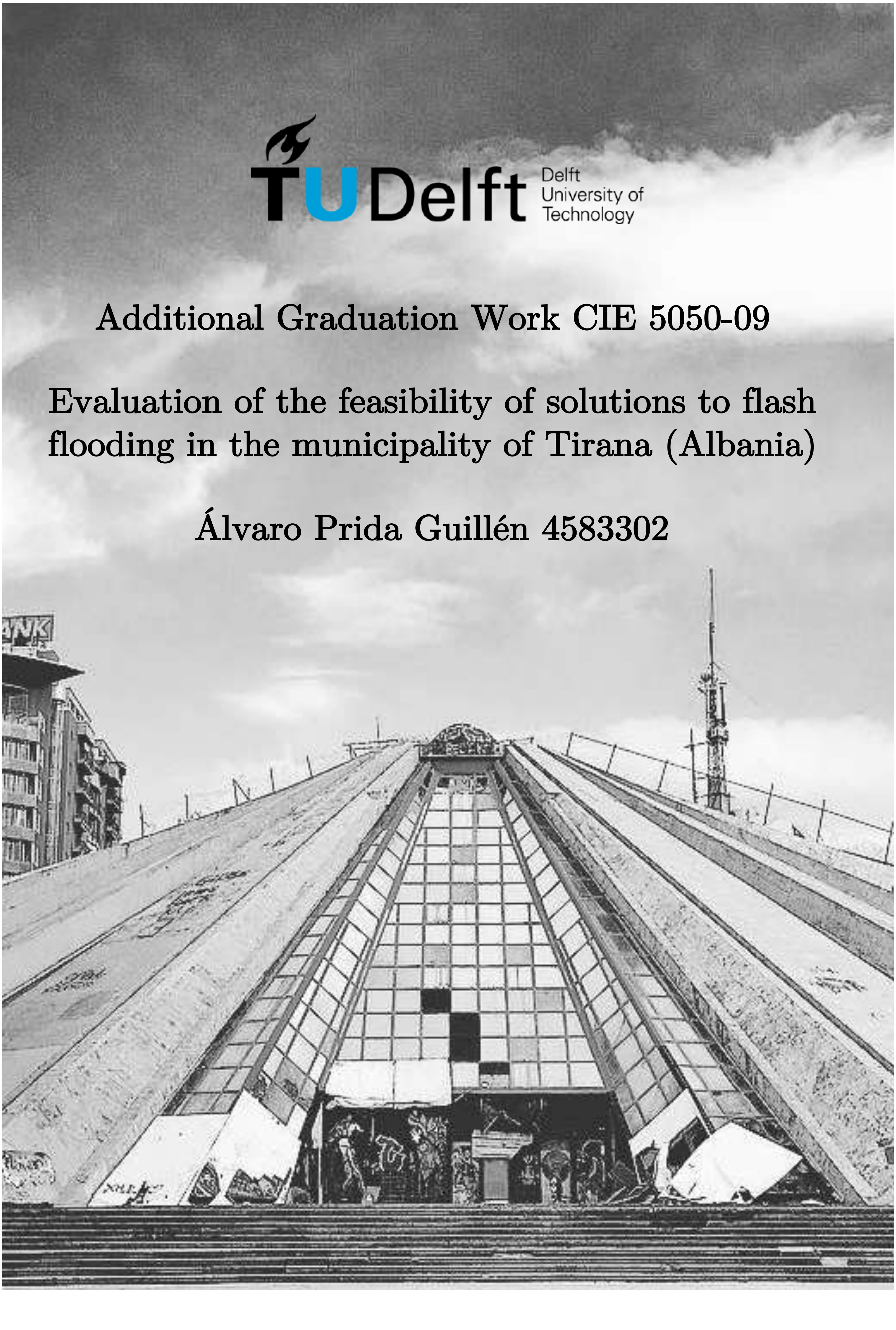

Additional Graduation Work CIE 5050-09

Evaluation of the feasibility of solutions to flash flooding in the municipality of Tirana (Albania)

Álvaro Prida Guillén 4583302

Delft University of Technology

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2018-2019

Preface

Right after my Bachelor of Science Graduation in July 2017, I went on a trip to the Balcans. This mountainous region fascinated me by its history. I was curious to meet their traditions, their people, and their links to the Otoman Empire in the past. Less than hundred kilometers separate the Italian coastal region of Puglia and the Albanian coastal county of Vlorë. However, despite the geographical proximity of the mentioned locations, the cultural differences are notorious. One of the most representative examples is the difference in the traditional religious confession of each country, which is the Christianity in the case of Italy and the Islam in the case of Albania. This represented an important contrast in Europe, a continent that is known as traditionally Cristian. During my trip to the Balcans, I could visit several neighbor countries like Macedonia, Serbia or Greece, but I did not visit Albania. However, the future had kept me the opportunity to visit this country sooner than expected.

In February 2018, I started to plan my second year as Master student at the Delft University of Technology. In order to enhance my profile as Hydraulic Engineer, I wished to gain some international experience in the Flood Risk field. On top of that, I was seeking for a project in which I could interact with students and experts from different technical fields and countries. My goal was to collaborate with students of different disciplines, in order to achieve an integrated design from the technical, the economical and the sustainability perspective.

The project in Tirana proposed by the Delft University of Technology focused on the two goals that I was looking for. The project consisted of two tasks. The first one was a master-plan, in which general design flood protection concepts were developed. In the master-plan, students from Architecture, Hydraulic Engineering and Transport and Planning collaborated together to elaborate on the design proposals. The second part was an individual project on the evaluation of the feasibility of different flood protection systems, which is presented in this report.

For the above mentioned reasons, I felt that the project in Tirana could improve my engineering skills on the first place, but it could also be very useful to incorporate knowledge from other disciplines, crucial to achieve an integral design. From the 26th to the 30th April workshops were carried out at the Polytechnic University of Tirana (UPT) to get a better perspective of the areas affected by the flood events during December 2017. The objective of the workshops was the evaluation of the current state of the Limuthi river system. The institutions involved in the workshops were TU Delft and BRIGAID, including experts from both entities, and students from Delft University of Technology, the Polytechnic University of Tirana, the Epoka University and the University of Tirana.

Álvaro Prida Guillén
Delft, 15th November 2018

Abstract

In the recent years, several cases of flash floods have been reported in the Municipality of Tirana, causing important material damage in the region. One of the most devastating events occurred in December 2017, which flooded the City Park area (West of the Municipality of Tirana). This area is strategical because it includes relevant centers of economic development, such as commercial and industrial areas, that are located nearby a main road connection between the Port of Durres, the Tirana International Airport and Tirana itself.

To mitigate the flood risk at this location, several hydraulic engineering concepts have been developed. However, the suitability of the concepts is limited by two main constraints: budget constraints and spatial constraints. With the aim to develop a flood protection system that is financially and spatially feasible, a system that combines dikes and flood walls is proposed. The industrial and commercial areas of the City Park are very close to the river. Hence, due to the spatial constraint, flood walls are used to protect these areas. Dikes are implemented in the river boundaries were agricultural and green land uses are found. Grass covers are implemented on the dike slopes to compensate the impact provoked by the water retaining structures in the local environment. The grass enhances the local visual amenity. Furthermore, bike paths are implemented on the dike crests and contiguous to the flood walls to improve the livability of the area.

In order to adapt further to the budget, the dimensions of the structures to be built are reduced by dredging the river beds, what increases the section of the river to convey water. A granular filter is placed on the river bed in order to reduce the soil erosion. Moreover, in order to reduce the financial and visual impact of the dikes, temporary structures of one meter height will be placed on the crest of the dikes in case a 100-year flood event occurs.

A Cost-Benefit Analysis (CBA) of the proposed solution has been carried out to evaluate the feasibility of the alternative. The resultant BC ratio of the solution is 0.84, which is close to what is considered feasible ($BC > 1$). However, this ratio is sensitive to the discount rate. According to the falling tendency of this rate in Albania in the recent years, as shown by the information published by the Central Bank of Albania, the rate is expected to fall from 1.25% to 1% in the close future. This drop would make the project feasible ($BC = 1.05 > 1$).

Acknowledgements

I would like to thank Dr. Mark Voorendt for the guidance along the project. From him, I have learnt the importance of the methodology and the structure when addressing an engineering problem, knowledge that will be very valuable for my future career. Also I would like to thank Dr. Olivier Hoes for his critical reflections on my progress and his support in the use of softwares.

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1 Introduction

1.1 Motivation for the current design

In the recent years, the municipality of Tirana (Albania) has been suffering flood events during the winter season. One of the most devastating ones occurred on the 2nd and 3rd December 2017. One of the most damaged spots was the City Park, an area that gathers industry and commerces located 14 km to the West of the city center.

The City Park is located in a very strategical spot, since it is next to one of the most important highways in Albania, which connects the port of Durres to Tirana. The direct effects of the flooding included the damage of industrial, commercial residential and agricultural areas. However, the indirect damage had also a relevant impact. The transportation of goods and the private users were affected during the December 2017 flood due to the interruption of the traffic, specially in the connections between the center of Tirana and the port of Durres, and the accesses to the airport.

Figure 1: Project location.

The first economic center developed in the City Park was a commercial mall, which was built in 2011. From this year on, the area has been characterized by an important economic growth. First, the mall and an Aquapark, and later private companies were established in the area. In this line, the land use of the nearby properties has been changing from 2011 (from agricultural to commercial or industrial), increasing the economic value of the land. Consequently, the risk of flooding has increased substantially since the economic potential loss is now much larger than before 2011. Furthermore, the flooding also harmed residential, agricultural and industrial areas located in the Limuthi watershed.

Based on the above mentioned reasons, the municipality of Tirana is interested in reducing the flood risk of the area, in order to reduce the material losses and the indirect damage in case a flash flood occurs. This project analyses the structural possibilities to achieve this end.

The project aims to set a precedent in flood protection strategies in the Western Balkans, where flooding events during winter seasons are frequent. This project can be extended to the whole Ishmi basin, which is the larger watershed to which the Limuthi river belongs, so as the solutions presented in this report can be extended downstream of the Limuthi river.

1.2 Report structure

Before starting the design process, a solid strategy to achieve the final solution is required.

Before tackling the problem, it is necessary to acquire a proper understanding of the problem and its causes. This will be the first step of the process. To do so, the precedents in the history of Albania related to flash flooding are studied (Subsection 2.1). An initial analysis at national level will show the relevance of these natural disasters in a large part of the Albanian territory. The figures exposed pretend to create awareness on the reader about the vulnerability to floods of the country. On the other hand, the study of previous floods allows to acquire knowledge about why the floods were produced and what measures were taken in the past. Based on the review of past events, improvements in the strategy to follow in future projects can be proposed.

Subsequently, the analysis is brought to a more local scale. The recent flood events in the Ishmi basin are analyzed (subsection 2.2). In this subsection, the Ishmi watershed is described. Finally, the effects of the last major flood in December 2017 are exposed.

The separate analysis in a national and local scale create a background to justify the development of the project. The project itself is defined in the following section (section 3). This section starts with the problem statement (subsection 3.1). Once it is known what problem needs to be faced, the objective of the project can be set (subsection 3.2).

The next step is defining a framework for the design. Two components delimit the mentioned framework: a list of requirements (subsection 3.3), which is the result of a stakeholder analysis, in which the interests of the different stakeholders are addressed, and the boundary conditions (subsection 3.4), which are given by the physical characteristics of the area of study.

After the design definition, the design methodology is proposed (section 4). In this section, the steps of the process to arrive to the final design are explained. Subsequently, it is proceeded to the analysis of the current situation, which consists on the flood assessment of the Limuthi basin, given a certain flood event (section 5). First, explanations about the model setup are given (subsection 5.1). Second, a flood event is simulated with with HEC-RAS. The results of the dynamic simulation determine which areas of the basin are more vulnerable to damage (subsection 5.2).

To mitigate the damage, flood protection concepts are proposed in section 6. Based on the requirements of the stakeholders and several constraints (subsection 3.3), some of the presented concepts will be selected as alternatives for the design (section 7). A Cost Benefit Analysis (CBA) is carried out to evaluate the alternatives (section 8). The cost corresponds to the investment and the benefit corresponds to the risk reduction. The Benefit-Cost ratio enables to set an order of preference of the alternatives. Subsequently, specifications and detailing of the selected measure will be shown in section 9. Finally, conclusions about the project and recommendations for further development are given (section 10).

2 Context

2.1 Albania: a country prone to flash floods

The Western Balkans is a region prone to natural disasters. Specifically in Albania, from 1850 to 2013, there have been recorded more than 4000 events. The 68% of the recorded disasters are related to hydrology, meteorology or landslides (Toto and Massab (2014)), and all three types of events are concentrated in the season from November to February (Figure 2), increasing the probability of simultaneous occurrence. This fact makes the country very prone to flash floods. In the case of Albania, the main causes of flash flooding are:

- **Heavy long-lasting precipitation**: it increases the water discharge of the river due to the direct rainfall in the course and the run-off.
- **Landslides and accumulation of debris**: the increase of the water level in the streams can lead to slope instabilities and transport of sediment and debris. These materials can be accumulated in a certain section of the river, creating a bottleneck. Consequently, there is less cross sectional area for the water discharge and the probability of overflow increases.
- **Dam bursting**: this aspect is also consequence of heavy rainfall and increased water discharge. When the water level in a reservoir exceeds a certain value, the resistance of the dam to the hydrostatic pressures cannot be guaranteed. Consequently, the gates must be opened, causing a sudden release of a large discharge that cannot be assumed by the main course of the river, provoking flooding of downstream locations.
- **Change of land use**: the deforestation and the change of agricultural land to construction land lead to slower filtration velocity of water to the soil, in case of heavy precipitation. In some cases, informal buildings interrupt the main course of the river, reducing the available river section to assume the discharge and hence increasing the flooding probability.

At the same time, the statistics about flash floods show an increasing trend for the last 20 years (Figure 3).

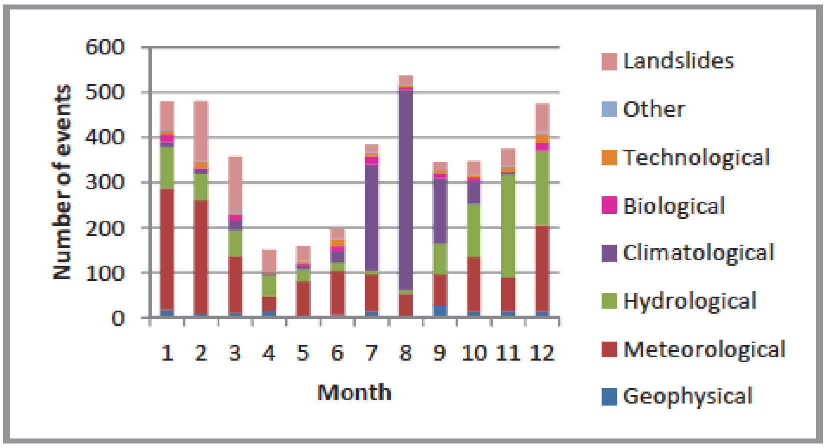

Figure 2: Seasonal distribution of the types of natural disasters from 1851 to 2013 (Toto and Massab (2014)).

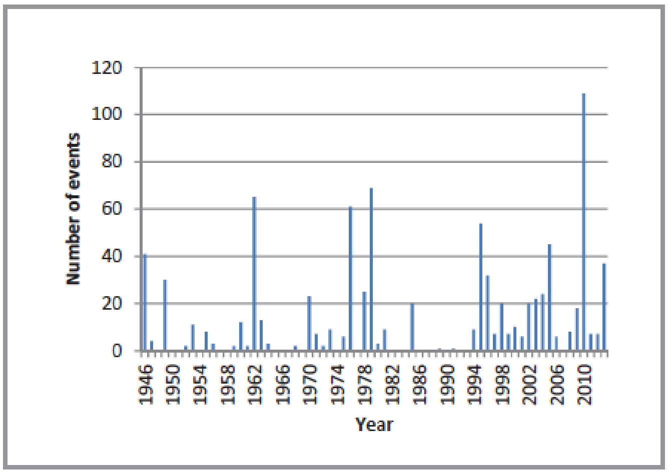

Figure 3: Flash floods trend for the period 1946 - 2013 (Toto and Massab (2014)).

The consequences of the flash floods in Albania are mainly material losses. The mortality due to flash flooding has been decreasing in the last 20 years and the maximum people killed in a flood in this period has been 5 casualties in 1995 (Kacinar - Mirdite district, northwest of Albania).

The main losses are related to the agricultural land uses. On the one hand, the flash floods are responsible for the 70% of the agricultural losses caused by natural hazards. Agriculture represents the main source of income for the population living in rural areas (46.3% of the total population of Albania). Even if it has been decreasing in recent years, agriculture represents an important part of the GDP of the country (19%, 2016) (Nations (2016)). Additionally, the 45% of the total employed citizens work in the agriculture sector. All these figures give an idea of the socioeconomical relevance of this sector in Albania. As an overall, the flash floods provoke 72% of the economic losses related to natural disasters, what includes all types of land uses. A summary of the figures recorded from 1993 until 2013 is shown below (Figure 4).

	Average for year	Maximum for year	Average for event	Maximum for event	% of communes (%)	Most affected communes
Events	22.4	109 (in 2010)	n.a.	n.a.	39	Balldren i Ri (14), Ana e Malit (11), Zejmen (11)
Mortality	1.05	9 (in 1995)	0.05	5	1.6	Bushtrice (5), Kacinar (5), Gramsh (2)
Houses destroyed/ damaged	40/ 1,136	539 (in 1995)/ 9,672 (in 2010)	1.8/ 50.7	300/ 7,000	11/ 21.1	Gur i Zi (300), Shkoder (150), Dushk (46)/ Dajc (2,977), Shkoder (2,221), Berdice (1,352)
Agriculture (Hectares)	7,419	43,739 (in 2010)	331	20,000	16.3	Levan (7,000), Balldren i Ri (3,000), Gradishte (2,600)
Economic losses	370 mln LEK	4,040 mln LEK (in 2010)	16.5 mln LEK	1,100 mln LEK	17.6	Dajc (1,000 mln LEK), Berdice (540 mln LEK), Bushat (480 mln LEK)

Figure 4: Summary of the consequences of flash floods and floods in the period 1993 - 2013 (Toto and Massab (2014)).

2.2 The Ishmi basin: recent flood events

The Ishmi basin is spreaded along the Tirana and the Durres counties, with an area of 688 km^2. Four tributaries lead to the Ishmi river: the Zeza, the Terkuza, the Lana, the Tirana and the Limuthi rivers (Figure 5). According to Otoman chronicles and more recent observations, the largest flood events in the past 150 years have occurred in 1854, 1905, 1937, 1962-63 and 1970-71, but there is only complete records about the peak discharges in the latest two events (Bogdani and Selenica (1997)). The flood in 1962-63 is the third with the largest inundations. The duration of the event was eight days, in which 5,000 ha were flooded. The peak discharge during this event achieved the 1,500 m^3/s.

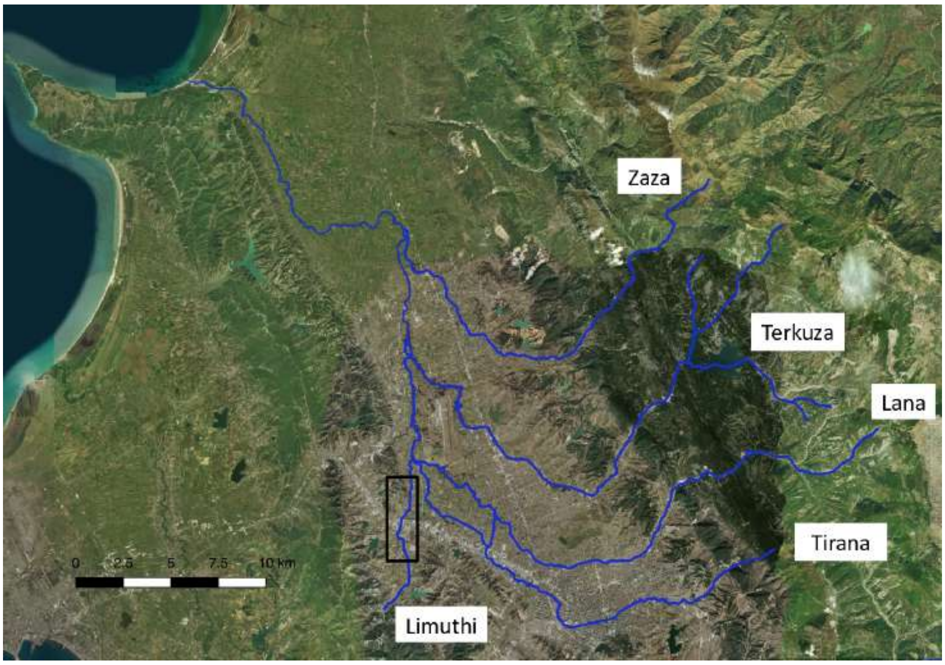

Figure 5: Ishmi basin. In black, the area of study, which was seriously damaged during the floods in December 2017.

After this severe event, the Albanian government reconstructed the breached embankments by the flood and constructed new levees along the Ishmi basin, which were supposedly designed to protect the adjacent areas for 50-year return period flood events. The confidence in the protection provided by the earthen levees fostered the economic development of areas nearby the river. This fact increased the value of the land and, consequently, the potential damage in case of flooding. The embankments reduced the damage caused by the flood events that were yet to come, but turned to be insufficient to guarantee an acceptable damage.

The lack of monitoring of the river bed and the accumulation of debris were unattended aspects, that would lead to fatal floods in the future. Additionally, informal constructions started to appear in the watershed, even blocking partially the conveyance cross section of the rivers. This situation was observed in several occasions in the Limuthi basin during the field trip to Tirana in April 2018. Due to the strong erosion of the watershed during the last large flood event, a larger cross section was created. Since the discharge in normal conditions of the Limuthi is relatively low, part of the cross section of the river becomes dry. This ares are used by some individuals to build unauthorized constructions.

The most recent overflow episode in the Ishmi basin with significant damage occurred in December 2017, when the Lana, the Tirana and the Limuthi river flooded an extensive urban area of the capital Tirana and other areas of the Municipality. The land uses affected were mainly residential and commercial in the more centric areas, while in the rest of the locations were industries and agricultural land. During this event, maximum rainfall levels of 121 mm in 24 hours were recorded in Tirana, which corresponds to a 10-year return period precipitation event (Davies (2017)). The Albania City Park (Figure 6), a shopping mall built in 2011, was flooded and 100 people needed to be rescued (?). Furthermore, the connections by road to the airport and the port of Durres were interrupted, isolating not only the population but also hindering the transport of goods (Figure 5). Hence, not only direct damage was provoked in this event, but also indirect damage.

The flooding was spread over 15,000 hectares (not only the Ishmi basin was affected) and the following overall damage was reported: 4,715 buildings, 41 businesses, 127 road sections, 177 schools, 78 bridges, 30 water supply stations, 11 dams, 26 electrical stations, 29 dikes and one water pumping station (Davies (2017)).

Figure 6: A tunnel is being flooded in the surroundings of the Albania City Park.

2.3 System description and backgrounds

The Limuthi basin is located to the West of the Municipality of Tirana. It is formed by three streams that born at two upstream lakes, the Gjokes and the Kusit lakes. These streams meet in one main river, the Limuthi. The surface area of the Kusit and the Gjokes lakes is approximately 28 ha and 29 ha, respectively. The reservoirs do not count with water retention systems, like dams. Therefore, the discharge is not regulated. The lakes are surrounded by earthen dikes with mild slopes and covered with mixed vegetation (Figure 7).

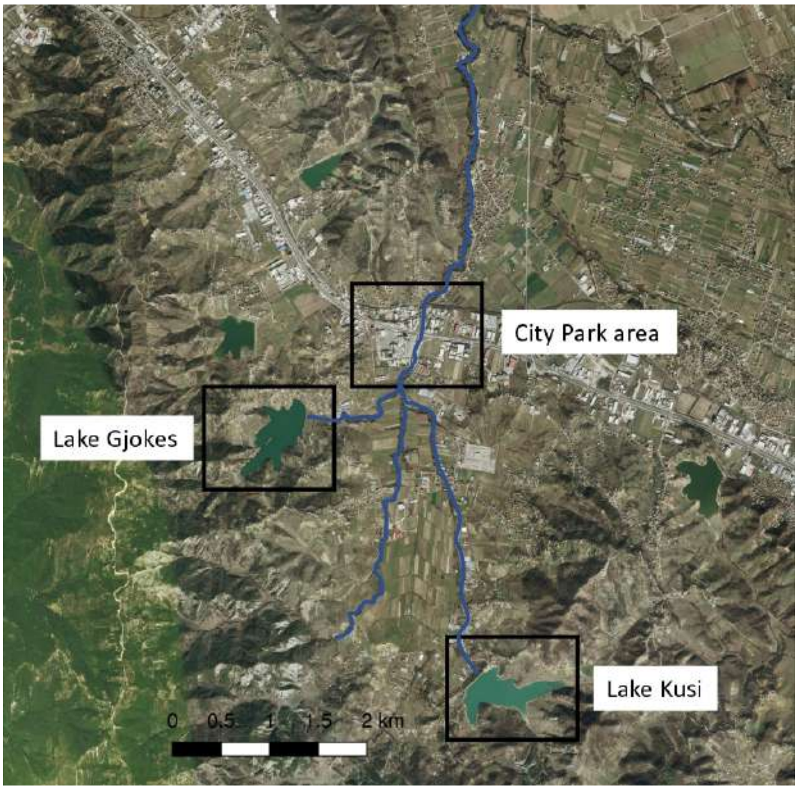

Figure 7: Sources of water of the Limuthi river.

The discharge of the Limuthi river is very seasonal. During summer, the Limuthi river remains almost dry, while during winter the occurrence of flash floods due to heavy rains is common. One of the main reasons for the occurrence of floods is the convergence of the three streams, as previously mentioned.

Concerning the structural features of the watershed, issues were detected in the design of the culverts at the City Park section. They seemed to have insufficient capacity. The flood of December 2017 blew up the soil cover of the tunnel through which the water used to flow. The tunnel has not been repaired, leaving the stream uncovered and the parking lot of the City Park is partially out of service due to the blow-up. The parking lot and the riverside remains in a very deficient state, with parts of several reinforced concrete structures and geotextiles visible.

Besides, some culverts carrying water from other areas irrupted in the main discharge, reducing the cross section of the river. The concrete structures went almost across the whole cross section of the river, in almost perpendicular direction. On top of that, the lack of slope stability enhances the erosion of the banks. The soil material eroded is accumulated at some downstream sections, increasing the local river bed level and reducing the effective cross section of the river.

During the flood in December 2017, the flood depth achieved 1.5 m up from the parking lot level. Marks of the water level in the façade of the Aquapark and the mall were visible. The Aquapark was being repaired during the field trip, and a new building was being constructed adjacent to it, invading to some extent the watershed. Informal buildings are located at several points in the watershed, hindering the water flow and increasing the risk of flooding.

Finally, human waste is present in the flow and in the riversides, what denotes unawareness of the population on the effects these debris can induce in the flood risk due to the pile up of waste in certain locations, what hinders the water flow.

3 Design definition

3.1 Problem statement

The economic development of the City Park area due to its strategical position has increased the flood risk in the Limuthi basin. Additionally, the lack of adequate flood management and river maintenance and monitoring makes the area very vulnerable to flash floods. The underestimation of the losses due to a flood lead to lack of investment in protection measures against flooding and maintenance of the watersheds. The consequences of flooding are direct damage, spread over residential, commercial, industrial and agricultural land, but also indirect damage, due to the interruptions in crucial road routes, such as the connection between the center of Tirana to the airport and to the port of Durres.

3.2 Design objective

The objective of this project is the evaluation of the feasibility of the different interventions that could be implemented in the Limuthi basin to reduce the risk of flooding. Furthermore, the integration of the solution to the local environment is crucial and should be included in the design, as long as the budget enables so.

3.3 List of requirements

The involvement of the different institutional stakeholders in the project is crucial to achieve an agreed solution by all the parties. Due to the impossibility to carry out interviews and meetings with the stakeholders in the area of study, it is taken as a reference case a report that gathers the social impacts of the flood events of February 2015 in the southwestern Albania (UNDP (2016)). The results of the stakeholder analysis provided by the mentioned reportare assumed analogous to the results that could be obtained if a stakeholder analysis was conducted for the current project, due to the proximity of the locations and the dates. The stakeholders participating in the emergency management were:

- Prefectures
- Quarks
- Municipalities
- Local residents
- City Park private investors
- Fire service
- Water and drainage boards
- Regional agriculture directorates

The analysis of the stakeholders developed in UNDP (2016) led to the following requirements:

- Full compensation of damages caused by floods and more incentives for the development of agricultural areas
- Construction and strengthening of embankments and other protective works
- Consistent maintenance and cleaning of collectors and drainage network
- Monitoring of river beds and operation of dams in catchments
- Maintenance of pumping stations
- Improvement of the notification system
- Creation of awareness among the public

- Provision of electricity supply
- Detailing the plan of civil emergencies

However, the accomplishment of these requirements is limited by three main constraints. First, the financial resources to execute these interventions are limited. The cost-effectiveness of each intervention must be analyzed in order to determine a rational distribution of the available capital resources among the proposals. In this project, only concepts related to the execution of flood defences are considered. It is assumed that the implementation of hydraulic structures is a highly cost-effective measure to prevent flood risk and therefore that this type of interventions are preferential.

The second constraint is related to the available space to construct in the area of study. Due to the presence of developed land in the surroundings of the Limuthi river, the space for the construction of flood defences is limited. Therefore, the final design should consider this aspect. The third constraint is related to the structural capacity of the retaining structures. The final design should withstand the high water levels that result from a 100-year return period flood event.

3.4 Boundary conditions

The flood analysis should be framed inside the following boundary conditions:

- **River stretch of study**: to set up the hydraulic model, a delimited stretch of the river has been considered. The main interest of the study are the economic consequences on the most valuable lands, which are assumed to be residential, public and commercial buildings, and the road infrastructure. The mentioned land uses are located in the surroundings of the City Park. The length of the stretch is 3.33 km (Figure 8).

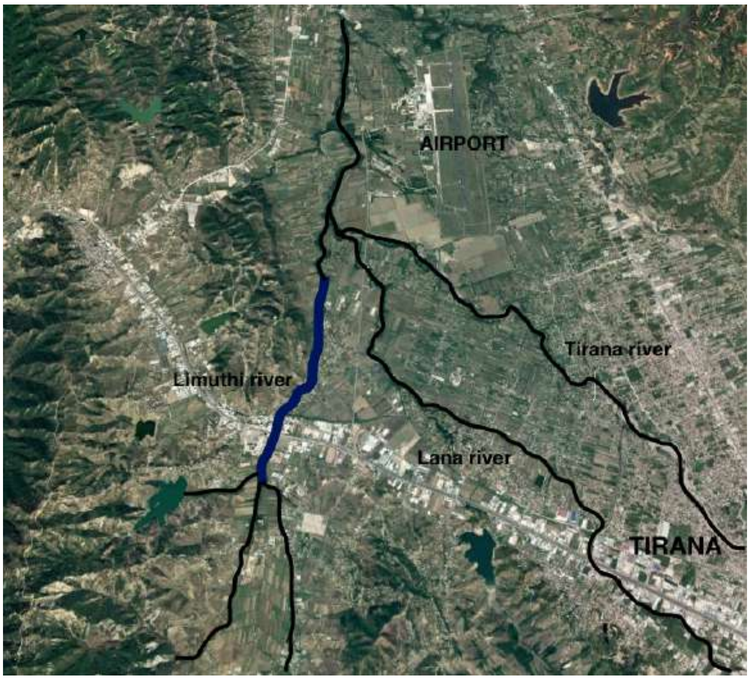

Figure 8: Stretch of the river considered (thicker blue line).

According to Google Earth, the elevation at the upstream reach of the stretch is 54 m and at the end of the stretch is 38 m. Hence, the average slope of the bed i_b is:

$$i_b = \frac{\Delta H}{\Delta L} = \frac{54-38}{3300} = 485 \ cm/km \tag{1}$$

- **Ground elevation**. A Digital Elevation Model (DEM) is necessary to develop the flood model (Figure 9). A DEM is a raster that includes the elevation of the terrain for every pixel. The trajectory of the water follows the direction of maximum slope. The DEM used has been downloaded from the Earth Explorer portal of the United States Geological Survey (USGS (2000)) and is provided by the Shuttle Radar Topography Mission (SRTM). It has a resolution of 30 meters × 30 meters.

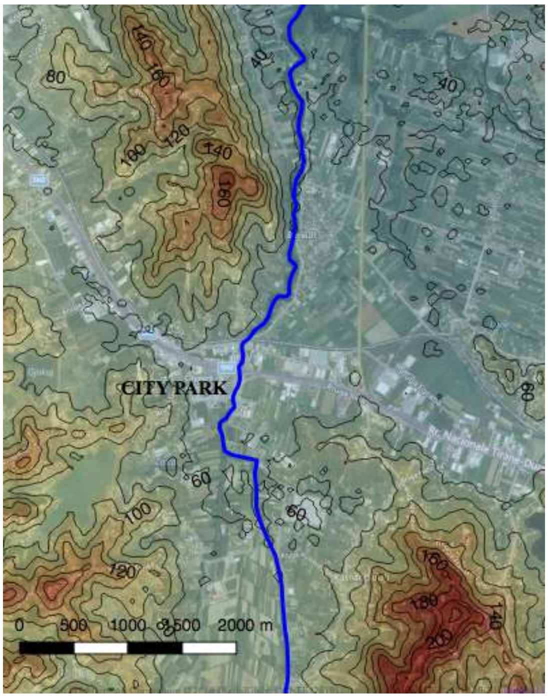

Figure 9: DEM of the region of study (altitude in meters).

- **Discharge of the Limuthi river**: data sets that include the discharge of the Limuthi river are not available. However, it is available the maximum monthly discharge of the Ishmi river for the period 1951-1992, at the Gjola station Hidrometeorologjik (1985). The Limuthi river is one of the four tributaries of the Ishmi river. Since it is desired to obtain a design discharge related to a certain return period, a probabilistic analysis of the available data is carried out.

  First, the histogram of the discharges is built (Figure 10). By means of the histogram, it is possible to make a first guess of which distribution of the maxima can fit the monthly maximum discharge.

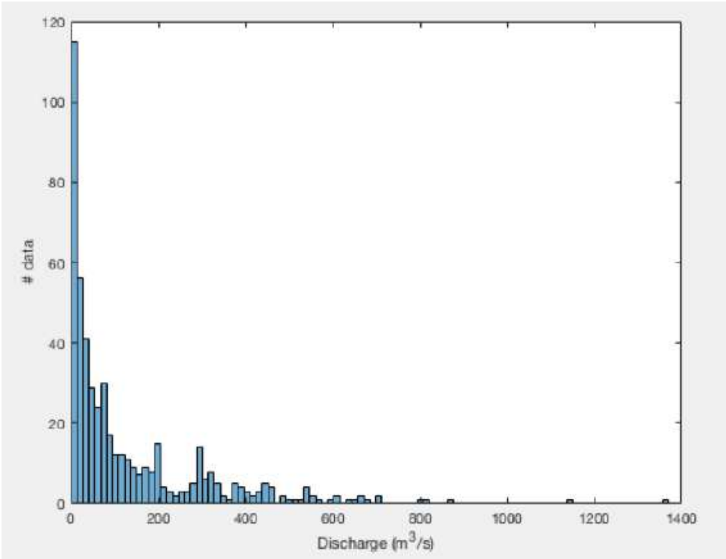

Figure 10: Histogram of the monthly maximum discharge of the Ishmi river at the Gjola hydrometric station (1951-1992).

As observed in Figure 10, the histogram shows positive skewness. Therefore, it is reasonable to fit the distribution of the monthly maximum discharges to a Gumbel distribution. Consequently, the distribution of the maximum annual discharges will also be Gumbel distributed, but shifted to larger values of the discharge (Morales-Nápoles (2018)). The cumulative distribution function (CDF) of the annual maxima discharge is expressed by:

$$F_{Gumbel}(Q) = exp(-exp(-\frac{Q-\mu}{\beta})) \tag{2}$$

where μ is the location parameter and β is the scale parameter. From the data, the empirical CDF can be obtained. Subsequently, the Gumbel distribution is fitted to the data by two methods (method of moments and maximum likelihood). The complementary probability of the Gumbel distribution (probability of exceedance) is shown below (Figure 11).

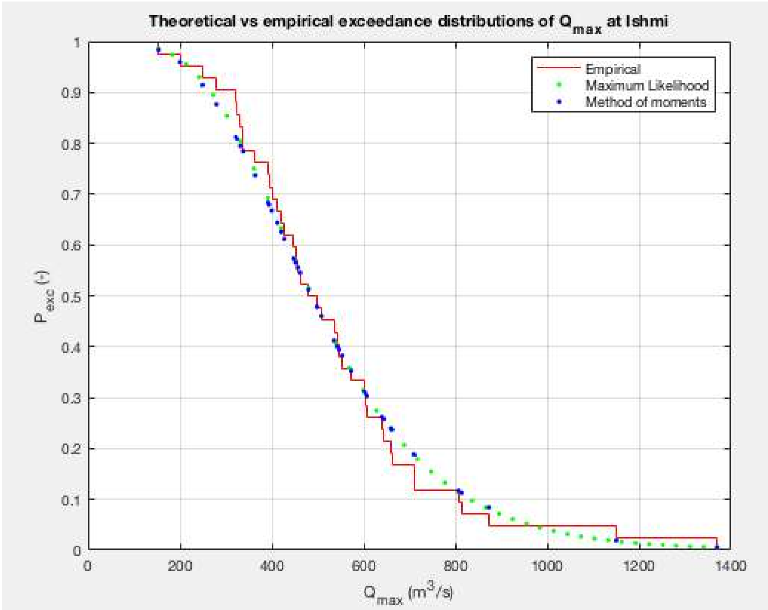

Figure 11: Probability of exceedance in function of the annual maximum discharge in the Ishmi river. Comparison of the empirical and the fitted distributions.

To assess which of the methods fits better the empirical distribution, the sum of squared errors in the probabilities for both methods is calculated. This results in 2.03 for the maximum likelihood and 0.04 for the method of moments. Hence, as an overall, the method of moments fits best the empirical distribution and this method will be used in the extrapolation of the annual discharges. The parameters of the fitted Gumbel distribution are $\mu = 418 \; m^3/s$ and $\beta = 187 \; m^3/s$.

Now it is possible to carry out an extrapolation of the fitted function, in order to obtain the discharge for a certain return period. The return period is defined as:

$$T_r = \frac{1}{f} \tag{3}$$

where f is the frequency of occurrence of the flood event. The probability of exceedance (P_{exc}) of a certain value of the discharge Q is:

$$P_{exc}(Q) = P(\underline{Q} > Q) = 1 - exp(-exp(-\frac{Q-\mu}{\beta}n)) \tag{4}$$

where n is the number of discrete years in which a certain Q is not exceeded. Since the interest lies in the design discharge that is not exceeded every single year, $n = 1$ in this case. On the other hand, for frequencies of occurrence smaller than 0.1, equation (4) converges to (Voorendt and Molenaar (2016)):

$$P_{exc}(Q) = 1 - exp(-fn) \approx f \tag{5}$$

Extrapolating the set of data to larger values of by means of the Gumbel fitted distribution, it is possible to obtain smaller probabilities of exceedance (larger return periods) related to larger discharges (Figure 12):

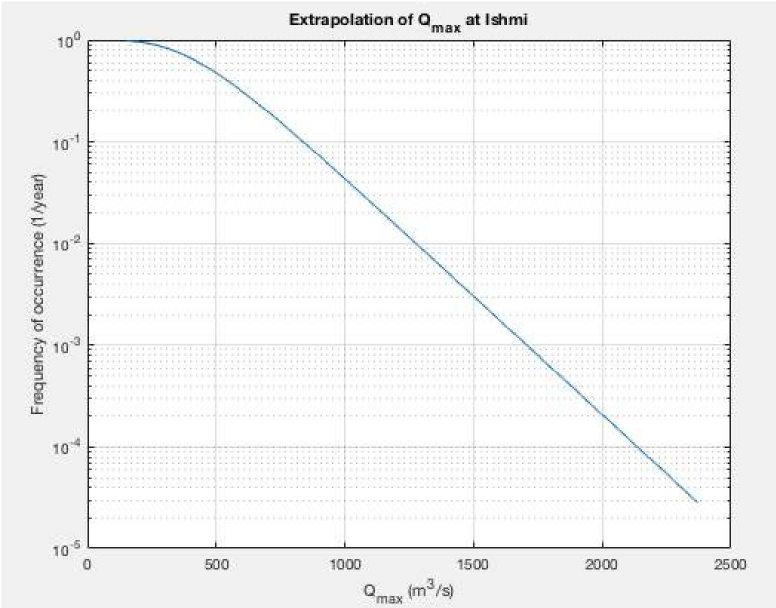

Figure 12: Frequency of occurrence ($1/T_r$) vs monthly maximum discharge (extrapolated function).

Assuming that the discharge and the precipitation are fully correlated, the discharge of the Ishmi river during the flash floods of December 2017 corresponded to a 10-year return period (the precipitation in the night of the 1st December 2017 was 121 mm in 24 h (Davies (2017)), which corresponds to a return period of 10 years, according to the data recorded in the Kamza pluviometric station (Hidrometeorologjik (1985)). Based on surveys made to the Albanian students that participated in the Workshops in Tirana, a return period of the event of 100 years is considered acceptable. Hence:

$$Q_{Ishmi,100yr} = 1276 \ \ m^3/s \tag{6}$$

Due to the lack of data about the discharges in the tributaries Lana, Tirana, Terkuza and Limuthi, it is assumed that their 100-year return period discharges are proportional to their watershed areas (Figure 13). The discharge recorded at the Gjola station is the sum of the four discharges.

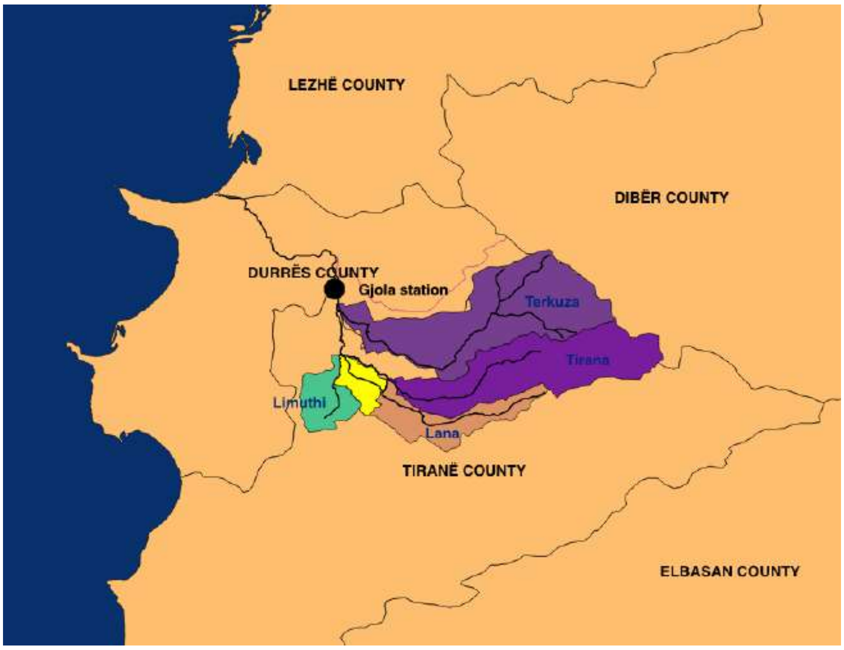

Figure 13: Watersheds of the rivers Terkuza, Tirana, Lana and Limuthi.

The sum of the watershed areas of every tributary is 421.419 km^2. The area of the Limuthi watershed is 31.585 km^2. Consequently, the Limuthi watershed area represents a 7.49 % of the total watershed. Therefore, the design discharge for the Limuthi river is:

$$Q_{Limuthi,100yr} = Q_{hr} = 0.0749 Q_{Ishmi} = 96 \ m^3/s \tag{7}$$

This discharge of design includes the runoff due to precipitation and it will also be called Q_{hr} (discharge during heavy rain).

- **Precipitation**: although the precipitation is included in the discharge of design for the Limuthi river, it is important to determine how it develops in time. To do so, the weather conditions of the last flood event in December 2017 are reproduced (Time and date (2017)). The period analyzed runs from the 29th November at 12 AM until the 5th December at 10 AM. Due to the relatively small area of the Limuthi watershed, it is reasonable to assume the same precipitation in its entire extension. Part of the precipitation that falls into the watershed areas of the rivers will run off into the river streams eventually. According to Time and date (2016), only one third of the precipitation finishes in the streams, while the other two thirds of the volume is evaporated or infiltrated into the subsoil.

Four levels of precipitation have been assumed, based on the floods of December 2017:

1. **No rain**: this level considers no precipitation.
2. **Light rain**: it is assumed that the precipitation associated to this level corresponds to the historical average precipitation in the month of December, which is 175 $mm/month$ (which is equivalent to 175 $L/m^2/month$ (Time and date (2017))).
3. **Rain or thundershowers**: this level assumes a 10-year return period precipitation, which is 128 mm in 24 h (Hidrometeorologjik (1985)).
4. **Heavy rain**: for this level it is considered a 100-year return period precipitation, which is 193 mm in 24 hours (Hidrometeorologjik (1985)).

The runoff that ends into the Limuthi stream (Q_{run}) during heavy rain is computed as follows:

$$Q_{run} = \frac{A_{wsh} Pr}{3} = 24 \ \ m^3/s \tag{8}$$

where A_{wsh} is the area of the watershed of the Limuthi river and Pr is the precipitation. However, we are analyzing a stretch of the Limuthi river that is upstream from the confluence with the other rivers. Hence, the runoff that enters through the upstream reach of the stretch of study is a fraction of total that represents the watershed area located upstream from the river stretch of study over the total area of the Limuthi watershed:

$$Q_{run,s} = \frac{A_{wsh,u}}{A_{wsh}} Q_{run} = 18 \ \ m^3/s \tag{9}$$

where $A_{wsh,u}$ is the area of the watershed that extends upstream from the river reach. By using spatial tools it has been obtained $A_{wsh,u} = 24.099 \ \ km^2$. If the additional river discharge due to precipitation is known, it is possible to calculate the discharge in case there is no rain (Q_{nr}):

$$Q_{nr} = Q_{hr} - Q_{run} = 78 \ \ m^3/s \tag{10}$$

A summary of the discharges in the Limuthi during the different levels of precipitation is provided in Table 1. The values exposed are introduced in the HEC-RAS model as upstream boundary condition in the stretch of study to simulate the flood event of design. The storm in December 2019 is taken as a reference to establish the development in time of the storm. The precipitation levels are set with a frequency of one hour and the resultant discharge is introduced in HEC-RAS accordingly.

Rain level	**Precipitation [L/m^2/s]**	**Run-off [m^3/s]**	**$Q_{Limuthi}$ [m^3/s]**
No rain	0	0	78
Light rain	6.75×10^{-5}	1	79
Rain / Thundershowers	1.48×10^{-3}	12	90
Heavy Rain	2.23×10^{-3}	18	96

Table 1: Summary of the discharges due to the different levels of precipitation in the Limuthi watershed.

The boundary condition downstream is the normal depth, which is automatically calculated by the program, given the friction slope (i_b).

- **Land use map**: this map includes the different land uses present in the area of study (Figure 14). This map has been obtained from the European Environment Agency Copernicus (2017). The land uses of the area of study are:
  - Discontinuous urban fabric
  - Industrial or commercial
  - Mineral extraction site
  - Agricultural - vineyards
  - Agricultural - pastures
  - Agricultural - complex cultivate patterns

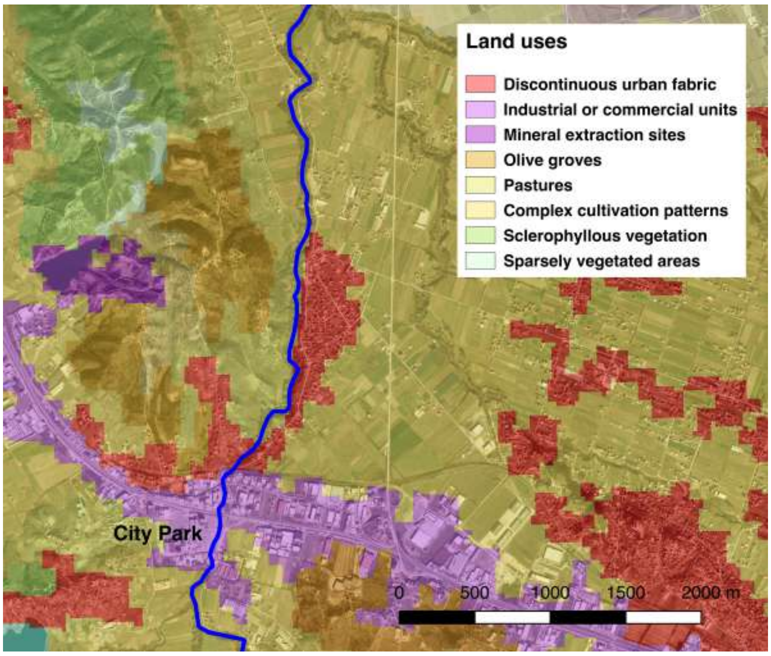

Figure 14: Map of land uses of the area of study. This raster has been aligned to the original DEM.

- **Damage curves**: the damage curves indicate the monetized damage in case of flooding for every land use, in function of the inundation depth. These curves are not available specifically for Albania. Instead, the mean European values based on Huizinga et al. (2017) are taken. These curves are provided by the Joint Research Center (JRC), an organism dependent on the European Commission whose objective is creating guidelines for the policy-making related to flooding in Europe.

  The curves available are related to industrial, commercial, residential, infrastructure and agricultural areas. Therefore, the land uses exposed before have been regrouped in three main categories, in order to be able to use the values given by the damage curves to evaluate the risk of flooding:

  - The discontinuous urban fabric is evaluated according to the residential damage curve.
  - The industrial/commercial areas and the mineral extraction sites are grouped, and are evaluated according to the commercial damage curve. The maximum damage is larger for the commercial areas than for the industrial areas, and the damage factor is also larger (Figure 15). Since there is no distinction between commercial and industrial uses, the damage values for the commercial are assumed for both land uses. This assumption leads to a conservative estimation of the damage in these areas.
  - The different types of agricultural and vegetation areas shown in Figure 14 are evaluated according to the agricultural damage curve.

  The curves corresponding to the different land uses are shown below (Figure 15).

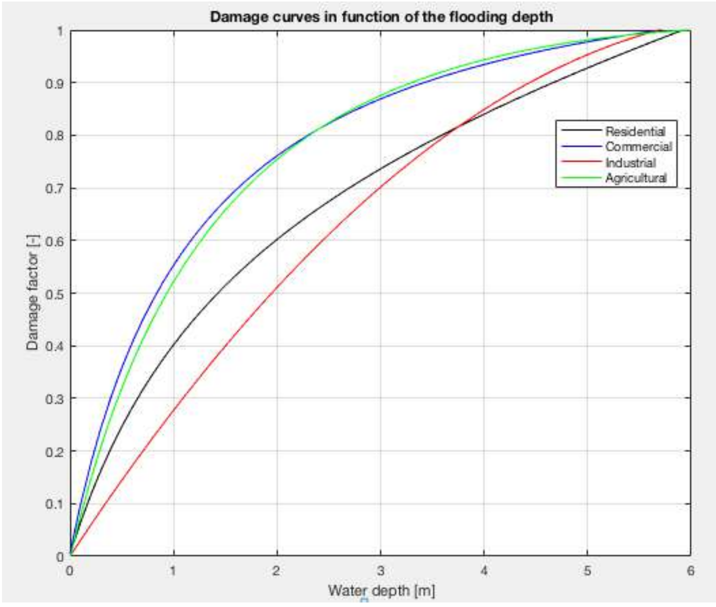

Figure 15: Normalized damage factor for Europe, in function of the flood depth, for industrial, commercial and agricultural areas.

In these curves, the damage factor is a function of the flooding depth. This value is comprised between 0 and 1, being 0 the case in which there is no flooding and 1 the situation of full direct damage. The full direct damage is assumed to occur for more than 6 m of flood depth. The curves have been fitted according to the discrete values given in Huizinga et al. (2017). The fitted curves are:

- **Residential**: it includes content damage.

$$f_r(d) = \frac{0.05109\ d^2 + 0.9519\ d + 0.006111}{d + 1.511} \tag{11}$$

- **Industrial**: it applies to an overall industrial area.

$$f_i(d) = \frac{-167.8\ d^2 + 2323\ d + 2.102}{d + 7780} \tag{12}$$

- **Commercial**: it includes content and inventory damage.

$$f_c(d) = \frac{-0.004135\ d^2 + 1.241\ d + 0.006376}{d + 1.248} \tag{13}$$

- **Agricultural**: based only on the flood depth.

$$f_a(d) = \frac{-0.0349\ d^2 + 1.515\ d - 0.006757}{d + 1.824} \tag{14}$$

Every land use has associated a maximum damage per square meter, as shown in Table 2.

Land use	$\mathbf{D_{max}}$ [€/$\mathbf{m^2}$]
Residential	750
Industrial	534
Commercial	621
Agricultural	0.77

Table 2: Maximum cost of the damage per land use (6 m of flooding depth).

The figures exposed above give room for a critical reflection. Several aspects must be discussed:

- First, the damage curve relative to the agricultural land presents a similar behavior than the commercial land. However, it is sensible that the difference between a flood depth of 2 or 3 m for a certain crop would not lead to very distinct damage values (actually, the damage must be already very large for this flood depths). Therefore, seems reasonable that the damage curve for the agricultural land would present a larger asymptotic behavior to $f_a = 1$, with already large damage factors for low flood depths.
- Second, the damage per hectare of agricultural land for a $f_a = 1$ ascends to 7700 €, which is a possible value for a crop of potatoes. Generally, other types of crops, such as corn, cereals or grass, have an approximate price per ha of 1500 €. Therefore, the difference in price per ha is very large, what can lead to overestimation of the damage in the agricultural land if a maximum damage of 0.77 €/ha is taken.
- Third, the maximum damage per square meter for the residential areas differs from the price proposed in Huizinga et al. (2017), although not so much for the case of Albania. According to Numbeo (2018), the price per square meter outside of the center of Tirana is approximately 650 €, which is approximately 85% lower than the price in the outskirts Amsterdam (4655 €$/m^2$). Probably, the low price per square meter is due to the consideration that only a certain area of the private properties corresponds to the residential building itself, what leads to a reduction of the damage per squared meter. In this case, if 750 €$/m^2$ is taken as the cost per square meter, the calculation would lead to an overestimation of the damage.

4 Design methodology

In order to achieve a successful outcome, it is fundamental to establish in the first place a methodology that gives an overview of the steps to follow towards the final design. With this strategy, the steps in the process are identified, what makes possible to set clear objectives in each phase of the project. The methodology is shown in a flow chart format in Figure 16.

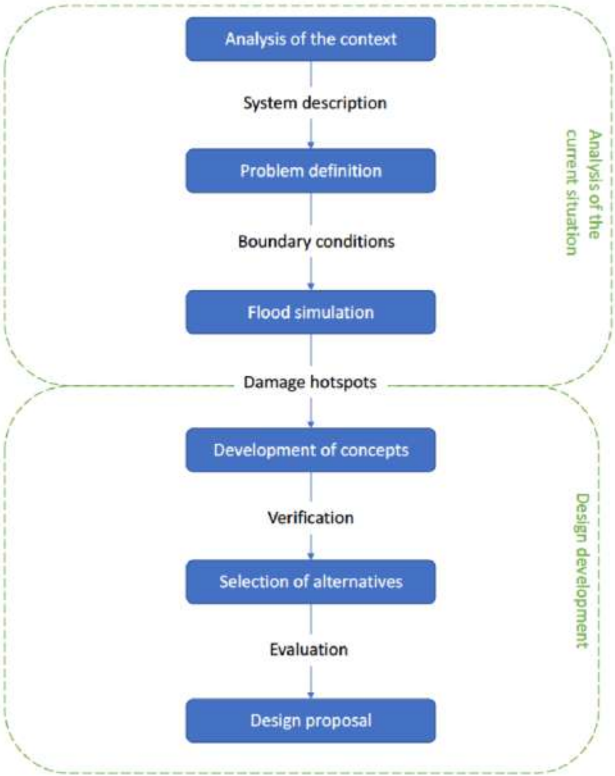

Figure 16: Methodology to achieve the final design.

The project starts by analyzing the current situation. First, the project location is investigated, from a large scale (broad analysis of the flash floods in Albania) to the small scale (description of the Ishmi basin and more specifically, the Limuthi basin). Based on the background of the Limuthi basin and the information related to the recent flood events, the problem definition is elaborated. Then, by identifying the boundary conditions of the project, an hydrodynamic model is set up and a flood event can be simulated, according to certain design conditions. The outcome of the simulation provides the geographical distribution of the damage, where the most affected areas are observed. These locations are called *hotspots* and they will be preferential targets to be protected.

The second part of the project is the development of a design to protect the *hotspots* from flooding. The first stage in the design is the development of concepts. To be considered alternatives, the concepts must satisfy a list of requirements. The list of requirements includes a summary of the demands of the project stakeholders, and structural and spatial requirements. This step in the process is called verification of concepts. The verified concepts are considered design alternatives, which are evaluated according to financial criteria. Finally, the most feasible alternative is selected and the design proposal is elaborated.

5 Analysis

5.1 Model setup

To be able to determine the locations that need preferential protection, it is required to analyze the effects of a flood of design in the current situation. To do so, an hydrodynamic model (HEC-RAS) is set up. The inputs of the model are:

- **Terrain**: first, the DEM of the area of study is implemented in HEC-RAS. The DEM has been pre-processed in QGIS to align the input rasters (land use map and DEM) and convert them to UTM projection. Accordingly, the output given by HEC-RAS (inundation map) will also be aligned with the inputs.

  The original raster has been downloaded from USGS (2000) and it is in WGS 1984 / Pseudomercator projection. To obtain a correct alignment when implementing the map in the HEC-RAS terrain interface, it is necessary that the raster is in a 2D projection. Otherwise, the distances will be modified in the process, what would lead to an incorrect outcome. To do so, first the original raster is transformed into a map of points (vector file), in which the value of every pixel is given to a point located at the center of each pixel of the raster. Then, the map of points is transformed from longitude/latitude coordinates (degrees) to UTM coordinates (meters). Finally, the map of points is converted into a raster, whose pixels have a cell size of $30 \times 30m$. Now, the processed map can be implemented in the terrain interface of HEC-RAS.

- **Geometry**: subsequently, it is delimited a spatial domain that might include the area that is expected to be flooded. Generally, this is fixed by checking the results of several model runs. If the model outcome shows that the edges are flooded, the inundation might reach an area beyond the initially established limits. Therefore, it is necessary to extend the flooding area.

  Then, a calculation mesh must be generated. At this point, a trade-off problem between level of detail of the outcome and computational time must be addressed. Since the program reaches the solution by means of an explicit time integration numerical scheme, the solution can be unstable. To obtain a stable result, the Courant-Friedrichs-Lewy (CFL) condition must be satisfied (Zijlema (2018)):

  $$\frac{\Delta t}{\Delta x}\; c_{prop} \leqslant 1 \tag{15}$$

  where Δt and Δx are the time and space steps, respectively. c_{prop} is the velocity of propagation of the solution. As it can be derived from 15, the larger the spatial level of detail of the solution (the smaller the space step), the smaller the time step required to satisfy the stability condition. This leads to a larger computation time.

  Several assumptions have been made in order to determine the time step required to achieve a stable solution:

  - The cross section of the river is assumed to be trapezoidal and prismatic, with an average width (B) of 12 meters (Google Earth). No flood plains are present in the stretch of the river analyzed. Therefore, the water flows only through a main stream.
  - Uniform flow conditions are assumed. Therefore, the Chézy formula is applicable. On the other hand, the friction coefficient for the coarse sand (c_f) is assumed to be 0.006 (Blom (2016b)). Therefore, the Chézy coefficient is:

    $$C = \sqrt{\frac{g}{c_f}} = 40 \;\; m^{1/2}/s \tag{16}$$

    However, the flow is not steady since the sudden presence of the flood wave leads to accelerations and decelerations in time. The water depth and the flow velocity are Battjes and Labeur (2014):

    $$d = \sqrt[3]{\frac{c_f \; Q_{hr}^2}{B^2 \; g \; i_b}} = 2 \;\; m \tag{17}$$

$$u = \frac{Q_{hr}}{B\ d} = 4\ \ m/s \tag{18}$$

- As stated in Amoroso (2016), the alluvial deposits in the river are formed, among others, by sands. Since also gravels and clays are present in the soil bed, it is considered an average uni-size sediment diameter of $D_{n50} = 0.9\ \ mm$, what corresponds to coarse sands.
- The Engelund-Hansen formulation is used to determine the sediment discharge (S). This formulation has been tested for values of the Shields parameter between 0.07 and 6 and when the nominal diameter is between 0.19 mm and 0.93 mm Blom (2016a). Due to the limited exceedance of the limit of these conditions (the calculation of the Shields parameter is shown some paragraphs below), the E-H is yet accepted to calculate the transport of sediment.
- The degree of non-linearity affecting c_{prop} (b) is assumed to have a constant value of 5 (Crosato (2016)).
- The density of the water (ρ_w) is 1000 kg/m^3 and the density of the sediment including pores (ρ_s) is 1855 kg/m^3 (Schiereck (2012)). A commonly used parameter that relates both densities is:

$$\Delta = \frac{\rho_s - \rho_w}{\rho_w} = 0.855 \tag{19}$$

- The 100-year return period is 96 m^3/s (Q_{hr}).

The velocity of propagation is defined as:

$$c_{prop} = \frac{b\ S}{Q_{hr}} \tag{20}$$

Therefore, the sediment discharge should be determined previously:

$$S = 0.05\ \sqrt{g\ \Delta\ D_{n50}^2}\ \frac{C^2}{g}\ \theta^{5/2}\ B \tag{21}$$

where θ is the Shields parameter:

$$\theta = \frac{u^2}{C^2\ \Delta\ D_{n50}} \tag{22}$$

Substituting values, it is obtained $\theta = 6.5$, $S = 136\ \ m^3/s$ and $c_{prop} = 7.1\ \ m/s$. It is assumed that a spatial step of 30 m will provide sufficient accuracy in the 2D flooding model, given that the DEM has also this cell size. Therefore, the required spatial step in the numerical computation to ensure a stable solution is, according to the CFL condition:

$$\Delta t \leq \frac{\Delta x}{c_{prop}} = 4.2\ \ s \tag{23}$$

- **Unsteady flow data**: in this step, the boundary conditions are imposed, assuming unsteady flow. As mentioned before, the upstream boundary condition is a discharge hydrograph and the downstream boundary condition is the normal depth, which is calculated automatically by the program, by just giving the friction slope.

5.2 Results of the simulation: flooded area for the *do nothing* alternative

In the figure below (Figure 17), the outcome of the hydrodynamic simulation is given. The output of the simulation is an inundation map of the area of study, supported by a raster format. It provides the inundation depth per pixel. On the far right side of the map, the inundation map shows a region with relatively large water depths. Aerial photographs show the presence of a river in that location. The ground elevation within the river trajectory is lower than in the adjacent locations. Therefore, since the water is conveyed in the direction of the maximum slope, the water runs off to the river.

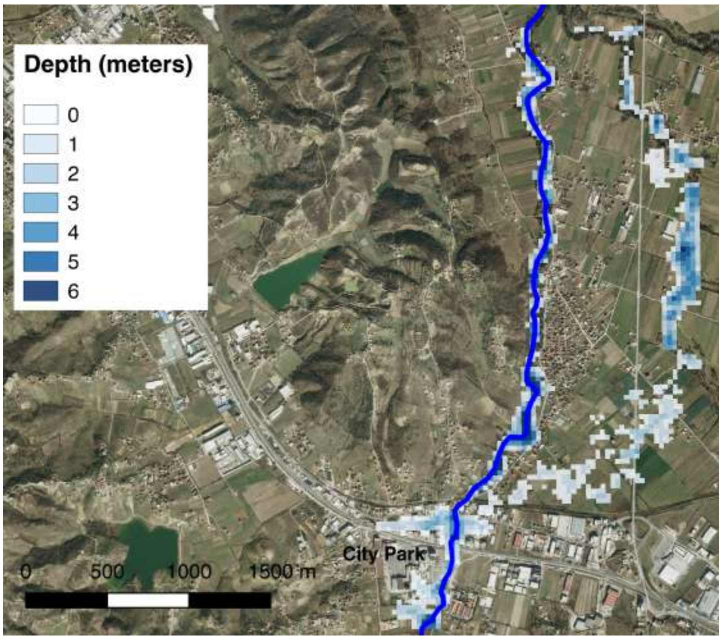

Figure 17: Flooded area for the *do nothing* alternative in the location of study.

The inundation map and the land use map are aligned to make their respective pixels coincide. The resolution of the maps is 1 arc-second, which is equivalent to approximately 30 meters. Hence, the total direct damage (D_{tot}) can be computed as:

$$D_{tot} = A_{pixel} \sum_{i=1}^{n_x} \sum_{j=1}^{n_y} D_{max,i,j} \; f_{i,j} \tag{24}$$

where:

A_{pixel}: area of one pixel ($30 \times 30 = 900 \ m^2$).
$D_{max,i,j}$: maximum damage corresponding to the land use in the row i and column j of the raster.
$f_{i,j}$: damage factor corresponding to the row i and column j of the raster. As previously mentioned, the value of the damage factor is a function of the depth in every pixel.

According to the simulation, the total direct damage in case a flood event of a 100-year return period occurs ascends to **90.4 million euro**.

It is remarked that the calculated damage just includes the direct effects on the area. Apart from the direct damage, the flood event would also cause indirect damage, for example as a result of the interruption of the traffic in the roads (interruption of the connections to the airport and the port of Durres). However, in this study only the direct effects have been considered.

In order to determine rationally where to intervene, an analysis of the geographical distribution of the damage is carried out (Figure 18).

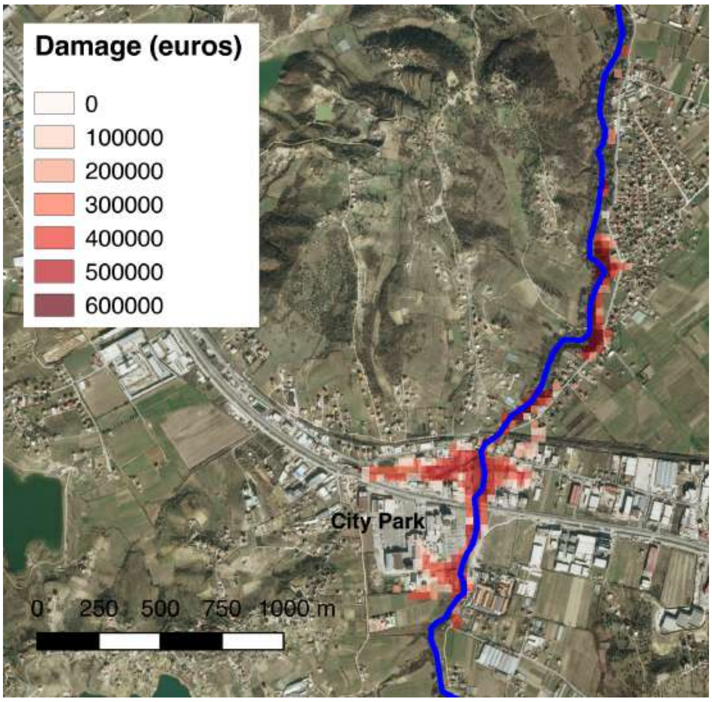

Figure 18: Damage (euros) produced in the area of study, given the flood event of design.

As observed, the City Park and the area right at the North of the highway are the *hotspots*, or areas where larger monetized damage is produced. The damage in this region ascends to 63 million euros, which represents the 70% of the total damage. Therefore, it is possible to reduce considerably the damage by protecting these areas, which are commercial in their majority. Also, it is fundamental to protect the highways and roads, to ensure the normal functioning of the network. This will mitigate to a great extent the indirect damage caused by the flood.

6 Development of concepts

In the previous section, the map of the geographical distribution of the damage enabled the identification of the areas which are subjected to larger flood risk (*hotspots*). In the present section, several concepts are proposed to reduce the (material) flood risk.

As explained further in this report (section 8), two types of strategies can be developed to reduce the flood risk. The first type of strategies aims for the reduction of the probability of failure of the system, which in this project is achieved by implementing a new structure or strengthening an existing one. The second type of strategies seeks the reduction of the damage due to flooding. A common plan of action to achieve so is the adaptation of the current spatial planning.

6.1 Synthesis of concepts

This report focuses on the evaluation of the performance of hydraulic engineering solutions. Therefore, the concepts developed are mainly related to the implementation of flood defences and barriers. However, in section 10 recommendations regarding the rest of the measures proposed by the stakeholders in subsection 3.3 are given.

According to Jonkman et al. (2018), several types of structures can be implemented to prevent fluvial flooding:

- **Dikes**: a dike is a water retaining structure consisting of soil with a sufficient elevation and strength to be able to retain the water under extreme circumstances (Figure 19).

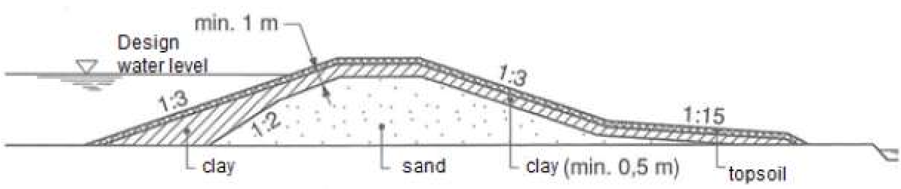

Figure 19: Example of the profile of a river dike (Leusden and van der Velden (1998)).

As shown in Figure 19, in general the core of the dike consists of sand and a thin layer of clay is applied on top of it. The clay guarantees the water tightness, but it deforms when it gets wet. On the other hand, the sand is relatively stable, but is not water tight. Therefore, a combination of both materials is generally used.

The inner part of the dike can include a berm, generally small than 1:3 to increase the stability during periods of high water and to prevent piping. However, sufficient space in the surroundings of the dike is required to apply the mentioned berm.

The type of dike revetment plays also an important role in the reduction of the run-up and the overtopping, but also in the integration of the dike in the local environment. Grass covers have been proved to increase mildly the roughness, but they induce visual amenity in the site. On the other hand, depending on the accessibility to certain materials, more effective revetments can be used. An example of an effective revetment is the rip-rap. However, the visual impact that this type of revetment creates is less desirable.

Moreover, dikes can have additional functions apart from protecting against flooding. Bike lanes and lookout points can be implemented on the dike crest, what enhances the recreational possibilities for the citizens of the area.

- **Flood walls**: water retaining structures generally made of concrete, but sometimes also made of steel (Figure 22). In order to resist the horizontal hydraulic loads, a shallow or deep (piled) foundation is necessary.

Figure 20: Flood wall in Machland (Austria).

According to Jonkman et al. (2013), the unit cost of flood walls are larger than the unit costs of earthen levees (5-9 million euro vs 1-5 million euro, per kilometer and per meter heightening, respectively). However, there are several reasons to select flood walls for the protection against floods. The first is that the footprint of the flood walls is much smaller. In case the space available for construction is limited, an earthen dike might not be applicable. The second reason is the lack of availability of material in the area to construct the dike.

- **Temporary flood defences**: these water retaining structures are deployed in case a flood event occurs. They are not permanently installed in a certain location. The performance of these structures is limited by their dimensions. Generally, temporary structures are effective for relatively shallow inundation depths (maximum 1 meter depth, in the case of BRIGAID solutions). In this case, a temporary structure called Slamdam is proposed (Slamdam (2019)), which is a barrier made of EPDM rubber that consists of two compartments, which are filled with water to guarantee the stability. The ProfilDam L model of this temporary structure is suggested to be used in this project, which retains water levels up to one meter and whose cross section is two meters wide.

Figure 21: Slamdam: temporary flood defence that is filled with water (Slamdam (2019)).

- **Dam**: a dam is a water retaining structure that separates two water bodies. The difference with a dike is that behind a dam water is located and behind a dike there is land. In Albania, these structures are extensively used for the production of hydro-power. Given the mountainous terrain of the country, large water jumps are formed in the river system which are used to transform the potential energy of the water into electric power. Apart from being suitable to produce clean energy, they are key structures to retain and regulate the discharge to downstream reaches of a river, by means of several gates. Hence, their flood prevention function is very relevant.

Figure 22: Banja hydro-power plant, located at the village of the same name (65 km southeast from Tirana).

- **Dredging of the river bed**. As observed during the field trip, the maintenance of the river is deficient. As a result, there are some spots in the trajectory of the river where sediment is accumulated, reducing locally the water depth. This fact increases the risk of overflow of the dikes in case a flood wave occurs. Additionally, the banks are eroded, what reduces the strength of the existing dikes. To increase the water conveyance section of the river, the bed of the river can be dredged. In order to increase the strength of the dikes, the slopes can be stabilized by installing a revetment.

7 Verification of concepts

The concepts developed in section 6 only become actual design alternatives if they satisfy the requirements of the stakeholders and they take into consideration the financial and spatial constraints (section 3.3).

According to the flood risk assessment of the current situation (Figure 18), the damage is concentrated in the industrial and commercial areas in the surroundings of the City Park (Figure 23). The affected areas downstream from the City Park represent a small share of the overall damage. Therefore, interventions in the downstream stretches of the Limuthi river are disregarded, for the sake of feasibility (Figure 24).

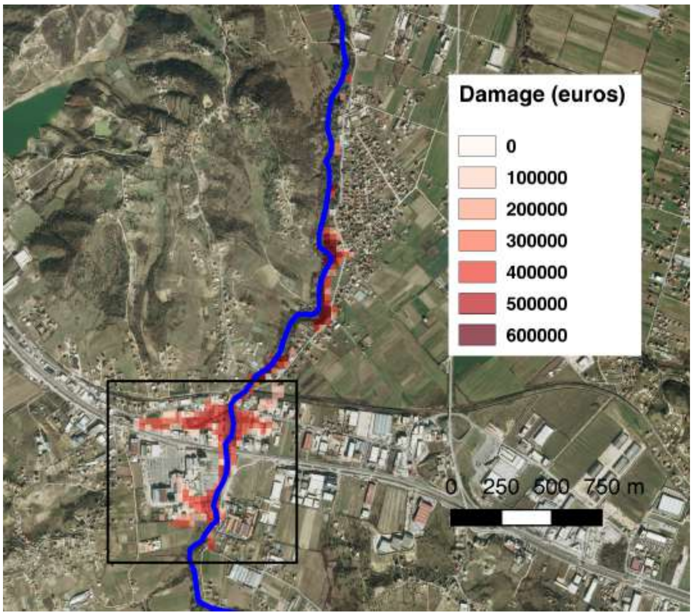

Figure 23: The City Park (inside the rectangle) is the hotspot of the damage in the Limuthi basin.

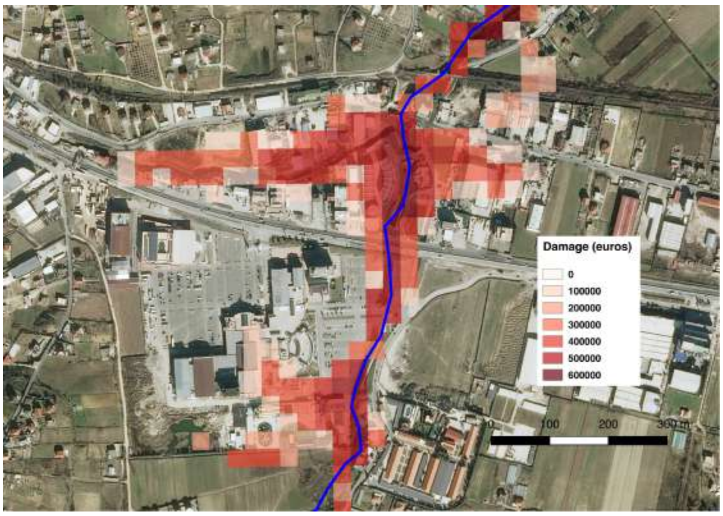

Figure 24: Target area to be protected from flooding.

The concepts proposed in section 6.1 are verified according to the constraints included in section 3.3:

- **Dikes**: the capital expenditures of the dikes are 1 million euros per kilometer long and per meter high (Jonkman et al. (2013)). The width of the dike increases proportionally to the height, given a certain width of the crest. Therefore, the space required to construct a dike can be relatively large. To guarantee the stability, the slope of the dikes should be at least 3 meters long per 1 meter high (1:3). In order to improve the livability of the City Park, a bike lane is implemented at the crest of the dike. To accommodate the bike lane, the width of the crest is 3 meters. Based on the inundation depths obtained from the simulation (section 5.2), a dike can be designed to withstand the water levels that result from a 100-year return period flood event.

- **Flood walls**: the capital expenditures of the flood walls are 7 million euros per kilometer long and per meter high (Jonkman et al. (2013)). The space required to build the flood walls is very small (e.g. 4 meters in the cross-sectional direction of the river, as mentioned in Jonkman et al. (2013)), compared to the dikes. Based on the inundation depths obtained from the simulation (section 5.2), a flood wall can be designed to withstand the water levels that result from a 100-year return period flood event.

- **Temporary flood defences**: the capital costs of the Slamdam equipment are 474 euros per meter length (Slamdam (2019)). The space required to deploy the Slamdam is two meters in the cross-sectional direction of the river. However, the maximum height of this temporary flood defence is fixed to one meter. Therefore, the Slamdam cannot withstand the water levels that result from a 100-year return period flood event by itself.

- **Dams**: the Limuthi river borns at the Gjokes and Kusi Lakes, whose surface is respectively $A_{lake} = 0.28\ km^2$. The City Park is right at the confluence of the two streams. Dams could be implemented in both lakes, in order to control the water discharge downstream and avoid flooding. However, due to the small dimensions of the lakes, extremely high dams and dikes would be necessary to retain a significant volume of water. To give a first impression of the required dam height, a fast calculation is carried out.

  Assuming that there is no rain, which is a conservative scenario (subsection 3.4), the discharge of the Limuthi river would be 78 m^3/s. This discharge would be continuous during the six days and five hours of event. As a first estimation, it is assumed that that $Q_{retained} = 24\ m^3/s$ are retained by each dam along this period, in order to reduce the discharge of the river at the City Park section to 30 m^3/s. Therefore, the total volume of water to store per dam is:

$$V_{total} = \frac{24\ m^3}{s} \times \frac{86400\ s}{1\ day} \times 6\ days = 12,873,600\ m^3 \tag{25}$$

  Assuming vertical walls at the perimeter of the lake, the height required to retain this volume of water is, at least:

$$h = \frac{V_{total}}{A_{lake}} = 45\ m \tag{26}$$

  The perimeter of every lake is approximately 3.7 km, which is approximately 1.5 times the required length of the dikes to protect the City Park area ($1.26 \times 2\ km$). Additionally, the required height of the dikes at the perimeter of the lake is much larger than the height of the dikes protecting the City Park. Therefore, the implementation of dams is disregarded due to the lack of feasibility of the design. Apart from that, the implementation of dams would have an unacceptable visual impact in the environment due to the vast dimensions of such an infrastructure.

- **Dredging of the river bed**: as shown in van der Horst (2018), the mobilization and demobilization costs of dredging are 500,000 euros. The running costs of the dredging works are 5 $euros/m^3$ and the lump sum costs of the bed and bank protection for the dikes (crushed stone) are 35 $euros/m^3$ for dry installation and 40 $euros/m^3$ for wet installation. It is assumed that dredging in the current river is possible up to two meters depth, since a larger depth would induce very quick morphodynamic changes in the river bed, which would oblige to carry out very frequent maintenance dredging activities.

Based on the analysis of the constraints, it is concluded that the dam is not a suitable alternative to prevent flooding in the City Park. The dredging of the channel is limited to two meters and the exclusive use of temporary flood defences is not effective for flood prevention. Furthermore, although being able to prevent flooding by itself, the implementation of dikes is limited by the space available, but it is more economic than the implementation of flood walls. According to this analysis, the concepts are combined in order to achieve feasible alternatives:

- **Combination of flood walls and dredging of the river bed**: the construction of flood walls is required in case developed land is located nearby the river. The design aims to minimize the relocation of developed land due to the execution of flood protection structures. The cost per kilometer long and per meter high is large compared to the construction of dikes, but in this case bank protection is not required to be implemented.

- **Combination of dikes, temporary flood defences and dredging of the river bed**: this solution is expected to be more economic than the flood walls and more environmental friendly. Apart from having a flood protection function, dikes can have a recreational function, due to the creation of cycle paths on their crest. Relocation of agricultural land is acceptable and, accordingly, dikes can be implemented in all those locations that are not used for commercial, industrial and residential purposes. Dredging activities are implemented also in this case up to two meter depth, in order to save 2 meters of heightening of the structure. Dikes also enable the implementation of temporary flood defences on top the crest, and therefore an additional meter of dike construction can be saved. The installation of a revetment on the dike slope is necessary to prevent erosion.

Several simulations of a 100-year flood event are carried out in order to determine the length and the height of the water retaining structures. The structures should guarantee that overtopping does not occur and that the industrial and commercial areas in the City Park are not affected. On top of that, the residential areas downstream of the City Park should not be affected. According to these requirements, the length of the water retaining structures and the water depth achieved in the river have been obtained (Figure 25).

Figure 25: Location of the water retaining structures (green) and water depth in the City Park area. The commercial and industrial area of the City Park are marked in purple and red, respectively.

The length of the water retaining structures is 2.532 km kilometers, considering both riversides (measured in QGIS). A digital elevation model (DEM) of the area of study is necessary to determine the required height of the retaining structures to avoid overtopping, based on the inundation depth (Figure 25). The height of the structures depends on the elevation of the locations were the water retaining structures will be installed (Figure 26). The retaining structures must withstand the maximum level of inundation of the pixels that are contiguous to every retaining structure location (including the diagonals). Furthermore, an addendum of one meter must be implemented in the structure.

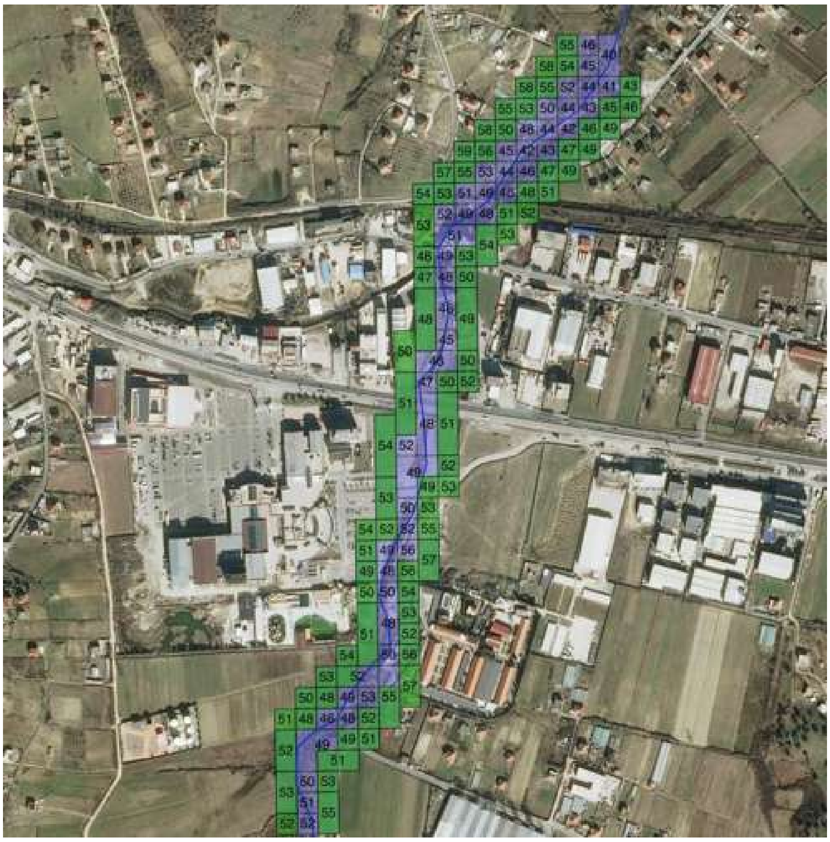

Figure 26: Ground elevation of the watershed of the Limuthi river in the City Park stretch.

Assuming that two meters of river bed are dredged, in Figure 27 the required height of the water retaining structures to satisfy the flood protecting function for a 100-year return period is shown.

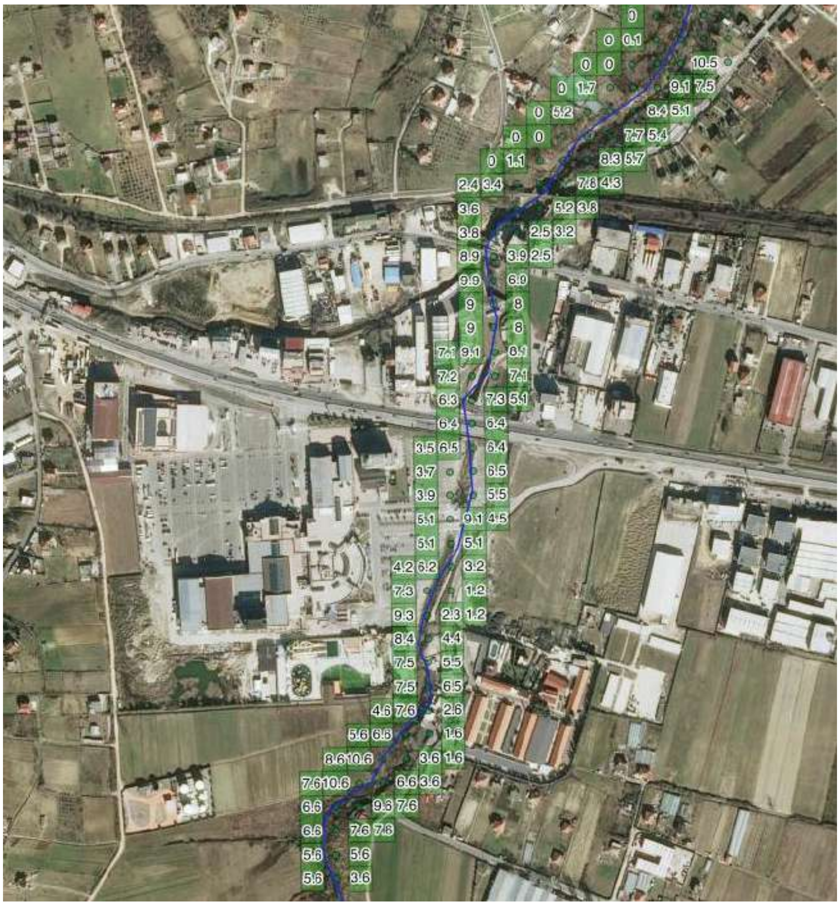

Figure 27: Height of the water retaining structures that protect the City Park area. The dredging depth and the addendum are already included in the calculation.

In the cross sections of the river that are contiguous to the industrial and commercial areas of the City Park, the minimum height of the structure required is 3.5 meters. If a dike slope of 1:3 and a crest width of 3 meters are applied, the horizontal distance from the vertical axis of the dike to the toe at the river results in 12 meters. Assuming that a distance of 13.5 meters is also required to build the dike in the hinterland direction, it is concluded that no space is available to the execution of dikes in these locations. Instead, flood walls are installed (polygons that include a number 1 in Figure 28). However, dikes can be implemented in the cross sections that are adjacent to agricultural land (polygons that include a number 2 in Figure 28).

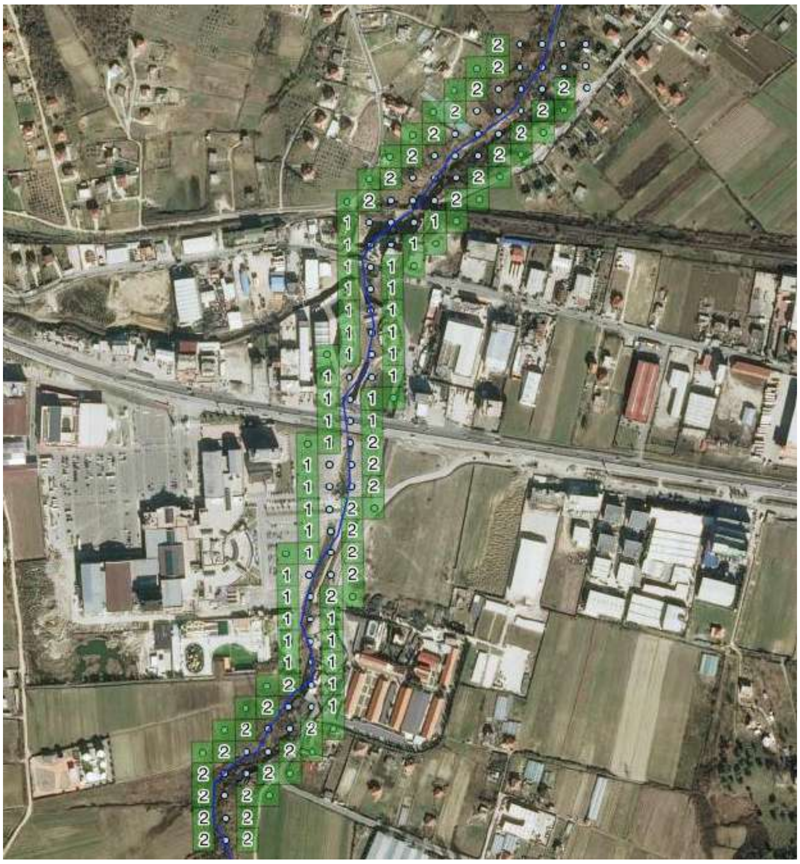

Figure 28: Determination of the type of retaining structure to be applied in every cross section of the Limuthi river, when passing through the City Park. '1' refers to the use of flood walls, in combination with dredging works in the river bed. '2' refers to the implementation of dikes, in combination of temporary structures (Slamdam) and dredging works in the river bed.

8 Evaluation of alternative solutions

8.1 Evaluation method: Cost Benefit Analysis (CBA)

The concepts that have overcome the verification procedure are considered as alternatives for the design. The feasibility of the alternatives is evaluated by assessing the values generated and the sacrifices needed to achieve these values (Voorendt (2017)). The current project can be framed in the macro-economic scale. Consequently, a Cost-Benefit Analysis (CBA) seems to be an appropriate method to support the decision-making in this project.

In a CBA, the effects of a project are quantified in monetary terms. In some cases, these effects are unambiguously quantifiable, such as in the case of the construction costs or economic risk reduction. However, the CBA also includes the monetization of not standardized effects, such as the mitigation of the life risk or environmental impact Jonkman et al. (2016).

The interventions proposed in the present case study aim to reduce the risk of flooding in the area of study. Therefore, the cost of the intervention is the investment (I, in euros), while the benefit is the reduction of the expected economic damage, expressed in present values ($\Delta E(D)$, in euros). An alternative is considered cost effective if:

$$\frac{\Delta R_{PV}}{I} > 1 \tag{27}$$

The ratio between the risk reduction and the investment is commonly known as Benefit-Cost ratio.

The flood risk can be mitigated in two manners:

- **Reduction of the probability of failure of the system**

  $$\Delta R = (P_{f,0} - P_{f,new})D \tag{28}$$

  where $P_{f,0}$ is the initial failure probability (per year), $P_{f,new}$ is the failure probability when the intervention has been implemented (per year) and D is the economic damage in case of failure (euros).

- **Reduction of the damage due to flooding**

  $$\Delta R = P_{f,0}(D_0 - D_{new}) \tag{29}$$

  where D_0 is the initial damage (euros) and D_{new} is the damage after investments in reducing the consequences (euros).

In Civil Engineering, typically large investments are made in the initial phase of the project, while the risk reduction benefits are spread over the lifetime of the infrastructure. For this reason, the benefits are commonly expressed in euros per year, while the investment is expressed in euros. Therefore, the probability of failure is generally given in units per year. To evaluate the benefits and the costs according to the current market prices, the cash flows that correspond to the future years of operation of the infrastructure are transformed into a net present value (NPV, euros):

$$NPV = \sum_{t=0}^{T} \frac{(Cash\ flow)_t}{(1+r)^t} \tag{30}$$

where C_t is the cost in the year t, T is the reference period and r is the discount rate, which represents a return on the investment. Since the reference period (lifetime of the infrastructure) in Civil Engineering projects is relatively long (i.e. decades), for a constant risk reduction per year it results:

$$NPV = I + \Delta R \sum_{t=1}^{T} \frac{1}{(1+r)^t} \tag{31}$$

where:

$$\sum_{t=1}^{T} \frac{1}{(1+r)^t} \approx \frac{1}{r} \tag{32}$$

Therefore, the Benefit-Cost ratio is:

$$BC = \frac{\frac{\Delta R}{r}}{I} \tag{33}$$

Based on the calculation of the Benefit-Cost ratio, a preference of interventions can be elaborated, being the alternative with the largest Benefit-Cost ratio the most feasible option.

8.2 Evaluation of the Benefit Cost ratio of the proposed alternative

In this project, the combined solution consisting of flood walls and dikes is the only solution to be evaluated. To carry out the evaluation of the proposed alternative, the investment and the risk reduction are calculated. The investment is calculated according to cost rates found in literature and the required dimensions of the structures.

The alternative proposed at the end of section 7 consists of dikes (including bank protections and temporary structures in case of extreme events) and flood walls, in addition to dredging works in the stretch of the Limuthi river that is being analyzed. The total length of dikes and flood walls is 1.25 and 1.17 kilometers, respectively. These distances have been measured by counting the number of pixels in which the structures are implemented (Figure 28). For the locations at which the trajectory of the river does not follow the South-North direction (S-N), the diagonal of the pixels has been taken (Southwest-Northeast (SW-NE) direction). Therefore, a pixel that follows the S-N direction measures 30 meters, while a pixel that follows the SW-NE direction measures 42.4 meters. Bank protections and temporary structures are only implemented on top of dikes, and therefore they are also used along 1.25 kilometers. The height of the dikes (and the dimensions of the bank protections) and the flood walls are a function of the location of every pixel (Figure 27). The bank protections are assumed to be one meter thick and are placed on the whole extent of the dike slopes, which have an angle of approximately 18 degrees.

Dredging activities should be carried out in 64 pixels. Every pixel has an area of 900 m^2. Hence, the dredging activities are developed along $57,600$ m^2. In Table 3, a summary of the cost rates and the dimensions of the different elements used in this alternative is given. In Table 4, a breakdown of the cost of every intervention for flood protection of the City Park is shown.

According to Kuriqi (2011), after the devastating inundations that occurred during the winter of 1962-63, the government of Albania decided to repair and strengthen existing embankments and construct new ones in the watersheds of the Albanian rivers, including the Ishmi river. The design return period of the water retaining structures was 50 years. The proposed system of water retaining structures in this report guarantees flood safety for events of at least 100-year return period. Therefore, the probability of failure before ($P_{f,i}$) and after ($P_{f,new}$) the implementation of the design proposed in this report is:

$$P_{f,i} \approx \frac{1}{50} = 0.02 \tag{34}$$

$$P_{f,new} \approx \frac{1}{100} = 0.01 \tag{35}$$

As mentioned in subsection 5.2, the monetized damage (D) in the City Park area in case of a 100-year flood event would be 63 million euros. The discount rate fixed by the Central Bank of Albania by the end of 2018 was 1.25% (CIA (2018)). Based on these figures, the risk reduction (benefit) of the proposed alternative (ΔE) is:

Intervention	Cost rate	Reference	Dimensions	Volume
Dike	$\frac{1\times10^6\ €}{km_{long}m_{high}}$	Jonkman et al. (2013)	$l = 1.25\ km$ $h = f(x, y)$ (Fig. 27)	-
Flood wall	$\frac{7\times10^6\ €}{km_{long}m_{high}}$	Jonkman et al. (2013)	$l = 1.17\ km$ $h = f(x, y)$ (Fig. 27)	-
Dredging works	$5\ €/m^3$	van der Horst (2018)	$A = 57,600\ m^2$ $d_{dredge} = 2\ m$	$115,200\ m^3$
Bank protection	$40\ €/m^3$	van der Horst (2018)	$l = 1.25\ km$ $h = 1\ m$ $\tan(\alpha) = 1/3$ $w = f(h(x, y))$ (Fig. 27)	$20,317\ m^3$
Temporary structures	$474\ €/m$	Slamdam (2019)	$l = 1.25\ km$ $h = 1\ m$	-

Table 3: Cost rates and dimensions of the elements used for flood protection.

Intervention	Cost
Dike	$6,424,600\ €$
Flood wall	$51,349,760\ €$
Dredging works	$576,000\ €$
Bank protection	$812,655\ €$
Temporary structures	$592,690\ €$
TOTAL	$\mathbf{59,755,704\ €}$

Table 4: Breakdown of the costs of the flood protection alternative.

$$\Delta E = \frac{(P_{f,i} - P_{f,new})D}{r} = 50,400,000\ € \tag{36}$$

Therefore, the BC ratio of the proposed alternative is:

$$BC = \frac{\Delta E}{I} = 0.84 \tag{37}$$

The observed tendency of drop of the interest rate (which is directly linked to the discount rate) in the recent years (4% in the last 8 years, as stated in Economy (2018)) leads to expect that by the time the project might be implemented, the discount rate would have fallen to 1%. In such a case, the BC ratio would rise to 1.05 and the project would become feasible.

9 Design of the selected alternative

In section 7, the dikes and the flood walls have been designed considering overflow as the governing failure mechanism. The required height of the water retaining structures has been calculated for each section, according to the results of the 100-year return period flood event. However, the design of these structures should consider other failure mechanisms. The goal of this section is to provide a guideline to develop a preliminary design of the dikes and the flood walls, by addressing other common failure mechanisms in these structures.

9.1 Design of the flood walls

The failure mechanisms to consider when designing the flood walls are:

- **Overflow**: as mentioned before, this is the failure mechanism governing in this design. The structures should be sufficiently high so as they are not overtopped during the 100-year return period flood event. Based on the simulations of such an event, the height of the structures have been determined in order to avoid overflow.
- **Structural failure of the flood wall due to shear or bending**: if the water level in the river is low, the shear and the bending moment exerted by the soil in the flood wall is maximal. In this case, the main function of the flood wall is the earth retention, rather than water retention. The calculation of the internal stresses is necessary to determine the thickness of the wall and the amount of reinforcement or prestressed steel required to withstand the loads. The use of reinforcing or prestressing steel is generally a trade off between material quantities and cost.
- **Horizontal stability**: the flood wall is a gravity structure, whose stability relies in great part in its self weight. The horizontal soil forces on the gravity structure are stabilized by friction between the soil and the structure, which increases proportionally to the self weight of the structure. Friction avoids the horizontal displacement of the structure. Therefore, the dimensions of the structure should also be determined according to the horizontal stability. The friction can also be enhanced by introducing prestressed concrete piles at the foot of the flood wall, since they act in downwards direction, increasing the normal force on the structure.
- **Vertical stability**: although self weight enhances the horizontal stability, it can compromise the bearing capacity of the soil. This should be enough to avoid the breaking of the soil underneath the structure. Hence, the dimensions of the structure are also limited by this failure mechanism.
- **Rotational stability**: the point of application of the external forces that act on the structure can be critical for the overturning of the structure. Large forces acting far from the center of rotation can lead to rotation of the structure. This stability condition should be taken into consideration when determining the shape of the structure, since the shape is influencing directly on the position of the centre of gravity of the structure. The center of gravity coincides with the point of application of the self weight, which is generally a governing force in the influence of the stability.
- **Piping**: the difference in water head between the river side and the land side induces water flow under the foot of the flood wall (impermeable structure), from the river to the land side. Due to this water pressure, a water pipe starts to develop under the foot of the flood wall, through which soil material is eroded. The migration of material under the dike can lead to an eventual instability and failure of the structure. In order to delay the creation of pipes under the flood wall foundation, the seepage length can be increased by installing an impermeable structure under the flood wall foundation. The impermeable structure can be a sufficiently long sheet pile (cutoff walls) or grout columns, which makes the soil impermeable and cohesive. Compaction works around the gravity structures also prevent the fast erosion of the soil.
- **Scour**: the scour of the soil in the vicinity of the foot of the flood wall can lead to macro-instability of the gravity structure. The erosion of the soil develops in time, until the point at which the shallow foundation losses the stability. The prevention of this failure mechanism is carried out by means of periodic inspection and maintenance activities. More details about the bed protection that surrounds the foot of the flood wall is given in subsection 9.3.

A preliminary design of the flood wall is shown in Figure 32 (bottom figure), which shows a general cross section of the Limuthi river where flood walls have been proposed, according to Figure 28.

9.2 Design of the dike

The failure mechanisms to consider when designing a dike are:

- **Overflow**: as in the case of the flood walls, this failure mechanism has been the one governing to determine the design of the dike. According to the water depths in the river obtained from the hydrodynamic simulation of the 100-year return period flood event, the height of the dike in each section of the river has been determined to avoid overflow.
- **Sliding of the inner slope**: this failure mechanism occurs mainly during flood events. Since the water level in the outside part of the dike remains high for long periods(days), the water is infiltrated in the dike body and the pore water pressures increase. This leads to a reduction of the effective stresses, what is translated into a reduction of the shear strength of the soil. Therefore, the probability of occurrence of macro-instabilities in the inner slope of the dike increases. This type of instabilities are generally approached by means of slip-circle analysis, for example by means of the Bishop method. This method uses the vertical and rotation equilibrium equations to calculate the most critical case, expressed in the form of a ratio between the strength and the load (factor of safety). In a subsequent stage of the design of the dikes in this project, it should be checked that the material used for the core of the dike and the dike slope is suitable for the stability of the structure.
- **Horizontal stability**: if the dikes are made of relatively light materials (specially peat), a notable difference in water level at both sides of the dike could result in horizontal shearing of the dike. In these cases, the relatively small effective stresses of the dike lead to a low friction force to compensate the hydrostatic pressures in the outside slope of the dike. The situation becomes particularly critical after severe droughts, when the water evaporates from the soil pores and the dike becomes even lighter. In the area of study, peat is not expected to be present. However, a more detailed soil investigation along the whole dike trajectory should be carried out in order to confirm so.
- **Sliding of the outer slope**: this failure mechanism can occur after a flash flood, when the water level in the river drops very quickly. The water level inside the dike does not drop at the same pace as the water in the river, what leads to the slide of the outer slope towards the water. To avoid this failure mechanism, the outer slope of the dike should be protected with a revetment (see subsection 9.3).
- **Micro-instability**: this failure mechanism is developed in time, when the phreatic level inside the dike rises until the inner slope. If an impermeable layer of soil (such as clay) is used, the rise in the water line inside the dike core can just push the layer apart. Once the internal pipe in the core is formed, the sand particles from the core of the dike are eroded, leading to an eventual instability of the dike. The migration of the sand particles of the core of the dike is avoided by means of the installation of berms or geotextiles in the inner slope.
- **Piping**: the development of pipes under the dike follow a similar mechanism as the explained for the case of the flood wall. If the difference in head at both sides of the dike is sufficiently large, water will start eroding the sand particles that are underneath the clay material of the dike. The reduction in the difference of heads mitigate the development of the piping mechanism. This can be achieved by the installation of a canal that runs parallel to the dike, together with a filter installed at the bottom of the canal in order to relieve water pressures but avoiding the erosion of sand particles from underneath the dike.

A preliminary design of the river dike is shown in Figure 32 (top figure), which shows a general cross section of the Limuthi river where dikes have been proposed, according to Figure 28.

9.3 Design of the bed and bank protections

The analysis of the flow velocities in the studied river stretch enable to assess if the erosion induced by the flow velocities is critical. According to Figure 29, the flow velocities exceed 1 m/s in all the locations.

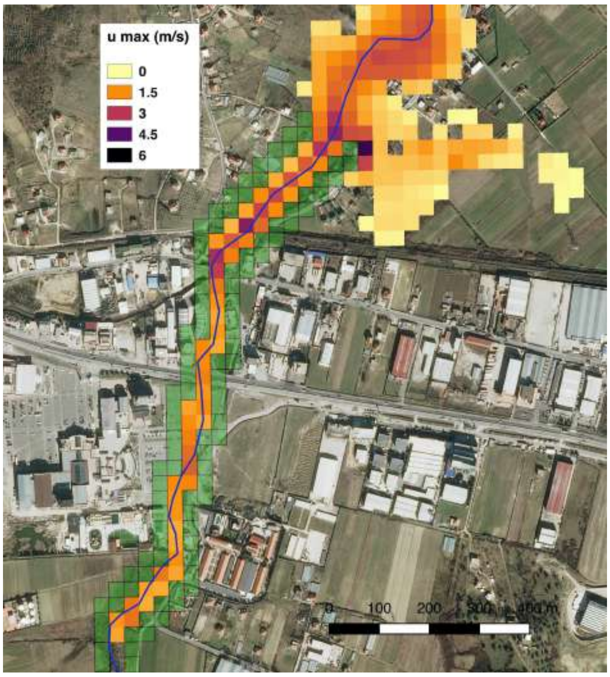

Figure 29: Maximum flow velocities in the analyzed stretch of the Limuthi river, during a 100-year return period flood event.

The minimum depth in the analyzed river stretch during a 100-year return period event is 4 meters. Therefore, the flow is turbulent since the Reynolds number is much larger than 2000:

$$Re = \frac{ud}{\nu} = \frac{1 \times 4}{10^{-6}} = 4 \times 10^6 \tag{38}$$

where u is the flow velocity (in m/s), d is the water depth (in m) and ν is the kinematic viscosity of the water (in m^2/s). Therefore, the flow velocity is almost constant along the depth (Schiereck (2012)) and the river bed is affected by velocities that achieve almost 4 m/s in some locations. Due to the large flow velocities, bed and bank protections are required. A statistical analysis of the maximum flow velocities in the analyzed river stretch has been carried out, in order to determine the flow velocity to design the bed and the bank protection. The mean and the standard deviation of the maximum flow velocities are 1.71 and 0.68, respectively. Assuming that the maximum flow velocities are normally distributed, the 95-percentile of the distribution corresponds to a velocity of 2.83 m/s. This value is used for the design of the bed and the bank protection (subsection 9.3.1).

The bed protection is implemented along the stretch of river that is being analyzed, while the bank protection is only implemented in the dike river sections. For the bed, a granular filter is used in the entire studied stretch. In the case of the bank, a granular filter is applied from the toe of the dike until the water level corresponding to the 95-percentile of the monthly maximum discharges (Hidrometeorologjik (1985)), which corresponds to a discharge of 69 m^3/s. Applying equation 17, this water depth is 1.6 meters. From this height on, the dike slope is covered by a grass layer (subsection 9.3.2). Generally, a height corresponding to the normal depth for an average flow discharge should be chosen. However, due to the the observed large flow velocities in case an extreme event occurs, the developed design is more conservative.

9.3.1 Design of the granular filters for the bed and bank protections

The bank protection consists of a rock armor layer, which lays on top of one or more filter layers. The granular layers of the bed protection are designed to be geometrically closed. According to Shields (Schiereck (2012)), the nominal diameter of the rocks to be placed in the armor layer, in order to achieve stability is:

$$d_{n50} = \frac{u_d^2}{\psi_c \Delta C^2} = 12 \ cm \tag{39}$$

where u_d is the design flow velocity (2.83 m/s), ψ is the Shields parameter (0.05) and C is the Chézy roughness (40 $m^{1/2}/s$). According to the standards gradings of rocks for the design of protections, as given in EN13383, the type of rock manufactured that should be used in this design is the type $LM_A \ 5-40$, which corresponds to rocks of $d_{n50} = 17 \ cm$, mass of 5 to 40 kg and a layer thickness of 25 cm.

The next step is the design of the filter layers, which are underneath the armor layer and on top of the original soil. The geometrically closed filters should guarantee stability and permeability:

- **Stability**: the space between the grains of one layer should be sufficiently small so as significant migration of finer grains through the filter does not occur. In order to guarantee this requirement, the following condition should be satisfied:

$$\frac{d_{15F}}{d_{85B}} < 5 \tag{40}$$

  where d_{15F} is the diameter of the grains of the filter (top) layer that correspond to the size of the sieve that enables to pass the 15% in mass of the filter layer soil sample, and d_{85B} is the diameter of the grains of the base (bottom) layer that correspond to the size of the sieve that enables to pass the 85% in mass of the base layer soil sample.

- **Permeability**: the space between the grains of one layer should be large enough so as critical water pressure buildup in the layer does not occur. The condition to satisfy in this case is:

$$\frac{d_{15F}}{d_{15B}} > 5 \tag{41}$$

Based on these rules, one filter layer and one armor layer are necessary (Table 5). It is assumed that the ratio between d_{85} and d_{15} for each layer is 2 (Schiereck (2012)).

	d_{15}	d_{85}	d_{n50}	**Type**	**Mass**	**Thickness**
Armor layer	110 mm	220 mm	170 mm	$LM_A \ 5-40$	5-40 kg	25 cm
Filter 1	11 mm	22 mm	18 mm	Medium gravel	-	20 cm
In situ soil	1.1 mm	2.2 mm	1.75 mm	Coarse sand	-	-

Table 5: Characteristics of the granular filter of the bed and the bank protection.

9.3.2 Design of the grass cover on the dike slope

A grass layer on a dike slope is considered a soft revetment. Grass is highly valued because because of its natural appearance. In some cases, dikes are protected with a hard revetment, which is covered again by a grass layer, in order to guarantee high resistance to hydraulic loads while preserving the visual amenity (Jonkman et al. (2018)). This type of revetments are called *hidden revetments*. Grass is placed in the areas that are not permanently submerged.

The visible part of the grass cover are the sward and the stubble, which correspond to the grass leaves. The turf is the part of the grass cover that provides the resistance against erosion and is located below the surface. The grass cover maintains the clay humidity and prevents it from drying out. At the same time, the clay provides the nutrients that feed the grass and keeps it healthy. The grass roots keep the connection between clay and grass (Figure 30). It is preferable that the grass layer is not uniform, because the bound between the grass and the clay is stronger if different sizes and shapes of roots are present. Regarding the maintenance of the clay cover, it is desirable that the grass is not treated with fertilizers and that the mowing of the grass is not frequent.

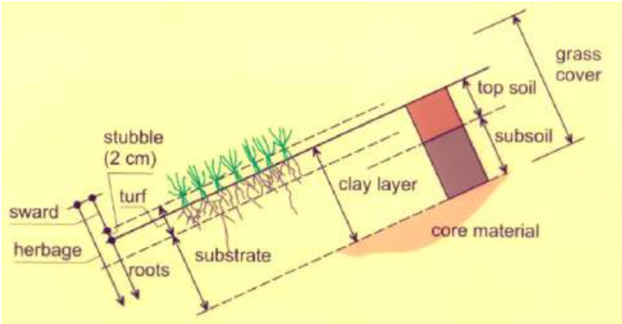

Figure 30: Composition and definitions of a grass on clay (Jonkman et al. (2018)).

If geotechnical failure can be prevented, notably large discharges can be resisted by a grass layer. The strength of the grass layer mainly depends on the duration of the overflow and the quality of the grass cover (Hewlett (1987)). In the figure below (Figure 31), the flow velocity, the duration of the overflow and the quality of the grass are related.

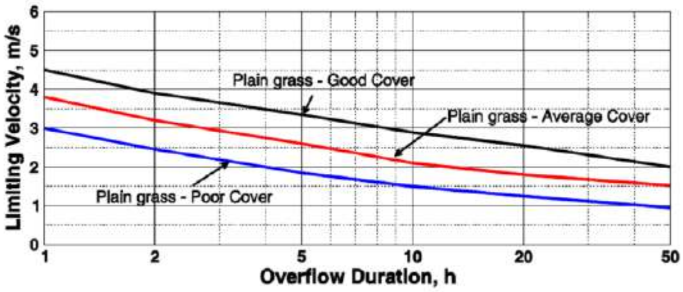

Figure 31: Relationship between the flow velocity, the duration of the overflow and the quality of the grass.

Assuming that a grass cover of high quality is applied, the cover would be able to resist more than ten hours, for the design flow of the bed and the bank protection (2.83 m/s). Therefore, it is considered that a grass layer on the higher part dike of the dike slope is enough to withstand the load. However, after the extreme events, repairs and maintenance activities should be carried out to recover the high quality of the grass cover.

9.4 Cross sections of the dike and the flood wall

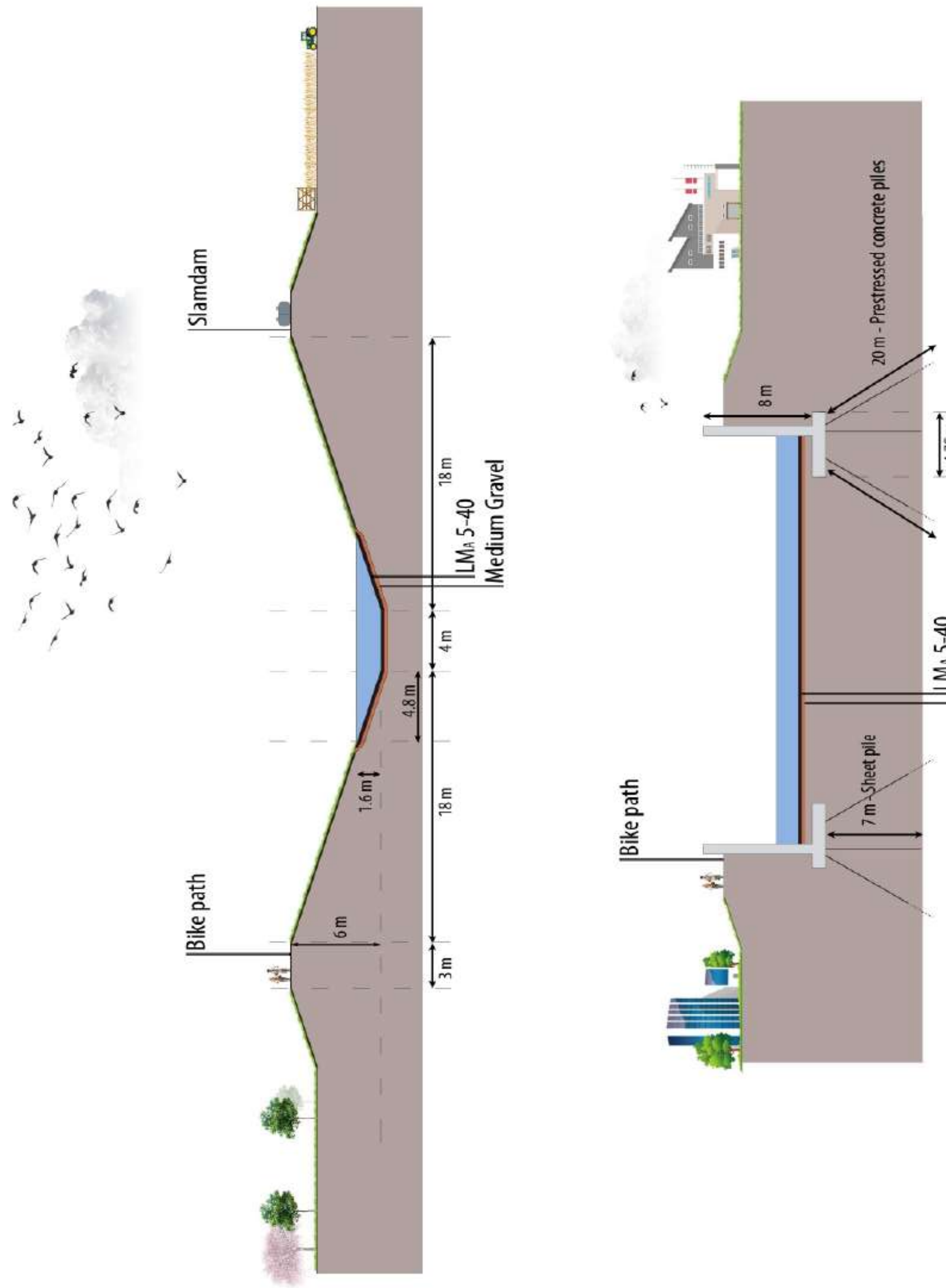

Figure 32: Relationship between the flow velocity, the duration of the overflow and the quality of the grass.

10 Conclusions and recommendations

10.1 Conclusions

In the recent years, several cases of flash floods have been reported in the Municipality of Tirana, causing important material damage in the region. One of the most devastating events occurred in December 2017, which flooded the City Park area (West of the Municipality of Tirana). This area is strategical because it includes relevant centers of economic development, such as commercial and industrial areas, that are located nearby a main road connection between the Port of Durres, the Tirana International Airport and Tirana itself.

The exceptional position of the City Park in the map has fostered the interest of the Municipality to potentiate the local economy. However, this area is in risk of flooding due to overflow of the Limuthi river, since the area is nearby the confluence of several river branches that come from upstream. During the field trip that took place in April 2018, several deficiencies in the river system were observed, such as the presence of informal buildings near in the river, lack of maintenance of hydraulic structures and drainage systems, uncontrolled waste disposal and sediment accretion at certain locations, what increased the probability of overflow of the river.

To mitigate the flood risk at this location, several hydraulic engineering concepts have been developed. The construction of dams to control the flow has been disregarded due to the small surface of the two upstream reservoirs. This fact leads to disproportionate dam heights in order to be able to accumulate the discharge of the river during a flash flood. The applicability of dikes and flood walls has also been addressed. Two main constraints in the development of concepts have been noticed: budget constraints and spatial constraints. The upside of the flood walls with respect to the dikes is the relatively small space required to implement them. The dikes have a larger footprint due to their maximum slope of 1:3 that guarantees the stability. However, the construction of dikes is notably more economic, compared to the construction of flood walls. Therefore, with the aim of developing a flood protection system that is financially and spatially feasible, a system that combines dikes and flood walls is proposed. The industrial and commercial areas of the City Park are very close to the river. Hence, flood walls are used to protect these areas. Dikes are implemented in the river boundaries were agricultural and green land uses are found. Grass covers are implemented on the dike slopes to compensate the impact provoked by the water retaining structures in the local environment. The grass enhances the local visual amenity. Furthermore, bike paths are implemented on the dike crests and contiguous to the flood walls to improve the livability of the area.

In order to adapt further to the budget, the dimensions of the structures to be built are reduced by dredging the river beds, what increases the section of the river to convey water. Dredging is more economic than executing structures, and therefore, increases the feasibility of the project. However, the dredging depth has been limited to two meters, in order to limit the morphodynamic changes of the river bed. Further research about the optimal depth to dredge should be carried out in further stages of the project. Moreover, in order to reduce the financial and visual impact of the dikes, temporary structures of one meter height will be placed on the crest of the dikes in case a 100-year flood event occurs. This measure reduces the dike height in one meter. The dikes also include a bank protection in order to reduce the soil erosion.

A Cost-Benefit Analysis (CBA) of the proposed solution has been carried out to analyze the feasibility of the alternative. The resultant BC ratio of the solution is 0.84, which is close to what is considered feasible ($BC > 1$). However, this ratio depends on the discount rate. According to the falling tendency of this rate in the recent years, as shown by the information published by the Central Bank of Albania, the rate is expected to fall from the 1.25% to a 1% in the close future. This drop would make the project feasible ($BC = 1.05 > 1$).

Beyond the design proposed for the specific case of the Limuthi river, this project can be taken as a guideline for the risk assessment and design of water retention structures in areas that are prone to suffer flash floods. Apart from providing a strategy to develop a design (section 4), this report shows the applicability of different methods to evaluate flood events, such as probabilistic methods (method of Montecarlo), decision-making methods (Cost-Benefit Analysis), hydrodynamic models (HEC-RAS) and geographic information systems (QGIS).

10.2 Recommendations

In the following paragraphs, a summary of suggestions given by the Regional Office for Europe of the World Health Organization are exposed (Menne and Murray (2013)). The interventions that are proposed help to mitigate direct and indirect damage. The proposals are classified into two categories of prevention: primary, secondary and tertiary.

Apart from the implementation of flood defences, there are more interventions that enable the flood risk reduction. The interventions for primary prevention aim to manage the flood risk by means of structural (engineering) and non-structural (policy and organization) solutions. The interventions for secondary prevention aim to mitigate the human and the material losses by identifying the most vulnerable population and land uses. Tertiary prevention includes the interventions that guarantee the availability of drinking water, the moving of personal belongings, rehabilitation of the infrastructure and the provision of medical care to the affected population.

Several interventions are proposed below, analogous to the interventions that the stakeholders of the project proposed in section 3.3:

- **Emergency plans**: they enhance the chances to evacuate the population on time, reducing the risk of human losses in case of flooding. The use of real time information systems that provide weather and water level forecasts, together with information about the strength of the dikes could improve the decision-making for evacuation (Schuurmans (2019)). Furthermore, the optimization of the evacuation routes can be carried out by means of network analysis. This can be done by means of the tool *closest facility analysis* in ArcGIS.
- **Land use management**: the material flood risk can be reduced by re-adapting the current spatial planning. The implementation of regulations that forbid the land development in flood-prone areas reduce the flood risk. Furthermore, the proposed design had an important spatial constraint for the implementation of dikes in the City Park area. If this spatial constraint would not exist and dikes could be implemented along the whole stretch of the river that has been analyzed, the investment would be reduced in a 70% (from almost 60 million euros to 18 million euros). Although this reduction in cost does not include the relocation costs, this option could be studied for further development due to its large feasibility margin (the BC ratio would rise to almost 3, without considering relocation costs).
- **Control of water resources and flow, including drainage systems**: an adequate functioning of the drainage systems mitigates the urban flooding. During the field trip, underground water collectors that conveyed water to the main river course were observed in a very deficient state. The steel reinforcement could be seen and the concrete was fractured. Moreover, in the confluence with the river course, the collectors occupied a great part of the river cross section, reducing suddenly the water conveyance cross section of the river. To improve the flow regime and the state of the river watershed, these damaged structures should be replaced by new ones. The constant monitoring of the river bed can determine the river sections that are prone to accumulate sediment coming from upstream, what reduces the water conveyance section. The sedimentation rates in the critical cross sections should be the basis of development of a maintenance dredging program. Furthermore, the regular inspection of the dike revetments and flood walls are key to prevent unexpected structural failure.
- **Design and architectural strategies**: building regulation and law enforcement prevents the development of urban uses in flood prone areas, contributing to the reduction of flood risk. The implementation of vegetation and trees in the flood prone areas increase the water filtration rates and hinders the spread of the flood. Furthermore, beyond the technical functionality, architectural strategies improve the adaption of the interventions to the landscape and enhance the livability of the area.
- **Flood insurance**: insurance can help to reduce the mental stress that the population experience after flooding, particularly with regard to the financial impact.

For a rational and further development of the project, it is recommended to evaluate the cost-efficiency and the added values of the proposed measures, in order to elaborate a list of preferences of interventions in the City Park area.

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