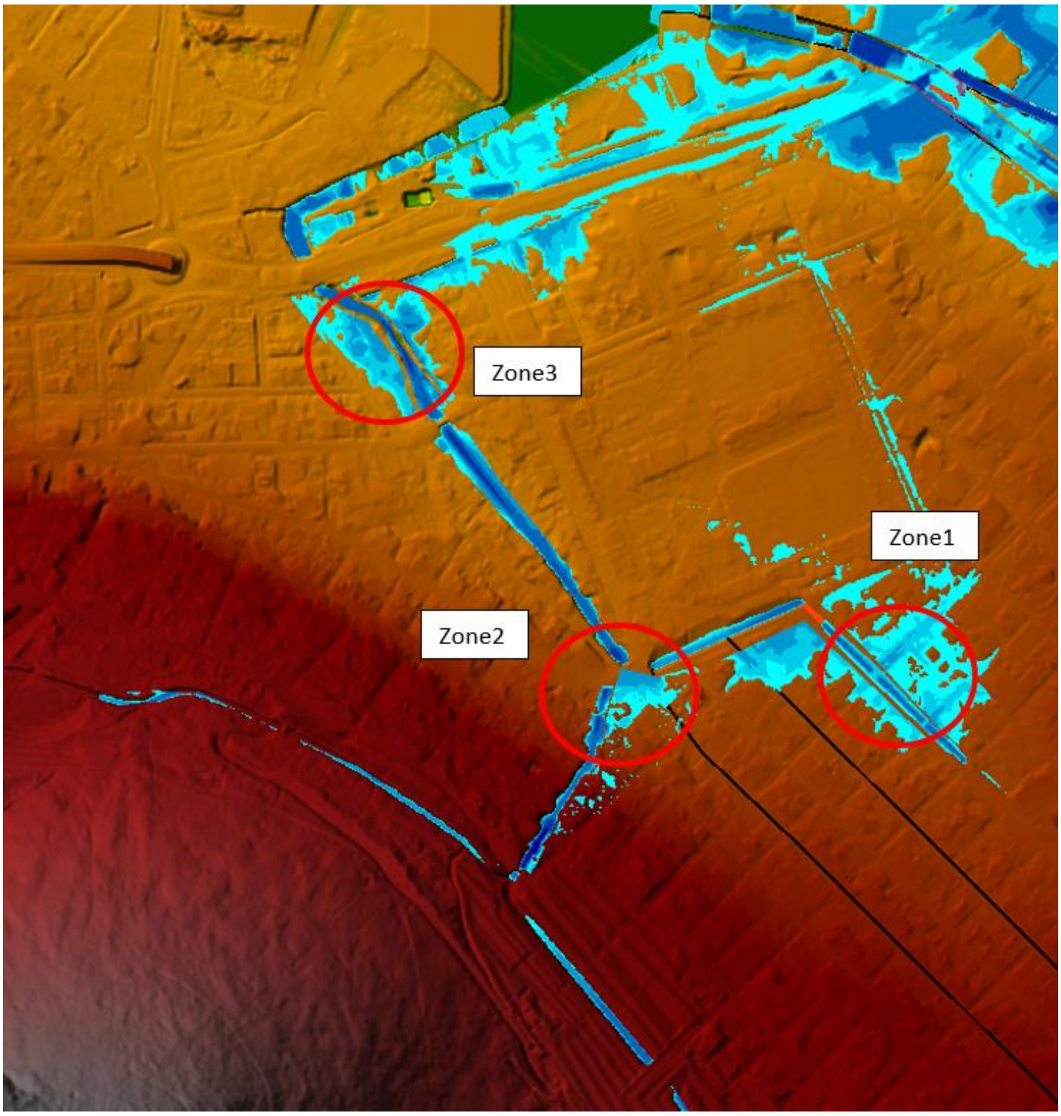


Figure 89 : Focus on Ruisseau Créole – Water depth (100 y event) and residual overflows location

- Duplication of the canal in Zone 1
- Increasing the gradient at the bridges in Zone 2
- Raising of both bank in Zone 3 by 1.20 m

Pouce Stream:

- Velocity acceleration ramp at the entry to the cover slab at the Hawker's market.

The screenshot below shows the results for a **100 year flood event** with the above described infrastructure.

Figure 90 : Pouce Stream, Poudrière and Ruisseau Créole – Water depth projected situation with additional works (100 y event)

The figure below shows the differences in water depth between the present state and after implementation of the proposed infrastructure (100 years event).

Figure 91 : Pouce Stream, Poudrière and Ruisseau Créole – Water depth impact with additional works (100 y event)

From the foregoing, it can be noted that:

Ruisseau des Créoles:

- Duplication of the drain in zone 1 enables this zone to be completely drained
- The velocity ramp in zone 2 decreases the standing water depth, but does not completely drain the zone.
- Widening of the drains in Zones 1 and 2 induces more flooding downstream.
- Raising of both banks in zone 3 enables this zone to be completely drained.

Pouce-Poudrière:

- Velocity acceleration ramps do not decrease significantly the standing water depths compared to what ER2C report contends.
- The decrease in the depth of standing water at La Poudrière canal remains almost the same as at present.

3.5.4 ALTERNATIVE APPROACH 1 - UPSTREAM PARTIAL CUT OFF DRAIN

A different approach to resolving the flooding issues in Central Port Louis is to transfer part of the stormwater run-off from its upper catchment into Canal Anglais in the adjacent catchment via a new cut-off drain between the foot of Le Pouce Mountain and Monneron Hill as shown on the drawing below.

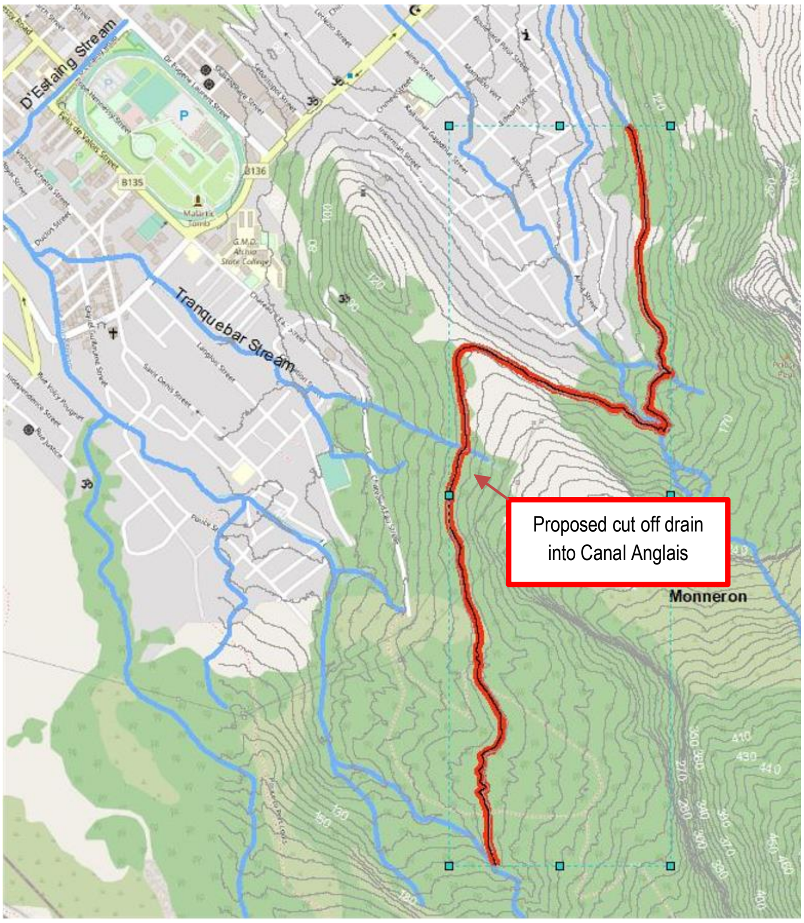


Figure 92 : Pouce Stream – Upstream cut off drain proposal

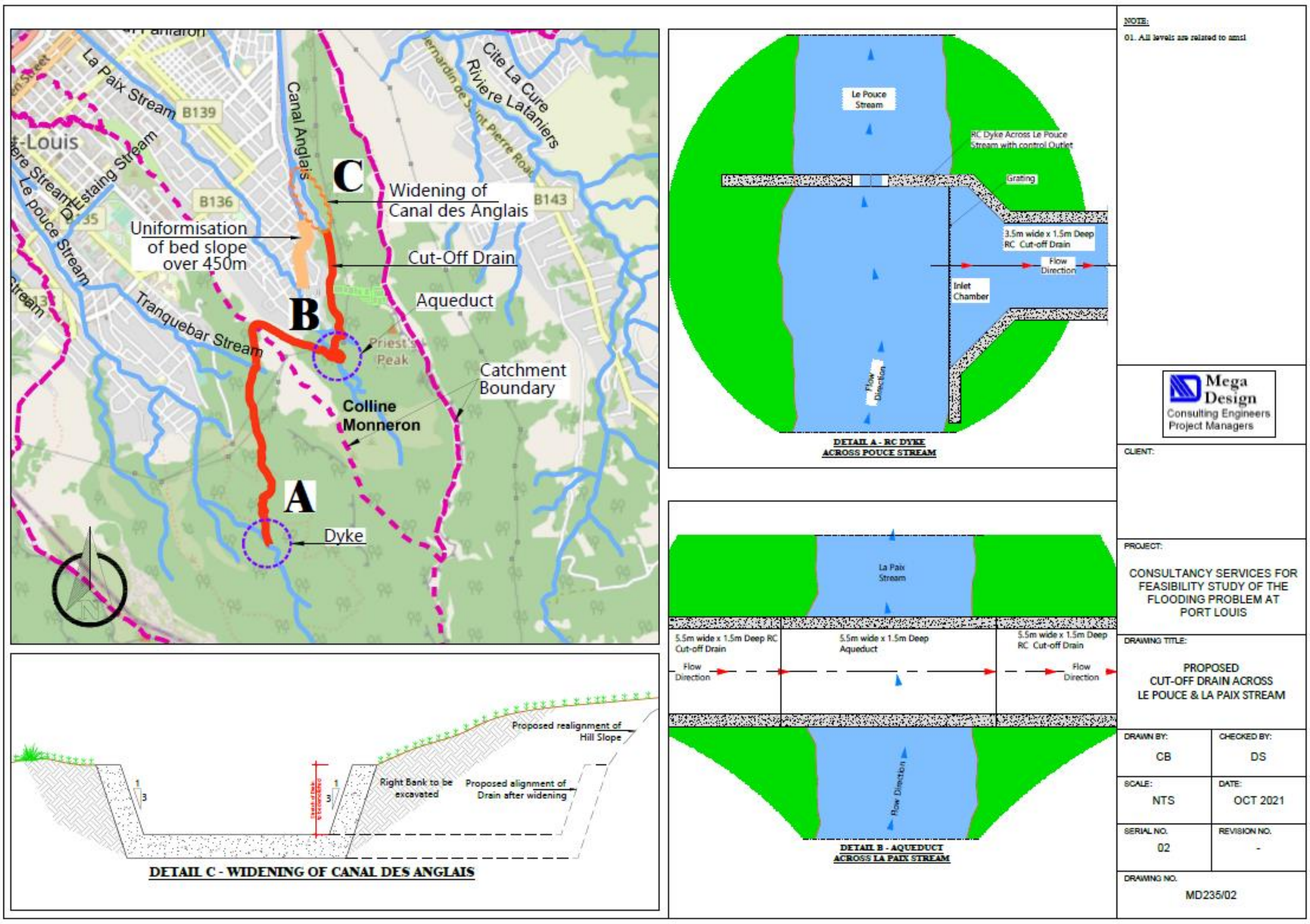


Figure 93 : Pouce Stream – Upstream cut off drain proposal – Detail

- **Initial design – Covered box/ trapezoidal section or pipe design:**

  The principle behind this scheme is the construction of a dyke or intake structure intercepting the entire flow of the Pouce Stream at the upper section of the catchment area, except for an outlet to allow for an environmental flow. This structure may be designed in the form of a reinforced concrete chamber with a side orifice with specified dimensions to divert a controlled flow up to approximately 28 m^3/s corresponding to the capacity of the Cut Off drain. Any inflow from Le Pouce Stream in excess of this 28 m^3/s will overflow back into the Pouce Stream via a spillway.

It is indeed through the creation of this type of intake chamber across the bed of Le Pouce Stream that the structure will be dimensioned and structurally constructed to resist the flows, including potential impacts from rocks or other foreign materials. It is to be noted that the flow of 120 m^3/s for Q50 is at the watershed outlet and not at the level of the water intake.

Figure 94 : Pouce Stream – Typical example of a water intake in a mountainous area

In accordance with the recommendations and decision of the LDA Board, this solution incorporating a box drain or a pipe has not been retained, reportedly due to problems associated with maintenance, eg debris or rocks obstructing the flow. It was requested to maintain a more classical cut-off open drain approach. This is presented below.

- **Retained design – Open Cut-off drain**

The principle adopted is an open trapezoidal concrete drain. This drain will intercept both water from the Pouce catchment at the level of the intake and direct stormwater runoff from the intercepted catchment.

The longitudinal profile of the existing topography along the proposed drain is shown below. The slope of the cut-off drain should be maintained at 1%. A free board of at least 300 mm will be maintained, lateral weirs along the downhiil side of the drain will maintain the flow through the aqueduct over La Paix stream and Canal des Anglais to a peak flow not exceeding:

- 32 m^3/s upstream of the aqueduct
- 40 m^3/s immediately downstream of La Paix stream up to the junction with Canal des Anglais

It is to be noted that open drain sections require much more space than a pipeline and the cost of implementation and costs associated with temporary works, plant and machinery are higher.

The calculations are carried out using a Manning roughness coefficient of 0.014, corresponding to smooth concrete in fairly good to fair condition (considering abrasion of the structure in the long term).

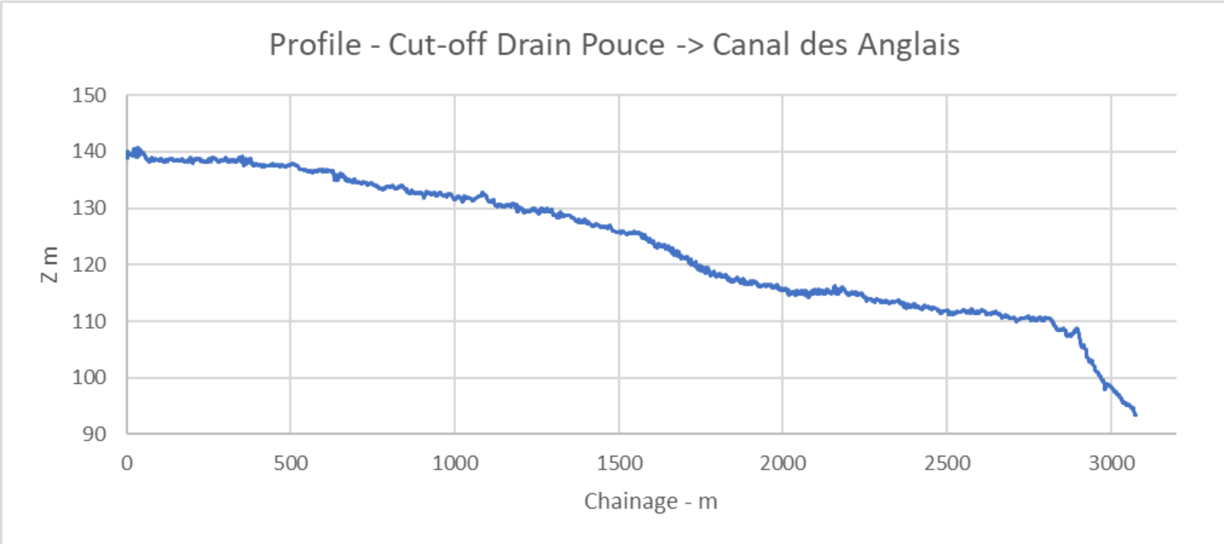


Figure 95 : Pouce Stream – Upstream cut off drain proposal – profile from Pouce Stream to Canal des Anglais

Futher details are provided below:

- **CEB High voltage tower**

The nearest high voltage tower is in excess of 40m, considering the sloping distance from the drain alignment. The collaboration of CEB should be sought when work is being carried out right underneath the cable (eg temporary power interruption or dismantling specific spans if really deemed necessary).

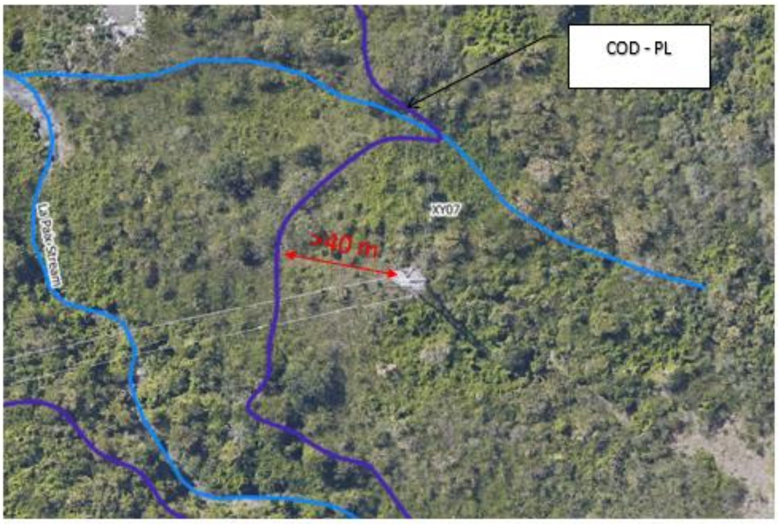


Figure 96 : Pouce cut off drain proposal - CEB High voltage tower

- **Ring Road phase 2 project Alignment**

With regard to the Ring Road phase 2 project, below is the proposed road alignment overlaid on the cut-off drain layout to give an idea of the feasibility of the project. Minor adjustment in alignment of both the drain and the road projects may be necessary at implementation stage. It is understood that the cost of the crossing of the road over the drain will be catered for within the road project budget.

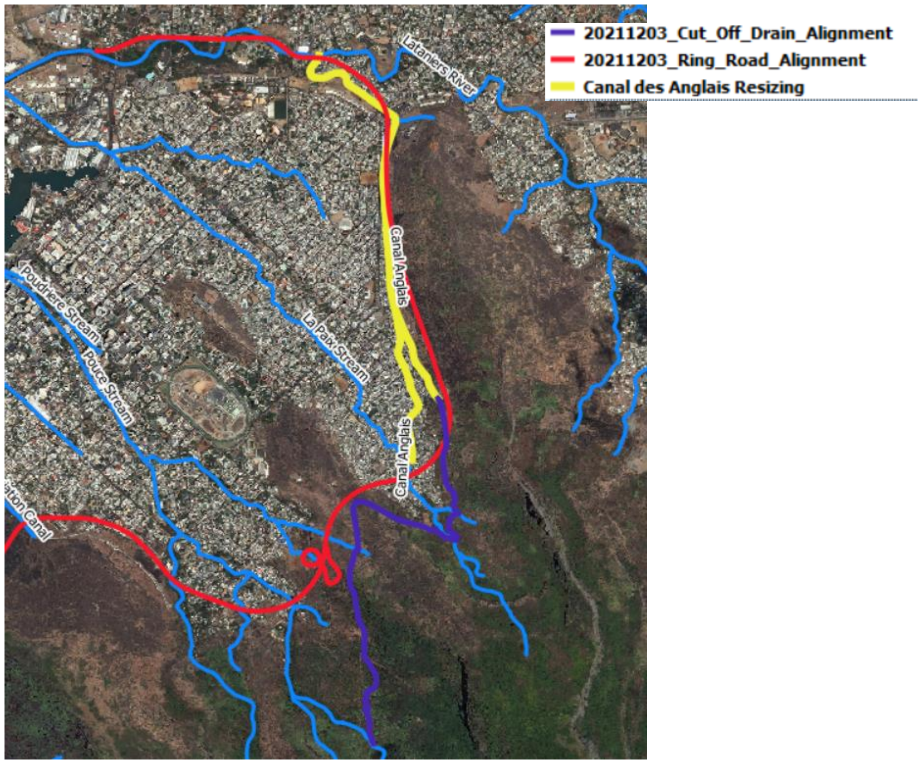


Figure 97 : Pouce cut off drain proposal - Ring Road phase 2 project Alignment

- **Constructability:**

The same construction methodology may be adopted as for a road project along mountain slopes. A typical example of terracing and slope stability is shown below. It is worth noting that the cross section of the hill along the proposed drain alignment does not exceed 30 degrees, with downstream stretches mostly at 12degrees.

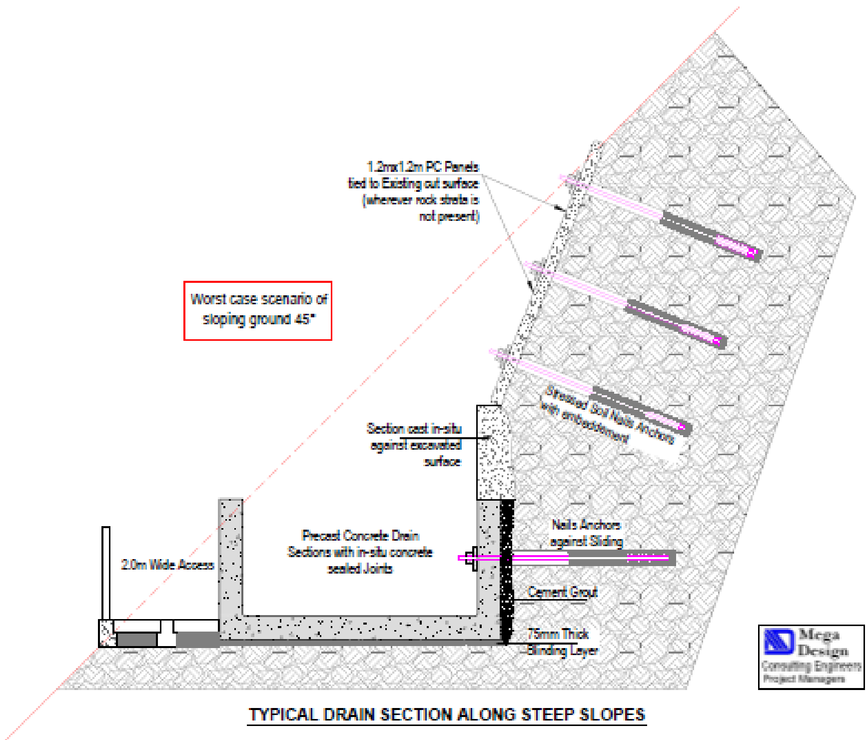


Figure 98 : Typical Drain Section along steep slopes

- **Proximity of residential zone:**

The proposed alignment did consider the proximity of residential zones which are in the main some 30m away. Constraints will however occur along the downstream end of the existing canal during its widening or duplication and this may need relocation of a few houses.

20211203_Cut_Off_Drain_Alignment

Downstream end of Canal Anglais

Figure 99 : Pouce cut off drain proposal - Proximity of residential zone

Canal des Anglais has spare capacity to varying extent at different sections as reported in Chapter 5 (La Paix stream and Canal Anglais).

Further studies will be required to determine to what extent Canal des Anglais could be upgraded or reconstructed to accommodate part of the flow from the Pouce catchment. This analysis is addressed in Chapter 3.9.

Widening of the canal will be effected to a maximum width of 9m with the width varying between 4.5 to 9m over the 2750m stretch according to the typical section shown below:

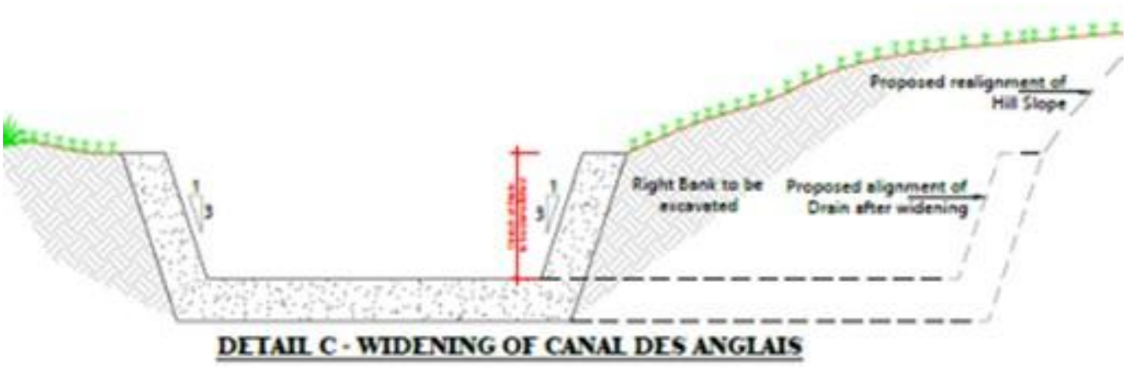


Figure 100 :–Widening of Canal des Anglais

Wherever possible velocity acceleration ramps producing velocities of up to 6 or 7m/s can be implemented in order to reduce the width of the drain.

Canal Anglais will have to be resized to accommodate this additional flow (refer to chapter 3.9.). This flow rate was determined iteratively from modelling.

Figure 101 shows the catchment areas contributing to the cut off drain.

The associated table also specifies the origin of the intercepted catchment areas, the peak flows for a T 100 y flood and the peak flows that will be collected by the cut-off drain.

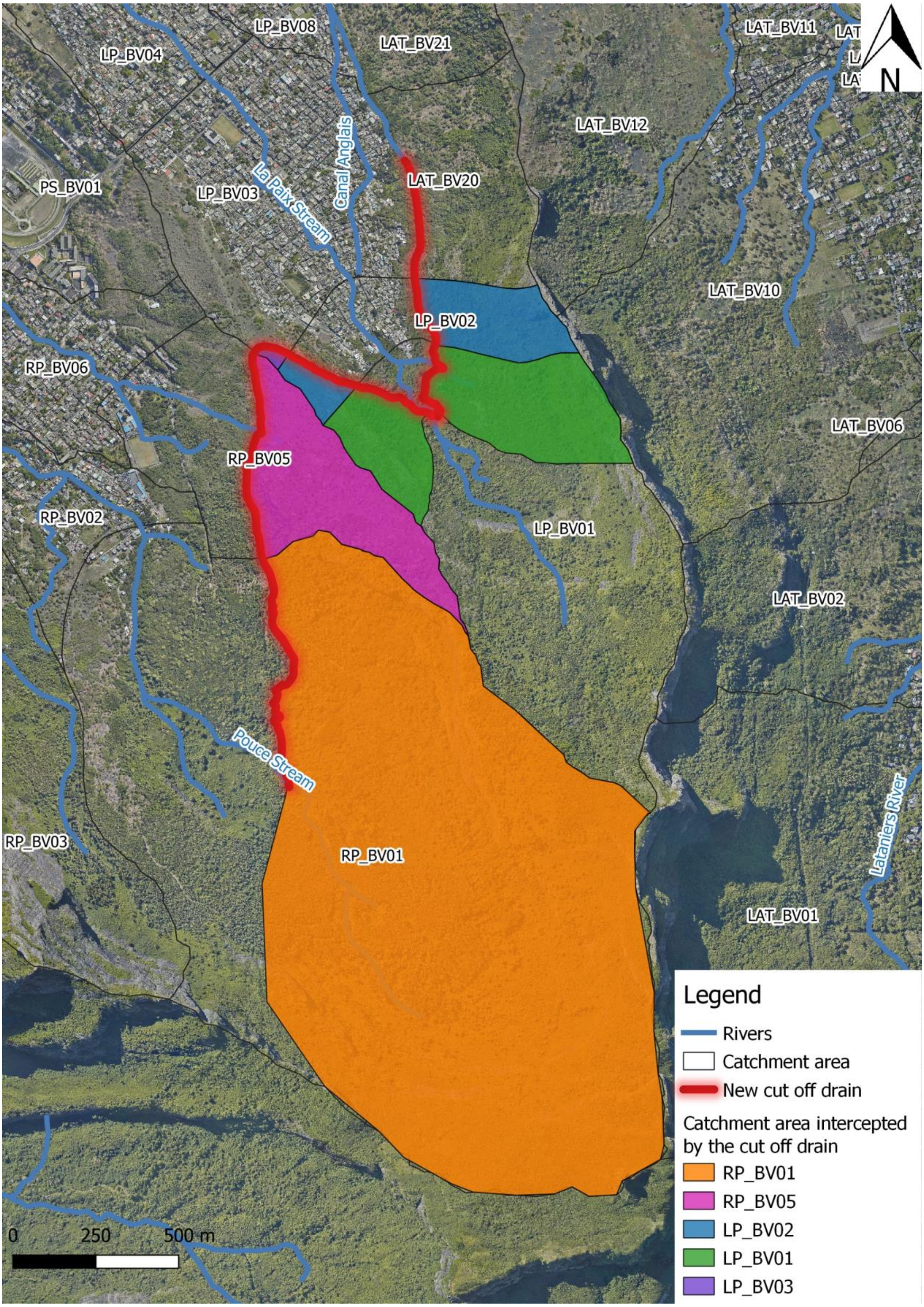


Figure 101 : Pouce Stream – Upstream open cut off drain proposal and intercepted catchment areas

Table 45: Diverted flow Port Louis – Pouce stream – La Paix and Canal des Anglais – peak flow 100 year

	Initial River Outlet	Initial peak flow (m³/s)	Projected Peak flow (m³/s)	Peak flow rate of the new cut off drain T100y (m³/s)	Total peak flow diverted
RP_BV01	Le Pouce	78.6	58.6	20.0	28.1
RP_BV05	Le Pouce	15.5	7.4	8.1	
LP_BV01	La Paix	24.7	18.0	Western part : 2.2 Eastern part : 4.5	11.0
LP_BV02	La Paix	7.5	3.4	Western part : 0.8 Eastern part : 3.3	
LP_BV03	La Paix	19.6	19.4	0.2	

In total, nearly 40 m³/s will be diverted from the catchment areas of Pouce stream (≈28 m³/s) and La Paix stream (≈11.0m³/s) to Canal des Anglais.

The Canal des Anglais will have to be resized to an equivalent section of 9 m wide by a minimum of 1.5 m deep to accommodate these additional flows.

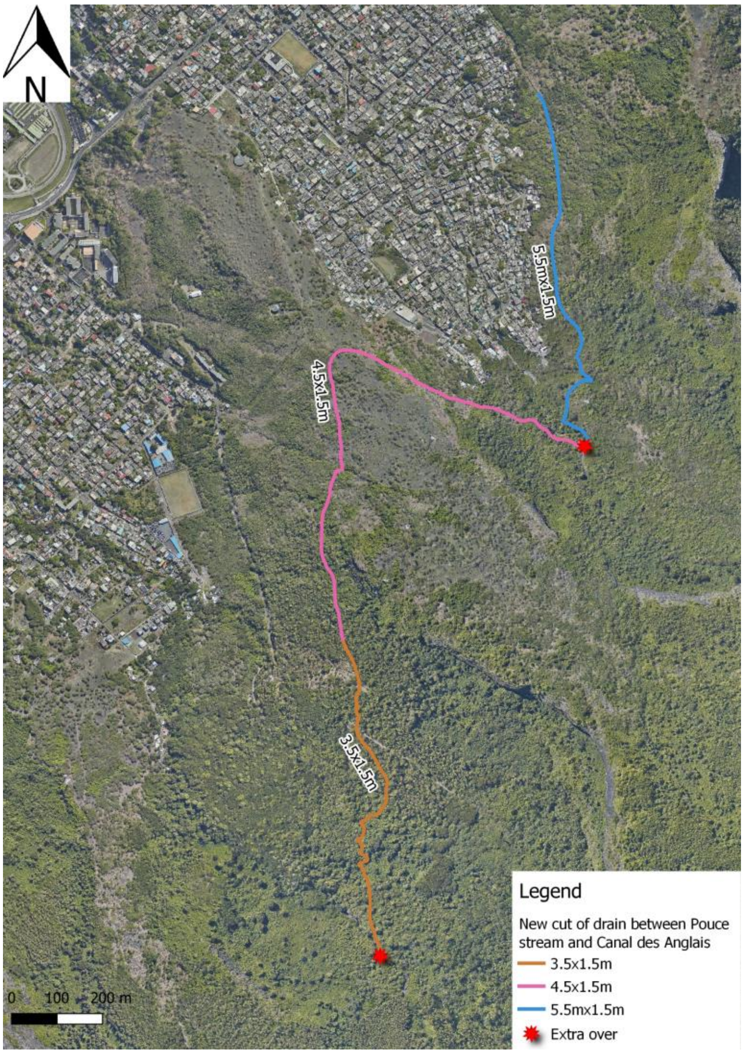


Figure 102 : Pouce Stream – Upstream open cut off drain proposal

3.5.5 ALTERNATIVE APPROACH 2 - FLOOD EXPANSION ZONE IN PORT LOUIS

An ambitious, yet feasible project is to convert the frequently inundated strip occupied by Jardin de la Campagnie and extending over the existing food court into a flood expansion zone and inter-connecting Pouce and La Poudriere streams.

All the facilities presently offered by these infrastructures can be used during normal weather, only to be evacuated during periods of intense rainfall events.

The extent of work involved are:

- A floodwall to a finished level of approximately 4.0m amsl and of aggregate length 900 m enveloping the two drains Poudrière and Le Pouce streams, Jardin de la Campagnie and the food court.
- Raising of Poudrière street over a length of 400 m with intermittent "culvert" crossings between Poudrière stream and Jardin de la Campagnie to balance the flow between the two streams.
- Raising of the lateral roads Brown Sequard, Mère Bertheremy and Descroches streets, each over a length of approximately 65 m.
- Widening of Pouce stream up to its crossing at Baden Powell street.
- Uncovering of Pouce stream downstream of La Chaussée street up to the motorway crossing.
- An alternative to raising Poudrière street is to alternate its alignment with Poudrière stream (i.e shifting Poudrière stream inside the confinement of Jardin de la Campagnie), in similar fashion to Rev Lebrun St.

The measures described above will generate an area of 25,000m^2 as flood expansion zone in addition to some 8,500m^2 along the widened Pouce stream up to Baden Powell street.

Cost estimate for APPROACH 2: FLOOD EXPANSION ZONE IN PORT LOUIS: 170 MRps (costs and alternative works not included in the CBA). This alternative has not been developed further.

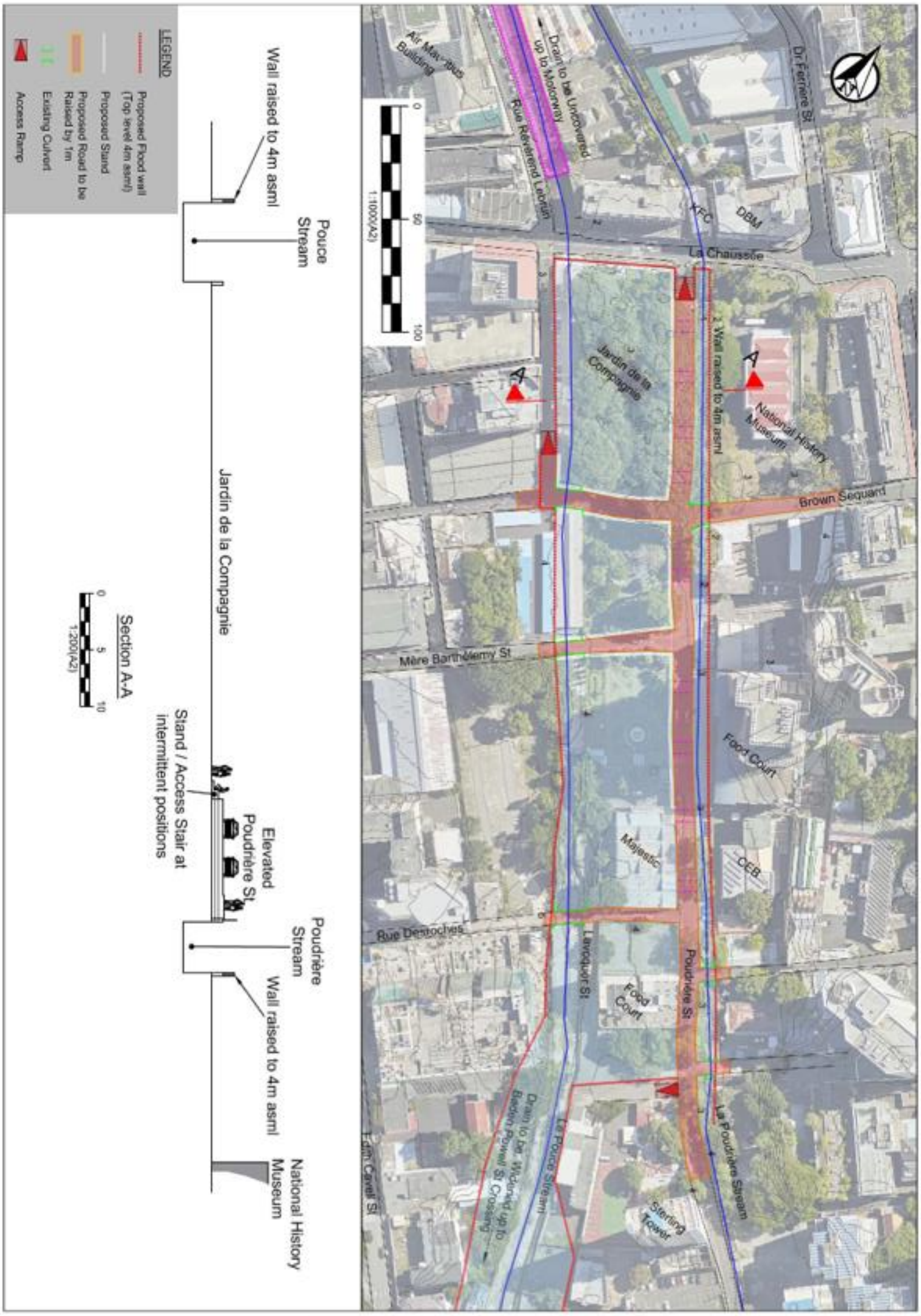


Figure 103 : Pouce Stream - Flood Expansion Zone at Jardin de la Compagnie

3.5.6 *MAIN IMPACTS, COST AND COST-BENEFIT*

The following table summarises the measures retained and their associated costs. The works item number are cross referenced in the plans provided in Annex 1.

It should be noted that the costs of the works being considered in the CBA of the Pouce Stream include all the costs associated with:

- The works related to the cut-off drain from Pouce stream, Ruisseau la Paix and the resizing of Canal des Anglais, and
- Other works specified downstream of Le Pouce and La Paix streams

Likewise, the extent of damage avoided within the same area of influence as a result of such measures is also assessed, viz

- Damage reduction for all protected areas around the Pouce and La Paix streams

The comparative flood maps and the cost-benefit analyses together with cost details are given in Annexes 2 and 3.

The flood maps for the current situation (ie “ Do Nothing Scenario”) are attached in Annex 0: they are also attached to Deliverable D3.

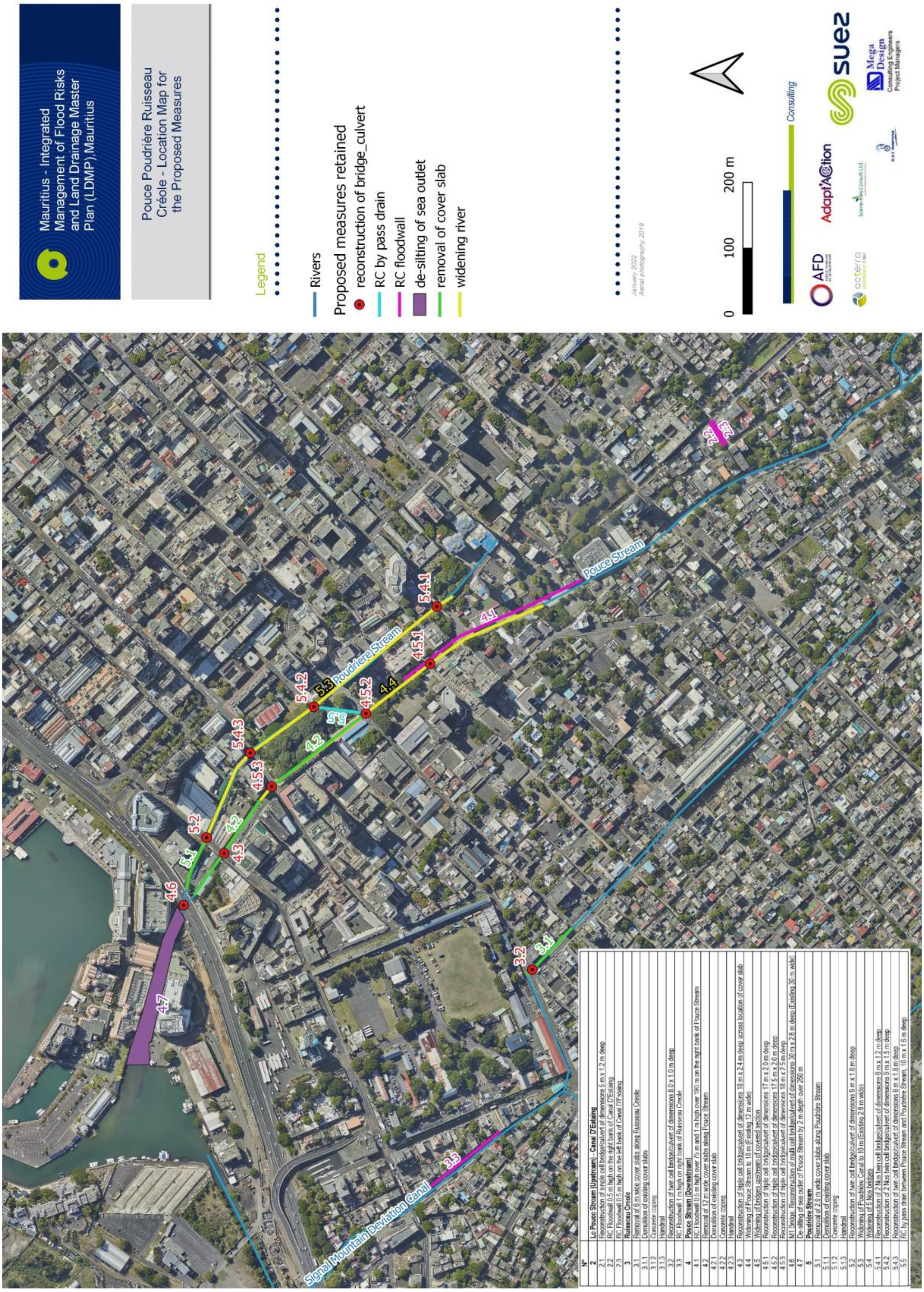


Figure 104: Port Louis – Rivière du Pouce, La Poudrière Stream, Ruisseau des Créoles & Cut-off drain – Location map for proposed measures (1/3)

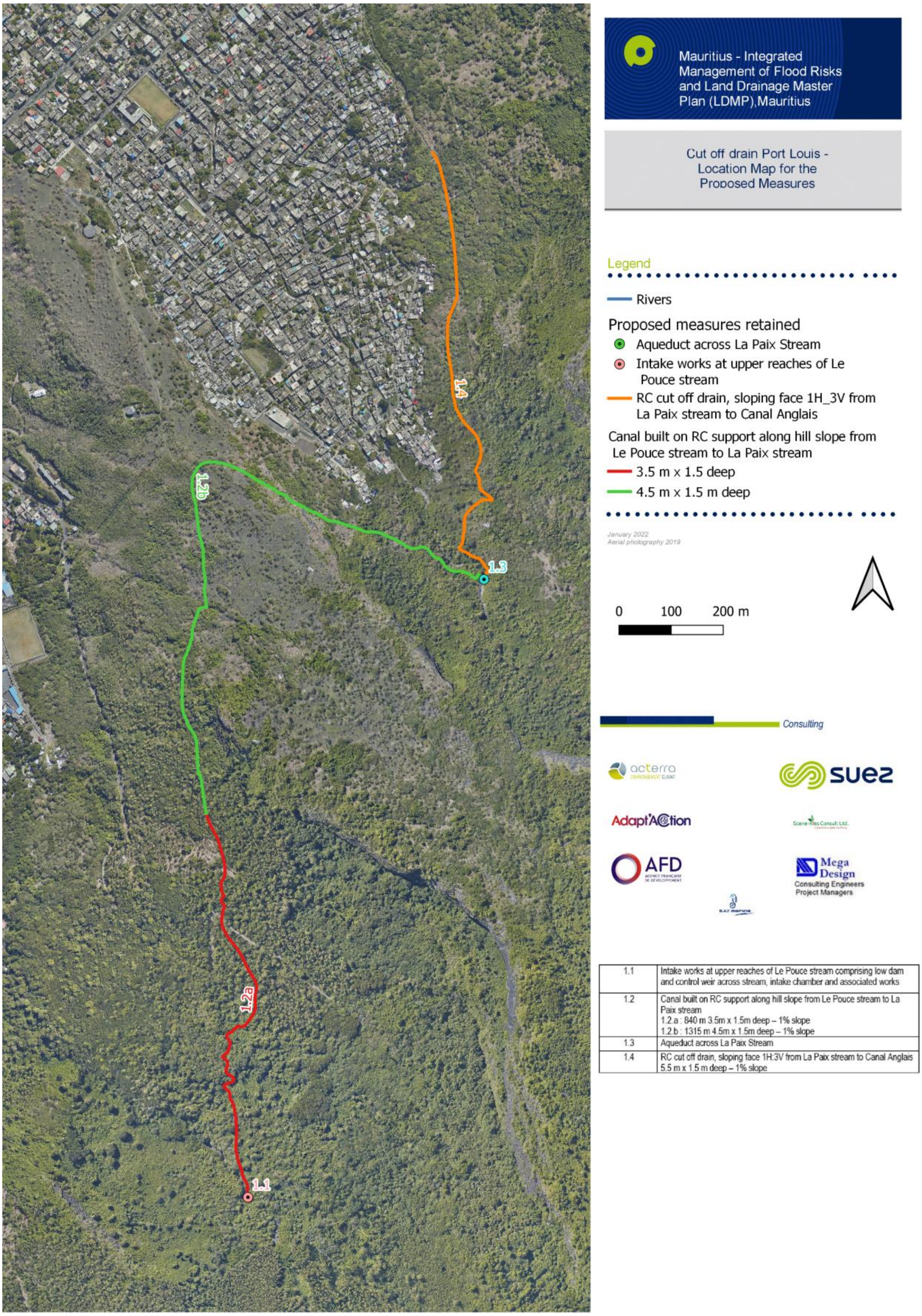


Figure 105: Port Louis – Rivière du Pouce, La Poudrière Stream, Ruisseau des Créoles & Cut-off drain – Location map for proposed measures (2/3)

Figure 106: Port Louis – Rivière du Pouce, La Poudrière Stream, Ruisseau des Créoles & Cut-off drain – Location map for proposed measures – Canal des Anglais (3/3)

Table 46: Port Louis - Rivière du Pouce, La Poudrière Stream, Ruisseau des Créoles & Cut-off drain Pouce and La Paix Stream – Measures and costs

COST ESTIMATE					
N°		**Unit**	**Quantity**	**Rate**	**Amount**
1	**Cut off conduit/drain between Pouce Stream and Canal des Anglais**				
1.1	Intake works at upper reaches of Le Pouce stream comprising low dam and control weir across stream, intake chamber and associated works	Sum			15 000 000
1.2	Canal from Le Pouce stream to La Paix stream				
1.2.a	RC cut off drain, sloping face 1H:3V from Le Pouce stream to La Paix Stream 3.5 m x 1.5 m deep @ 1% Slope, inclusive of soil nail anchors against sliding @ 1.0 m intervals and maintenance access way and handrail	m	840	250 761	210 639 240
1.2.b	RC cut off drain, sloping face 1H:3V from Le Pouce stream to La Paix Stream 4.5 m x 1.5 m deep @ 1% Slope, inclusive of soil nail anchors against sliding @ 1.0 m intervals and maintenance access way and handrail	m	1315	274 350	360 770 250
1.3	Aqueduct across La Paix Stream	Sum			25 000 000
1.4	RC cut off drain, sloping face 1H:3V from La Paix stream to Canal Anglais 5.5 m x 1.5 m deep – 1% Slope	m	940	297 939	280 062 660
Add 1.2 to 1.4)	Slope stabilisation where required, inclusive of precast concrete panels and soil anchors	m	500	52 500	26 250 000
	Sub Total				**917 722 150**
2	**Le Pouce Stream (Upstream) - Canal D'Estaing**				
2.1	Reconstruction of single cell bridge/culvert of dimensions 5 m x 1.2 m deep	Sum			9 500 000
2.2	RC Floodwall 0.5 m high on the right bank of Canal D'Estaing	m	40	12 000	480 000
2.3	RC Floodwall 0.5 m high on the left bank of Canal D'Estaing	m	40	12 000	480 000
	Sub Total				**10 460 000**
3	**Ruisseau Creole**				
3.1	<u>Removal of 6 m wide cover slabs along Ruisseau Creole</u>				
3.1.1	Demolition of existing cover slabs	m	100	3 000	300 000
3.1.2	Concrete coping	m	200	2 800	560 000
3.1.3	Handrail	m	200	3 500	700 000
3.2	Reconstruction of twin cell bridge/culvert of dimensions 8.0 x 1.0 m deep	Sum			19 800 000
3.3	RC Floodwall 1 m high on right bank of Ruisseau Creole	m	170	16 000	2 720 000
	Sub Total				**24 080 000**

COST ESTIMATE					
N°		Unit	Quantity	Rate	Amount
4	**Pouce Stream (Downstream)**				
4.1	RC Floodwall 0.5 m high over 75 m and 1 m high over 190 m on the right bank of Pouce Stream	m	265	16 000	4 240 000
4.2	Removal of 12 m wide cover slabs along Pouce Stream				
4.2.1	Demolition of existing cover slab	m	350	6 000	2 100 000
4.2.2	Concrete coping	m	700	2 830	1 981 000
4.2.3	Handrail	m	700	3 500	2 450 000
4.3	Reconstruction of triple cell bridge/culvert of dimensions 18 m x 2.4 m deep across location of cover slab	Sum			34 000 000
4.4	Widening of Pouce Stream to 18 m (Existing 12 m wide)	m	800	79 000	63 200 000
4.5	Widening of bridges upstream of covered section				
4.5.1	Reconstruction of triple cell bridge/culvert of dimensions 17 m x 2.0 m deep	Sum			30 000 000
4.5.2	Reconstruction of triple cell bridge/culvert of dimensions 17.5 m x 2.0 m deep	Sum			30 500 000
4.5.3	Reconstruction of triple cell bridge/culvert of dimensions 18 m x 2.5 m deep	Sum			34 000 000
4.6	M1 Bridge: Reconstruction of multi cell bridge/culvert of dimensions 30 m x 2.6 m deep (Existing 30 m wide)	Sum			75 000 000
4.7	De-silting of sea outlet of Pouce Stream by 2 m depth over 250 m	Sum			7 000 000
	Sub Total				**284 471 000**
5	**Poudrière Stream**				
5.1	Removal of 2.6 m wide cover slabs along Poudrière Stream				
5.1.1	Demolition of existing cover slab	m	85	1 300	110 500
5.1.2	Concrete coping	m	170	2 830	481 100
5.1.3	Handrail	m	170	3 500	595 000
5.2	Reconstruction of twin cell bridge/culvert of dimensions 9 m x 1.8 m deep	Sum			21 000 000
5.3	Widening of Poudrière Canal to 10 m (Existing 2.6 m wide)	m	640	95 000	60 800 000
5.4	Widening of 5 Nos bridges				
5.4.1	Reconstruction of 2 Nos twin cell bridge/culvert of dimensions 8 m x 1.2 m deep	Sum			40 000 000
5.4.2	Reconstruction of 2 Nos twin cell bridge/culvert of dimensions 9 m x 1.5 m deep	Sum			41 000 000
5.4.3	Reconstruction of twin cell bridge/culvert of dimensions 9 m x 1.8 m deep	Sum			21 000 000
5.5	RC by pass drain between Pouce Stream and Poudrière Stream, 10 m x 1.6 m deep	m	55	135 000	7 425 000
	Sub Total				**192 411 600**
	Total				**1 429 144 750**

COST ESTIMATE					
N°		**Unit**	**Quantity**	**Rate**	**Amount**
	ADD:				
	Relocation of buildings				40 000 000
	Provision for wayleave and Land Acquisition				50 000 000
	Contingencies 15%				214 371 713
	Project Management 5%				71 457 238
	Grand Total				**1 804 973 700**

COST ESTIMATE					
N°	**CANAL DES ANGLAIS**	**Unit**	**Quantity**	**Rate**	**Amount**
1	**Canal des Anglais**				
1.1	Widening of Canal des Anglais from existing 4 m to 9 m	m	2750	80 000	220 000 000
1.2	Lowering of invert of Canal des Anglais by a maximum of 1 m (Reconstruct)	m	450	45 000	20 250 000
	Total				**240 250 000**
	ADD:				
	Provision for wayleave and Land Acquisition				10 000 000
	Contingencies 15%				36 037 500
	Project Management 5%				12 012 500
	Grand Total				**298 300 000**

3.6 Port Louis – Canal Dayot and the drains within its urbanised zone

3.6.1 Background

The Saint Louis river drains two large catchments, that englobing the south part of Signal Mountain to the north and the other englobing the Moka mountain range to the south. At Canal Dayot located on its right bank, the catchment area is 14.5 km^2.

The Urbanised Zone of Canal Dayot located west of Port Louis and bordering St. Louis River suffered intense flooding on 30th March 2013.

St Louis River was narrowed down into a stone masonry drain and the reclaimed area parcelled out for construction purposes. This interference with the natural watercourse, coupled with Roussel Bridge constructed at a very low elevation along the A1 road caused backflow and were the main cause of flooding, this compounded with high tide during the March 2013 event.

Figure 107: Sub-division of catchment area drained by Canal Dayot and the drains within the urbanized zone into sub-catchments (Orthophoto 2019)

Table 47: Canal Dayot and the drains within the urbanized zone – Physical Characteristics of individual Sub-catchments

Name	Area (ha)	Area (km²)	Low level (m)	High level (m)	Length	Slope (m/m)	Slope (%)
CD_BV01	250.85	2.51	102	795	2837	0.24	24.44
CD_BV02	298.85	2.99	60	631	2929	0.19	19.49
CD_BV03	194.29	1.94	28	414	2582	0.15	14.95
CD_BV04	351.28	3.51	40	678	4759	0.13	13.41
CD_BV05	269.15	2.69	40	630	4506	0.13	13.10
CD_BV06	7.41	0.07	28	69	785	0.05	5.23
CD_BV07	75.64	0.76	3	225	1290	0.17	17.24
CD_BV08	31.29	0.31	0	39	1245	0.03	3.11
CD_BV09	43.95	0.44	42	411	1145	0.32	32.24
CD_BV10	38.20	0.38	10	50	929	0.04	4.35
CD_BV11	35.31	0.35	32	328	821	0.36	36.04
CD_BV12	27.83	0.28	10	40	1033	0.03	2.87
SWCoD_BV01	4.02	0.04	102	795	401	0.56	56.17
SWCoD_BV02	6.41	0.06	60	631	623	0.41	41.33
SWCoD_BV03	6.74	0.07	28	414	600	0.47	47.28
CD_Global	**1641.22**	**16.24**	**0**	**795**	**10187**	**0.08**	**7.80**

Table 48: Canal Dayot & the drains within its urbanized zone – Flows for sub-catchments and at outlet of catchment for return periods of 10, 25, 50 and 100 years

BVs	Q10 (m3/s)	Q25 (m3/s)	Q50 (m3/s)	Q100 (m3/s)
CD_BV01	27.81	37.50	44.23	51.25
CD_BV02	33.60	45.20	53.10	61.50
CD_BV03	25.54	33.40	38.65	44.13
CD_BV04	45.95	59.86	68.94	78.54
CD_BV05	29.07	38.92	45.73	52.92
CD_BV06	1.01	1.33	1.49	1.67
CD_BV07	10.75	13.89	16.04	18.20
CD_BV08	4.37	5.61	6.48	7.32
CD_BV09	5.04	6.76	7.97	9.20
CD_BV10	5.63	7.23	8.27	9.31
CD_BV11	3.78	5.14	6.08	7.01
CD_BV12	4.12	5.23	5.98	6.80
SWCoD_BV01	0.42	0.57	0.70	0.84
SWCoD_BV02	0.67	0.95	1.10	1.26
SWCoD_BV03	0.84	1.14	1.30	1.46
Outlet of Canal Dayot	**188.6**	**250.1**	**291.5**	**335.4**
Q /A (m³/s/km²)	***11.6***	***15.4***	***17.9***	***20.6***

3.6.2 Previous Proposals and Works Implementation

Following the flood event of 2013, Mega Design carried out an assessment and proposed, inter-alia, the following flood alleviation measures to cope with a 100 year flood event.

- Floodwall along the river banks
- Enlargement of the river bed and lining with Rock Revetment
- Relocation of a few houses built alongside the river bank
- Raising of Roussel Bridge or its enlargement.

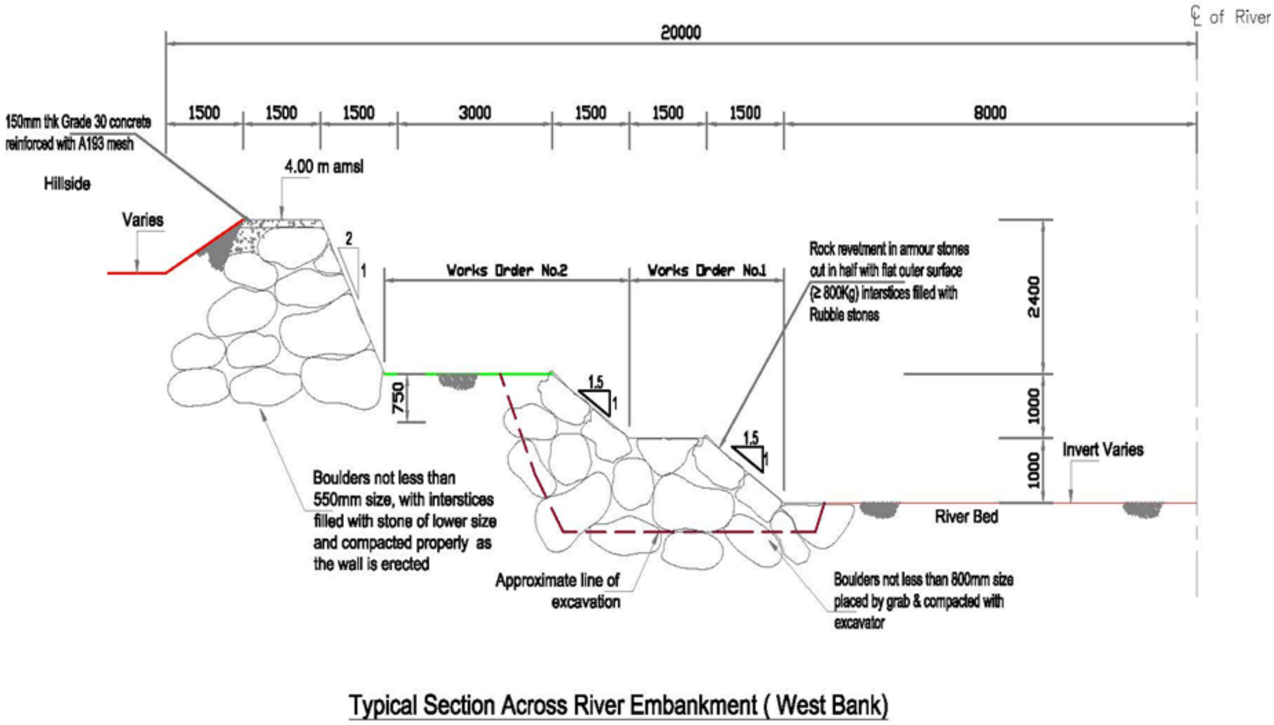


Figure 108: Typical section across river embankment (West Bank)

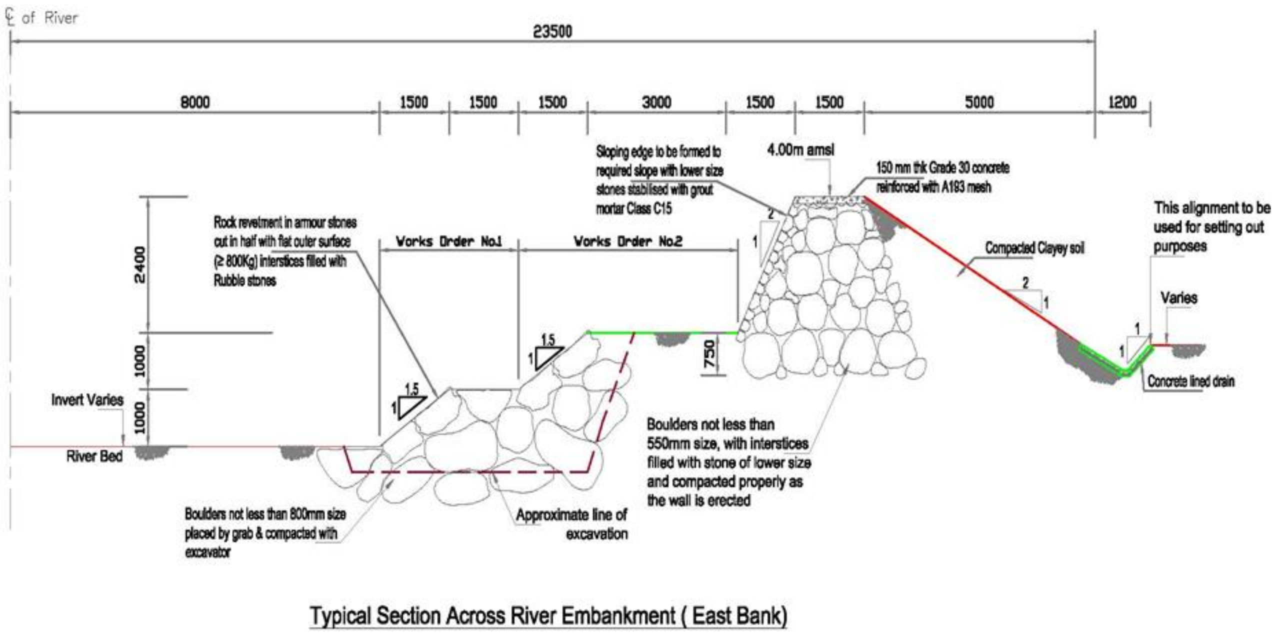


Figure 109: Typical section across river embankment (East Bank)

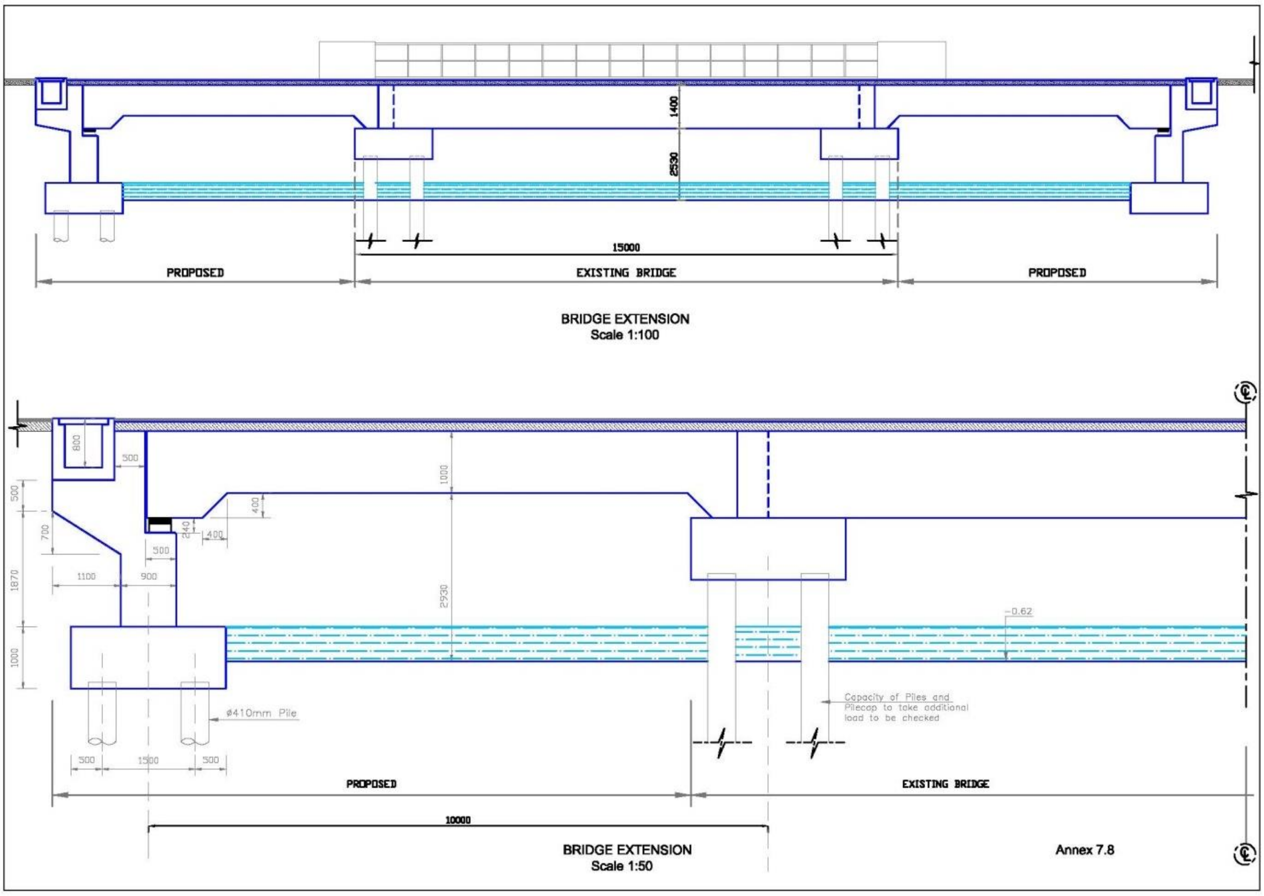


Figure 110: Typical bridge extension

Only part of the proposed Flood Alleviation Works got implemented thereafter, viz

- Enlargement of the river bed between 9 and 17 m
- Rock revetment works

3.6.3 Hydraulic Modelling of Existing Infrastructure

Hydraulic Capacity Modelling elaborated in D3.2.1 on the infrastructure as it stands at present shows that:

A. For a 10 year flood recurrence period, no overflow occurs at Canal Dayot. On the other hand, some overflows can be observed downstream of the drains within the urbanised zone causing flooding of a few assets located to the north of A1 road.

B. For a 25 year flood recurrence period, Canal Dayot still has adequate capacity with no observed overflows. On the other hand, overflows can be observed on the right bank of the drains within the urbanised zone upstream of M1 motorway, which receive flow from the cut-off drain, and this affect several office buildings. The water depths are however less than 0.25 m. The drain located along the M1 motorway collects some of the overflow and channels it to the outfall of Ruisseau des Créoles. There also exist numerous overflow points on the downstream part of A1 road.

C. Canal Dayot overflows as from a 50 year flood recurrence period on its right bank upstream of the A1 bridge, as well as downstream on the right bank, affecting in particular the natural habitat. Increasing overflows within the urbanised zone can be observed.

D. During a 100 year flood recurrence period, flooding becomes more widespread, with an increase in flood water depths, and the number of assets being affected. The M1 gets flooded over part of its length.

The ER2C report contains the following proposals:

- the resizing of Roussel bridge,
- a halt to the construction in the floodplain,
- the relocation of the most vulnerable houses bordering the stream, and
- the construction of a dyke running parallel to the existing river revetment and as close as possible to the housing units to be protected.

The report also recommends the monitoring of river bed silting and regular desilting operations if found necessary.

The extent of works proposed at Canal Dayot is shown on the plate overleaf.

To completely mitigate flooding in this sector for a 50 or 100 year period, the discharge capacity of Roussel Bridge across the A1 road should be increased by either raising the deck or constructing an additional waterway alongside.

Figure 111: ER2C measures: Canal Dayot Short term priority measures

The South West Signal Mountain cut-off drain was built after the March 2013 flooding to complete the protection of the urbanised area on the western slope of Signal Mountain from overland flow.

This cut-off drain was connected to an existing natural drain followed by a small rectangular concrete canal (2 m width and about 1.60 m depth) flowing through the M1 motorway and the Plaine Lauzun industrial area and along the A1 road up to the St. Louis river/GRNW outlet bay.

Priority measures recommended in the ER2C report consist in the resizing of drains and bridges from upstream A1 bridge to the sea outlet, in accordance with the capacity of the cut-off drain and the contributions of the natural catchment area of the drain, viz

- Enlargement of drain sections
- Enlargement of A1 and M14 road bridges
- Enlargement of other bridges

3.6.4 Hydraulic Modelling of Proposed Works

3.6.4.1 Synthesis

A summary of the impacts of the proposed infrastructure is shown in the Table below. Details on the impacts of the works are given below.

Table 49: Canal Dayot & urban drains - measures detail

Location	Operation	Result
Canal Dayot	Widening and raising of bridge	Floods are contained by the floodwall
	Construction of floodwall	
Urban Drain	Drain widening, Zone 1	No flooding
	Drain widening, Zone 2	No flooding

The infrastructure proposed is adequate to contain flooding.

Note: Hydraulic modelling reveals that only sectional parts of the cut-off drains need to be widened in contrast to ER2C report proposing widening of the whole stretches.

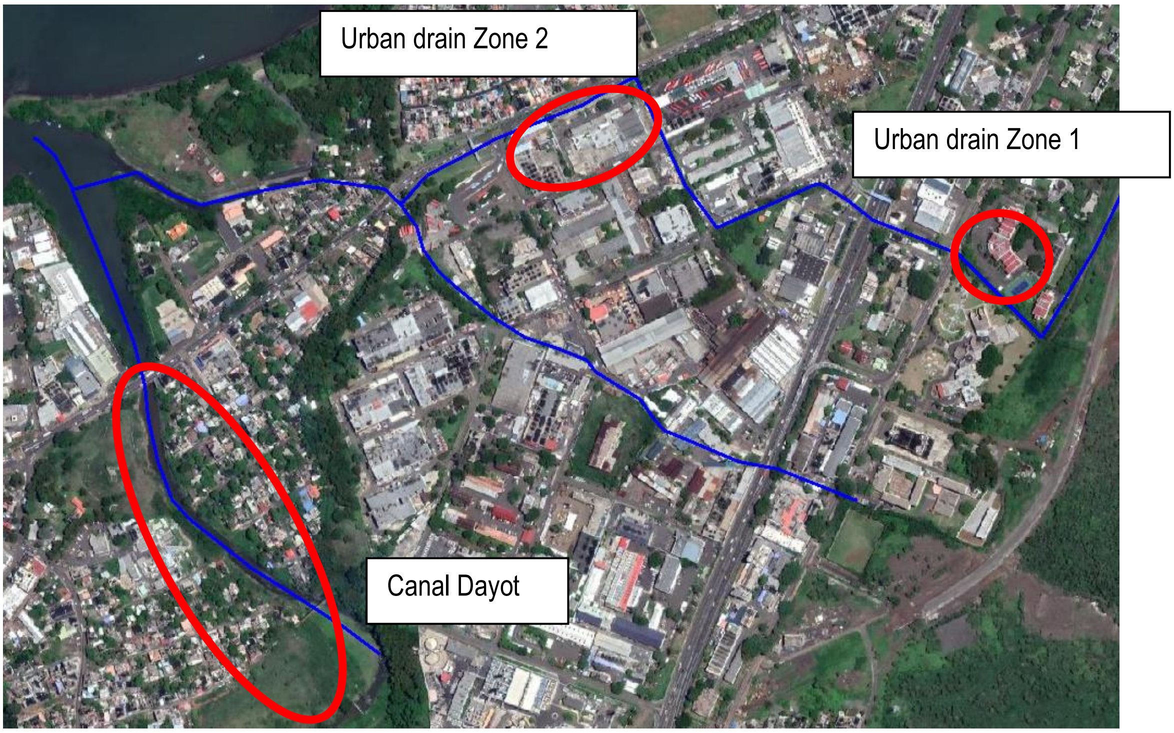
A second outlet to the South West cut-off drain will prevent overflowing from the left bank of the existing outlet, restricting overflow only from the right bank onto open ground, which will not impact on built-up areas.

3.6.4.2 Basis proposal

3.6.4.2.1 Description

Modelling of flood alleviation schemes have been carried out within 3 zones as shown below:

CANAL DAYOT



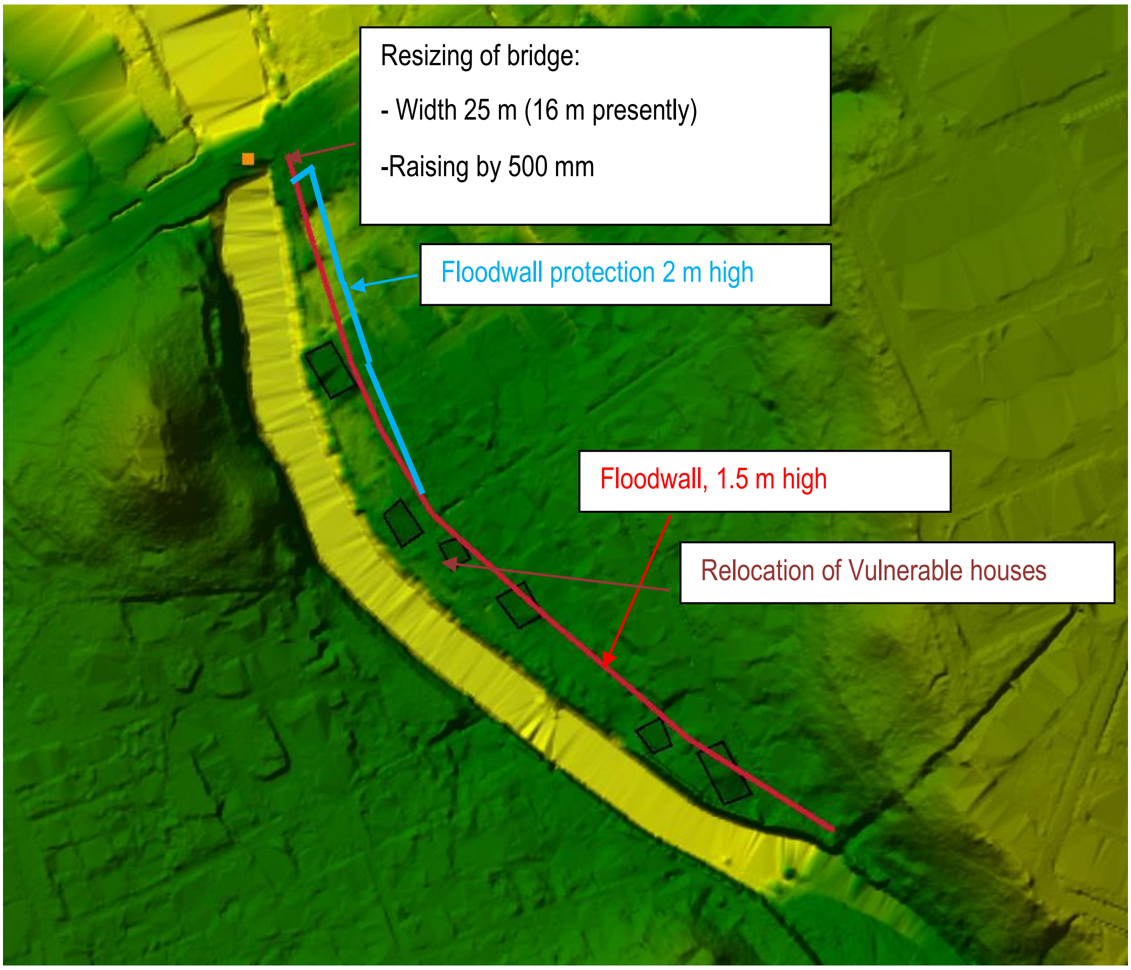
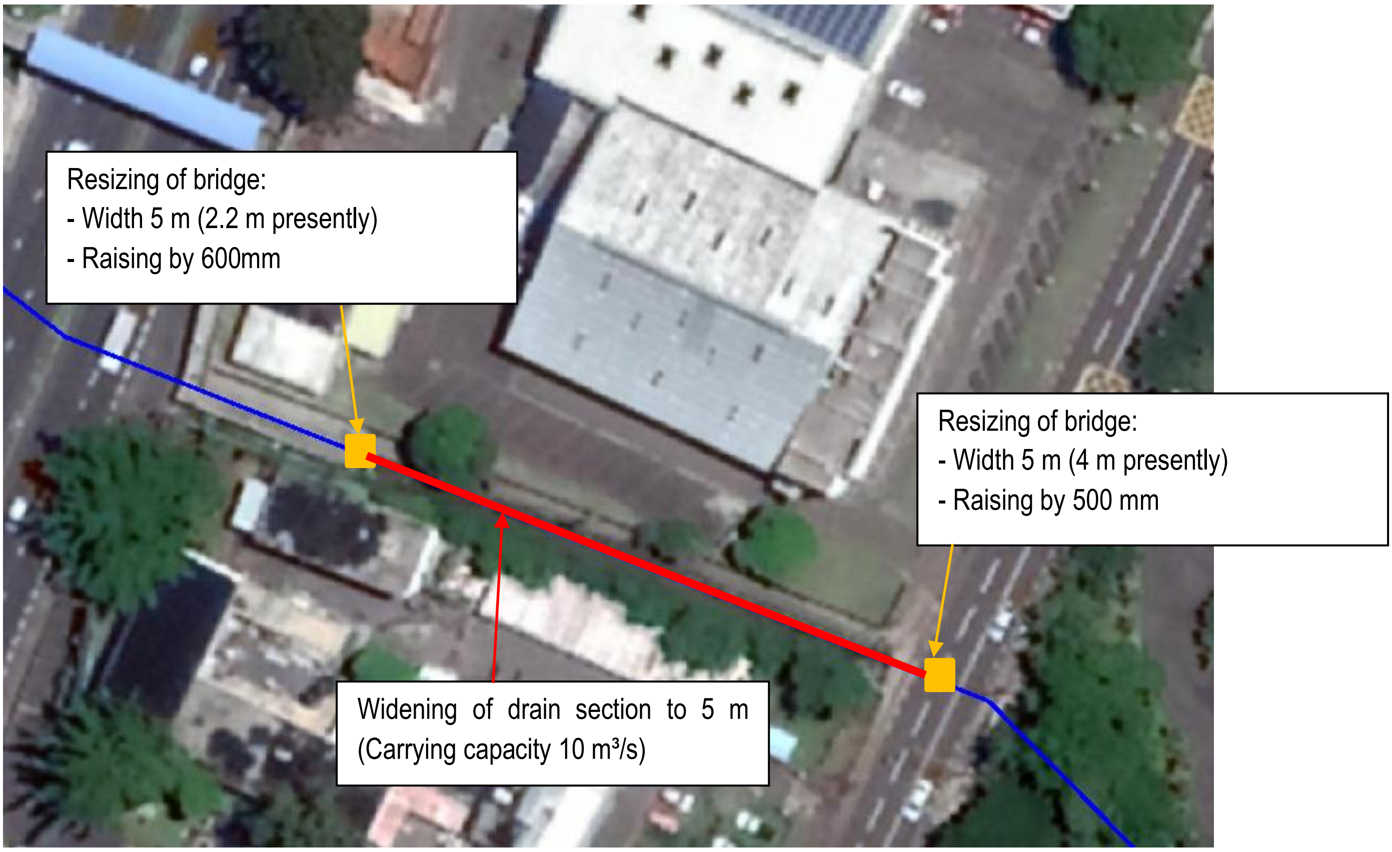


Figure 112: Basis proposal – Canal Dayot and urban drains

URBAN DRAIN ZONE 1



URBAN DRAIN ZONE 2

Figure 113: Basis proposal –Urban drains

3.6.4.2.2 Results

The screenshot below shows the results for a 100 year flood event with the above described infrastructure.

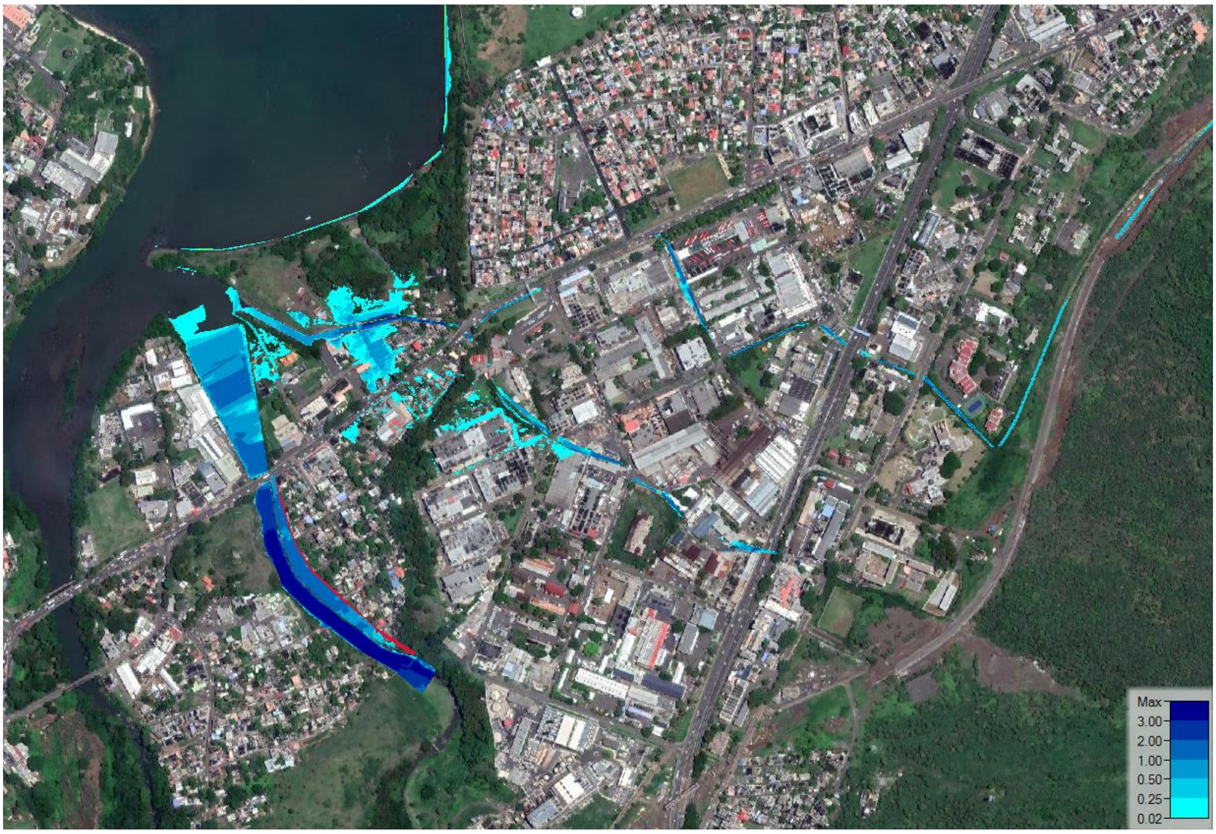


Figure 114: 100 year flood with measures (Canal Dayot and urban drains)

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Table 50: Canal Dayot & urban drains - measures detail

Location	Operation	Result
Canal Dayot	Widening and raising of bridge	Floods are contained by the floodwall
	Construction of floodwall	
Urban Drain	Drain widening, Zone 1	No flooding
	Drain widening, Zone 2	No flooding

The infrastructure proposed is adequate to contain flooding.

Note: Hydraulic modelling reveals that only sectional parts of the cut-off drains need to be widened in contrast to ER2C report proposing widening of the whole stretches.

3.6.4.3 Additional Infrastructures

3.6.4.3.1 Description

In addition to the infrastructure proposed in the ER2C report, the following infrastructure has been tested:

- Zone 1: Increase capacity to 24m³/s of bridge and drains
- A second outlet at Sable Noir to shed off part of the South West cut-off drain flows. Carrying capacity of this cut off drain is 7m³/s (slope 0.3% - sloping face 1H:3V, 3.0 m x 1.0 m deep).

Figure 115: Urban drains – Zone 1 and second outlet location

Zone 1

Flooding within Zone 1 occurs where the canal bed width gets constricted from 5 m to 2.5 m. This section as well as the culvert within should be widened to prevent localised flooding.

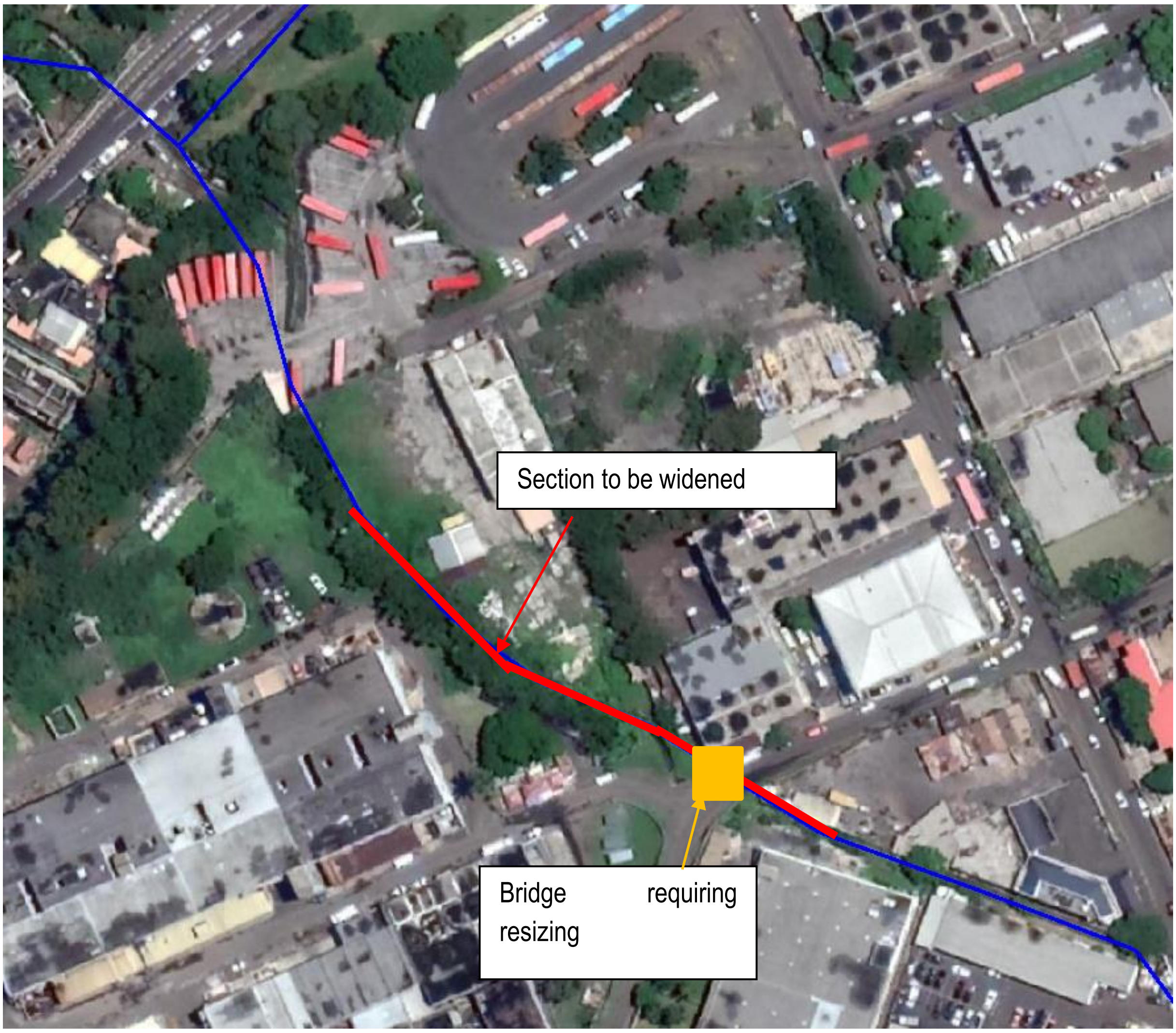


Figure 116: Urban drains – Zone 1 location

3.6.4.3.2 Results

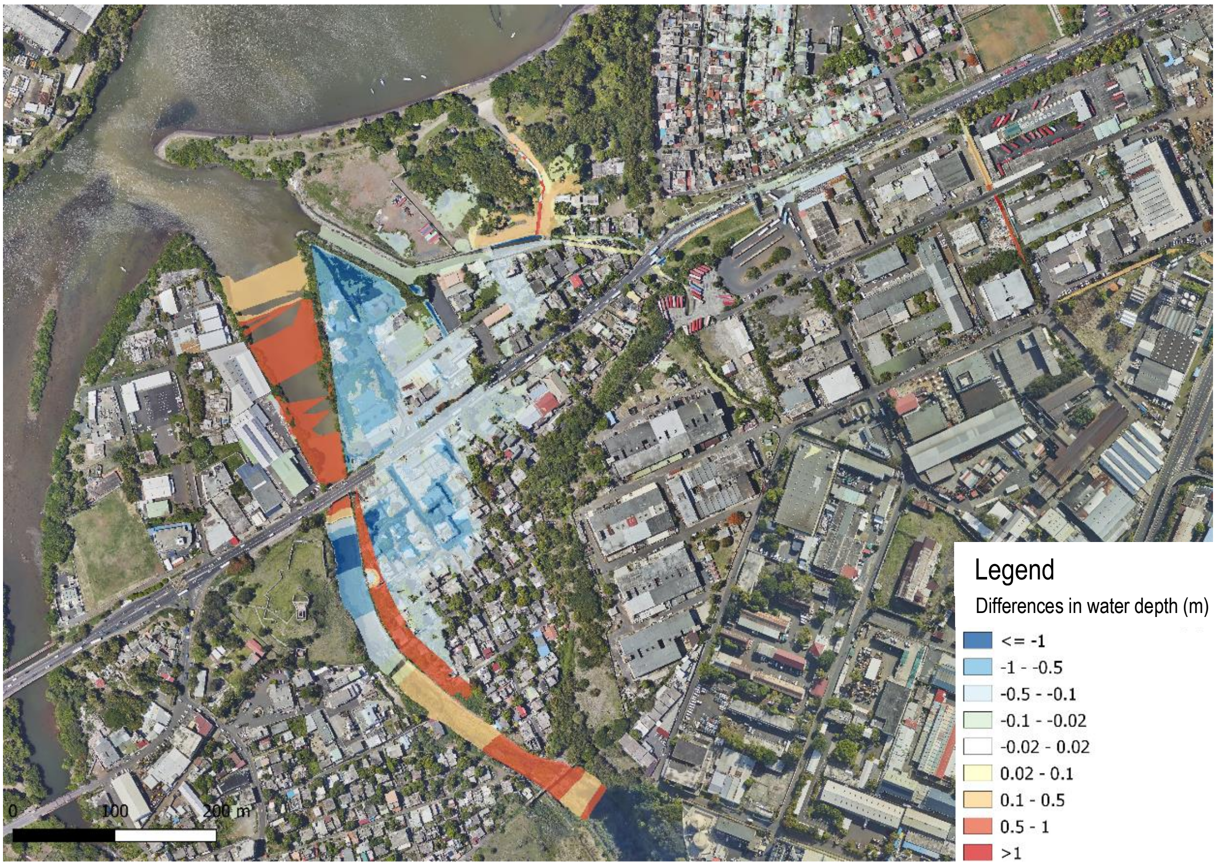
Figure 117: 100 year flood (Canal Dayot and urban drains) with additional measures

Raising of the right bank of Canal Dayot prevents inundation of this area.

The second outlet to the South West cut-off drain prevents overflowing from the left bank, restricting overflow only from the right bank onto open ground.

Although the second shedding canal will not suffice to contain a 100 year flood, any overflow is over the right bank on to open ground and will not impact on built-up areas.

The map below shows the differences in water depth between the present state and that after implementation of the proposed infrastructure.



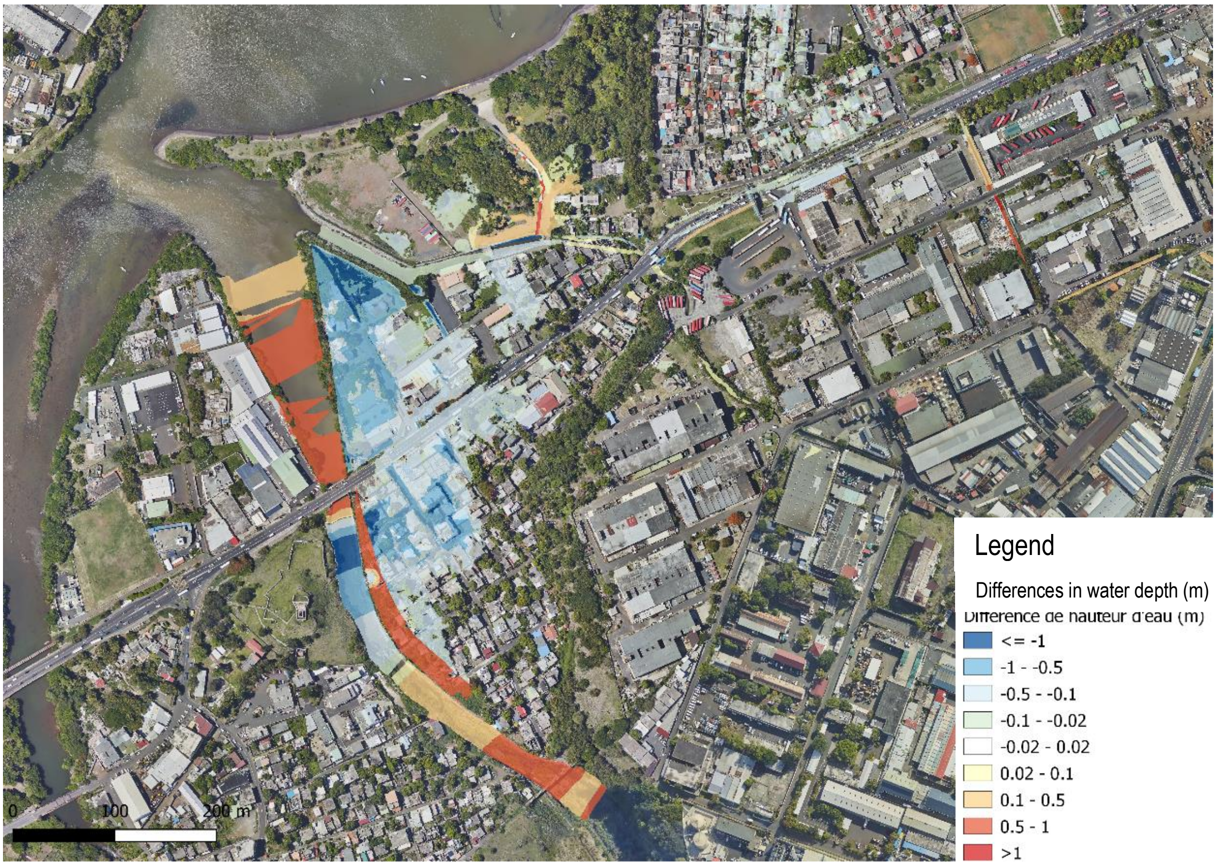


Figure 118: 100 year flood - Differences in water depth (m) – Canal Dayot and urban drains

3.7 Proposed Solutions

3.7.1 Floodwall along the East Bank

A floodwall along the east-bank will provide additional flow capacity and will protect the township from flooding from a rare event of 1/100 years.

3.7.2 Reconstruction/Rehabilitation of Roussel Bridge

The Roussel Bridge is very low and the small discharge area under the bridge is further restricted by the bridge supports and the beams. Seemingly, no consideration had been taken during design of the bridge on the restricted hydraulic area imposed by the obstructive supports and beams on the one hand and the resistance to flow from the tail water from the sea on the other hand. This restriction to flow will be further exacerbated during cyclonic weather conditions when the surge level will reach as high as 0.76m amsl.

The ideal solution would be to reconstruct the bridge at a much higher level so that the underside of the bridge beam is at least 1.0 m above the maximum flood level.

Should rehabilitation of the existing bridge be considered, the hydraulic capacity under the bridge can be increased to $300m^3/s$ to cope with the design peak discharge through either of the following measures:

(i) Raising of the bridge deck

(ii) Enlargement of the flow area by building a waterway on either side of the existing bridge. However due to restriction imposed by the existing building on the northern side (Port Louis) enlargement could be undertaken on only one side (GRNW side).

The following considerations should be addressed in formulating additional rehabilitation works at and upstream of the bridge:

(i) Peak flow under the raised bridge should be such that a minimum freeboard of 500 mm and preferably 1000 mm is allowed to enable the passage of large floating debris under it.

(ii) To generate energy to force flow water under the bridge, the upstream water surface can be raised to provide a headwater. Raising of water upstream of the bridge should be such as not to cause substantial backflow to overrun the floodwall protecting the village.

(iii) Flaring of the inlet and outlet will be necessary to increase flow under the bridge. While there is enough space upstream of the bridge for wing walls, some land should be reclaimed at the downstream end where the fencing and a temporary warehouse within Lising properties have encroached onto the estuary.

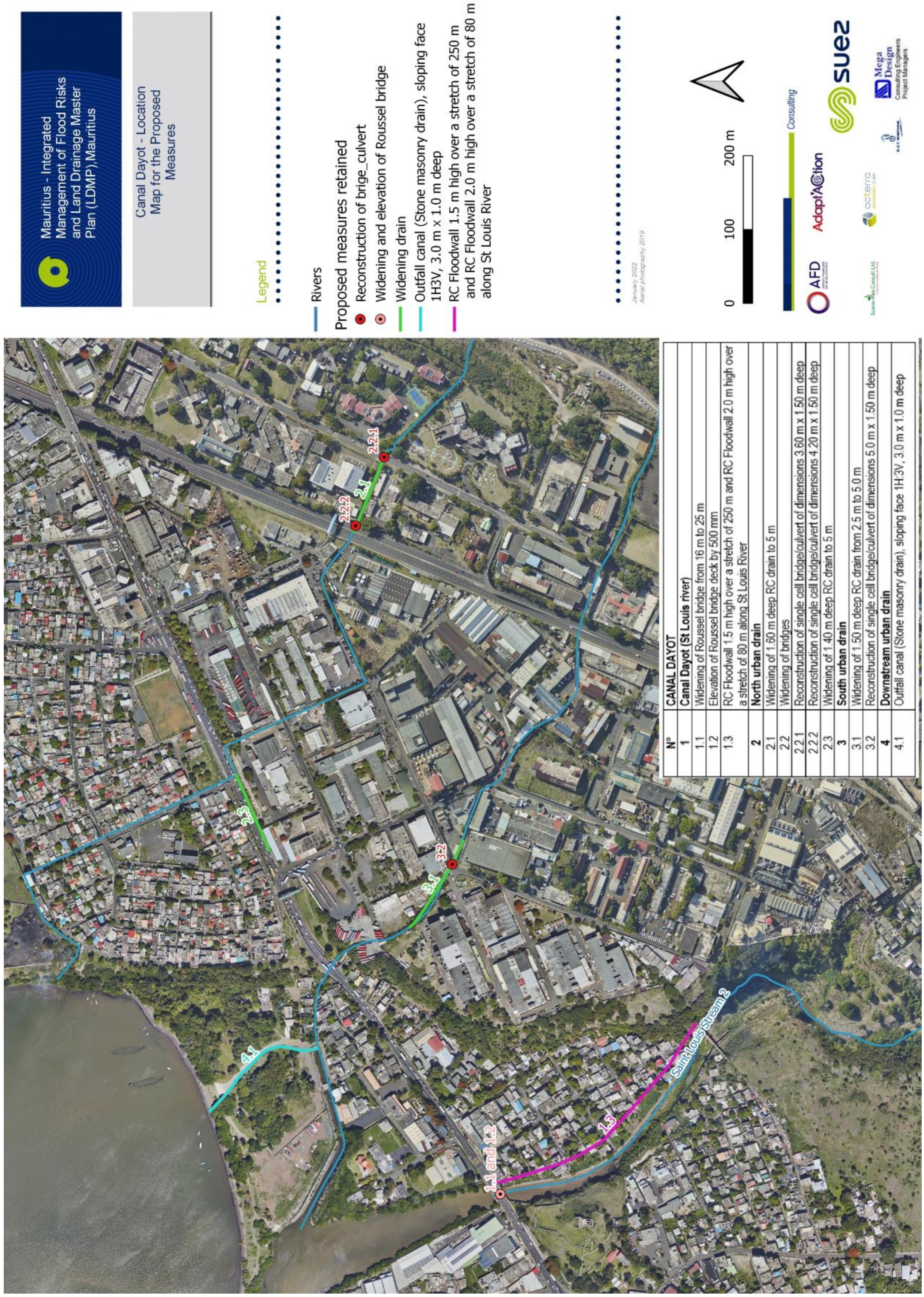
(iv) A similar principle of headwater could be used upstream where sufficient detention storage is available, thereby attenuating flood peaks. Flood peak attenuation could be effected through the construction of an outlet control structure at the level of the aqueduct or further upstream.

3.7.3 MAIN IMPACTS, COST AND COST-BENEFIT

The following table summarises the measures retained and their associated costs. The works item number are cross referenced in the plans provided in Annex 1.

The comparative flood maps and the cost-benefit analyses together with cost details are given in Annexes 2 and 3.

The flood maps for the current situation (ie “ Do Nothing Scenario”) are attached in Annex 0: they are also attached to Deliverable D3.



N°	CANAL DAYOT
1	Canal Dayot (St Louis river)
1.1	Widening of Roussel bridge from 16 m to 25 m
1.2	Elevation of Roussel bridge deck by 500 mm
1.3	RC Floodwall 1.5 m high over a stretch of 250 m and RC Floodwall 2.0 m high over a stretch of 80 m along St Louis River
2	North urban drain
2.1	Widening of 1.60 m deep RC drain to 5 m
2.2	Widening of bridges
2.2.1	Reconstruction of single cell bridge/culvert of dimensions 3.60 m x 1.50 m deep
2.2.2	Reconstruction of single cell bridge/culvert of dimensions 4.20 m x 1.50 m deep
2.3	Widening of 1.40 m deep RC drain to 5 m
3	South urban drain
3.1	Widening of 1.50 m deep RC drain from 2.5 m to 5.0 m
3.2	Reconstruction of single cell bridge/culvert of dimensions 5.0 m x 1.50 m deep
4	Downstream urban drain
4.1	Outfall canal (Stone masonry drain), sloping face 1H:3V, 3.0 m x 1.0 m deep

Figure 119: Port Louis – Canal Dayot and the drains within its urbanised zone – Location map for proposed measures

Table 51: Port Louis - Canal Dayot and the drains within its urbanised zone – Measures and costs

COST ESTIMATE					
N°	**CANAL DAYOT**	**Unit**	**Quantity**	**Rate**	**Amount**
1	**Canal Dayot (St Louis river)**				
1.1	Widening of Roussel bridge from 16 m to 25 m	Sum			22 000 000
1.2	Elevation of Roussel bridge deck by 500 mm	Sum			42 000 000
1.3	RC Floodwall 1.5 m high over a stretch of 250 m and RC Floodwall 2.0 m high over a stretch of 80 m along St Louis River	m	360	25 625	9 225 000
	Sub Total				**73 225 000**
2	**North urban drain**				
2.1	Widening of 1.60 m deep RC drain to 5 m	m	120	38 000	4 560 000
2.2	Widening of bridges				
2.2.1	Reconstruction of single cell bridge/culvert of dimensions 3.60 m x 1.50 m deep	Sum			10 300 000
2.2.2	Reconstruction of single cell bridge/culvert of dimensions 4.20 m x 1.50 m deep	Sum			11 500 000
2.3	Widening of 1.40 m deep RC drain to 5 m	m	150	40 000	6 000 000
	Sub Total				**32 360 000**
3	**South urban drain**				
3.1	Widening of 1.50 m deep RC drain from 2.5 m to 5.0 m	m	175	35 000	6 125 000
3.2	Reconstruction of single cell bridge/culvert of dimensions 5.0 m x 1.50 m deep	Sum			12 000 000
	Sub Total				**18 125 000**
4	**Downstream urban drain**				
4.1	Outfall canal (Stone masonry drain), sloping face 1H:3V, 3.0 m x 1.0 m deep	m	180	37 000	**6 660 000**
	Total				**130 370 000**
	ADD:				
	Provision for wayleave and Land Acquisition				20 000 000
	Contingencies 15%				19 555 500
	Relocation of buildings at Canal Dayot	No	5	5 000 000	25 000 000
	Project Management 7.5%				9 777 750
	Grand Total				**204 703 250**

3.8 Port Louis – Rivière Lataniers

3.8.1 Background

The Rivière Lataniers drains an elongated catchment of total area 14.83 km^2, stretching from the foot of Pieter Both mountain in the south at an elevation of some 500 m to the sea within the Port Louis harbour. On its terminal end it receives flows from another catchment through Canal Anglais and La Paix Stream. At the level of Cité La Cure, the catchment area is 10 km^2. A tributary flowing across Cité La Cure joins Rivière Lataniers downstream of the main access road and Marjolin bridge into the Cité.

Recent works comprised the reconstruction and raising of Marjolin bridge at the entrance to Cité La Cure to prevent overflow at the bridge and a floodwall in the form of a rock revetment to protect the village of Marjolin adjacent.

The catchment area drained by Rivière Lataniers and Canal Anglais, having an approximate area of 14.8 km² (out of which 14 km^2 by Rivière Lataniers), has been sub-divided into 26 sub-catchments.

The catchment of Rivière Lataniers has the longest drainage path of approximately 11km with an average slope of 7.1%.

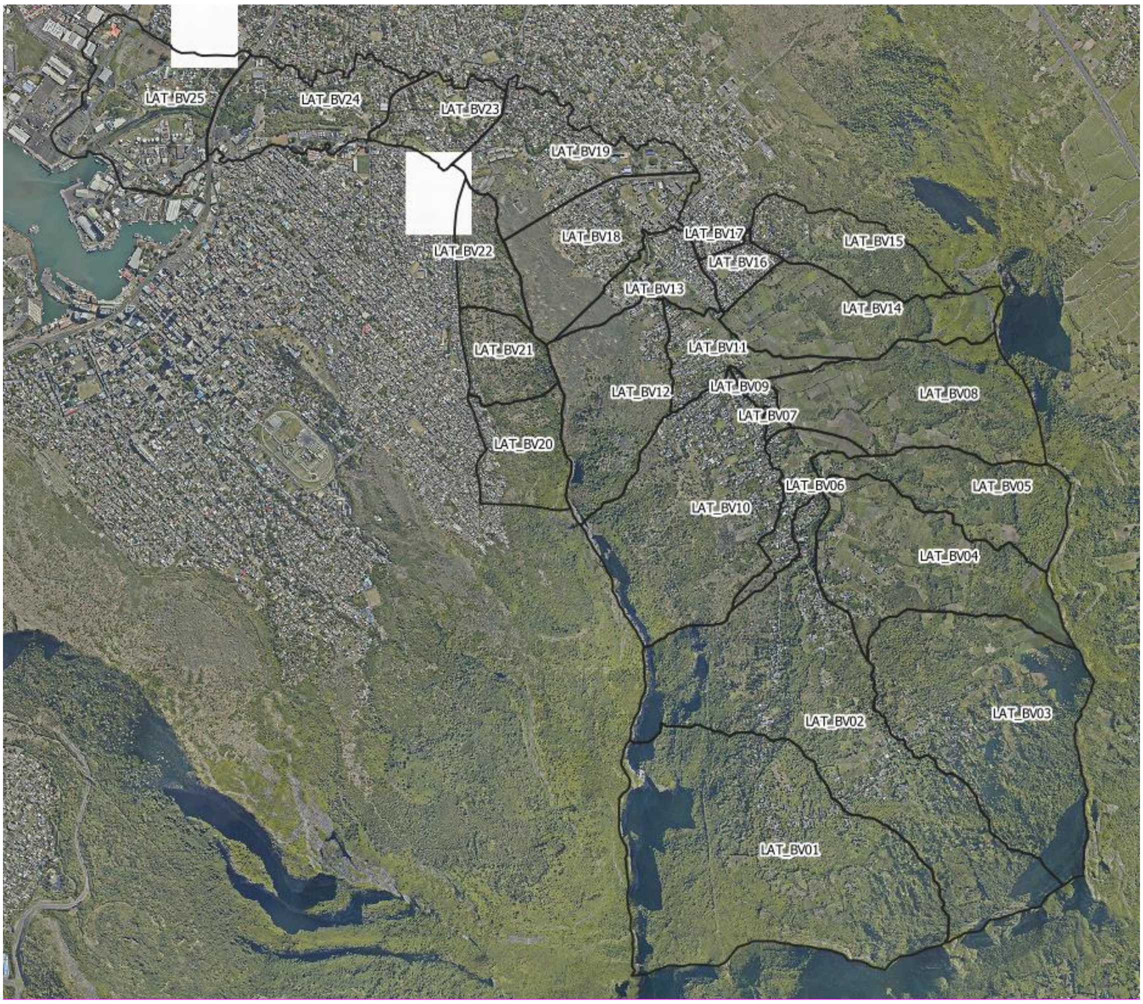


Figure 120: Sub-division of catchment area drained by Rivière Lataniers and Canal Anglais into sub-catchments (Orthophoto 2019)

Table 52: Rivière Lataniers and Canal Anglais – Physical Characteristics of individual Sub-catchments

Name	Area (ha)	Area (km²)	Low level (m)	High level (m)	Length	Slope (m/m)	Slope (%)
LAT_BV01	208.63	2.09	98	775	1862	0.36	36.33
LAT_BV02	188.10	1.88	45	784	3727	0.20	19.81
LAT_BV03	138.97	1.39	77	821	2320	0.32	32.04
LAT_BV04	86.87	0.87	45	543	2075	0.24	23.96
LAT_BV05	54.03	0.54	43	513	1721	0.27	27.26
LAT_BV06	21.21	0.21	40	167	1309	0.10	9.72
LAT_BV07	4.16	0.04	36	44	586	0.01	1.43
LAT_BV08	85.68	0.86	36	461	2070	0.21	20.52
LAT_BV09	0.79	0.01	33	39	237	0.03	2.77
LAT_BV10	117.21	1.17	33	445	1986	0.21	20.74
LAT_BV11	26.00	0.26	28	83	1548	0.04	3.57
LAT_BV12	64.66	0.65	28	339	1739	0.18	17.90
LAT_BV13	24.65	0.25	25	269	1030	0.24	23.78
LAT_BV14	61.20	0.61	32	555	1823	0.29	28.69
LAT_BV15	39.30	0.39	38	241	1420	0.14	14.27
LAT_BV16	10.81	0.11	30	60	581	0.05	5.13
LAT_BV17	3.92	0.04	28	40	448	0.03	2.63
LAT_BV18	57.92	0.58	21	303	1252	0.23	22.53
LAT_BV19	57.23	0.57	14	155	1880	0.07	7.49
LAT_BV23	28.10	0.28	62	306	1226	0.02	1.63
LAT_BV24	51.40	0.51	54	303	1390	0.01	1.26
LAT_BV25	70.92	0.71	33	277	1279	0.01	0.80
LAT_Global (Excluding Canal des Anglais)	**1401.76**	**14.02**	**1**	**775**	**10901**	**0.07**	**7.10**
LAT_BV20	33.73	0.34	62	306	1246	0.20	19.54
LAT_BV21	25.00	0.25	54	303	836	0.30	29.82
LAT_BV22	22.64	0.23	33	277	1289	0.19	18.91

Area - LAT_Global Add Cut Off Drain - Canal des Anglais): 1483.13 ha

The flows obtained for Rivière Lataniers and Canal Anglais for return periods of 10, 25, 50 and 100 years are shown in the table below.

Table 53: Rivière Lataniers and Canal Anglais – Flows for sub-catchments and at outlet of catchment for return periods of 10, 25, 50 and 100 years

BVs	Q10 (m3/s)	Q25 (m3/s)	Q50 (m3/s)	Q100 (m3/s)
LAT_BV01	32.46	43.28	50.53	58.33
LAT_BV02	30.01	39.44	45.63	52.39
LAT_BV03	22.00	29.15	33.96	39.10
LAT_BV04	13.37	17.86	20.84	23.98
LAT_BV05	7.73	10.39	12.29	14.26
LAT_BV06	4.18	5.35	6.04	6.80
LAT_BV07	0.82	1.11	1.25	1.40
LAT_BV08	12.00	16.24	19.07	22.14
LAT_BV09	0.27	0.30	0.31	0.32
LAT_BV10	22.19	28.65	32.71	37.16
LAT_BV11	5.27	6.66	7.61	8.53
LAT_BV12	11.27	14.73	16.98	19.44
LAT_BV13	5.64	7.16	8.02	9.07
LAT_BV14	8.46	11.40	13.44	15.66
LAT_BV15	6.64	8.68	10.00	11.45
LAT_BV16	2.55	3.23	3.65	4.10
LAT_BV17	0.91	1.21	1.35	1.51
LAT_BV18	12.82	16.14	18.23	20.52
LAT_BV19	12.18	15.33	17.29	19.44
LAT_BV20	6.36	8.27	9.48	10.80
LAT_BV21	4.55	5.85	6.77	7.78
LAT_BV22	4.09	5.35	6.15	7.02
LAT_BV23	5.91	7.46	8.33	9.40
LAT_BV24	10.46	13.11	14.79	16.53
LAT_BV25	14.28	17.96	20.11	22.57
Outlet of Rivière Lataniers and Canal Anglais	**156.3**	**207.2**	**241.4**	**277.7**
Q /A (m³/s/km²)	***11.2***	***14.8***	***17.2***	***19.8***

3.8.2 Review of Previous Studies

The ER2C report recommends, as a first priority, a halt in the construction of houses within the flood plain and the relocation of vulnerable dwellings, and thereafter to follow up with the construction of a lateral dyke to protect the urbanised areas of Cité La Cure for a 100 year return event and to preserve as much as possible the flood expansion fields. The proposed infrastructure works are indicated in the plate below.

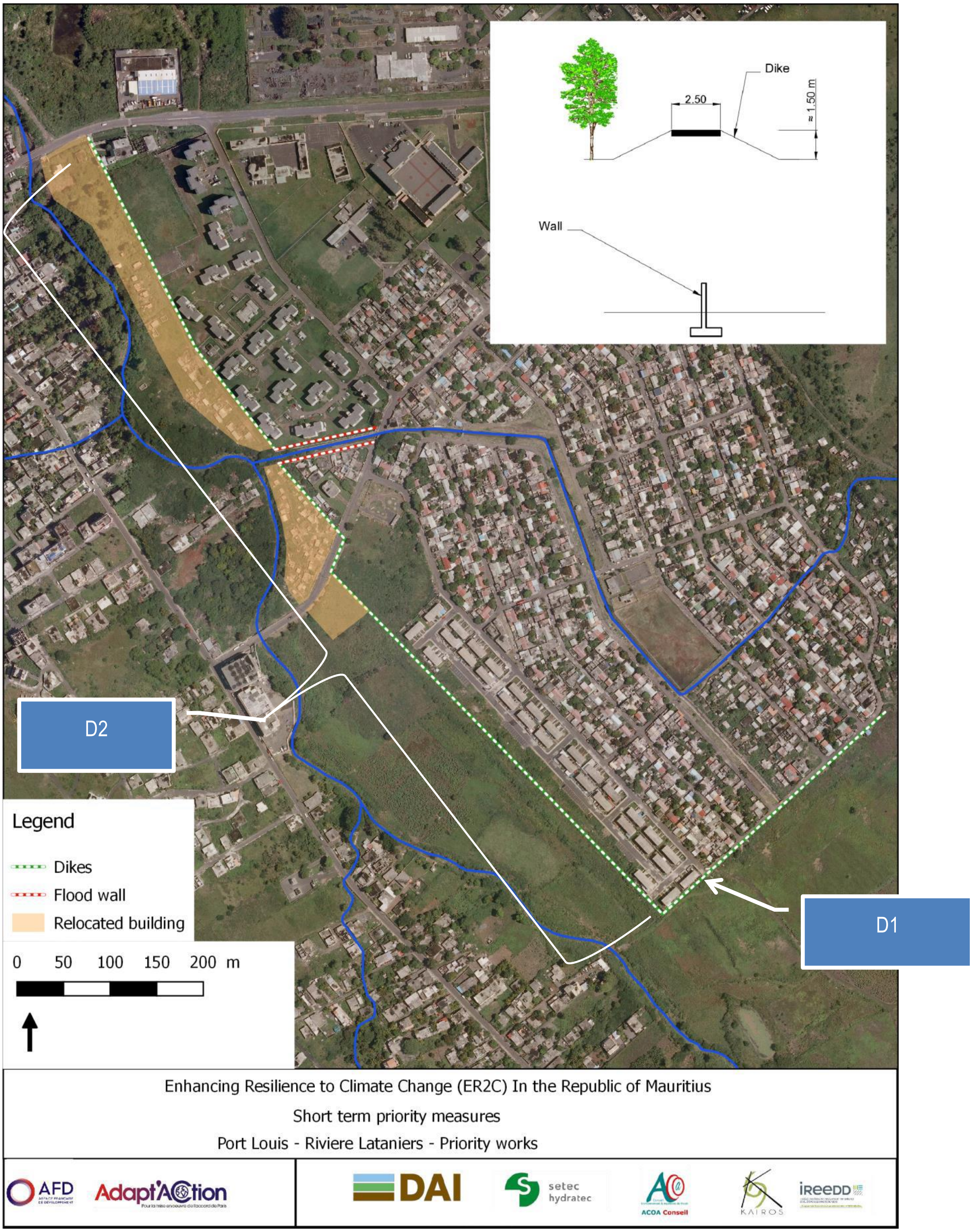


Figure 121: ER2C measures: Rivière Lataniers - Short term priority measures

3.8.3 Hydraulic Modelling of Proposed Works

3.8.3.1 Synthesis

A summary of the impacts of the proposed infrastructure is given below. Details on the impacts of the works are given below. Details on the impacts of the works are given below.

- The cut-off drain to Marjolin Estate (dyke No 1 = D1) will prevent flooding
- Only the downstream end of dyke No 2 (D2) at Marjolin bridge needs to be raised to contain river flows

3.8.3.2 Basis proposal

3.8.3.2.1 Description

Infrastructure proposed:

- D1 : A dyke of height 1.60 m
- D2: A dyke of height 1.70 m

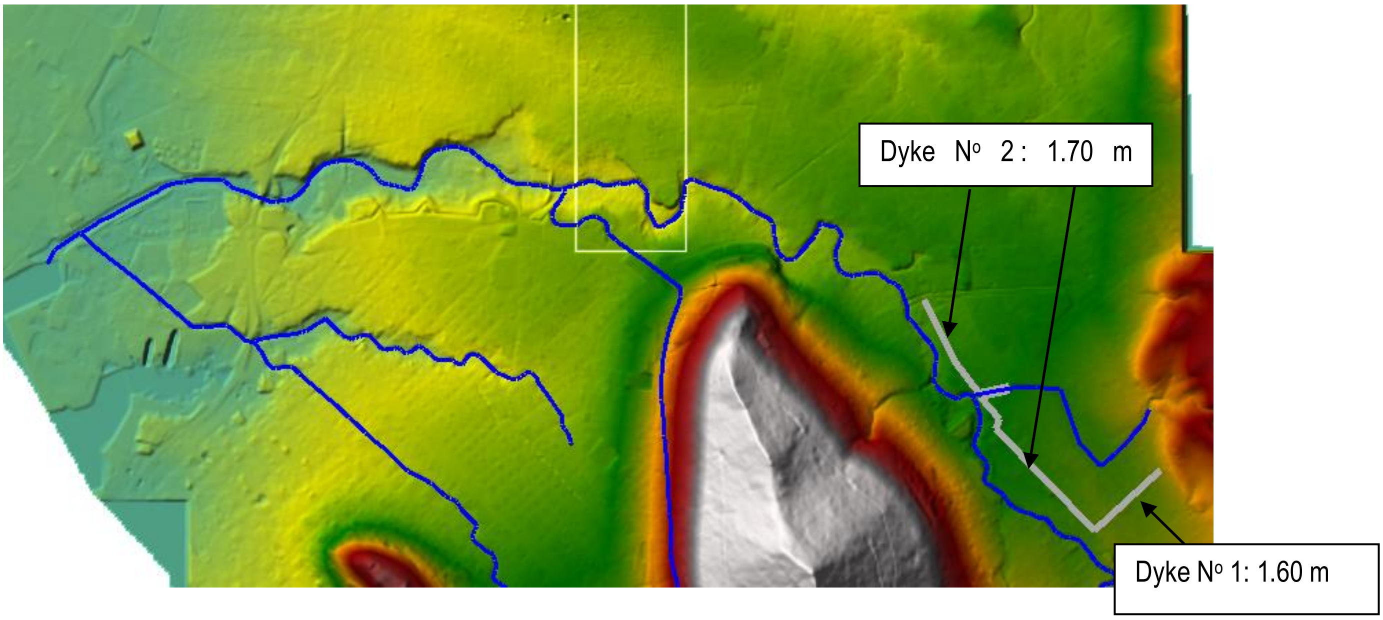


Figure 122: Rivière Lataniers – Dykes D1 and D2

3.8.3.2.2 Result

A screen-shot of the trial run showing the results for a Q_{100} flood with the proposed infrastructure are:

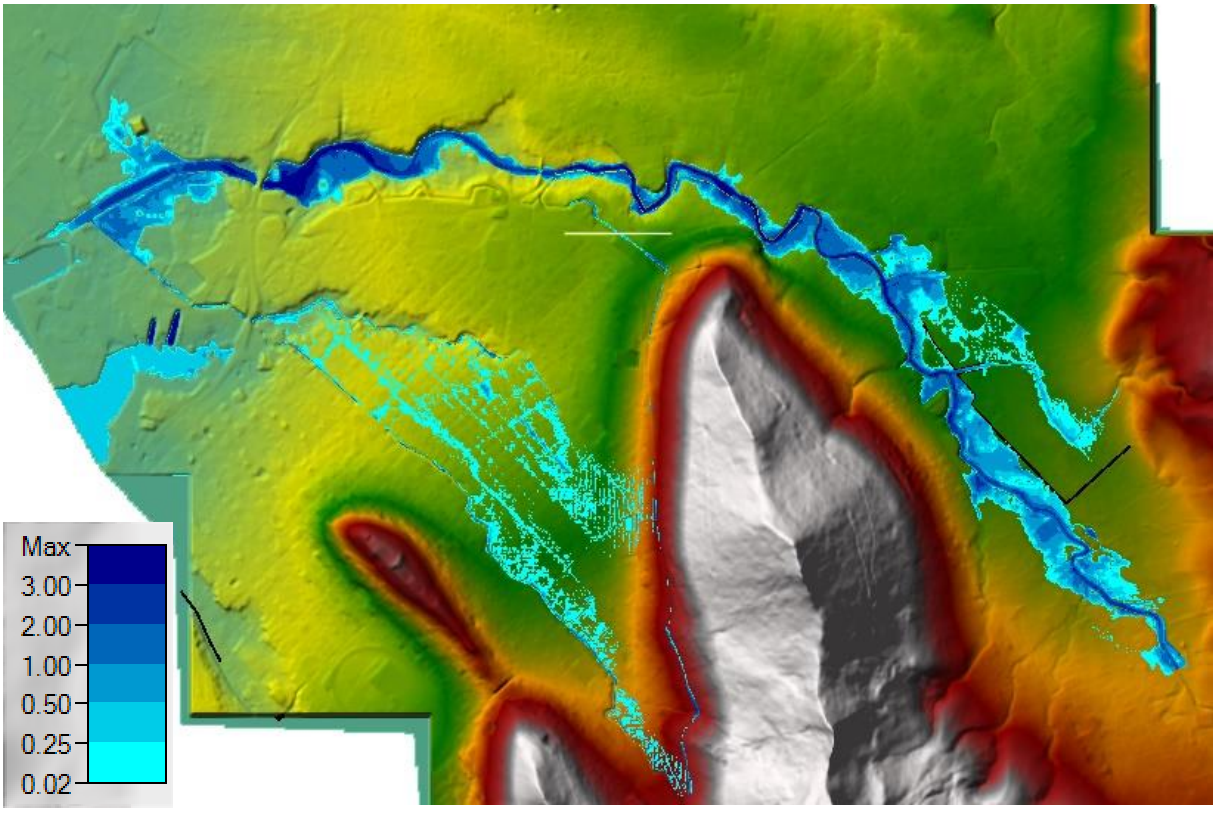


Figure 123: Rivière Lataniers – Dyke D1 and D2 – 100 year flood

The map below shows the differences in water depth between the present state and after implementation of the proposed infrastructure.

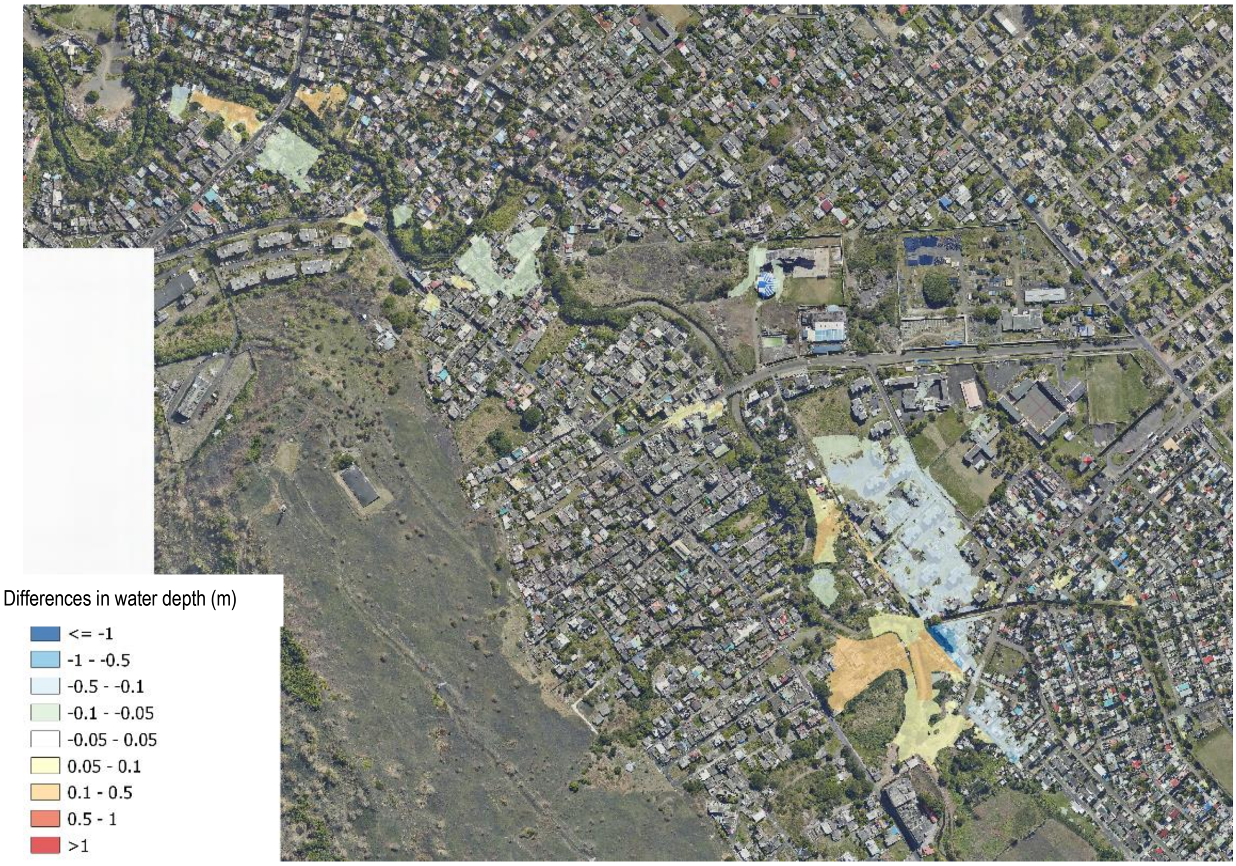


Figure 124: Rivière Lataniers – Dykes D1 and D2 – Differences in water depth (m) (100y)

Dyke No 1 contains the stream flow. However:

- Only the downstream section of the dyke needs to be raised by 1.60 m to contain river flows (Green line on the sketch overleaf)
- A section of the dyke shown as a redline serves no purpose, not being subject to flooding
- Another section shown as an orange line can be lowered to 500 mm

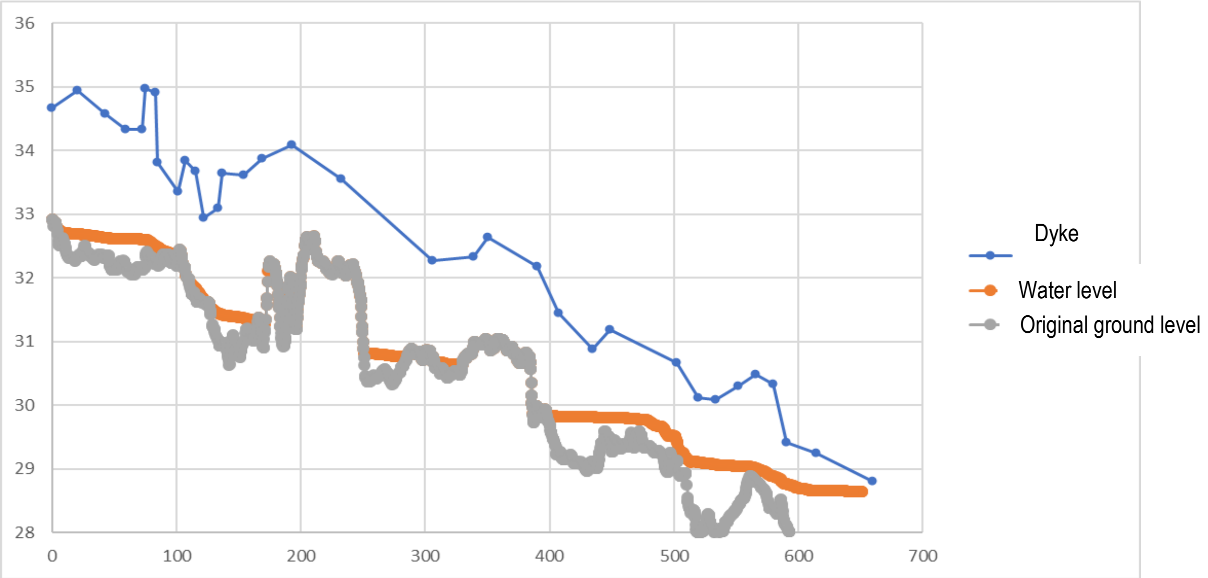


Figure 125: Rivière Lataniers – Dykes D1 and D2 – water level (100y) and ground level

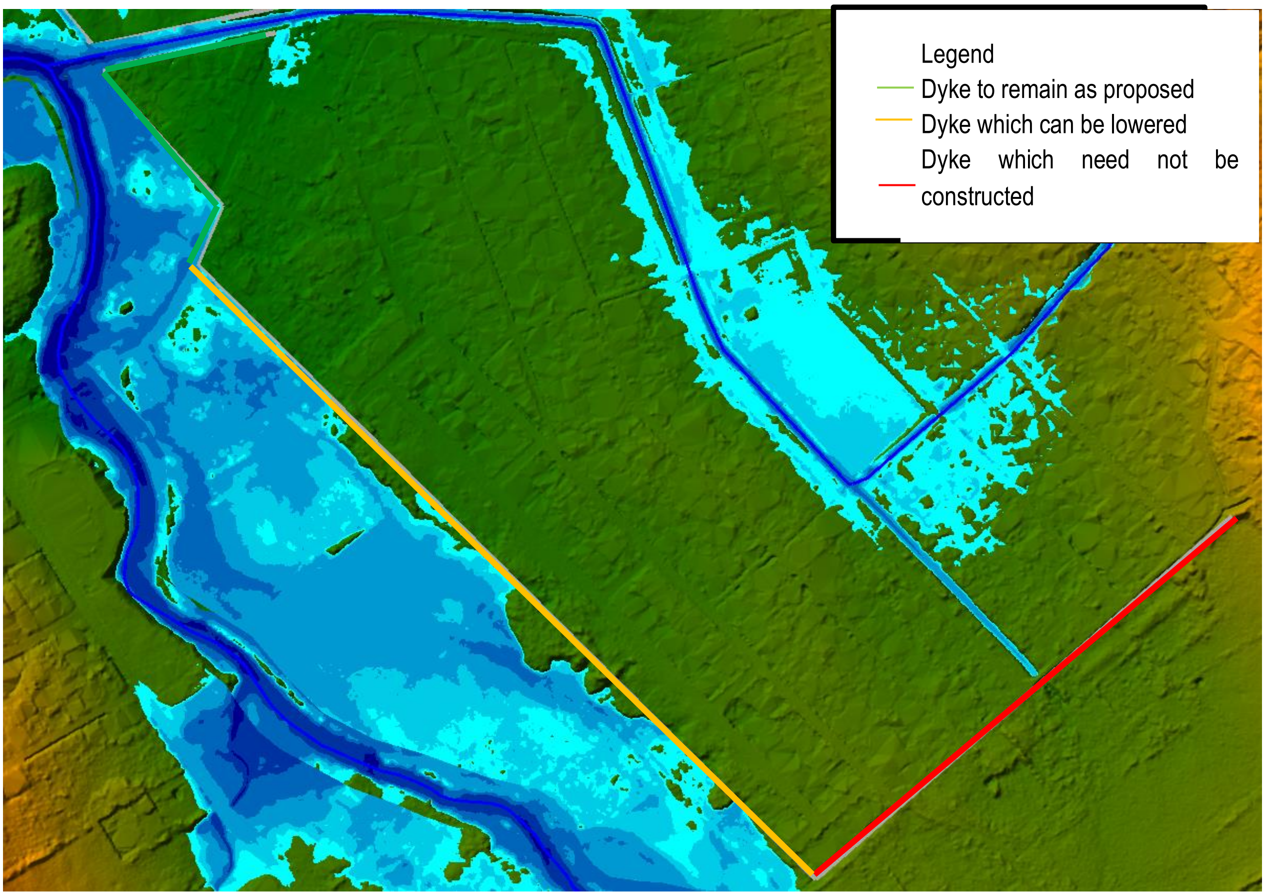


Figure 126: Rivière Lataniers – Dyke 1 location with 100 years flood water level

Dyke No 2 will also serve to contain the flood. However, it's not an ideal alignment since it does not contain water spill at the bridge. The zone is therefore not completely shielded from stormwater flow.

Moreover, the return flow will not find its way back to the minor bed.

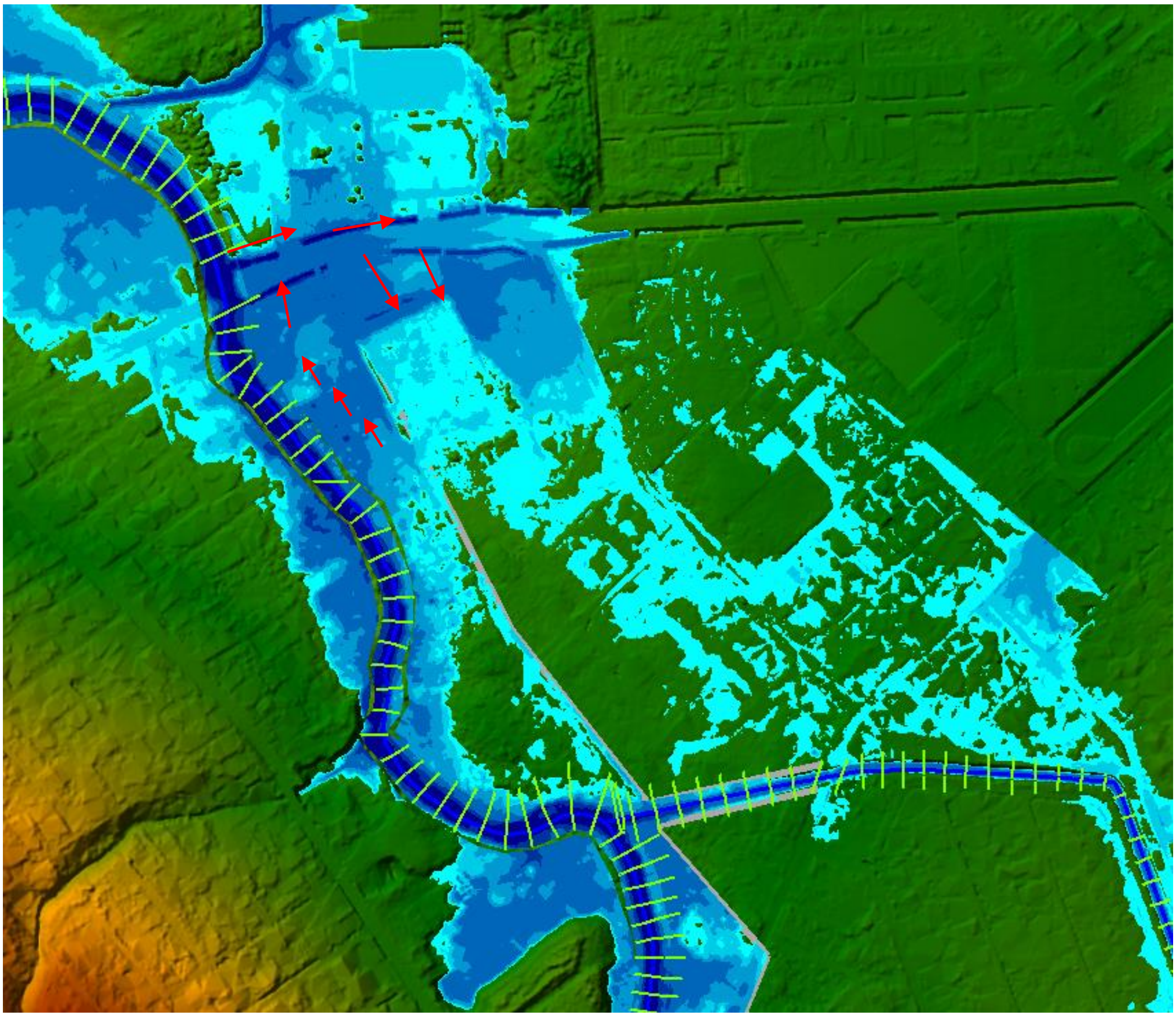


Figure 127: Rivière Lataniers – DykesD& and D2 location with 100 years flood water level

3.8.3.1 Additional Infrastructures

3.8.3.1.1 Description

The infrastructure comprises:

- A dyke of 1.60 m
- A dyke of 1.70 m

In addition, the following modifications are made:

- A cut-off drain of 2.5 m x 1.0 m deep to transfer flow from the upstream catchment directly into Latanier river (Stone masonry cut off drain, sloping face 1H:3V, Slope 1.2% - Carrying capacity: 9 m^3/s by modelling.
- Extension of the dyke proposed in the ER2C urban drain
- A new dyke of 2.0 m high along the urban drain
- Creating of a detention basin near the football ground

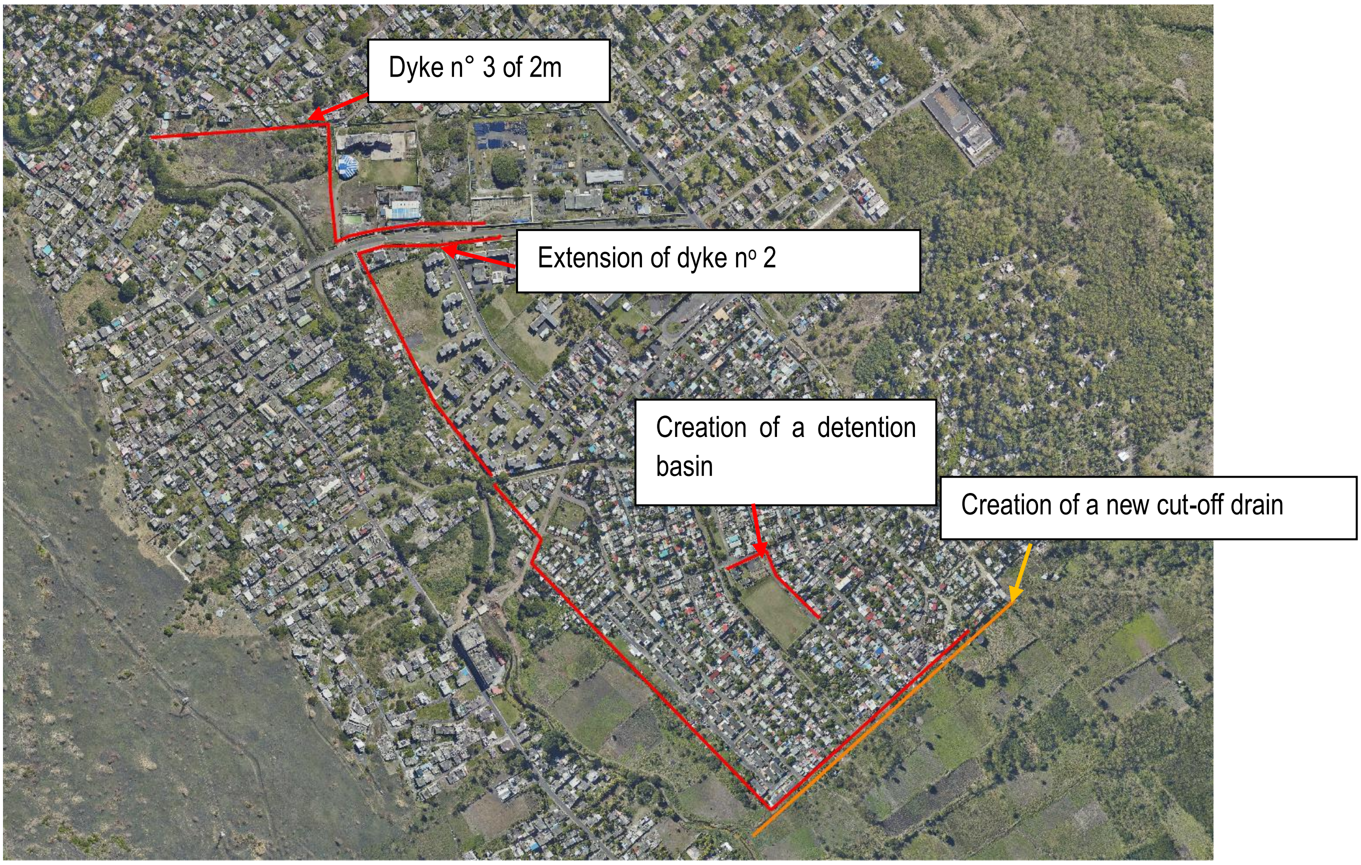


Figure 128: Rivière Lataniers – Additional infrastructures location

3.8.3.1.1 Result

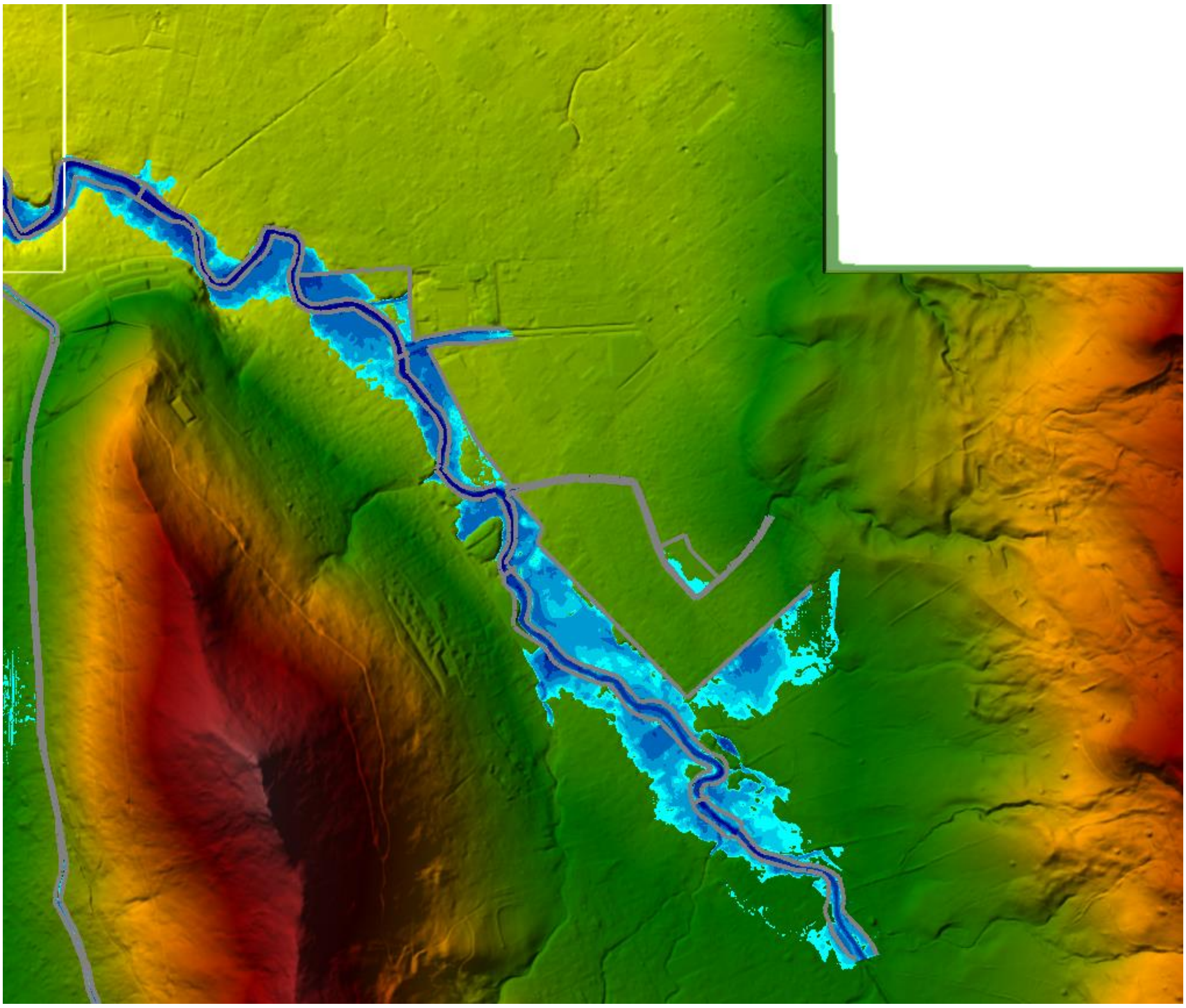


Figure 129: Rivière Lataniers – 100 year flood with additional infrastructures

It can be observed that:

- Extension of dyke no 2 will prevent the road being flooded
- Any over spill at the new cut-off drain will not have an impact on new assets.

The map below shows the differences in water depth

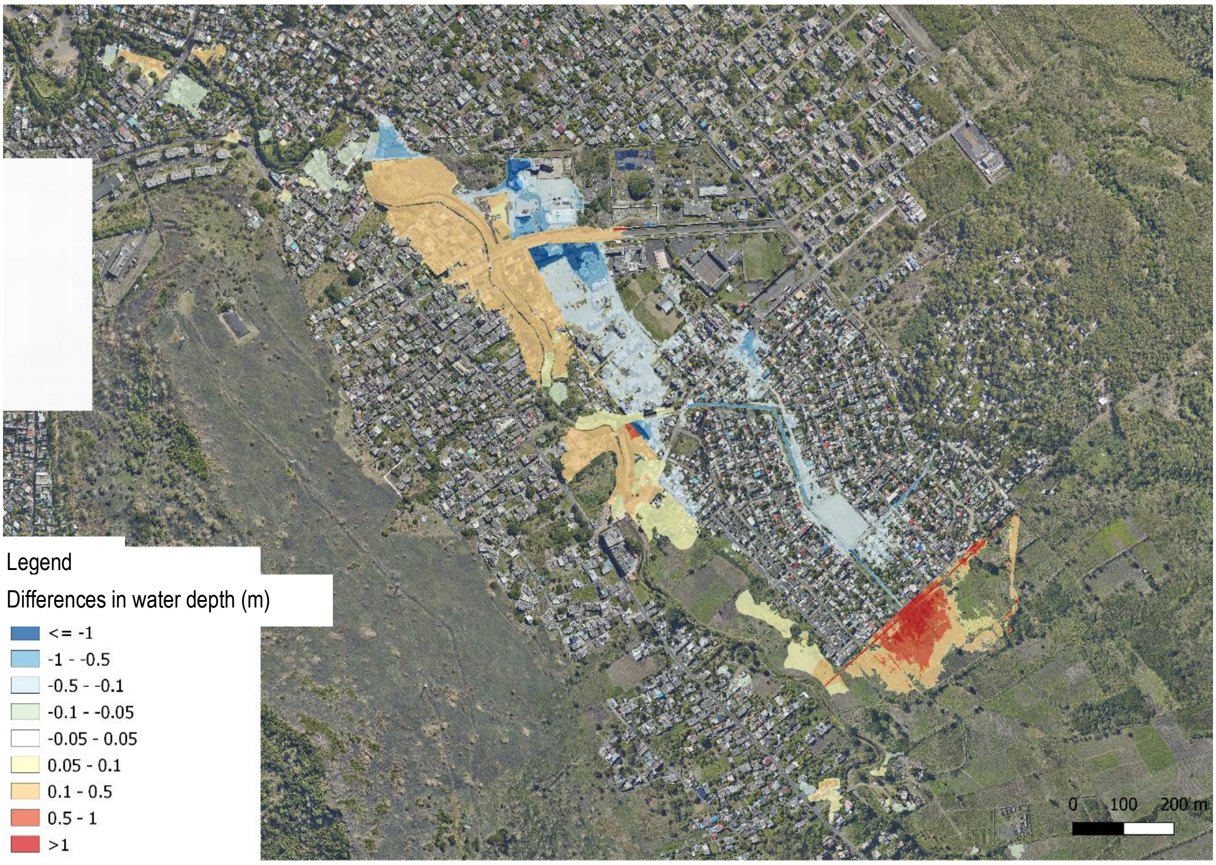


Figure 130: Rivière Lataniers – Differences in water depth (m) (100y)

3.8.4 MAIN IMPACTS, COST AND COST-BENEFIT

The following table summarises the measures retained and their associated costs. The works item number are cross referenced in the plans provided in Annex 1.

The comparative flood maps and the cost-benefit analyses together with cost details are given in Annexes 2 and 3.

The flood maps for the current situation (ie " Do Nothing Scenario") are attached in Annex 0: they are also attached to Deliverable D3.

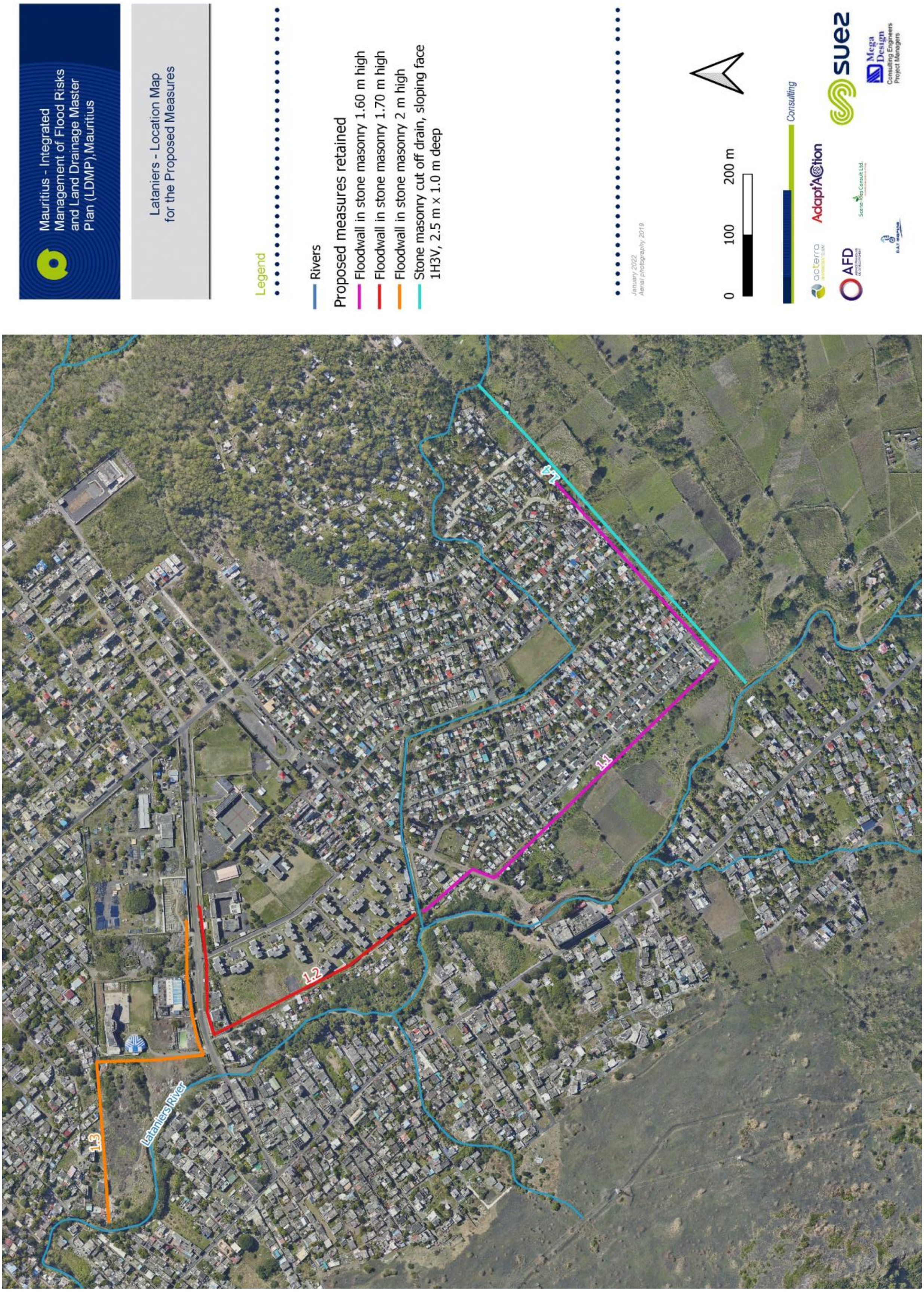


Figure 131: Port Louis – Rivière Lataniers – Location map for proposed measures

Table 54: Port Louis - Rivière Lataniers– Measures and costs

COST ESTIMATE					
N°	**RIVIERE LATANIERS**	**Unit**	**Quantity**	**Rate**	**Amount**
1	**Riviere Lataniers**				
1.1	Floodwall in stone masonry 1.60 m high	m	1050	33 000	34 650 000
1.2	Floodwall in stone masonry 1.70 m high	m	600	35 000	21 000 000
1.3	Floodwall in stone masonry 2 m high	m	660	45 000	29 700 000
1.4	Stone masonry cut off drain, sloping face 1H:3V, 2.5 m x 1.0 m deep	m	650	29 725	19 321 250
	Total				**104 671 250**
	ADD:				
	Provision for wayleave and Land Acquisition				10 000 000
	Contingencies 15%				15 700 688
	Project Management 7.5%				7 850 344
	Relocation of buildings along Riviere Lataniers	Sum			50 000 000
	Grand Total				**188 222 281**

3.9 Port Louis – Sector 74 - La Paix and canal des Anglais

Note: the additional site of La Paix has been included in D4.1 as the proposed works concern drains shared with the priority sites (Canal des Anglais and proposed extension of the cut off drain to Pouce Stream).

3.9.1 Overview

The catchment area of Sector 74 Port Louis La Paix, having an approximate area of 3.65 km², has been sub-divided into 11 sub-catchments. This catchment has a long drainage path of approximately 4.95 km and an average slope of 11.7 %.

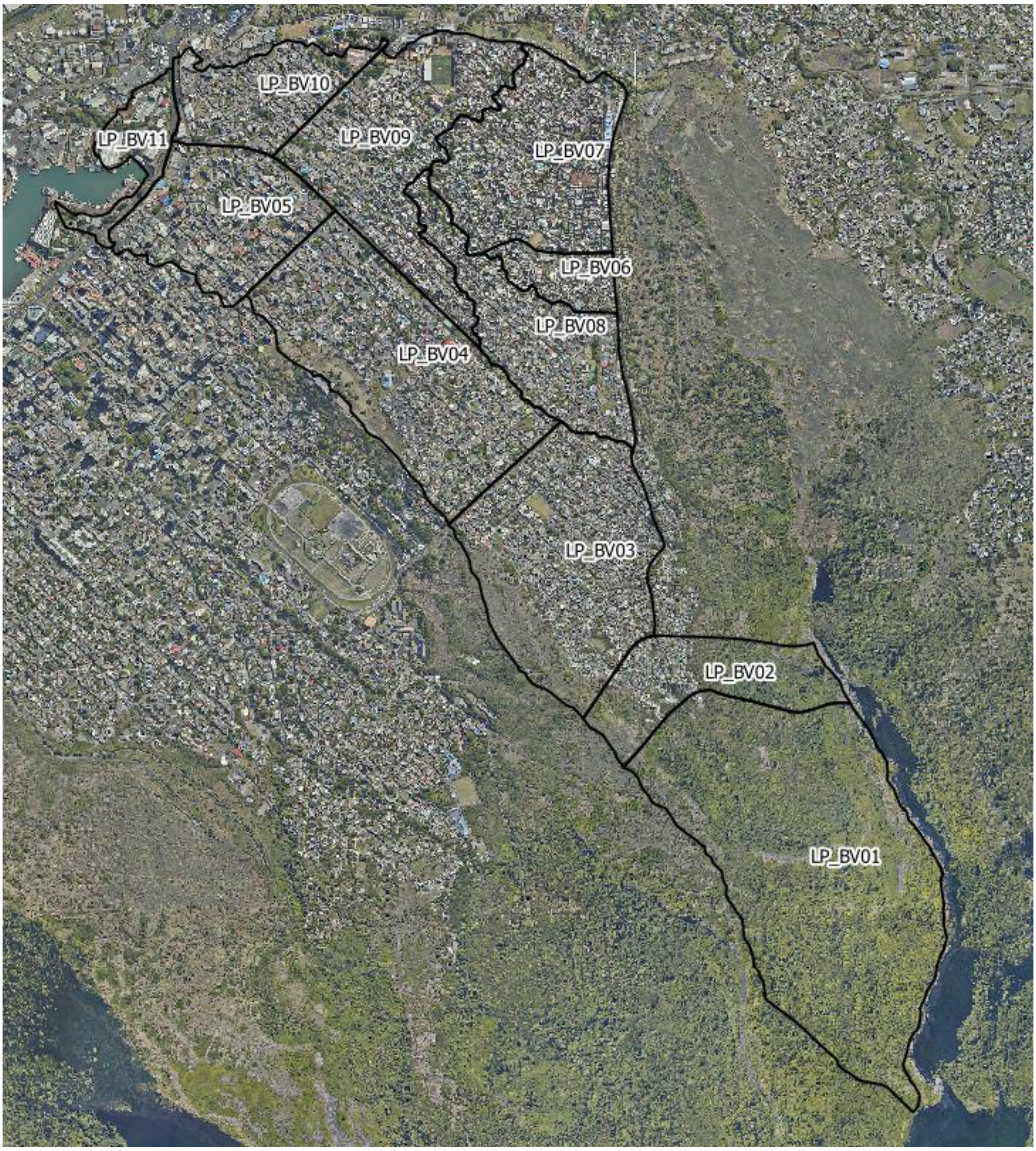


Figure 132: Sub-division of catchment area of Sector 74 Port Louis La Paix into sub-catchments (Orthophoto 2019)

Table 55: Sector 74 Port Louis La Paix – Physical Characteristics of individual Sub-catchments

Name	Area (ha)	Area (km²)	Low level (m)	High level (m)	Length	Slope (m/m)	Slope (%)
LP_BV01	89.07	0.89	77	579	1796	0.28	27.99
LP_BV02	20.39	0.20	59	365	905	0.34	33.75
LP_BV03	48.18	0.48	32	154	1239	0.10	9.86
LP_BV04	50.88	0.51	13	83	1507	0.05	4.68
LP_BV05	26.27	0.26	0	17	884	0.02	1.89
LP_BV06	7.20	0.07	21	54	426	0.08	7.92
LP_BV07	34.38	0.34	11	49	802	0.05	4.80
LP_BV08	25.03	0.25	11	64	1265	0.04	4.21
LP_BV09	37.59	0.38	4	26	1423	0.02	1.51
LP_BV10	17.84	0.18	1	18	927	0.02	1.77
LP_BV11	8.81	0.09	0	9	514	0.02	1.78
LP_Global	**365.63**	**3.66**	**0**	**579**	**4950**	**0.12**	**11.70**

The flows obtained for Sector 74 Port Louis La Paix for return periods of 10, 25, 50 and 100 years are shown in the table below.

Table 56: Sector 74 Port Louis La Paix – Flows for sub-catchments and at outlet of catchment for return periods of 10, 25, 50 and 100 years

Name	Q10 (m3/s)	Q25 (m3/s)	Q50 (m3/s)	Q100 (m3/s)
LP_BV01	14.4	18.4	21.5	24.7
LP_BV02	4.9	6.0	6.7	7.5
LP_BV03	13.5	16.0	17.7	19.6
LP_BV04	13.6	16.1	17.8	19.6
LP_BV05	7.0	8.3	9.2	10.0
LP_BV06	2.2	2.5	2.8	3.1
LP_BV07	9.8	11.6	12.8	14.0
LP_BV08	6.8	8.0	8.9	9.7
LP_BV09	9.4	11.1	12.2	13.4
LP_BV10	4.8	5.7	6.2	6.9
LP_BV11	2.6	3.0	3.4	3.7
Outlet of Ruisseau La Paix	**75.2**	**90.5**	**101.6**	**112.8**
Q /A (m³/s/km²)	***20.6***	***24.8***	***27.8***	***30.9***

3.9.2 Canal Anglais

Canal Anglais is a 2.5 km long open drain along the foot of Priest Peak Mountain constructed in 1990 mostly in reinforced concrete and stone masonry of width varying between 1.5 m and 4.5 m. It starts at Vallée Pitot at ground level 94.0 and discharges into Latanier river at approximate level 8.0 m. Some 400 m of the downstream end of the canal to its connection with Rivière Lataniers, hitherto unlined, is presently being concreted with a 4.5 m wide drain.

The decision to concrete line the downstream part of the earth-lined drain is considered as being inappropriate in that this will disturb the water exchange process between the stream and the adjoining land (bank and ground water recharge), more so as this stretch is not prone to flooding. Restricting the flow within the concrete lined drain will also attract illegal squatting along its bank, like elsewhere upstream. To discourage illegal occupation of the reserve and flood plain it is recommended to restructure the reserve into a terrace (lit majeur) as shown below.

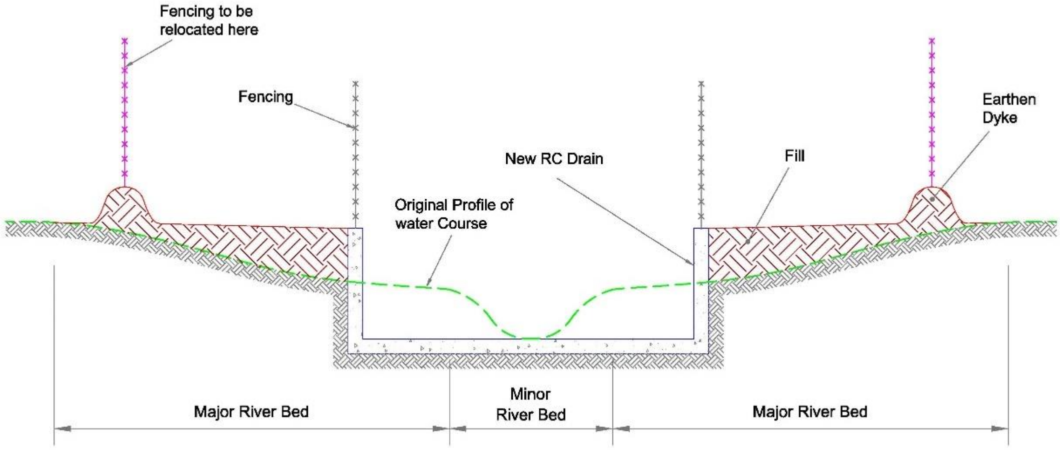


Figure 133: Canal des Anglais – typical section

3.9.2.1 Review of Previous Studies

The only previous study on Canal Anglais made available by NDU is a draft design report[4] by Gibb in January 2020.

The objective of the assignment was to check the existing capacity of the canal and subsequently to identify sections thereof which have been damaged with time and/or structurally non-performing in view of repairing/upgrading them. Gibb's findings were that the flood levels for the 1 in 50 year return period does not affect the surrounding areas and the existing culvert has adequate flow capacity to carry the required flood flow as per the computed flow data reproduced below.

[4] Draft Detailed Design Report, Jan 2020, Consultancy Services for Design, Supervision and Management of the Upgrading of Canal Anglais, Port Louis.

Sub-catchments contributing flows into Canal Anglais are:

Table 57: Physical characteristics – Canal des Anglais

Sub-catchment	Area (km^2)	Peak Flow Q_{50} m^3/s
A (upstream)	0.08	1.305
B	0.19	3.098
C	0.16	2.609
D	0.12	1.957
E	0.06	0.978
F	0.03	0.489
G	0.08	1.547
Total	0.72	11.983

The physical characteristics and the carrying capacity of the Drain and Culverts reported are as follows:

Table 58: hydraulics characteristics – Canal des Anglais

Chainage	Type of Construction	Peak Discharge Q_{50} m^3/s	Bed Slope	Maximum Discharge m^3/s (Carrying Capacity)
0 –140	Stone masonry	0.65	0.05	4.98
140 – 250	Stone masonry	1.30	0.06	10.22
250 – 500	Stone masonry	4.40	0.05	14.36
500 – 1000	Concrete	7.01	0.01	32.81
1000 – 1400	Concrete	8.97	0.04	63.57
1400 (Culvert)	Concrete	9.21	0.02	56.73
1400 – 1789	Concrete	9.95	0.02	43.35
1789 (Culvert)	Concrete	10.44	0.03	30.90
1789 – 2020	Concrete	10.82	0.01	24.26
2020 – 2325	Concrete/ unlined	11.60	0.02	42.17
2325 (Culvert)	Concrete	11.98	0.01	18.68

A layout of the existing drainage system from the report is reproduced overleaf.

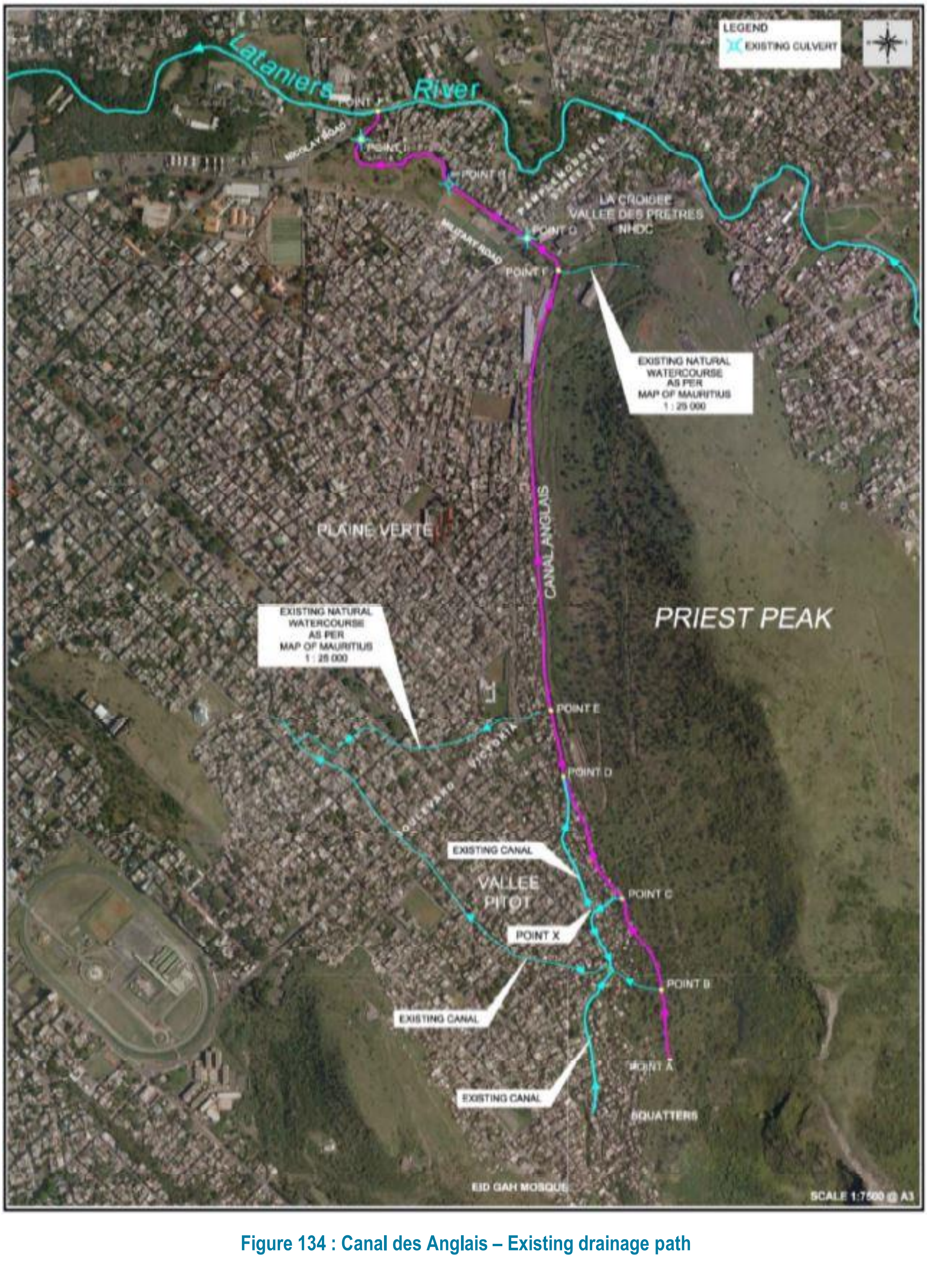


Figure 134 : Canal des Anglais – Existing drainage path

3.9.2.1 Hydraulic Modelling

3.9.2.1.1 Description

The upstream part of the canal is presently being cleaned and damaged sections repaired.

The downstream part of the canal over a stretch of some 400m hitherto earthlined is being rehabilitated with a rectangular reinforced concrete drain of size 4.45m x 1.5m depth with a carrying capacity of 20 m³/s

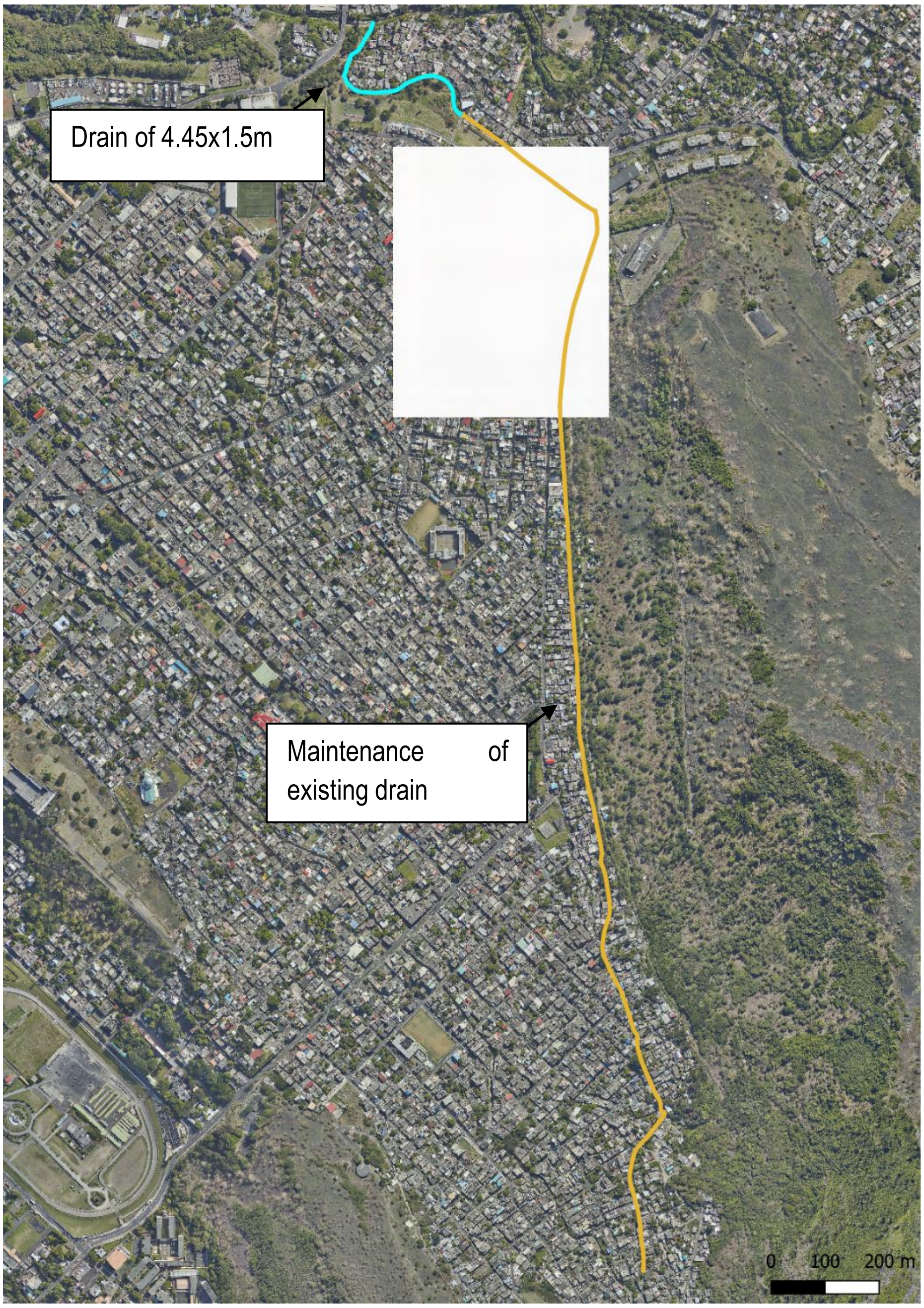


Figure 135 : Canal des Anglais – Works in progress (2021)

Within the downstream part a concrete drain of dimension 4.45m x 1.5m is being constructed in replacement to the earth lined stream which had a width varying between 5m and 7m and a depth of about 1.20m. This implies that the replacement drain is smaller in size. It was therefore decided not to model this change which will not have a positive impact on flooding.

The existing pedestrian bridge, which is planned to be pulled down, has also not been input into the model.

3.9.2.1.2 Increasing the capacity of Canal des Anglais to accommodate the Pouce Stream Cut off drain project

Cf 3.5.4

In total, nearly 40 m^3/s will be diverted from the catchment areas of Pouce stream (28.1 m^3/s) and La Paix stream (11.0 m^3/s) to Canal des Anglais.

The Canal des Anglais will be resized to a section of 9 m wide by a minimum of 1.5 m deep to accept these additional flows. The total carrying capacity will be 55 m^3s.

3.9.2.1.3 Additional Infrastructure Proposed

Along the mid stretch of the canal, even minor flooding will adversely affect the densely built areas and raising the drain wall at this location will alleviate flooding.

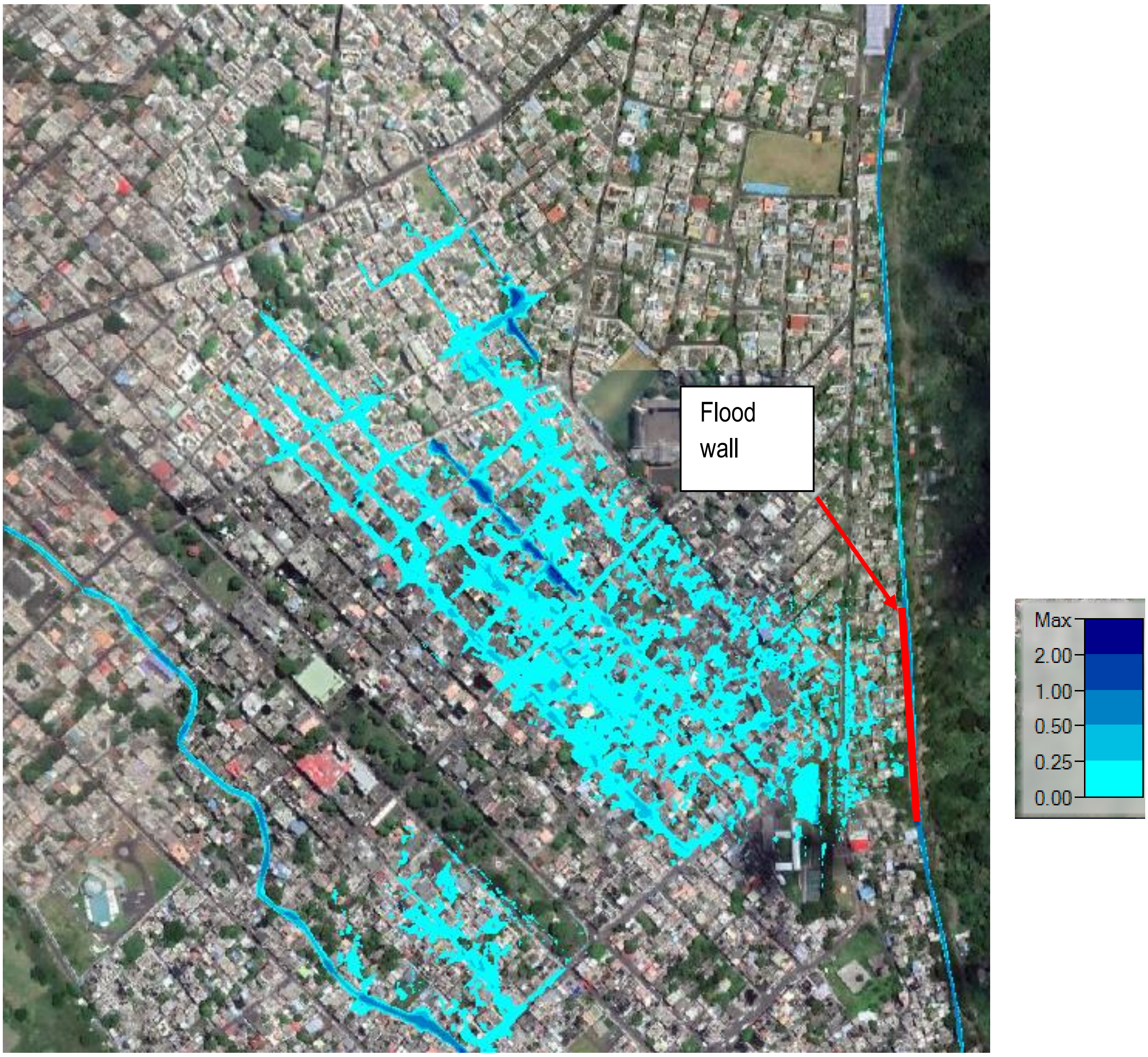


Figure 136 : Canal des Anglais – Current hydraulic insufficiency

In order to accommodate partial flows from the cut-off drain at the upstream stretches of Pouce and La Paix streams canal des Anglais will have to be resized to a revised section 9 m wide by a minimum of 1.5 m deep. This resizing will eliminate overflows at this location.

3.9.3 La Paix Stream

3.9.3.1 Background

La Paix stream drains the suburban regions of Vallée Pitot, Plaine Verte and Camp Yoloff. It is joined by a tributary canal Fanfaron just upstream of the M1 motorway.

Flooding problems are mostly associated with overflowing of the banks at bridges and are accentuated in the region of Plaine Verte and at the stream confluence with Canal Fanfaron. Overflowing at bridges is attributed principally to:

(i) Insufficient hydraulic capacity and freeboard
(ii) Walls of multi-cell culverts trapping floating debris during floods
(iii) Utility services spanning across the water way
(iv) Encroachment of buildings onto the stream

Multi-cell Culvert at Gravier Street

Trespassing by Utility Services

Encroachment at Mariamen Temple

Skewed crossing across Barbeau Street near M1

3.9.3.2 Hydraulic Modelling

Hydraulic modelling of the existing infrastructure indicates flooding induced by undersized bridges which need raising or widening at the level of Plaine Verte and upstream of the M1 motorway.

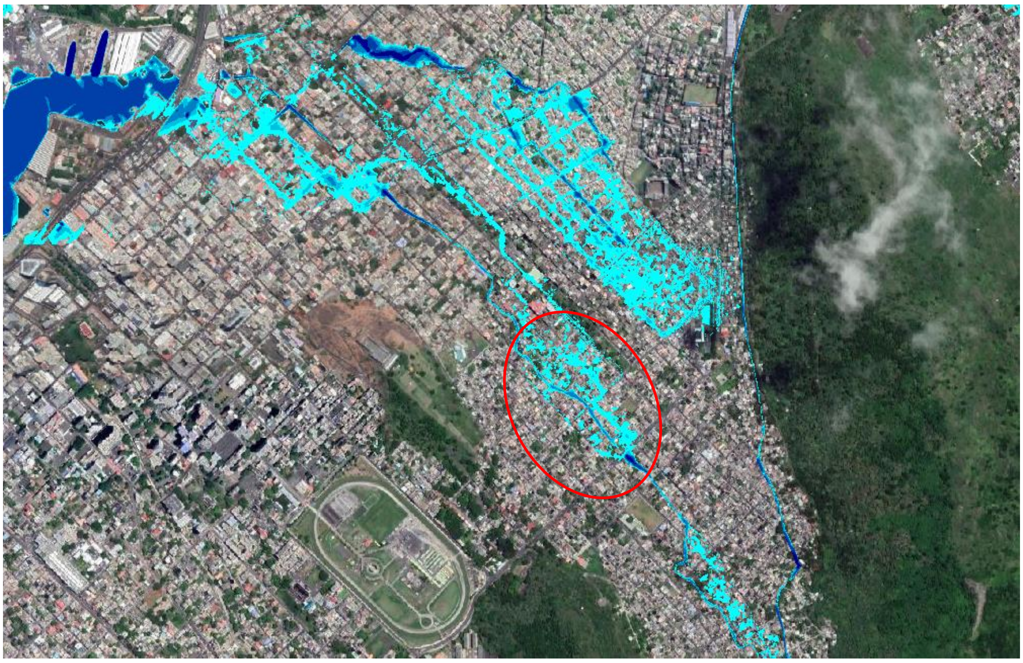
Bridge across Nyon Street

Bridge across Diore Street

Bridge across China Road

Bridge across Blvd Victoria

Along the upstream section of La Paix stream, the flooding shown encircled in red on the screenshot below is induced by undersized bridges which need raising or widening.



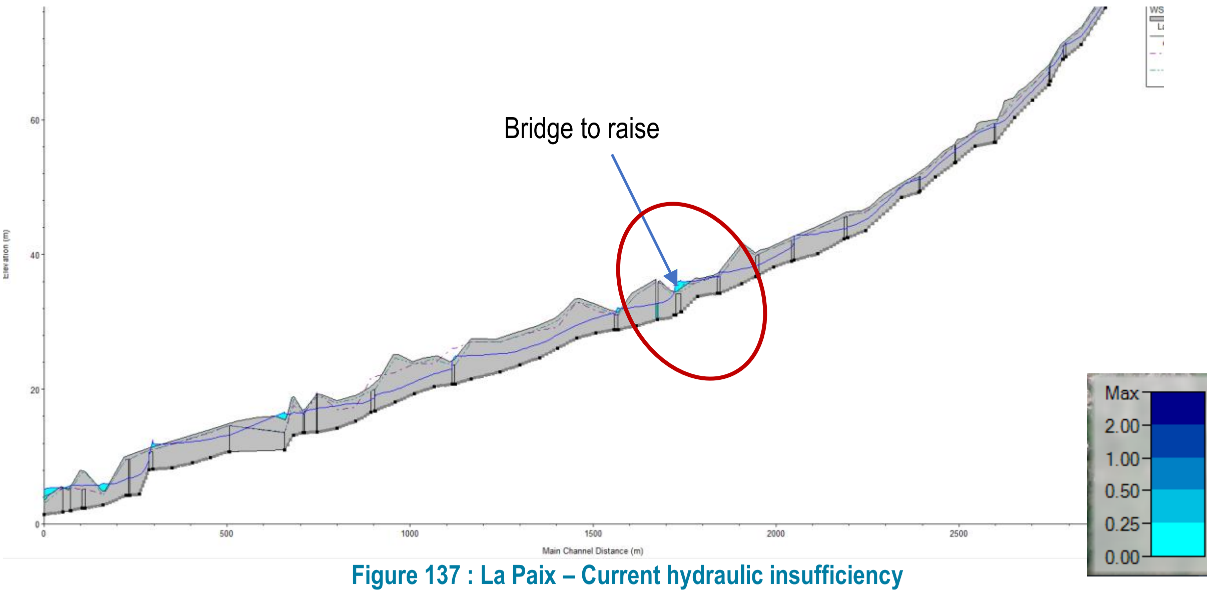


Figure 137 : La Paix – Current hydraulic insufficiency

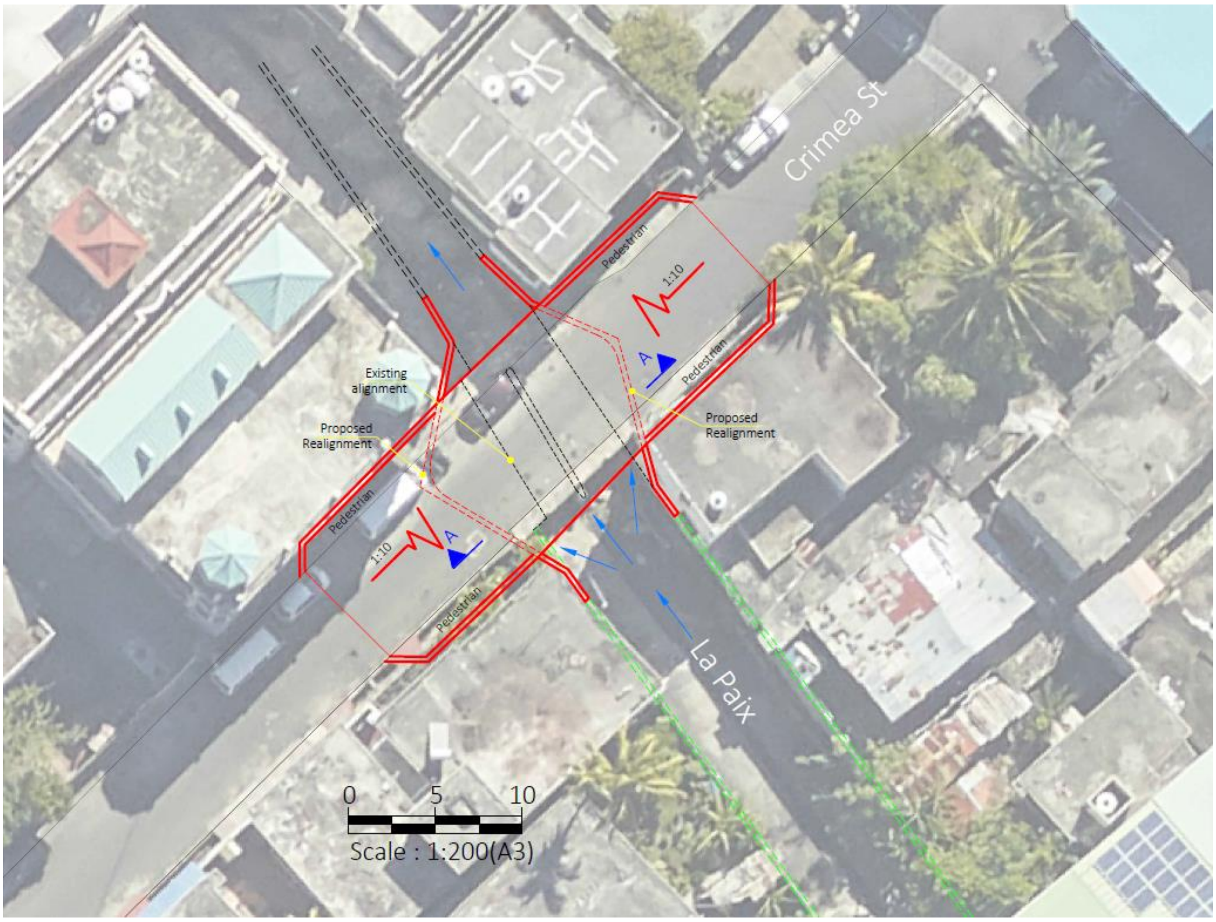
Along the downstream section of the stream and the upstream section of the tributary to the north (Canal Fanfaron), numerous flood prone areas can be observed. Considering the high urban density, general widening of the drain is not feasible. The only flood alleviation scheme which could be considered are:

(i) Reducing upstream inflows by extending Canal des Anglais upstream to intercept a part of La Paix catchment (cf. 3.5.4)
(ii) Raising culverts where feasible.

It is proposed the resizing of the green plain bridge: installation of an 8m wide and 3.5m high frame: Carrying capacity: 60m³/s. This structure is located on the figure below.

Figure 138 : La Paix – Resizing of the Green Plain Bridge

Below are the drawings for the reconstruction of bridges in confined space such as Port Louis - La Paix.



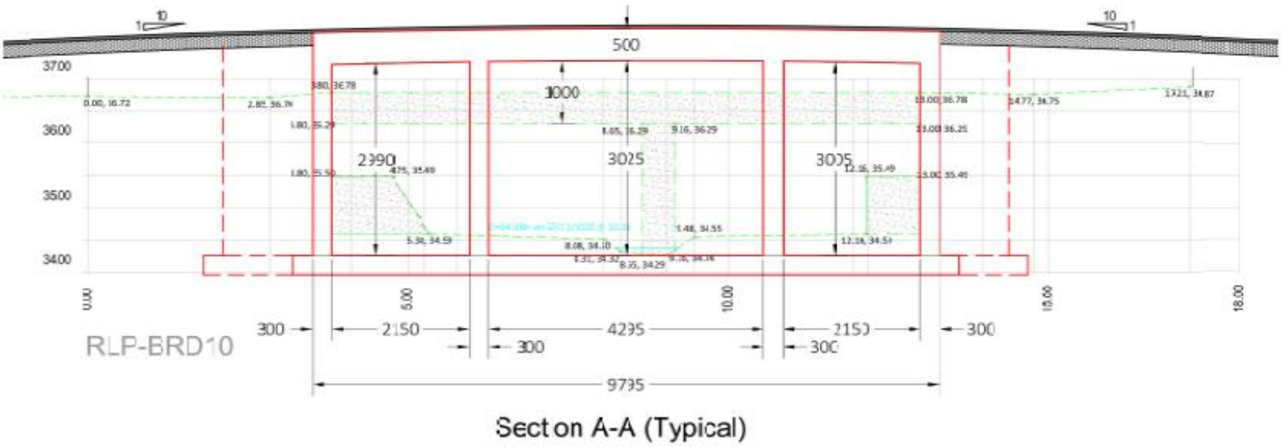


Figure 139 : La Paix – Typical Resizing of Bridge within confined space– Drawings and section

3.9.4 MAIN IMPACTS, COST AND COST-BENEFIT

The following table summarises the measures retained and their associated costs. The works item number are cross referenced in the plans provided in Annex 1.

It should be noted that the incidence and costs of the works to be considered here in the CBA of the Ruisseau la Paix include in a global way.

- The works related to the cut of partial drainage of the Pouce river, Ruisseau la Paix and resizing of the Canal des Anglais
- The additional works specific to the Pouce and the Paix streams
- Damage reduction for all protected areas, i.e. the Pouce River and La Paix

Thus, the CBAs for Pouce Stream and La Paix are to be analysed in a global way.

The comparative flood maps and the cost-benefit analyses together with cost details are given in Annexes 2 and 3.

The flood maps for the current situation (ie " Do Nothing Scenario") are attached in Annex 0: they are also attached to Deliverable D3.

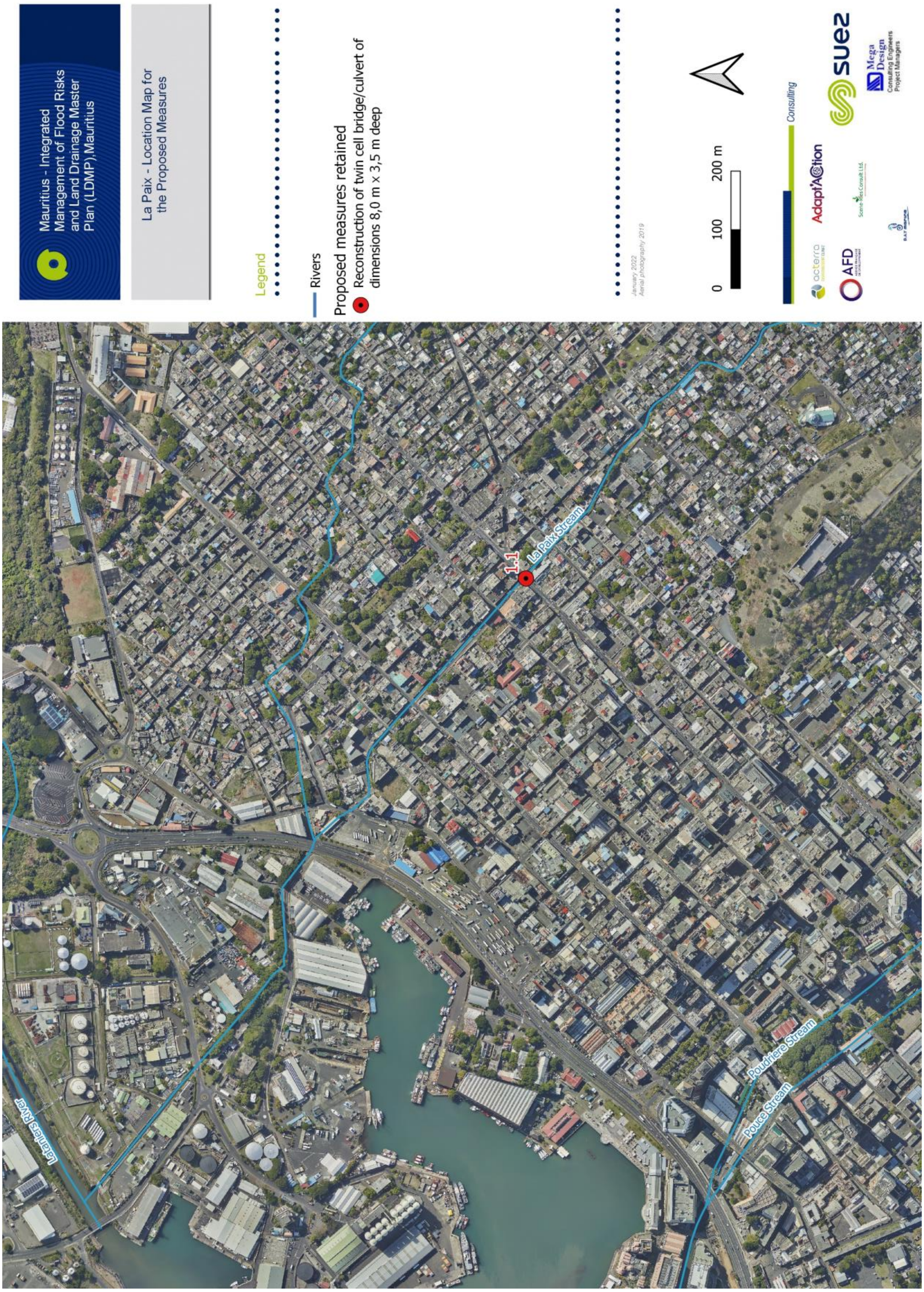


Figure 140: Port Louis – Sector 74 - La Paix and canal des Anglais – Location map for proposed measures

Table 59: Port Louis – Sector 74 - La Paix and canal des Anglais– Measures and costs

COST ESTIMATE					
N°	**74 - LA PAIX STREAM**	**Unit**	**Quantity**	**Rate**	**Amount**
1	**La Paix Stream**				
1.1	Reconstruction of twin cell bridge/culvert of dimensions 8.0 m x 3.5 m deep	Sum			**22 000 000**
	ADD:				
	Provision for wayleave and Land Acquisition				5 000 000
	Contingencies 15%				3 300 000
	Project Management 7.5%				1 650 000
	Grand Total				**31 950 000**

COST ESTIMATE					
N°	**CANAL DES ANGLAIS (Reminder – Le Pouce Stream)**	**Unit**	**Quantity**	**Rate**	**Amount**
1	**Canal des Anglais**				
1.1	Widening of Canal des Anglais from existing 4 m to 9 m	m	2750	80 000	220 000 000
1.2	Lowering of invert of Canal des Anglais by a maximum of 1 m (Reconstruct)	m	450	45 000	20 250 000
	Total				**240 250 000**
	ADD:				
	Provision for wayleave and Land Acquisition				10 000 000
	Contingencies 15%				36 037 500
	Project Management 5%				12 012 500
	Grand Total				**298 300 000**

ANNEX 0

MAPPING: WATER DEPTHS FOR CURRENT SITUATION (“DO NOTHING SCENARIO”)

ANNEX 1

MAPPING: LOCATIONS OF MEASURES

ANNEX 2

MAPPING: IMPACTS AND RESULTS ASSESSMENT DUE TO FLOOD RISKS

ANNEX 3

COST BENEFIT ANALYSIS (DIGITAL FILES)