Master Thesis

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Assessing the impact of urbanisation on surface runoff peak flows in Bogota

A study based on historical change in impervious cover and increase in drainage efficiency

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Gerhard Barmen

Gerhard Barmen

Local MFS Programme Officer

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Abstract

Due to internal conflicts and an economical conversion in the mid 1900's, the Colombian society has seen a great social transition from rural to urbanised in the last 50 years. Apart from positive impacts of urbanisation on social factors, there are hydrological issues coupled to it. With the aim to assess the impact of urbanisation on runoff peak flows in Bogota, the historical change in impervious cover and increase in drainage efficiency through the years 1797-2013 have been studied. The curve number- and triangular unit hydrograph-methods were used to determine pre-development flows, while the urban peak discharge formulas (USGS, 1984) were used to determine post-development flows. Eight sub-basins located in the most urban parts of Bogota have been subjected to the study, and results show an increase of at least 100% in peak flows in all watersheds, with one exception, and up to 500% in others. Solutions of both constructional and political sorts have been suggested, including subventions for investments in sustainable urban drainage measurements, and green roofs and permeable pavements. However, good social conditions, equality, and knowledge among the population is key to a better water resources management. It has been concluded that the urbanisation and increase in drainage efficiency in Bogota has had great impact on the runoff peak flows in Bogota, and that different types of measures are required to withstand further impact on the hydrological cycle.

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Abbreviations

BDF	Basin Development Factor
CN method	Curve Number method
FDC	Flow Duration Curve
Pte Bosa	Puente Bosa
SCS	Soil Conservation Service
SUDS	Sustainable Urban Drainage System
UPD Formulas	Urban Peak Discharge Formulas
USGS	United States Geological Survey

Chapter 1
Introduction

According to the United Nations Population Fund, UNFPA (2016), the world is currently undergoing its *largest wave* of urbanisation ever. Towns and cities are estimated to currently host half of the world's population – a figure that is expected to increase to 5 billion people in the upcoming 13 years. Apart from economic growth (Rogers, 2012), the transition to an urbanised society is believed to bring great potential for well-being, in the form of social mobilisation and access to education and health services (UNFPA, 2016). However, the effects of urbanisation are not only significant from a social point of view, but also eminent from an environmental standpoint, making it an issue that deserves more attention.

1.1 General background

The Merriam-Webster Learner's Dictionary (2017) defines urbanisation as *the process by which towns and cities are formed and become larger as more and more people begin living and working in central areas*. The urban population first exceeded the rural one in 2007, and the world has since then remained principally urban. Projections show a steady increase in urbanised settlements, expecting 66% of the world's population to live in urban settlements by 2050 (United Nations, 2015).

For the past half century, the Colombian society has experienced a great social transition from agricultural to urbanised. Massive migration to cities has transformed the country to one of the most urbanised nations in Latin America (Nationalencyklopedin, n.d.). Colombia is also a country that has seen war and conflict for several decades, where deadly violence, sexual assaults, and threats have forced its own population to displace both internally and externally. In 2010, between 3.3 and 4.9 million people were estimated to be refugees in their own country (Landguiden, 2017). According to the UNHCR (2003), the majority of these have rural background, and up to 50% end up in cities. These displacements initiated the urban transformation, which was boosted in the 1970's when Colombia transitioned to an export-based economy, creating industrial jobs (del Ama, 2013). Even though cities such as Bogotá, Medellín, Cali and Barranquilla have all seen increased populations, increased economic activity, access to better education and health-care systems, and accelerated technological development for the last 50 years, the benefits of an urbanised society have not reached the whole population. Social divides between rural and urban areas, and within cities as well, have seen an increase. Intense growth of inner cities has forced many people to live in makeshift settlements, where almost one third of the population are considered poor. As an example, almost two thirds of the population in Medellin, the second largest city in Colombia, live in slums (del Ama, 2013).

The population increase in Bogota for the years between 1810 and 1980 is deemed remarkable – increasing from 20,000 to four million inhabitants. The latter amount nearly doubled until 2014, to a population size of eight million (Nationalencyklopedin, n.d.) with an urban area of almost 560 km^2, according to the maps that this study is based on. Bogota is therefore not only an interesting site for studying urbanisation and the impact of it, but also a very important one. Since today's prognosis predict a major increase in migration from rural to urban areas all over the world in less than two decades (UNFPA, 2016), a city like Bogotá makes the perfect case study for future reference.

1.1.1 Studied impacts of urbanisation on hydrological systems

One of the most direct and noticeable environmental consequences of urbanisation is the change in land use and land cover, in the form of increased impermeable surfaces, affecting the hydrological cycle. This is made apparent through changes in several hydrological features, including those in surface and groundwater levels, surface runoff patterns, and water pollution (Arup, et al., 2006).

According to the USGS (2016a), land-use change due to urbanisation impacts water systems in a variety of ways. In the initial phase of large scale urbanisation, an increase in storm runoff and erosion can be noted. This is due to the decrease in vegetation acting as a hinder for storm runoff, allowing an amplification on its pace. Consequently, the quantity of sediment being washed into streams is increased. Additionally, alterations to water-drainage patterns – common in this initial phase – can cause flooding. Alterations can include the introduction of drainage pipes, and the channelization of rivers.

During continued urbanisation, more roads, houses, and buildings of both commercial and industrial kind are added to the area. More people can now live together on a closer area, consequently increasing wastewater amounts being discharged into local water bodies. The impermeable surfaces in form of pavements and constructions hinder the infiltration of water into ground water reserves, giving less water for groundwater recharge. Furthermore, an increase in storm water runoff can also be noted. Water that was to be infiltrated, will instead reach water bodies with the risk of causing flooding (USGS, 2016a).

According to a study made in North Virginia (Anderson, 1970) on the effects of urban development on floods in that area, the general conclusions that could be drawn were that the installation of sewer systems, independent of impervious development, led to an increase in flood-peak magnitudes. Another conclusion was that completely impervious surface areas increase average-sized floods. However, no significant impacts could be shown on larger floods, such as 100-year floods. Other findings include that the lag time, defined as *the time from the centroid of rainfall excess to the centroid of direct runoff,* seems to be the basin characteristic mostly affected by urbanisation. Supported by Shuster et al. (2005), it is concluded that the lag time decreases with urbanisation, giving higher runoff peaks and thus higher volumes of water added to receiving water bodies.

Kent et al. (2010) emphasize the effect that urbanisation has on flow velocities, as vegetation is replaced by impermeable surfaces, such as streets and gutters. This has a generally decreasing effect on the travel time through the watershed, as runoff is being transported downstream much faster. Similarly, overland flow lengths are decreased due to runoff being conveyed to channels. These are constructed with high hydraulic efficiency, once again increasing flow velocities. It is also mentioned that urbanisation can have both an increasing or decreasing effect on slopes. Supported by USGS (2003), peak discharges of floods are strongly influenced by changes in the natural landscape, as storage capacities are decreased with land use change, and the movement of water is increased, as described above. This results in higher peak discharges in urban streams, and also larger volumes of discharged water during floods. An example of this has been presented by the USGS (2003), where the streamflow in two creeks – one urban, one rural – have been compared during the same flood. The urban creek saw earlier and faster increase in streamflow, together with higher peak discharge and overall volume during the same flood.

A typical impact of urban development on recipient waters is increased amounts of water entering them, and a lower time of concentration. The combination of these two events can result in altered stream paths due to erosion of streambeds and streambanks, and more rapid runoff. Rapid runoff and impermeable surfaces also prevent infiltration to soil and aquifer recharge, which can lead to lower sustained stream flows, especially during dry periods (Coles, et al., 2012).

Finally, it has been shown that urbanisation has a great impact on the quality of water. Litter, lawn and garden chemicals, paints and oils, and detergents high in phosphorus, are some acknowledged releases made into storm- and wastewater sewers, harming water sources and the surrounding environment (USGS, 2016b).

1.1.2 Sustainable Urban Drainage Systems (SUDS)

SUDS are used to approach the issue with increasing urbanisation and the studied hydrological effects coupled to it. They include approaches that aim to manage surface water, and consider flooding aspects, pollution, and biodiversity. They are usually local solutions aiming to mimic nature, and can be designed to, for instance, convey and/or attenuate runoff, and store water to increase infiltration or evapotranspiration. SUDS are considered more sustainable than traditional systems because of many reasons, including for instance that they are better at managing runoff volumes from impermeable surfaces, they make it possible to use runoff where it falls, and they reduce pollution from runoff. Additionally, they provide attractive environments for the wildlife and for the local community, as well as protecting natural habitats (Susdrain, 2017).

One example of a SUD is the harvesting and recycling of storm runoff as non-potable water. Cities and communities suffering from water scarcity can thus save drinking water, by using storm water for watering plants, toilet flushing, or the washing of clothes. Additionally, the redirected water would take pressure off the drainage system (State of Green, 2015).

One method used to improve the water quality of storm water runoff before it is drained into a natural water body, stream, or channel, is the so-called infiltration trench. This is a submersion in the landscape covered with grass, with a top-layer possible of retaining pollution. This layer can for instance consist of a mixture of sand and humus, which removes a wide range of pollution. Beneath the top soil, trenches can be placed to act as storage space as the water infiltrates into the ground (Birch, et al., 2008). The principal of an infiltration trench is visualised in figure 1.1

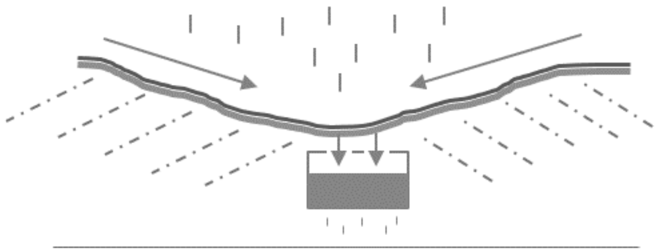

Figure 1.1 The principal of an infiltration trench, allowing for storage of and infiltration of runoff into the ground

Green roofs, another form of SUDS, are used to delay and decrease the total volume of runoff. Additionally, they can be used as insulation against warmth, and provide natural habitats for some insects and birds. A typical green roof consists of a growth medium, a drainage layer, and a water-proof membrane. Water that is retained in the roof eventually evaporates (State of Green, 2015).

Permeable pavements are a simple way of introducing infiltration-increasing methods without changing too much of the landscape or the use of it. They can be used as walking streets or for driving, and they allow for rainwater to infiltrate. The infiltration capacity varies between places, as it is dependent on, for instance, the soil characteristics around it (State of Green, 2015). This method is considered rather effective, but it also requires maintenance for it to function well. Tire particles or humus particles of garden soil can have a negative impact on the infiltration properties of the permeable pavement, due to clogging of the pores. This can be prevented by regular cleaning, but it can be cost intensive (Birch, et al., 2008).

A more direct measurement towards decreasing peak-flows and flood-risks is the installation of retention and/or detention ponds. The retention ponds always have a constant pool of water, while detention ponds are dry between rain or snow events. Both types aim to slow down water flow by detaining water, and can also be used to improve water quality as they allow nutrients, pathogens, and metals in the sediment to settle out. The detention pond holds water for a shorter period of time, only allowing larger grains to settle, while smaller grains move further on in to water courses. The retention pond holds water for a longer period of time, allowing for the settlement of finer grains as well. (Laramie County Conservation District, 2016). Both methods require rather large areas, and for the retention pond, the safety hazard of standing water must be taken into account. Additionally, mosquitos can be attracted by standing water, causing an un-attractive environment for the locals (Leber, 2015).

1.2 Project aim and limitations

The aim of this thesis work is to investigate and assess the impact of urbanisation on runoff peak flows in Bogota, using historical change in impervious cover between the years 1979-2013, and increase in drainage efficiency.

The project is limited to the change in impermeable cover in the most urbanised and populated areas of Bogota, and their respective catchment areas. These consist of eight main river basins: Fucha, Salitre, Tunjuelo, Torca, Tintal, Jaboque, Quebrada Yomasa, and Quebrada Yerbabuena. Soil types and changes in land use will be considered to enable fair representations of the areas of study, together with basin development factors, which include the introduction of sewer systems and channelization of rivers. Change in impervious cover is limited to the years 1797, 1848, 1923, 1938, 1958, 1976, 1985, and 2013, as it is from these years that maps showing the urban development in Bogota have been retrieved and used. The main matter is to show a correlation between change in land-use cover and the increase in drainage system efficiency over time, both aspects coupled to urbanisation, and runoff peak discharges. This is done by comparing runoff peak flows from the pre-development scenario with the post-development scenarios. The pre-development scenario represents a scenario where no significant changes in peak flows can be noted due to urbanisation, and is connected to the method used for post-development methodology. One of the main limitations of this comparison is the lack of measured peak flows for all the years considered.

The maps and areal information of the studied catchments have all been retrieved from a background study which is a part of this project, using ArcMap and maps from each corresponding year, and will be used as such in this essay. The process for retrieving this information is explained in the appendix, section I.I.

Chapter 2
An overview of Bogota

Chapter 2 briefly overviews the city of Bogotá; it defines the study area, introduces the relevant catchment areas and the historical change in impervious cover for each of them, and additionally it dives into the past and current water management systems of the city.

2.1 General background

Bogota was funded in 1538 by Gonzalo Jiménez de Quezada, almost four decades after the outset of the Spanish Conquest (Landguiden, 2017a). In the 18th century it was made the capital of the then so-called Vice Kingdom of New Granada, and in 1830, it became the capital of today's Colombia (Nationalencyklopedin, n.d.). With its almost eight million inhabitants, the District Capital of Bogota makes up the largest city in Colombia – having a population almost three times larger than Medellín, the second largest city in the country (Landguiden, 2017b).

The majority of these inhabitants populate an area of approximately 560 km^2 (according to our study), giving the city a population density of more than 14,000 people per km^2. Apart from being the capital city, it also constitutes the centre of finance and trade, including both the stock market and the major banks.

Bogota is located approximately 2,600 m a s l (Nationalencyklopedin, n.d.), and belongs to the so called cold region of Colombia. Temperatures vary between 12-18 degrees Celsius, and there can be rather significant contrasts between night and day temperatures (Landguiden, 2017c).

Figure 2.1 shows the current borders of Bogota and its division into catchments. The light grey area with red borders shows the most urbanised areas of Bogota, which make up the subject of this study, and the respective catchments that this area is divided into. The main river-courses and water bodies are also included in this figure, marked in light-blue. The main river that can be observed running along the upper west border of the city is the Bogota river.

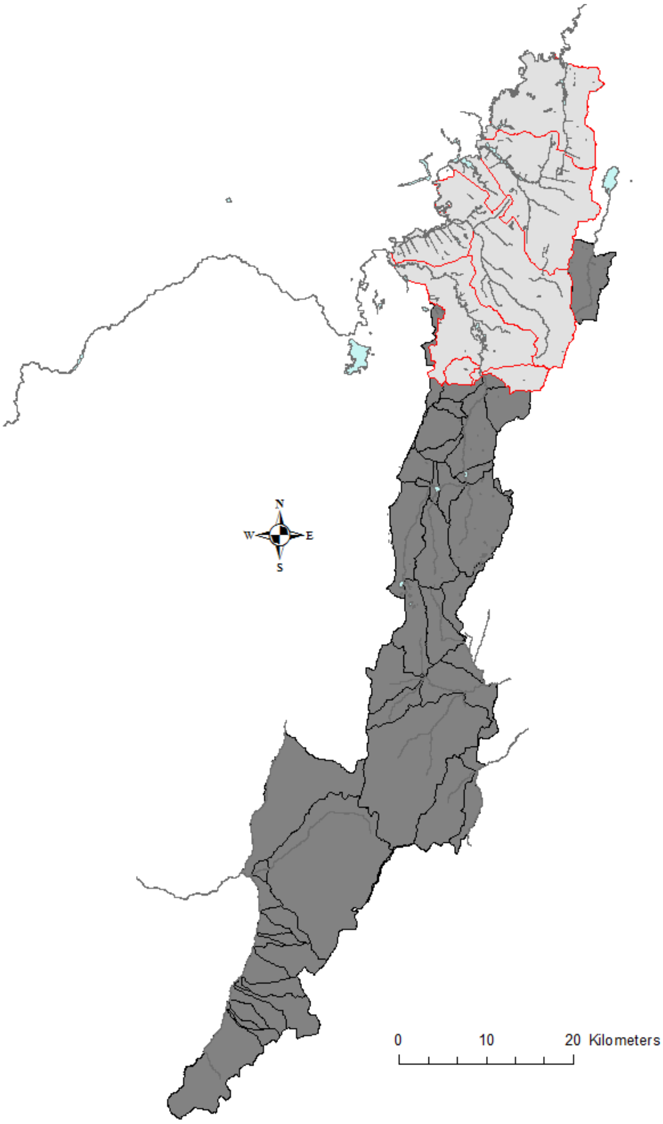

Figure 2.1 The Capital District of Bogota. The area of the study is the light-grey area, which corresponds to the urban parts of the city. The dark-grey area is the rural part.

The geological formation in Bogota is known as the Bogotá Formation (Acosta & Ulloa, 2002), and is composed of poorly stratified mudstone and silty claystone, with fine- to medium-sized grains (McLaughlin Jr & Arce H, 1970).

Figure 2.2 below shows the saturated permeability in mm over the urbanised parts of Bogota.

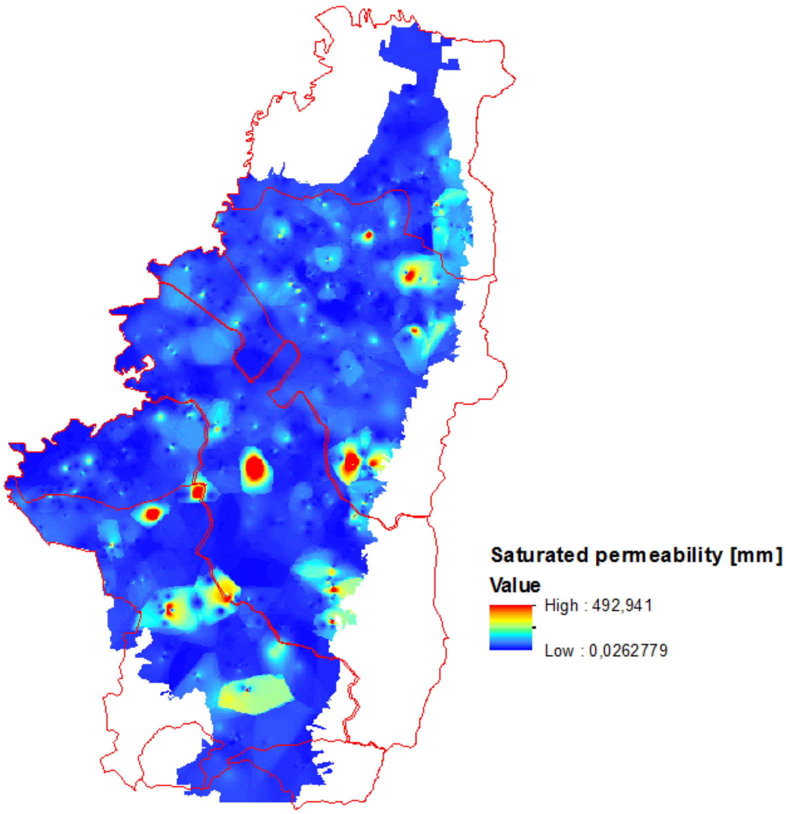

Figure 2.2 Saturated permeability in mm over the urbanised parts of Bogota (Figure source: Bogota Water Utilities)

As mentioned in chapter 1.4, the population increase in Bogota for the past century has seen remarkable numbers. The figure below, 2.3, shows the increase over time for the urban agglomeration of Bogota since the 1950's, and the projected values until 2030.

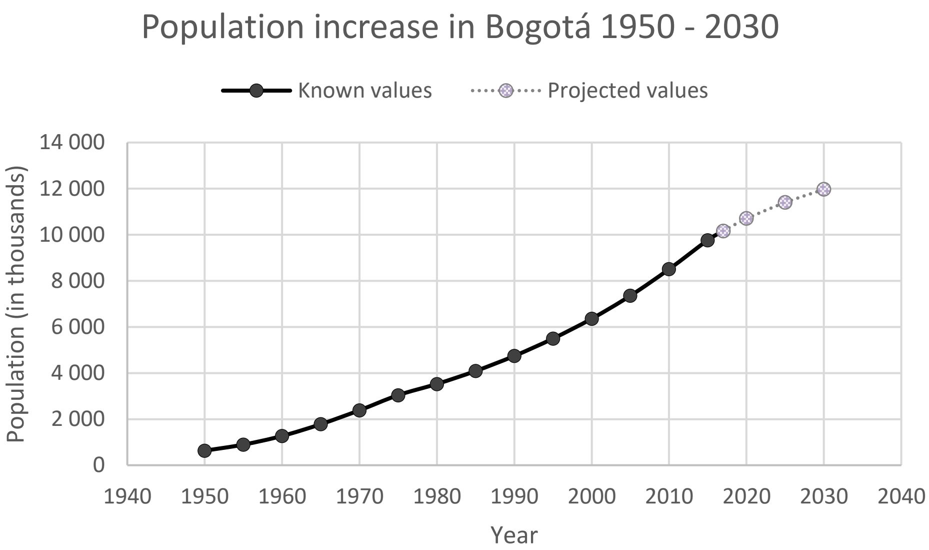

Figure 2.3 The population increase in the urban agglomeration of Bogota since 1950-2015, and the projected values from then until 2030. Figure source: http://worldpopulationreview.com/world-cities/bogota-population/

Visualised in figure 2.4 below, is the historical change in impervious cover, i.e. the urban development, over the time-span 1797-2013 in Bogota. The red borders indicate the currently urban area of Bogota divided into its corresponding catchments. The overall increase in urban development is indicated in grey.

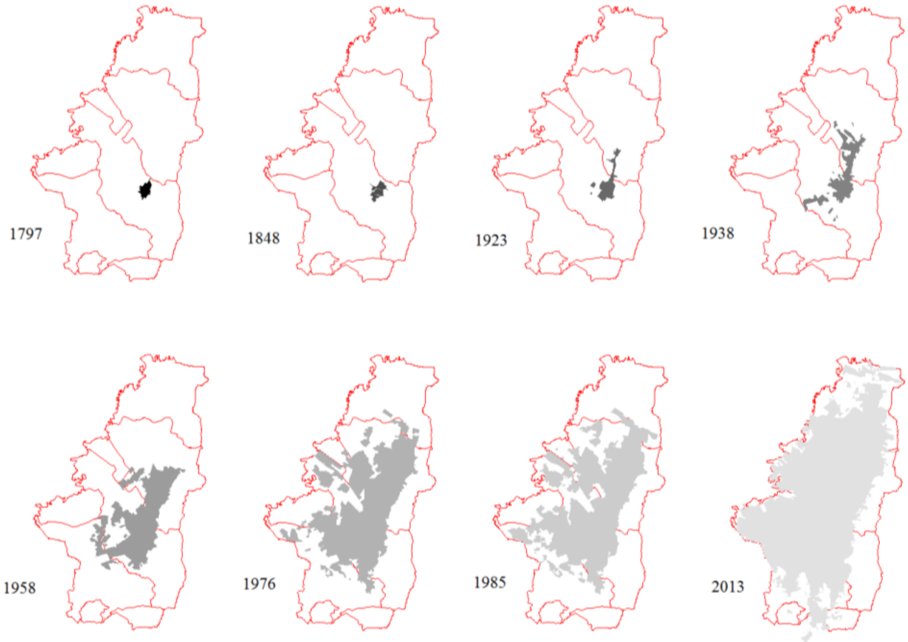

Figure 2.4 The urban development in Bogota through the years 1797-2013

2.2 Water-management through the years

Bogota has gone through significant changes in storm water management since before the Spanish conquest in the 1500's. Both the technical changes and the changes in attitude towards water, will be generally overviewed in this section, starting with a historic overview.

2.2.1 An historic overview

Before the Spanish colonisation, the wet soils and natural flooding cycles of Bogota were considered good resources, and were consequently exploited to construct extensive agricultural systems. Fields were ridged to manage extremely wet soils, and together with canals they improved drainage, served as buffer area, they controlled temperature differences, and provided good habitats for fish farming. The ridged fields came in different forms, with their own specific function. Some were believed to retain water, while others were used to drain excess water after floods. Based on the extension of the fields, and the level of maintenance required, it is believed that the construction of the fields took centuries to finalise, and required large populations. Back then, when inhabited by the indigenous population of the Muiscas, water was not only a tool for the improvement of food production, as it also had symbolic meanings, where two of the most symbolic representations of Muisca mythology were tied to water. Both symbolic representations tried to, among other things, explain the flood events and their natural cycles.

Shortly after the Spanish inquisition, in the 16th century, a change in settlement patterns could be noted. Foreign, modern agricultural techniques were introduced and replaced the traditional ones, which resulted in a change of land tenure. The growth of new villages occurred slowly, but nevertheless they over-exploited the landscape potential, increasing simple water supply systems, and increasing the effect on ecosystems.

Despite changes to agricultural systems, the changes to water management systems were deemed low and insignificant up until the 19th century. At the end of said century, laws were introduced to allow for drainage of lakes, wetlands, and marshes to enable for agricultural and livestock production, and as a measurement towards flood control. The construction of ditches made it possible to use and exploit flood prone areas. In 1887, it was established that water was for public use only; private individuals could use it but not claim rights of it. Unlike water, private ownership over land was possible, which enabled land-users to manipulate water streams and allowing them to drain water bodies to occupy them. Other measurements taken to dry land was to introduce water-consuming plants, such as the eucalyptus plant (Rojas, et al., 2015).

Additionally, tracing back to as far as to colonial times, water bodies and sources were used to rid of urban waste, up until late 20th century when the modern waste management system was developed. The waste was dumped into the surface gutters and washed away by rain showers (Gallini, et al., 2014a). Furthermore, since bathrooms were not a common feature in everyday household before the development of the domestic aqueduct system in the beginning of the 20th century, either makeshift holes or latrines excavated around the house were used, or the relief of the people occurred on the streets. This was expected to be flushed away with the rain water to nearest sewer, which in turn would be discharged into the nearest water streams. This resulted in the rivers and water bodies of the city acting as a multipurpose water network, ridding the city of human and urban waste (Gallini, et al., 2014b).

As the population grew larger, the increasing demand for fresh water led to decreased volumes of water supplying fountains and aqueducts. This in turn generated decreasing amounts of water washing the wastes away. One consequence of the then existing water and waste management system was an increase in deaths due to water borne diseases. In 1920, 76 times more people died of the waterborne disease typhoid fever in Bogota than in United States cities. This issue was addressed in different ways. For instance, the domestic aqueduct system was implemented and expanded, which changed the relationship to water in general. Waters originating from rain or waste were now seen as *unnecessary, undesirable*, and *eventually dangerous to urban life* (Gallini, et al., 2014a). Water became easily accessible through the households of the city's inhabitants, meaning that local rivers and rainwater were no longer needed for bathing. Consequently, the water demand increased, but also the amounts of used, misused, and polluted waters. The continued growth of the city caused previously unused and untouched water bodies to be included within the city limits; wetlands that used to be visited during holidays for recreational purposes were now dried out, constructions took over fertile farmland, and the nearby rivers were integrated into the urban landscape, receiving discharges of wastewater (Gallini, et al., 2014b). Other consequences of the increased waste amounts in water bodies included bad odour and unpleasant appearance around the river and river banks. To address these issues in the first decades of the 20th century, large rivers were channelized, and the constructions of underground sewerage systems were initiated. However, due to scarce municipal funds and weak coordination of the works from the municipal's side, the sewerage systems were found to be *weak, defective, and incomplete* (Gallini, et al., 2014a).

Figure 2.5 shows three different parts of the city where a river has been channelized. As can be seen, these channels come in different sizes, ranging from rather small channels, suited for residential areas with closely living populations, to larger ones in the middle of traffic. Discharges are added through pipes, as can be noted most distinctly in the picture to the upper right and the picture below.

Figure 2.5 Parts of the same channelized river in different parts of the city.

In 1959, the first large-scale water treatment plant in the Bogota river-basin was introduced. It was constructed upstream the city, and has since been upgraded to increase its capacity from 3.5 to 5 m^3/s. Despite this, the river has seen declining water quality and increasing flows, due to the rapidly growing city of Bogota (The World Bank, 2012).

2.2.2 Water, waste- and storm-water management today

Today, an average of 130 litres of water are consumed per person and day in Bogota. This can be compared to the average figure of 250 l/day and capita in Latin America, where some cities reach over 400 l. The reason for this, in comparison, low consumption of water is the extended development of the water management systems in the city, finalised in 1997, which drew up the tariffs on water consumption. The tariffs payed in Bogota are some of the highest in all Latin America.

The water and sewerage company, *Empresa de Acueducto y Alcantarillado de Bogota* (EAAB), is the organ responsible for water supply and sewage management in Bogota. The public company was founded in 1955, and the initial years of its establishment were focused on securing sufficient raw water supplies for the rapidly growing city. Additionally, they were responsible for providing water supply, sewerage, and bulk drainage services. Today, more than 99% of Bogota is supplied with potable drinking water through EAAB. The effort put on improving water supply and sanitation has given great results. However, the new approach towards water management did not take into account neither domestic nor industrial wastewater treatment, or sources of pollution leading to the degradation of water bodies. One of the main environmental issues that Bogota is suffering from today is untreated wastewater being discharged into the Bogota river. Wastewater is added through three main tributaries: Salitre, Fucha, and Tunjuelo. The average dry flow receives 22 m^3/s of additional water as it moves through the city, and only 20% of these flows are subjected to primary treatment at the Salitre treatment plant, located in the northern parts of the city (The World Bank, 2012).

2.3 Defining the current state of the area of study

As mentioned previously, the study is limited to the most urbanised and densely populated areas of Bogota. This area of Bogota covers eight river basins which all eventually drain into the main river of the city, the Bogota River, flowing along the west border of Bogota. The river basins and their respective rivers, including the Bogota River, are shown in figure 2.6.

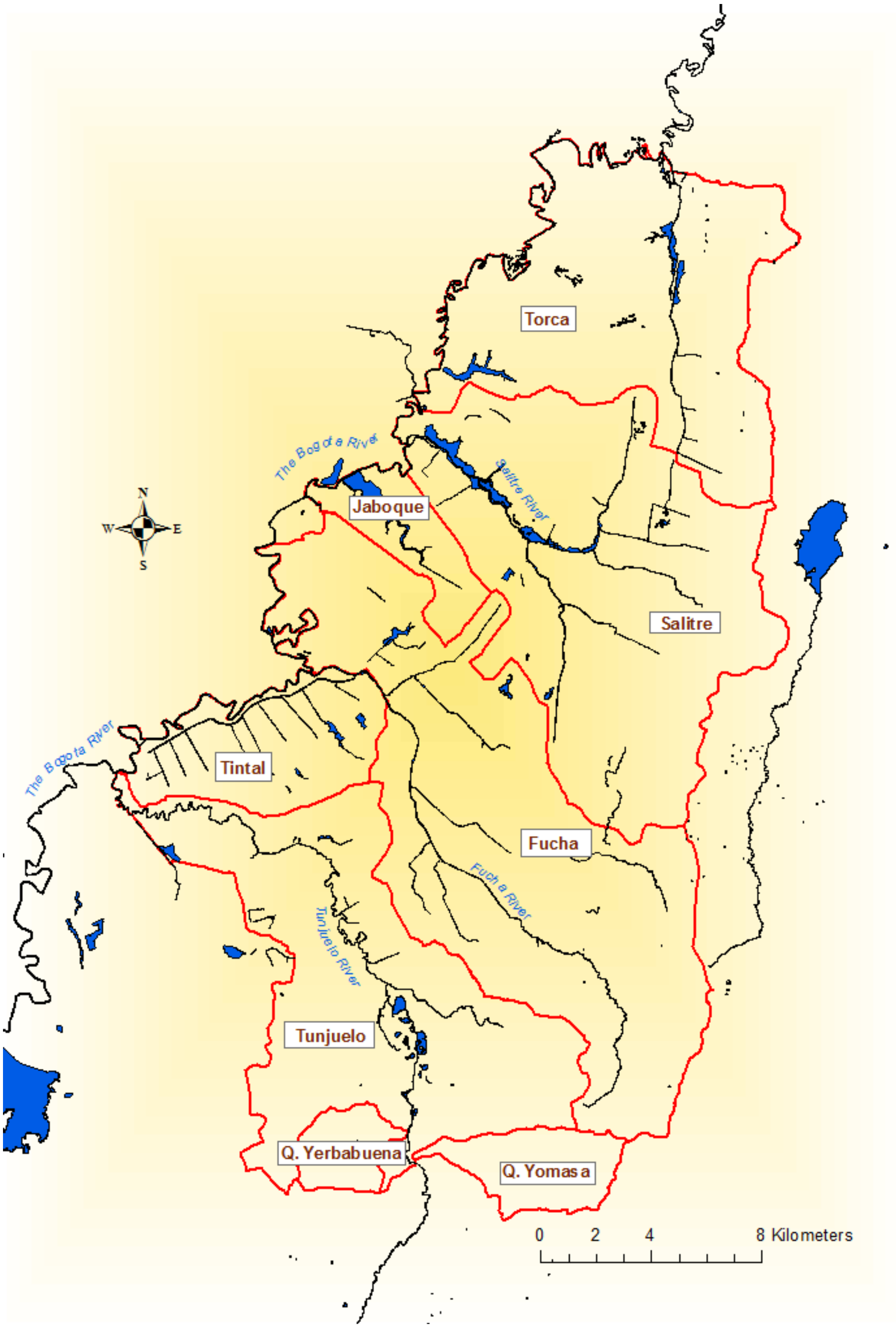

Figure 2.6 The study area divided into its corresponding catchments, and the most important water bodies marked out in blue.

The wetlands in the Torca, Salitre, and Tunjuelo basins will not be taken into account in the present analysis. More information regarding this will be presented later on in the report.

Table 2.1 gives detailed areal information about each river basin, in km^2.

Table 2.1 Areal information in km^2 about the studied river basins

Basin	Area [km^2]
Fucha	148
Salitre	133
Tunjuelo	107
Torca	98
Tintal	34
Jaboque	16
Quebrada Yomasa	15
Quebrada Yerbabuena	9

From the table, it can be noted that there is a rather significant difference in size between some of the catchments. For instance, the Fucha basin constitutes the largest river basin, with its 148 km^2, while Quebrada Yerbabuena makes up the smallest one, with its 9 km^2.

Fucha, Salitre, Tunjuelo, and Torca are the main catchments in Bogota, as they together cover almost 87% of the total surface considered in this study. Additionally, they include the main tributaries to the Bogota river with the Fucha river, the Salitre river, the Tunjuelo river, and the Torca canal, respectively. Due to this, focus in information gathering has been put on these four drainage areas. However, all basins as mentioned in table 2.1 have been included in the study. A more thorough overview of these catchments will be presented below, including an overview of the historical change in impervious cover for each respective area.

The urbanisation in Bogota was initiated in the Fucha river basin. It eventually evolved through the Salitre and Tunjuelito river basins, before reaching Tintal, Jaboque, and Torca respectively. The smallest river basins, Quebrada Yomasa and Quebrada Yerbabuena, did not see any development of urbanisation until sometime after 1985.

Below, some information regarding the different sub-basins will be presented. For the four largest basins – the Fucha, Salitre, Tunjuelo, and Torca basins – the information presented is retrieved from *Calidad del Sistema Hídrico de Bogota* (Alcaldía Mayor de Bogotá & Empresa de Acueducto y Alcantarillado de Bogotá, 2008), which have based their figures on studies made in 2006.

The following information includes water quality figures, among other things, which is a way of giving some background information regarding the current state of the watersheds and water bodies, and how they are used.

2.3.1 Fucha river basin

With an area of 148 km^2, the sub basin of the Fucha river constitutes the largest sub basin in the studied area. Additionally, it is the catchment where the urban development of Bogota first began, as shown in figure 2.4 The catchment borders with the Andean mountain chain to its East, and the Bogota river to its West. Water is drained from both the west and the east, to the southern-centrally located river of Fucha. Through this, the water is eventually delivered to the Bogotá River. Some parts of the river are channelized in concrete. Content is added to the river through several connections, as can be noted in figure 2.6 above.

In this area, both combined and separate storm water- and sanitary networks are used. The total length of these systems amounts to about 1,787 km. The combined sewage system dominates the eastern part of the area, draining to the areas with separate systems. Similarly, combined systems dominate the oldest parts of the basin as well.

Throughout the river of Fucha, the main recipient in the Fucha basin, a general increase in BOD_5 concentration can be noted as the river runs through the city. The levels of BOD_5 in the river are generally relatively high, and seen as a consequence of discharges of wastewater of both domestic and industrial origin being made to the river throughout its path. In a similar manner, an increasing pattern of the amounts of COD in the river have been noted.

The main uses of water resources in the Fucha river basin include agricultural, livestock, recreational, and uses for the preservation of aquatic life.

The increase in urbanised area in the Fucha basin has increased from 2% in 1797 to 69% in 2013. The gradual change in urban development can be observed in figure 2.7 below:

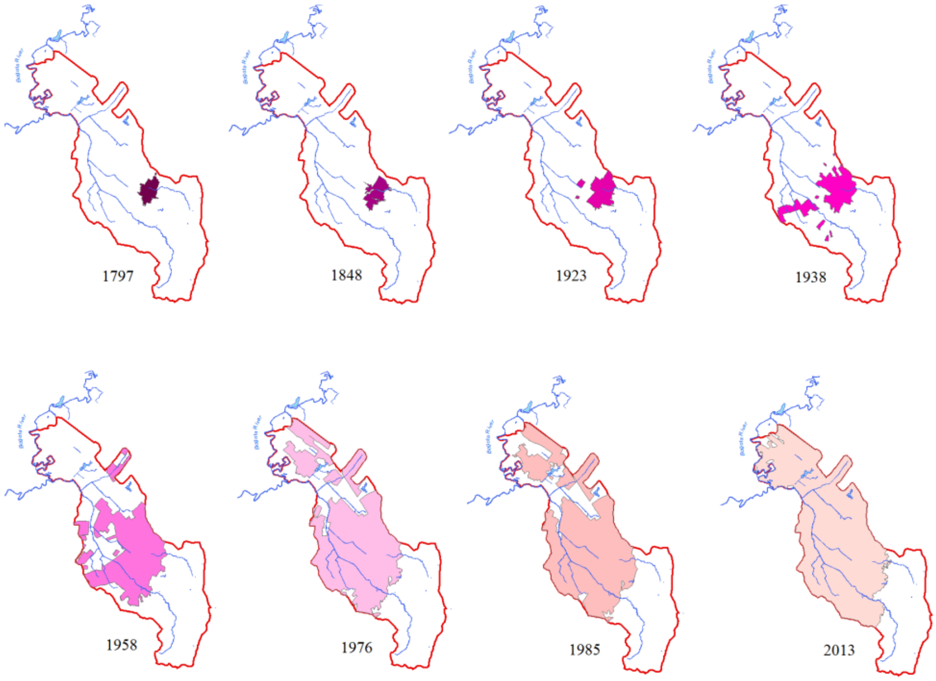

Figure 2.7 The urban development of the Fucha basin through the years 1797-2013

2.3.2 Salitre basin

The Salitre basin, the second largest in this study with an area of 133 km^2, is located in the central-northern parts of Bogota – just north of the Fucha basin. The main sources of pollution in this area is domestic wastewater being discharged into the river, contributing with organic matter, TSS, total coliforms, and *E.coli*.

The BOD_5 levels in the river are said to be held at relatively low levels. Similarly, as in the Fucha river case, the higher levels of BOD_5 were coupled to the points at which the main discharges of domestic wastewater into the river occur. A general increase in COD levels along the river could also be noted. The levels of COD indicate, in this case also, anthropic influence on the water body in the form of discharges of domestic residual origin. Additionally, other water bodies discharge into the river, contributing with polluted waters. Due to the amounts of organic matter present in the river, a general decrease in dissolved oxygen has been noted along the path of the it. This is because of the consumption of oxygen by the organic matter present in the water. Higher values of DO could be noted during the night, which is explained as a respond to the decreased amounts of residual water entering the river.

Generally, the water quality in the river sees a decrease along the river, with good quality water in the higher parts of it, and poor levels in the end. The poor-quality water is then delivered into the Bogota river.

The urban development between the years 1923-2013 is noted to 77%, and can be seen in figure 2.8 below:

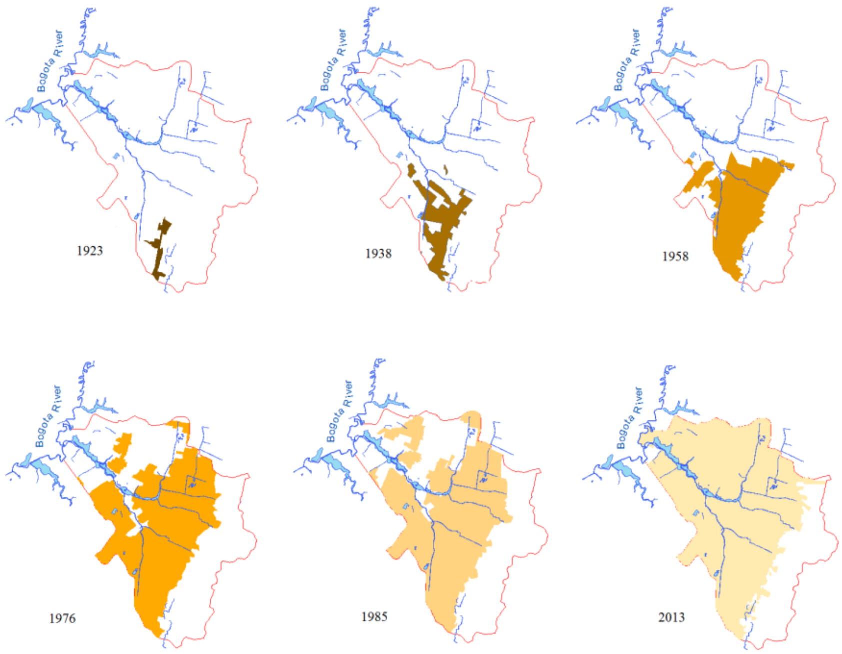

Figure 2.8 Urban development of the Salitre basin through the years 1923-2013.

2.3.3 Tunjuelo basin

The Tunjuelo river basin can be divided into two parts: the Western flatter part known as Tunjuelito Bajo, and the Eastern-Southern part which corresponds to the mountainous part known as Tunjuelito Alto. The events and developments in the Tunjuelo river basin are believed to have had a lot of impact on erosion, particularly due to the intense use of extractive materials for construction, and due to the inadequate housing developments, that this basin is characterized by.

The main source of pollution in the Tunjuelo River is the discharge of wastewater. Additionally, leachates from the treatment plant at the landfill Doña Juana generates discharges into the river, as well as large solid inputs from quarries near the river. Throughout most of the river, wastewater of both domestic and industrial origin is discharged into the river. If not discharged directly to the river, pollution is reached through the discharge into streams and various water bodies connected to the river.

Studies of BOD_5 amounts in the river show lower amounts in the initial parts of the river but higher in the end, similar to previously studied river basins and rivers. The DO showed a decreasing trend over the course of river, until it reached anoxic conditions in the end.

Similar to the previously mentioned rivers, a spatial gradient of the water quality in the river with a tendency towards degradation along its path, from its initial point to its final point discharging in the Bogota river, occurs.

From 0.09% urban areas in 1938, the urban development in the Tunjuelo basin reached 65% in 2013. This can be visualised in figure 2.9 below:

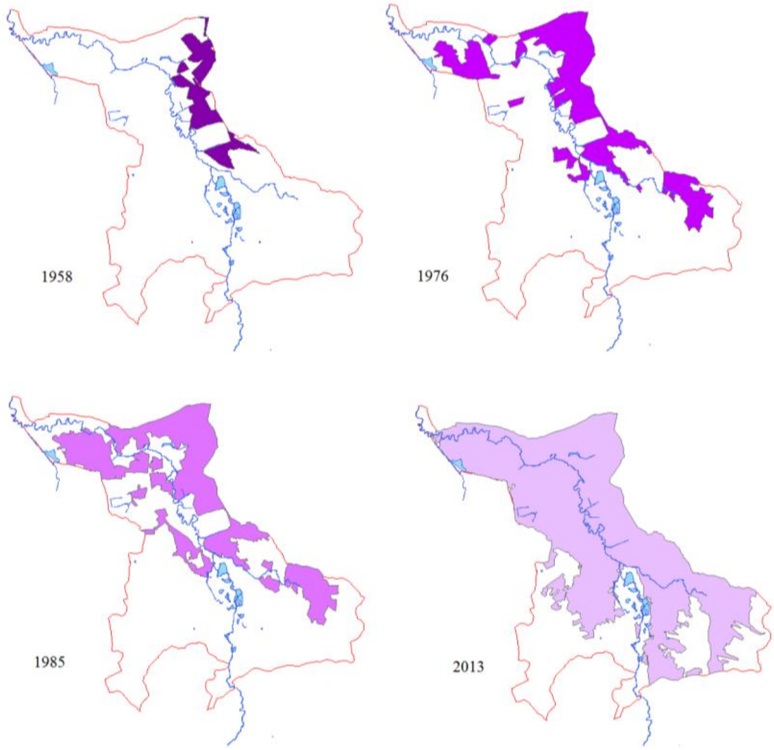

Figure 2.9 Urban development of the Tunjuelo basin through the years 1958-2013

2.3.4 Torca basin

The Torca basin area amounts to 98 km^2, and is located in the most northern parts of Bogota. In the western part of the basin, the sewage system consists of a sanitary system based on the interception of the Bogota-Salitre-Torca rivers, which bring wastewater to the Salitre treatment plant, and a rainwater system which receives water from some channels before moving to the Torca wetland, which eventually delivers the water to the Bogota River.

The main source of pollution in the Torca channel consists of sewage contamination from both sanitary and storm water origin. The quarries located in the north-eastern parts of the city constitute an additional source of pollution. Large amounts of solids are transported from these quarries, through various streams, into the Torca channel.

Its gradual increase in urban areas, from 4% in 1976 to 45% in 2013, can be noted in figure 2.10 below:

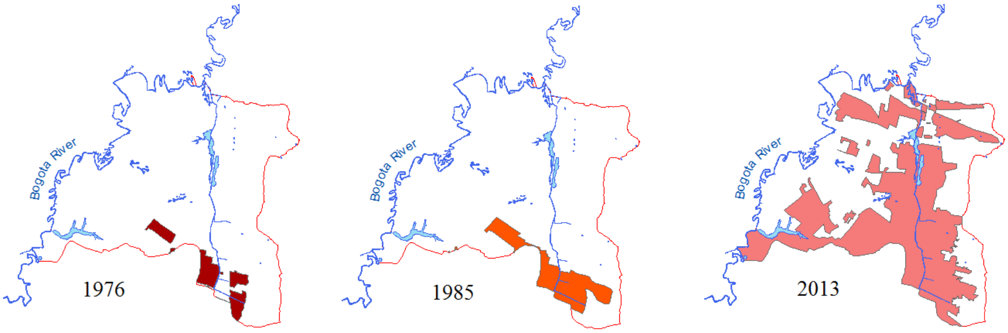

Figure 2.10 Urban development of the Torca basin through the years 1976-2013

2.3.5 Tintal, Jaboque, Quebrada Yomasa and Yerbabuena basins

The urban development of the Tintal basin, with its 34 km^2, has seen figures increase from 12% in 1976 to 81% in 2013, which can be visualised in figure 2.11 below:

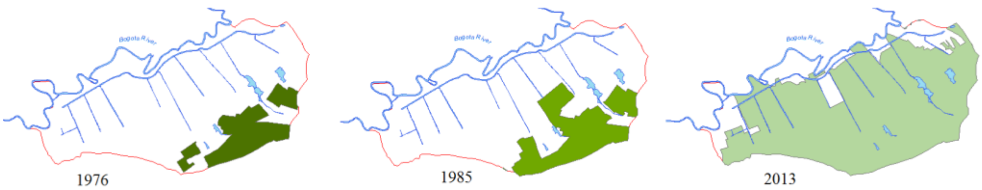

Figure 2.11 Urban development of the Tintal basin through the years 1976-2013

The main canal in the Tintal watershed, also known as the Cundinamarca canal, runs parallel to the Bogota river. The constructional work for the development of this canal began in 2000. It collects rainwater from the perpendicularly connected canals, which can also be viewed in figure 2.11, and wastewater through sewerage interceptors. The main canal depends on pumping stations for the water to be evacuated into the Bogota river. Additionally, three of Bogota's 14 remaining wetlands are situated in the watershed (Rojas, et al., 2015).

The area of the Jaboque basin amounts to about 16 km^2, making it one of the smallest in the studied area. Its urban development has increased from 26% in 1976, to 80% in 2013. Figure 2.12 below shows the urban development of Jaboque between said years.

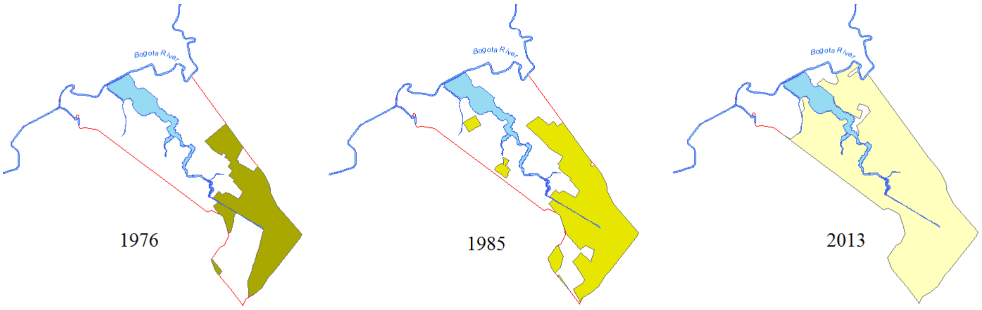

Figure 2.12 Urban development of the Jaboque basin through the years 1976-2013

The major water stream in the Jaboque basin mainly consists of a wetland.

Regarding Q. Yomasa and Q. Yerbabuena, no urban development was detected in those areas before 2013, for the years studied. Thus, their respective urban development has gone from none to 27% for Q. Yomasa, and to 3% for Q. Yerbabuena. The figure below gives a visual understanding of how much of the total basins that these figures represent.

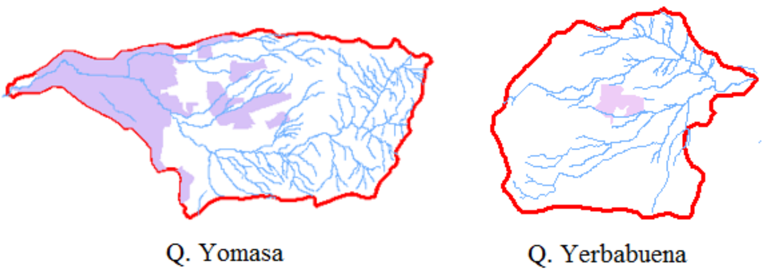

Figure 2.13 Urban development of the Q. Yomasa and Q. Yerbabuena basins, respectively, in 2013

In table 2.2 below, data regarding the urban development in percent from each year and for each catchment is summarised:

Table 2.2 Summation of the urban development of each catchment for each year, in %

	% Urbanised area of total basin area							
Year	Fucha	Salitre	Tunjuelo	Tintal	Jaboque	Torca	Q. Yomasa	Q. Yerbabuena
1797	2							
1848	2							
1923	4	2						
1938	8	8	0,09					
1958	28	21	6	0,15				
1976	51	49	20	12	26	4		
1985	52	51	26	18	37	7		
2013	69	77	65	81	80	45	27	3

Completed with figure 2.14 below, showing the increase in percentage urban development for each respective catchment:

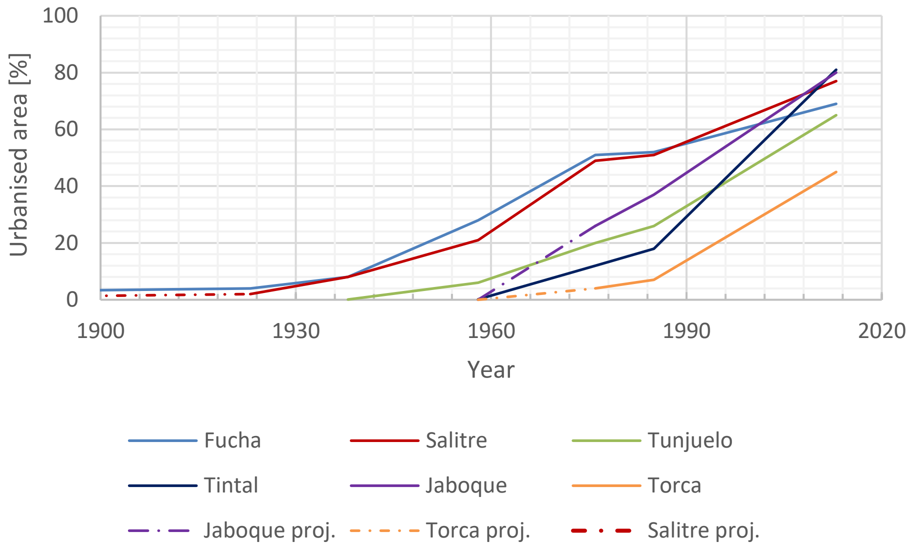

Figure 2.14 Urban development, in %, for the studied years for each respective catchment

The dotted lines show the estimated development until the solid line, since the maps used in this study did not cover the change in land use cover for those basins, before that.

Chapter 3
Methodology

This chapter aims to give a brief review of the theory behind the formulas and relationships used to estimate runoff peak flows, for both the pre- and post-urban scenarios. In connection to this, the methodology used in this project will be presented.

3.1 Runoff and the general water balance

Runoff is created after precipitation has evapotranspired or intercepted (Bengtsson, 2004). It occurs in three different main flow paths: Q_o=overland flow, which is the *water that flows over the ground surface to stream channels [...],* throughflow, Q_T, and groundwater flow, Q_G (Ward & Robinson, 2000).

The general water balance for a catchment is a simple, mathematical way of describing the hydrological cycle, mainly defined as the following equation:

$$P = R + E \pm \Delta S \qquad \textit{Eq. 3.1}$$

Where P = precipitation, R = runoff, E = evaporation, and ΔS = difference in storage.

Which can be summarised in the following formula:

$$I - O = \Delta S \qquad \textit{Eq. 3.2}$$

Meaning that I = input into the system, minus O = output from the system, gives the difference in storage (Ward & Robinson, 2000). This has been visualised in figure 3.1 below:

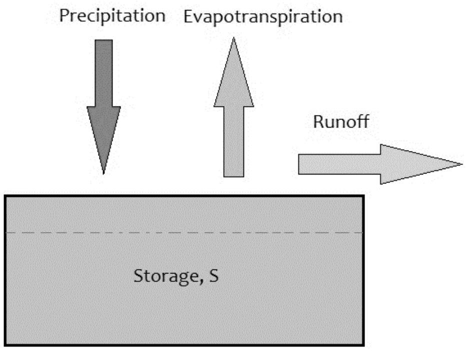

Figure 3.1 A simple visualisation of the general water balance

The runoff that has been studied in this project, and on which calculations have been made and shown for below, is the overland runoff, meaning that only surface runoff is taken into account. More specifically, the peak flows of this runoff have been studied.

3.2 Estimating effective precipitation using the Curve Number (CN) method

The CN method was established by the Soil Conservation Service (SCS) in the United States in 1986. It uses the total depth of a rainfall, P, represented by a design storm hyetograph, to calculate the depth of the corresponding surface runoff. It includes the following parameters:

- I_a= initial abstractions: all losses occurring before the initiation of surface runoff, such as interception, infiltration, and depression storage [in, mm]
- S= maximum possible retention [in, mm]
- P_e= effective precipitation: the part of precipitation that becomes surface runoff, i.e., the runoff expressed in depth [in, mm]

Which can be summarised in the following formula, used to calculate the total depth of surface runoff created by a specific precipitation event:

$$P_e = \frac{(P - I_a)^2}{(P - I_a) + S} \qquad \textit{Eq. 3.3}$$

The soil type and land-use conditions of a drainage basin are taken into account in the S-parameter, which does not include any initial abstractions (Durrans & Haestad Methods, 2003). However, it is assumed that the initial abstractions are a fraction of the maximum possible retention, giving the following relationship:

$$I_a = \gamma S \qquad \textit{Eq. 3.4}$$

Where γ is a dimensionless figure, representing the fraction of S that becomes initial abstractions. With this relationship, eq. 3.5 becomes (Soulis & Valiantzas, 2012):

$$P_e = \frac{(P - \gamma S)^2}{P + (1 - \gamma)S} \qquad \textit{Eq. 3.5}$$

On average, $I_a = 0.2S$, giving an average value of $\gamma = 0.2$. However, it might be appropriate to adjust γ depending on the hydrological conditions of a catchment. In heavily urbanised areas, where interception, depression storage, and infiltration – the parameters included in I_a – can increase the amount of rainfall being subjected to direct runoff, the portion of maximum retention becoming initial abstractions may be lower. The opposite goes for the reversed situation (Durrans & Haestad Methods, 2003).

CN is a parameter defining the land use conditions and surface conditions, which have a significant effect on the produced runoff (Durrans & Haestad Methods, 2003). This information, together with information regarding soil type, makes it possible to determine the fitting CN for a basin. CN can be expressed in terms of S, as seen in eq. 3.6 (expressed in SI-units):

$$CN = \frac{25{,}400}{S + 254} \qquad \textit{Eq. 3.6}$$

Consequently, if the CN of an area is known, S can be expressed in terms of the CN, as seen in eq. 3.7 (Soulis & Valiantzas, 2012):

$$S = \frac{25,400}{CN} - 254 \qquad \textit{Eq. 3.7}$$

To specify the soil type of an area, four groups representing different soil types have been set up. The groups range from A through D, where group A includes soils that have been classified as well-drained sand and gravels, with high infiltration rates. Group D includes the opposite types of soils; those having poor infiltration properties, commonly composed of clays, shallow soils over nearly impermeable material, and soils with a high water-table. Group B and C are in the midrange. Table 3.1 below gives the CN and soil group for different types of agricultural lands.

Table 3.1 Cover description and CN for hydrologic soil group for agricultural land, for CN determination, as explained in Durrans & Haestad Methods (2003)

Cover Description		CN for Hydrologic Soil Group			
Cover type	Hydrologic condition	A	B	C	D
Pasture, grassland, or range-continuous forage for grazing	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow continuous grass, protected from grazing and generally mowed for hay		30	58	71	78
Brush - brush-weed grass mixture with brush the major element	Poor	48	67	77	83
	Fair	35	56	**70**	77
	Good	30	48	65	73
Woods-grass combination	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30	55	**70**	77
Farmsteads - buildings, lanes, driveways and surrounding lots		59	74	82	86

Using retrieved precipitation maps with isohyets over Bogota (INGETEC - Acueducto, 2015) average data on precipitation, P, for 3-, 5-, 10-, and 25-year events for durations of 15, 30, 60, and 90 minutes respectively, could be established. These figures were obtained as average numbers for the total area of each catchment, meaning that constant and even rainfalls over the catchments were assumed. The isohyetal maps gave information in mm fallen precipitation. The 30-min rain event was chosen for the model, as this gave reasonable figures of rainfall events with neither too high nor too low intensity, and it is these figures for each catchment that are presented in table 3.2 below.

Table 3.2 Average precipitation information for T_r=3, 5, 10, and 25 years, for D=30 min, for each catchment

T_r	3	5	10	25
	P [mm]	P [mm]	P [mm]	P [mm]
Torca	48	53	60	70
Salitre	58	60	68	75
Fucha	48	53	58	70
Tunjuelo	38	40	45	50
Q. Yomasa	43	48	53	65
Q. Yerbabuena	38	38	45	50
Tintal	38	40	50	55
Jaboque	48	53	63	75

The SCS CN method was then used to determine the effective precipitation for each of the rainfalls, for each of the catchments. With an assumed value of CN=70 for pre-development conditions, according to historical descriptions of the land use before urbanisation, and an average γ=0.2, S could be determined and put into equation 3.7 above. This enabled the calculation of effective precipitation, P_e, for pre-developent conditions using equation 3.5.

3.3 Dividing the catchments into sub-catchments

To obtain flow-data giving a fair representation of each catchment, the catchments of Fucha, Salitre, Tunjuelo, and Torca were divided into sub-catchments. Additionally, the Q. Yomasa and Q. Yerbabuena catchments were included in the greater Tunjuelo sub-catchment. The division depended on the water movement in the basin, as flow-measurements in one specific point in the catchment were to be made where all water from upstream finally gathered. Contour curves and the drainage network were used to determine the drainage divide for the different sub-catchments.

No division into sub-catchments was deemed necessary for the Jaboque and Tintal basins, as the flow seemed to follow straight-forward patterns. Even though the main water path in the Jaboque basin includes a wetland, it has been taken into account due to the, in comparison, small size of the basin. This is further discussed later.

Dividing the Tunjuelo basin

In this case, the placement of existing measurement-stations helped find well-placed borders for the sub-catchments. The division of the Tunjuelo basin was probably the most interesting one, since it included a measuring-station with enough information to validate the Urban Peak Flow Formulas that will be used later on. This station, the Chiguaza station, is marked in figure 3.2 below. The sub-basin was given the same name as that station.

The dotted lines in the same figure, show the inclusion of Q. Yomasa and Q. Yerbabuena in the greater basin, here called the Tunjuelo basin. The measuring station downstream, also marked as a pink dot, is called Puente (Pte) Bosa, and this name will be used when needed to specify which station is used. However, Pte Bosa includes the total area from upstream, including Q. Yomasa, Q. Yerbabuena and Chiguaza, up until that point. The data from the Pte Bosa station will later be used to show another correlation between urbanisation and runoff flows.

Figure 3.2 Division of the Tunjuelo basin into sub-basins, with the Chiguaza and Puente Bosa stations marked with pink dots

The runoff produced below the Pte Bosa station was not studied, as those flows could not be centred to one point in the post-development scenarios, since this area was drained by means of sewer systems. The area that was excluded is filled in grey in the figure above.

Dividing the Salitre basin

The Salitre basin was divided into two parts, which both reached the Salitre wetlands located in the centre of the basin. The lower part, below these two basins, was not studied for similar reasons as for the lowest part of the Tunjuelo basin: the runoff did not have a single gathering-point because of the elevation, and additionally, the wetlands which initiated that part made it hard to get a fair representation of the flow-patterns. For that reason, the catchment was divided into two parts in the upper basin: Salitre North, $Salitre_N$, and Salitre South, $Salitre_S$.

Figure 3.3 The division of the Salitre basin into sub-basins

As can be seen above in figure 3.3, $Salitre_N$ represents the area with red borders, while $Salitre_S$ represents the area with black borders. Just like with the Tunjuelo basin, the area filled with grey is the area excluded from the study.

Dividing the Fucha basin

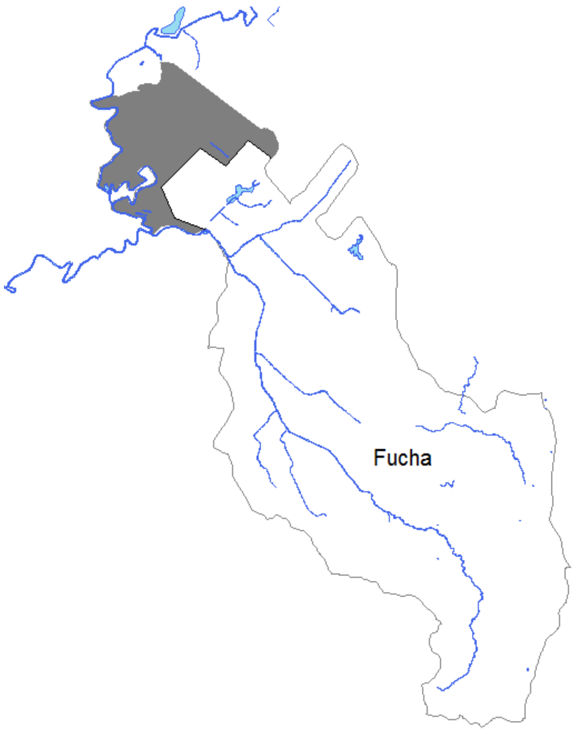

Figure 3.4 The division of the Fucha basin into sub-basin

In a similar manner, a part of the Fucha basin was excluded from the study due to flow-patterns being affected by storm-sewers being introduced in post-urban scenarios. The grey area in figure 3.4 above marks the area that was excluded.

Dividing the Torca basin

Due to certain differences in elevation and to the wetland in the lower part of the Torca basin, the division shown in figure 3.5 represents the part of the basin subjected to this study. The grey area indicates what part of the basin was excluded from the study.

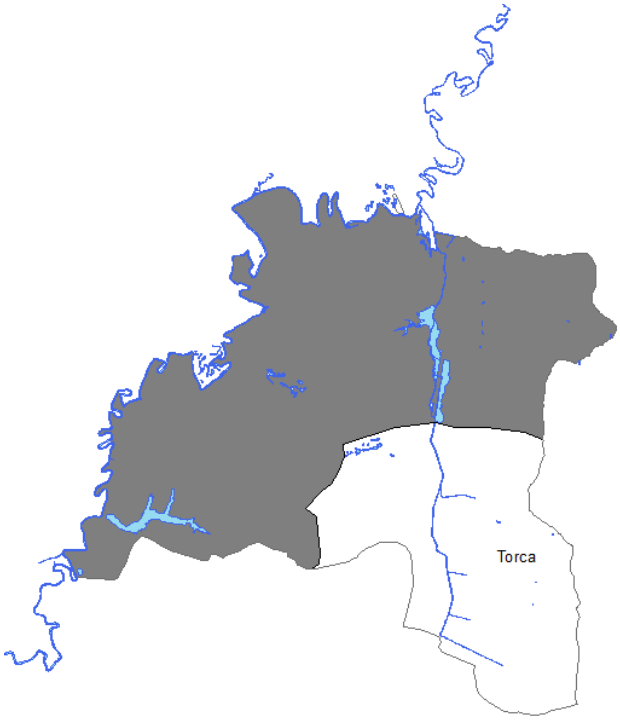

Figure 3.5 The division of Torca into sub-basin

Areal information about the new sub-catchments

The division of the basins into sub-catchments meant new areas for the study. These are compiled in table 3.3 below:

Table 3.3 Areal information in km^2 about the sub-basins subject to the study after division

Sub-catchment	**Area [km^2]**
Tunjuelo (Pte Bosa)	126
Chiguaza	14
Salitre$_N$	49
Salitre$_S$	58
Fucha	131
Torca	31
Jaboque	16
Tintal	34

It is these sub-basins that have been subjected to the study, and all information after this is applied to these areas.

3.4 Taylor-Schwarz (1952) for determination of average basin slope

There are some methods and formulas developed to determine the average basin slope, many of them depending on the basin length and at least two elevation points. The Taylor-Schwarz formula from 1952 accounts for the basin length, L_w, and divides it into lengths, l. The total average basin slope, Y, depends on the different slopes of each l.

Firstly, the basin length is defined as *the total distance [...] from the point of interest to the highest point on the basin boundary following the main-channel route* (Granato, 2012). It is primarily measured by following the *fork* that has the largest drainage area. In figure 3.6 below, the basin length is visualised as the dotted line going along the whole catchment:

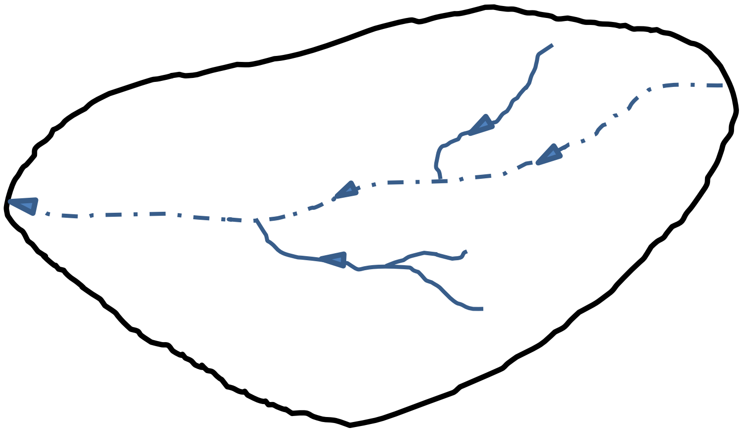

Figure 3.6 The basin length of a catchment, according to Granato (2012), visualised as the dotted line

The Taylor-Schwarz formula is defined as:

$$Y = \frac{L_w}{\left(\frac{l_1}{\sqrt{y_1}} + \frac{l_2}{\sqrt{y_2}} + \frac{l_3}{\sqrt{y_3}} + \cdots + \frac{l_n}{\sqrt{y_n}}\right)^{0.5}} \qquad \textit{Eq. 3.8}$$

Where Y is the average basin slope, and y_n is the slope of each l_n.

The principle of the formula can be visualised in figure 3.7 below, showing the cross section along the basin length of a drainage basin:

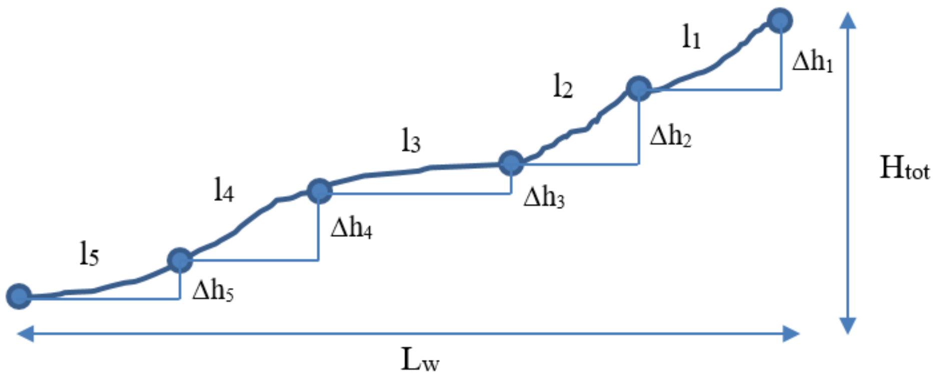

Figure 3.7 The principal of the Taylor-Schwarz formula along the cross-section of a drainage basin

Where the slope for each l is calculated using equation 3.9:

$$y = \frac{\Delta h}{l} \qquad \textit{Eq. 3.9}$$

The basin length of each sub-catchment was determined as explained above for all catchments – with a reminder that Q. Yomasa and Q. Yerbabuena were both included in the Tunjuelo (Pte Bosa) river basin after the division – with the focus on finding the fork with the largest drainage area.

By using maps of each catchment in ArcMap, with their corresponding main water course, and additional water stream information when needed, and elevation lines, the basin length could be determined using the program's measure-tool. The basin length was determined according to figure 3.7 above.

To determine the basin slope, the obtained L_W of each catchment was divided into lengths l of 10% of its total length, and the point of the beginning and end of each l was marked, see figure 3.7. Using elevation curves, the elevation at each point could then be determined and used to estimate the slope y of each l. These figures were then used in the Taylor-Schwarz formula, eq. 3.8. It was believed that this was a good method to use in order to take into account large variations in elevation.

Average basin slope of Jaboque and Tintal

Due to their comparatively small areas and relative flat composure, the average basin slopes of Jaboque and Tintal were determined in a more similar manner than that explained for the catchments above. Following the basin length of each of the basins, and taking the start-point and end-point of it, noted h_1 and h_2 respectively, the slope was calculated with formula 3.10:

$$Y = \frac{h_1 - h_2}{L_W} \qquad \textit{Eq. 3.10}$$

3.5 SCS triangular unit hydrograph for pre-development peak flow estimations

The unit hydrograph is a tool commonly used to describe the resulting runoff from a 1-inch (or 1 cm) rainfall, generated uniformly over the catchment at a constant rate for an effective duration. It was first proposed by Sherman in 1932, and should be used only with surface runoff determinations (Chow, et al., 1988). The model is linear and can be used to derive the hydrograph from any excess rainfall amounts. Additionally, the model is based on a few basic assumptions that under natural conditions cannot be perfectly met. Nevertheless, the results from the unit hydrograph are generally deemed acceptable for practical purposes, given that carefully selected hydrological data has been used. According to Chow et al. (1988) the following assumptions are made when generating a unit hydrograph:

1. The excess rainfall has a constant intensity within the effective duration
2. The excess rainfall is uniformly distributed throughout the whole drainage area
3. The base time of the duration of direct runoff (DRH) resulting from an excess rainfall of given duration is constant
4. The ordinates of all DRH's of a common base time are directly proportional to the total amount of direct runoff represented by each hydrograph
5. For a given watershed, the hydrograph resulting from a given excess rainfall reflect unchanging characteristics of the watershed

The model was intended for larger watersheds when developed, but has been successfully applied to basins smaller than 0.5 hectares to 25 km^2. It is also considered not applicable for runoff originating from snow or ice.

Figure 3.8 below shows a general unit hydrograph and the different parameters included.

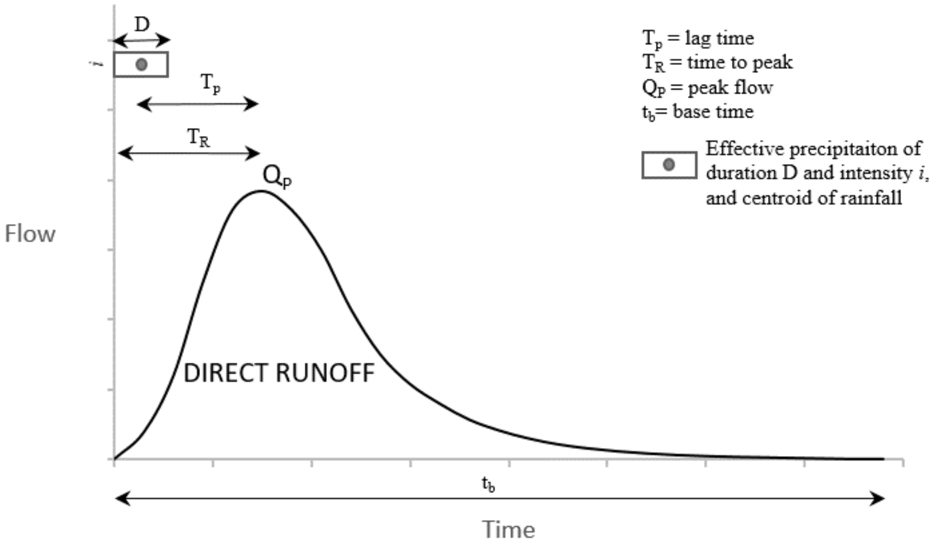

Figure 3.8 A general unit hydrograph

The area under the graph represents the amount of direct runoff produced for the studied effective rainfall. Since the unit hydrograph describes the resulting runoff from a 1-inch rainfall event, spreading out the amount of direct runoff over the area, should give a height of 1 inch (Durrans & Haestad Methods, 2003).

There are several ways of generating a unit hydrograph, all depending on what type of information is available. One of them is the SCS triangular hydrograph, which gives a triangular form on the unit hydrograph with t=0, $Q=Q_p$, and $t=t_b$ being the three corners of the triangle. Even if the total hydrograph is not of interest, it makes it possible to determine the peak flow of a certain rainfall for a certain catchment. The method used below follows the instructions from example 2-9 in Hydrology and Floodplain Analysis (Bedient, et al., 2008).

The time lag, T_p, is first calculated using the following formula:

$$T_p = \frac{L_w^{0.8} \cdot (S+1)^{0.7}}{1900 \cdot Y^{0.5}} \qquad \textit{Eq. 3.11}$$

Where:

L_w = basin length in ft

S = maximum possible retention in inches, as from the SCS CN formula from chapter 3.2

Y = average basin slope in %

From this information, the time to peak can be determined as:

$$T_R = \frac{D}{2} + T_p \qquad \textit{Eq. 3.12}$$

Where D is the duration of the effective rainfall expressed in hours. This can in turn can be used to determine the peak flow, Q_P, as:

$$Q_P = \frac{484 \cdot A}{T_R} \qquad \textit{Eq. 3.13}$$

Where A is the area, expressed in mi^2.

From this, a unit hydrograph similar to the one visualised in figure 3.9 is retrieved:

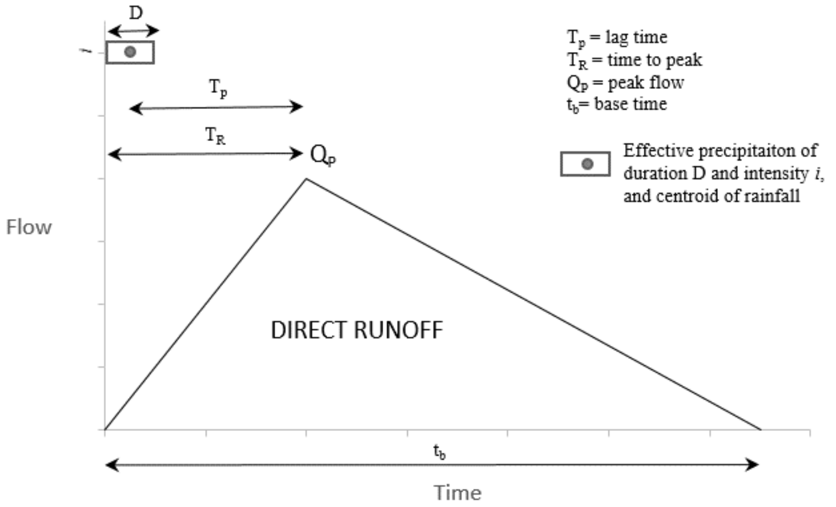

Figure 3.9 A general SCS triangular unit hydrograph

The obtained unit hydrograph is that of a 1-inch rainfall. Since the hydrograph is linear (Chow, et al., 1988), it is possible to adjust for this, and make it representative for the effective rainfalls calculated according to chapter 3.2. By calculating the share of effective precipitation of a 1-inch rainfall, the share of peak flow determined for the 1-inch rainfall, which fit with the right effective precipitation number, could be determined.

When determining the peak flows of pre-development conditions for the different catchments, P, S, and Y from equations 3.8 and 3.10, together with the known areas of the sub-basins, were used in the formula for peak flow estimations using the triangular unit hydrograph method. A D of 30 min was used. All units were in US-units, before being converted into the SI-units presented in the results section.

3.6 USGS Peak Flow Discharge Formulas for urban peak flow determinations

To calculate peak discharges in different urban stages (Urban Peak Flow, UQ) of a respective catchment, the SGS Urban Peak Discharge (UPD) formulas can be used.

The method was developed in the US and is based on a study made on 199 catchments all over the country. Variables regarding topography, climate, and land-use, were used to describe the characteristics of each river basin; the size of the water shed, channel length, valley length, stream slope, SCS soil classification, and SCS curve number classification for land-use, to mention some. Parameters describing the extent of urbanisation were also evaluated. These include for instance percentage of basin covered by impermeable surfaces, population and population density, and lag time of the basin. Additionally, the basin development factor, BDF, was developed. The BDF is a way of describing the efficiency of the drainage system of a catchment, taking the following four aspects into consideration:

I. Channel improvements
II. Channel linings
III. Storm drains, or storm sewers
IV. Curb-and-gutter streets

More detailed information about these four aspects has been compiled in appendix, chapter I.IV.I.

The BDF of a catchment is determined by dividing the catchment into thirds. If each third of the catchment fulfils the four aspects, one point per aspect is given. The points for the total catchment are then summed up as the total BDF. Four aspects and three divisions means that one catchment can obtain maximum 12 points.

For a catchment to be considered for this study, a few criteria had to be met; for instance, the watershed had to have at least 15% coverage of industrial, commercial, or residential development; reliable flood-frequency data had to be available; and, the period of calibration must have had relatively constant urbanisation, which was defined as *a change in development of less than 50% during the period of record.*

Through linear multiple regression techniques, known values of peak discharges for 2-, 5-, 10-, 25-, 50-, 100-, and 500-year floods could be related to seven independent variables. These included: area, A; channel slope, SL; basin rainfall, RI; basin storage, ST; BDF; impervious surfaces, IA; and rural peak discharge, RQ (which will later on be noted *pre-development flows*). The last one is used to account for regional differences in runoff. Of these seven variables, A, BDF, and RQ are deemed the most important variables. To reduce the amount of data and effort required for the application of the formulas, the less significant variables can be neglected, giving formulas including only three parameters – giving the so called 3-parameter equations. This does increase the standard error of regression, but for peak flows of two to 100-year magnitudes, the coefficient of determination, R^2, remains over 90%. Table 3.4 below shows the R^2 and the standard error of regression for the 3-parameter equations:

Table 3.4 Difference in and the standard error of regression for the 3-parameter equation

	UQ2	UQ5	UQ10	UQ25
R^2 3-parameter eq.	0.91	0.92	0.92	0.92
	Standard error of regression			
Log-units	0.1797	0.1705	0.1720	0.1802
Average %-units	$\pm$43	$\pm$40	$\pm$41	$\pm$43

A high value of R^2 means that the deviations and standard errors of the regression lines are small, giving a good fit of the line (Khan Academy, 2010).

The three-parameter equations derived for 2-, 5-, 10-, and, 25-year peak discharges are compiled below:

$$UQ2 = 13.2A^{0.21}(13 - BDF)^{-0.43}RQ2^{0.73} \qquad \textit{Eq. UQ2}$$

$$UQ5 = 10.6A^{0.17}(13 - BDF)^{-0.39}RQ5^{0.78} \qquad \textit{Eq. UQ5}$$

$$UQ10 = 9.51A^{0.16}(13 - BDF)^{-0.36}RQ10^{0.79} \qquad \textit{Eq. UQ10}$$

$$UQ25 = 8.68A^{0.15}(13 - BDF)^{-0.34}RQ25^{0.80} \qquad \textit{Eq. UQ25}$$

Where UQ2 is the formula for the 2-year recurrence period, UQ5 is the formula for the 5-year recurrence period, etc.

The BDF of each sub-catchment was subjectively determined using the method explained above. Additionally, the UQ2-formula was used to determine UQ3-flows, something what will be justified later. The pre-development peak flows determined in the previous chapter, using the triangular unit hydrograph, were used as RQ-values in these formulas, together with previously determined areas for each sub-basin. A BDF of 0 indicates pre-development flows, while a BDF of more than 0 gives post-development scenario results.

3.6.1 Verifying the formulas

To verify the formulas above, information about maximum flows from the different flow measuring stations were collected from the Bogota Water Utilities. The only data deemed sufficient and/or representative for the studied basin, was that of the Chiguaza station, located in the upper Tunjuelo basin. Here, more than ten years' worth of maximum flows were available, in addition to the station not being subjected to flows from outside the Chiguaza catchment. Thus, this information was used in a Gumbel-distribution to determine the recurrence period of the maximum flows.

Firstly, the Gumbel distribution uses historic data of annual maximum flows to tell the extent of a flood of recurrence period T. 12 annual peak flows were available from the Chiguaza station, giving N=12. These were ranked from highest to lowest (M=1, 2, …, 12) before probabilities were assigned to each peak flow. The probability, P, was determined using the Weibull probability formula:

$$P = \frac{M}{N+1} \qquad \textit{Eq. 3.14}$$

The exceedance probability of each flow was then calculated by putting the respective M of each known flow into the formula above, with N=12. By multiplying it with 100, the probability of a flow exceeding that studied flow in percent was obtained (Bagley, 2017). The flows vs. their exceedance probability $\left(P(x \geq x_i)\right)$ were then fitted to a straight line, making it possible to determine probable peak flows for rainfalls of other recurrence periods, using the following formula:

$$P(x \geq x_i) = \frac{1}{T_R} \qquad \textit{Eq. 3.15}$$

This gave known values of urban peak flows for the Chiguaza sub-basin, which enabled validation of the formulas expressed above. The urban peak flows according to the UQ-formulas could be determined using information regarding BDF, RQ for each studied recurrence period, and A, of the Chiguaza sub-basin. The percentage error between the estimated and actual urban peak flows could then be determined, using the following formula:

Eq. 3.16

$$\% - error = \frac{UQ_i - Q_{act,i}}{Q_{act,i}}$$

Where

UQ_i = the calculated value of the urban peak flow for recurrence period i using the urban peak flow formulas

$Q_{act,i}$ = the actual value of the urban peak flow for recurrence period i retrieved from the Gumbel distribution

The percentage error for each recurrence period i was then compared to the standard error of regression in %-units shown in table 3.4 above. The formulas were assumed valid as long as the percentage-error was in between or very close to the interval for the corresponding error of regression.

3.7 Additional data showing correlation between urbanisation and increased runoff flows

As an additional measurement towards showing the impact of urbanisation on storm water runoff flows, information regarding daily mean flows from the Puente Bosa station in Tunjuelo for the years 1970-2016 were used to construct two Flow Duration Curves (FDCs) for comparison.

An FDC uses measured data to show how often a certain flow, or flows exceeding that, occur for a specific site (Renewables First, 2017). The figure below gives a simple visualisation of the principle of a flow duration curve:

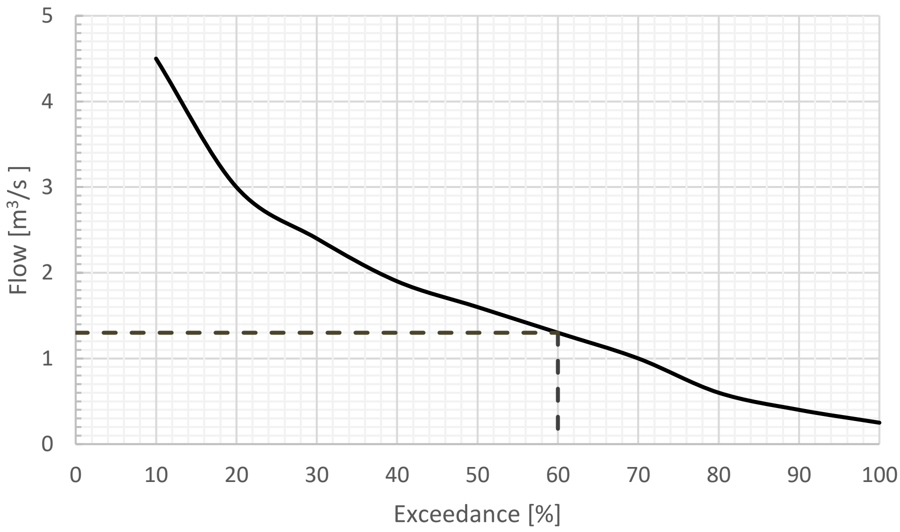

Figure 3.10 The principle of a Flow Duration Curve (FDC).

According to figure 3.10 above, a flow of 1.3 m^3/s has an exceedance occurrence of 60%. This means that the flow of 1.3 m^3/s or higher happens 60% of the time according to measured data.

The principal behind creating an FDC is similar to that explained for the Gumbel-method, as all known daily mean flows are ranked from highest to lowest values. Observe that the difference between the methods is that daily mean flows are used instead of annual peak flows. Using the Weibull-probability formula previously explained, the exceedance probability could be determined. The data with daily mean flows was divided into two parts: the flows from the period 1970-1985, which would represent less-urbanised data, and the flows from the period 1985-2016, which would represent more-urbanised data. Two separate FDCs for the respective periods were compiled into one figure, allowing for a straight-forward comparison and overview of the difference in mean flows between the two periods.

Chapter 4
Results

This chapter includes the most important results for the assessment of the impacts of urbanisation on runoff flows. A straight-forward analysis of the obtained results can be found below each sub-chapter, while further discussion on the matter is found in the next chapter. The calculated values for effective precipitation, basin lengths and slopes, and tables of rural peak flows, have all been put in the appendix, section I.V.

4.1 Verification of the UPD formulas

The UPD formulas were verified by studying the %-error with real data from the Chiguaza station, Q, and calculated data using the UPD formulas, UQ. Firstly, the Gumbel-distribution graph for the Chiguaza station was studied to obtain peak flows for the post-development scenario, which is shown in figure 4.1 below:

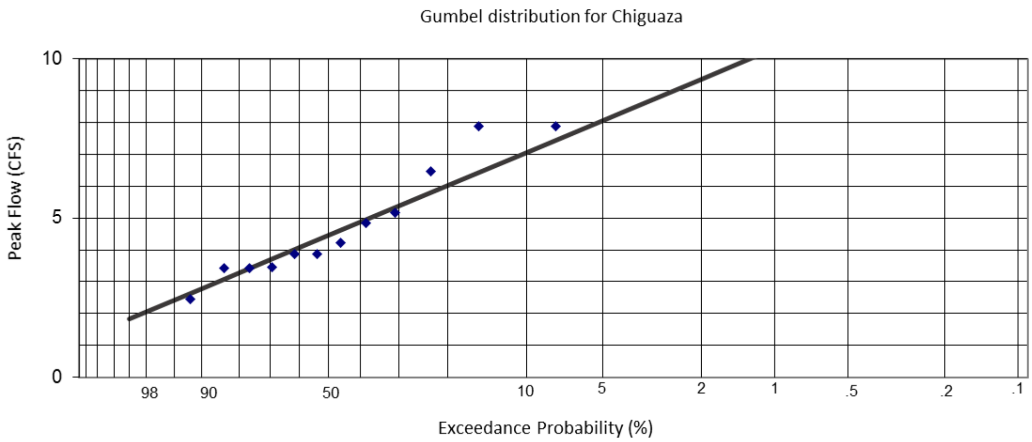

Figure 4.1 Gumbel distribution graph for the Chiguaza Station, where the flow is shown in Cubic Feet per Second (CFS)

When comparing these values to the ones obtained through the use of the UPD-formulas, the difference in %-error was compared and used to verify the formulas. The figures obtained from this comparison have been compiled in table 4.1 below:

Table 4.1 %-error for the UPD formulas used to verify them with data from the Chiguaza station. The bolded figures show results far outside the average %-error interval

Tr	UQ	Q	%-error	Av. %-error
3	6.1	5.2	18	±43
5	7.4	6.0	22	±40
10	10.3	7.1	45	±41
25	**13.3**	**8.4**	**58**	±**43**

The bolded figures in the table above show the figures which were far outside the average %-error interval.

Analysis

When studying table 4.1, it becomes evident that the reliability of the study decreases with higher recurrence periods. When verifying the UQ-formulas, the percentage error increased with every recurrence period, and reached too high figures for events above the 10-year recurrence period. This decreases the credibility of the 25-year events. However, comparing the results from lower recurrence periods, a similar trend can be noted. For this purpose, it is the individual numbers that are mainly regarded unreliable.

Additionally, since the results from the 3-year event rainfall fall well within the average %-error interval, the decision to use equation *UQ2* for studying 3-year events is deemed justified and a good enough simplification.

4.2 Peak discharge development

Figure 4.2 gives information regarding the peak flow development for the Tunjuelo basin. Similar trends can be noted for the other water sheds as well, which is why the corresponding information for them has been put in the appendix, section I.V.I. In this case, no significant changes in peak flows before 1958 could be noted.

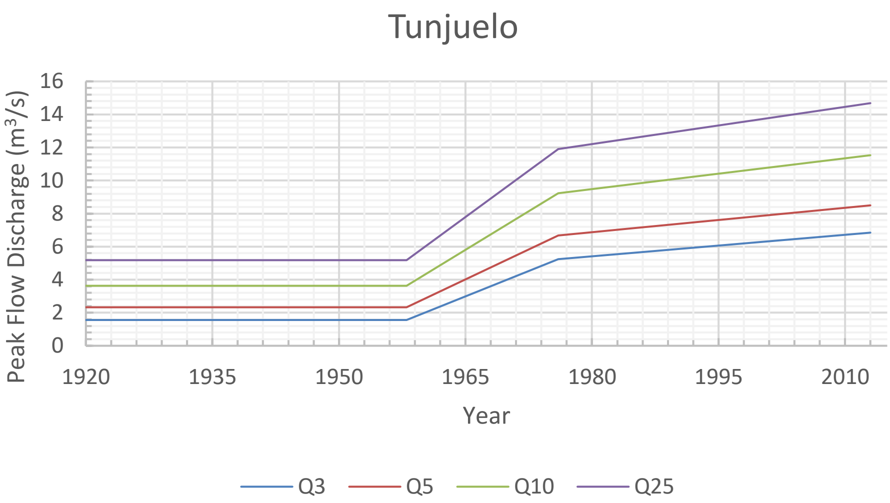

Figure 4.2 Increase in peak runoff flows through the studied years for Tr=3, 5, 10, and 25 years for the Tunjuelo basin.

Difference, in %, between pre-development and latest post-development (2013) peak flows for each basin

Below, in table 4.2, the percental increase in peak flows between the pre-development scenario and the latest post-development scenario, from 2013, for the Chiguaza and Tunjuelo basins respectively has been compiled.

Table 4.2 The percental increase in peak flows between the rural scenario and the latest urban scenario (2013) for Chiguaza and Tunjuelo

	Chiguaza			Tunjuelo		
T_r	**RQ**	**UQ**	**%-increase**	**RQ**	**UQ**	**%-increase**
3	2.53	6.14	143	1.56	6.85	339
5	3.13	7.36	135	2.33	8.50	265
10	4.89	10.27	110	3.63	11.53	218
25	6.89	13.27	93	5.18	14.68	183

Similar patterns can be noted for the other sub-basins, which is why those tables have been put in the appendix, chapter I.V.II.

Analysis

As can be seen in figure 4.2, and the corresponding figures for the other water-sheds in the appendix, chapter I.V.I, all results point towards an increase in runoff peak flows with increasing urbanisation, when considering the change in land use and increase in basin development factor. This is also what has been expected, according to previous studies made on the subject. For instance, the USGS (2016a) report that an increase in population increases the water discharged into sewage- and water systems, that way increasing runoff flows as well. Additionally, alterations to drainage patterns, by introducing pipes and channels, increase storm runoff flooding. Anderson (1970) is on the same track, meaning that sewer installations alone increase flood peak magnitudes, and that completely impervious surfaces increase average sized floods. Even if the percental increase in impervious cover is somewhat included in the parameters for BDF-determination, the results show a very strong correlation between drainage efficiency and peak flows alone, supporting Andersson (1970) further. Almost all basins have seen at least an 100% increase in peak flows for all studied recurrence periods, with the 25-year rain-event for the Chiguaza basin being the only exception. Additionally, studying figure 4.2 and figures I.1-I.7 in chapter I.V.I, linear relationships can be noted between the measured years. It is important to note that this must not be the case in reality.

An interesting notification that can be made by studying the Chiguaza and Tunjuelo results, for instance the figures in table 4.2, is that the peak flows between the two sub-basins do not differ a lot, even though the difference in area is rather evident and the Chiguaza basin is included in the total Tunjuelo basin. This is believed to be due to the very high slope of the Chiguaza basin, of almost 7 %, as compared to the slope of the Tunjuelo basin that amounts to 0.13%. Studying the SCS triangular unit hydrograph, it is evident that the slope plays a major role in the determination of the pre-development peak flows, as steeper slopes give higher flows. Since the pre-development peak flows are strongly connected to the urban peak flows in the urban peak discharge formulas, a similar pattern in the urban peak flow determinations can be expected. Additionally, the Tunjuelo basin has a much longer L_w than the Chiguaza basin, which has a lowering effect on the peak flows according to eq. 3.11 and 3.13. It is important to distinguish runoff flows from runoff amounts, which take the time-factor into account, meaning that the Chiguaza watershed does not necessarily produce the same amount of runoff as the Tunjuelo basin, even if they do have similar flows.

4.3 Flow Duration Curve for Tunjuelo, Puente Bosa

The flow duration curve, produced to show the difference in average flows over the Tunjuelo basin between the periods 1970-1985 and 1986-2016, is shown in figure 4.10 below:

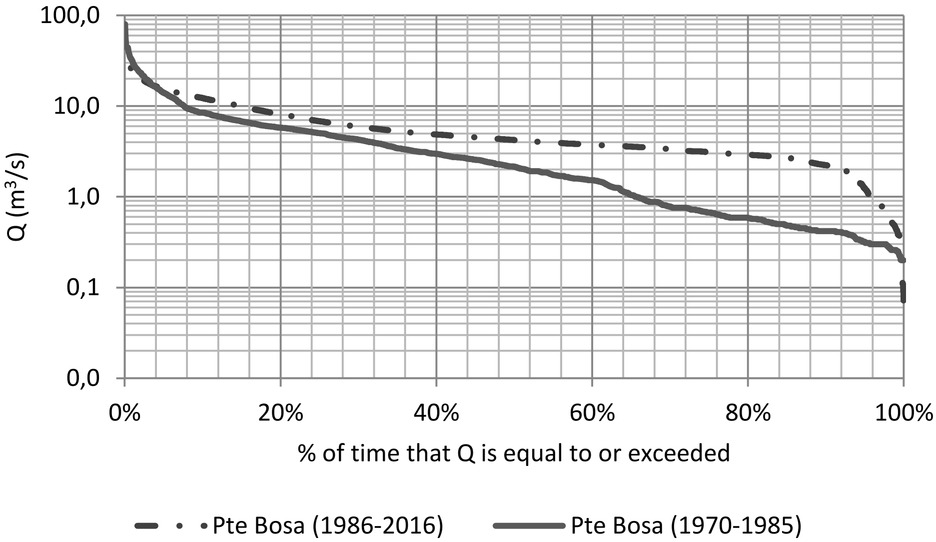

Figure 4.3 FDC for the Puente Bosa station for the two periods 1970-1985 and 1986-2016.

Analysis

The FDC for the Pte Bosa station is another way of showing a correlation between urbanisation and increase in flows patterns, in this case the mean daily stream flows. Figure 4.3 shows that the average flows reach higher figures more often and are more common today, than they did before 1985. This can be correlated to figure 2.9, showing that the urbanisation in the Tunjuelo basin saw great development between the years 1985 and 2013.

Chapter 5
Discussion

This chapter aims to analyse and discuss the limitations and uncertainties of the study, as well as suggest improvements on it. Furthermore, possible approaches towards urbanisation and the issues coupled to it will be discussed.

Limitations and uncertainties

One of the main restrictions of this study is the limited information about the basin characteristics for the pre-development scenario that were available for use. For this reason, a lot of the input data is based on average values, assumptions, and simplifications. For instance, rather uniform land cover and soil properties in the pre-development state for the corresponding peak flow determinations are assumed, as a constant CN of 70 is used for the total study area. Since more detailed information regarding soil properties and land use characteristics of each catchment from pre-development periods is lacking, this was considered the best simplification. It is believed that further assumptions regarding land use cover and soil type for the different sub-basins might have impacted the credibility of the study, which is why the assumption of a uniform CN of an area of 560 km^2 was deemed a simple and good enough solution to the issue of lacking information.

A similar approach was taken when using isohyetal maps for determining precipitation patterns over the catchments. Completely uniform and evenly distributed precipitation events over the catchments, which include areas of the magnitude of 148 km^2, are a simplification deemed necessary for the sake of restricting the input of information. Dividing each catchment into several sub-catchments, to describe the precipitation events more precisely over smaller areas, would have required a lot more input data, and resulted in large amounts of data to be studied, assessed, and presented.

It is important to point out that the UQ-formulas used to determine the urban peak flows are based on a regression analysis made on catchments in the United States. The ideal solution would of course be to use formulas that have been fitted to the Colombian geography, geology, and background. Since this was lacking, using these formulas from the US were considered a good approximation. The formulas have been developed studying almost 200 catchments all over the US with no restriction towards catchment type or basin characteristics except for degree of urbanisation and data available. Since the US is a very diverse country regarding geography, it is believed that these formulas are made to fit a very wide range of basins with different characteristics, meaning that they might as well be fit for use in Colombia as well, until more progress is made on the subject. This was also partly verified with the data from the Chiguaza station, showing good enough correlation for at least 2-, 5-, and 10-year events.

Furthermore, when using the UQ-formulas, the BDF was very subjectively decided for all sub-basins. Due to lacking information about when measures included in the BDF-aspects were taken in Bogota, the BDF was decided based on subjective notifications and observations from the studied maps, which had no detailed information on how the drainage system was developed. This part is therefore also considered an uncertainty of the study.

Regarding the Tintal watershed, the certainty of the results is affected by the man-made canal developed in 2000 and the pumping station situated at the end-point of the canal, which is used to drain the canal of its content into the Bogota river. This makes it fairly hard to reach concluding results regarding the change in peak flow between pre- and post-development scenarios in the Tintal watershed, since the calculations do not include the involvement of a pumping station, nor do they take into account the great change in watershed characteristics by the development of a canal with including interceptors.

With the Jaboque watershed, there may be some uncertainties in the results coupled to the wetland situated in it. Since the wetland affects the movement of water in the watershed, slowing it down, the results may give higher values on the peak flows than what would have been obtained if the wetland was taken into account in the calculations. The decision to include the total area of the Jaboque basin was based on the relatively small size of the basin.

Additionally, it is important to note that a linear urban development is assumed from year to year, which is not necessarily true.

It is important to note that the purpose of this study was to create a simple overview on how the changes in urban cover and drainage efficiency could have had an impact on runoff peak flows in Bogota. Even if many parameters are based on assumptions and simplifications, the formulas used have been verified and still show correlations that have been supported by previous studies. This study is seen as a very good first step towards facing the issues coupled to urbanisation, and should be complemented with further studies on the subject.

Possible improvements

Firstly, to reduce the amount of uncertainties in the study, more information and measured data regarding the different sub-basins would have been needed. That way, the input parameters could have been adjusted accordingly, thus hopefully giving more spatial output data. However, it is important to note that more data does not necessarily mean better results. It is believed that it is much more important to have data that is reliable and manageable, even if simplifications and assumptions are necessary. One can get into as many details as one could, but it is up to oneself to decide how much time and effort should be put on the study, and how much it will actually help the study.

Secondly, one good improvement would be to adjust and adapt the UQ-formulas to Colombian conditions. This can be done by principally gathering as much information on urban peak flows, factors regarding basin development (see p. 49), rural peak flows, and area, from as many basins in the country as possible, and do a similar regression analysis as the one made in the United States. This way, the formulas can be used more righteously, and a more thorough input database can be established.

Additionally, one interesting thing would be to study different precipitation patterns over time, with respect to climate change. Throughout this study, constant precipitation patterns over time have been assumed. If possible, precipitation information from different time periods could have been studied, to see 1) if they have changed over time due to climate change and 2) how that has played a part in the increasing runoff flows. This would expectantly require a lot of data gathering and work, but would absolutely give a certain depth to the study. Furthermore, possible changes in basin characteristics over time, such as basin slope, have not been taken into account either due to lack of data. This additional information would have been interesting to study as well, since the slope of a basin seems to have great effect on runoff flows according to equation 3.11.

Approaching the issue – how to deal with increasing urbanisation and the issues coupled to it

As can be concluded from this study, both the runoff peak flows and average flows, as seen in figures 4.2 and I.1-I.7, have seen an increase due to urbanisation. Additional impacts of urbanisation include less infiltration into groundwater reserves (USGS, 2016a), pollution of water bodies, and a negative impact on storm water quality (USGS, 2016b). A first approach towards dealing with these issues could be to implement SUDS.

One of the main structural SUDSs for the reduction peak flows is the detention and retention ponds. These are a good way of introducing green areas, with or without standing water, increasing the recreational value of a site. Additionally, they can be designed for different storm events. However, considering the large areas required for these types of basins to be installed, it might not be the best solution in a city like

Bogota. It can, however, be seen as a more long-term solution, where land can be gradually invested in by the municipality, and eventually be turned into detention ponds. The areas could be turned into green-parks to be enjoyed during sunnier days, while they at the same time would decrease flow velocities and result in cleaner storm water during rain events.

Green roofs seem like a very good solution to many issues in Bogota, much due to its heavily urbanised character. Firstly, they can be installed on already existing buildings, given that the buildings are adapted to handle the installation. This would not only delay and decrease the amount of runoff generated, but also provide natural habitats for some species (State of Green, 2015) while eventually serving some recreational purpose as well. Secondly, no extra space would be required, and neither would space be needed to be converted. Simply put, the constellation of the city would not be affected.

Since urbanisation prevents the infiltration of storm water and groundwater recharge, harvesting and recycling it could be one good solution to the increased amounts and flows of it. Storm water can be a good resource, making it possible to conserve drinking water and preserve its sources. The storm water could then be used for more crude use, such as toilet flushing and irrigation (State of Green, 2015). It could be stored in tanks underneath the house, on rooftops, or in tanks in the backyard if one exists. This would of course result in major reconstruction of the current water management systems, as drinking water and storm water pipes would have to be separated. Additionally, the tenants would have to be informed and educated to secure proper usage of the new system.

Approaches that can be taken on a more municipal level, where changes in constellation would be required, include the installation of wadis and permeable pavements. The implementation of green spots all over the city should always be encouraged, much because of the impact it has on infiltration properties and the recreational purpose it brings. Wadis would not only provide green areas, but also improve infiltration and cleaning properties of them (Susdrain, 2017). Additionally, they could decrease the amounts of surface runoff. Infiltration trenches can be installed in already green areas, meaning that the installation of one does not necessarily mean a change in land cover, as it is installed under-ground. Permeable pavements are also a good way of utilising roads in a more efficient way, as they both increase the infiltration properties, thus decreasing the generation of surface runoff, and serve a good purpose in an urbanised society, allowing for cars and pedestrians to use them as usual.

Of course, there is always the economic aspect of every change in constellation or addition to it. A plan for costs should always be well-prepared before any changes or implementations are made. However, major investments could be valuable in the long run. Seeing the historical development of the water management systems in Bogota, where various water bodies have been dried out and replaced by urban and industrial areas, rivers have been exploited to the point where they can no longer be utilised for purposes other than carrying storm water and wastewater, and where storm runoff peak flows have reached more than a 500% increase in some areas, it is evident that approaches must be taken to not only stop this development, but maybe even attempting to reverse it. Since an increase in population is expected, see figure 2.3, further stress on the existing water systems can be expected. For this reason, economic investments should be encouraged. Taking the successful story of the implementation of high drinking water tariffs, which has resulted in very low water consumption per capita compared to the rest of Latin America (The World Bank, 2012), economic incentives towards the degradation of water bodies, and work towards managing increasing amounts of storm water could be one implementation to encourage further improvement in water management. Examples of these could be expensive repercussions for those involved in draining and polluting water bodies, whereas cost- or tax-reductions for those investing in sustainable urban drainage solutions could be implemented or further developed.

However, to reach real changes in water management, the attitude towards water in general must change. By bringing the significance of water and its scarcity, its meaning, into schools and workplaces, water can be handled in more sustainable ways. According to Rojas et al., (2015), water had a great value to the population before the colonisation of Colombia, where every aspect of the water cycle was appreciated and exploited. As more modern water handling systems were introduced, urbanised areas grew larger, and new sanitation systems were installed, the overexploitation of water bodies increased. Waste and wastewater polluting storm-water and water bodies decreased the value of it, which resulted in investments in channelizing rivers and underground sewerage, instead of approaching the issue with pollution and overuse of water resources (Gallini, et al., 2014a). However, considering the long history of war and conflict in Colombia, with possibly four million internally displaced inhabitants (UNHCR, 2003), the huge social divides all over the country (del Ama, 2013), and the very rapid population increase for the last decades, it is understandable that questions regarding storm-water quality and amounts might not have been the political (or general) focus.

Chapter 6
Conclusion

It can be concluded that the last decade of intense urbanisation in the forms of increase in permeable surfaces and increased basin drainage efficiency, has led to greatly increased runoff peak flows in all studied parts of Bogota. It is very important to point out that it is not only the change in land-use cover that impacts the storm water flow patterns, as drainage efficiency measures in the forms of channelization of rivers and introduction of drainage systems are shown to have great impact on peak flows as well. It is believed that both political and constructional measures are needed to face the issues coupled to urbanisation. Due to the densely populated character of Bogota, constructional changes that require large areas, such as detention ponds, may not be the most suited solutions to begin with. Instead, installations that are easily integrated into the environment without greater impact on the constellation of the city, such as green roofs and permeable pavements, can be an easier way of introducing sustainable urban drainage systems. Additionally, the public and their views on water need to be involved in the process of making the water management systems of Bogota more sustainable. This, however, requires a change in social conditions where issues with poverty, safety, and water availability are handled.

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Appendix

I.I Digitalisation of maps

This chapter includes a brief overview of the digitalisation process and the materials needed for it.

I.I.I Materials for digitalisation

The maps that were to be digitalised were acquired physically from the cadastral survey of Bogota. From the Capital District's Spatial Data Infrastructure (IDECA) of Bogota, ArcGIS-files of constructions, water bodies, water streams, elevation lines, basin borders, and such (IDECA, 2017), of Bogota could be collected and used. To enable the digitalisation, the software ArcMap was used.

I.I.II Methodology

A shape-layer (SHP-layer) called Construcciones, showing all constructions in Bogota by 2013, retrieved from IDECA (2017), was used as a base layer on which each of the maps was projected. The projected layers were estimated through thorough studying of the printed maps, using existing constructions and eye measurement to mark constructions from the older maps.

For each map, a corresponding SHP-layer was created in the same file. These layers were made as polygons, where each polygon corresponded to an area from the respective map, but projected onto the Construcciones-file. Each layer was given a corresponding colour to make it possible to differentiate the layers, i.e. the maps, from one another.

For each layer, the coordinate-system of the Construcciones-layer was imported, so all the layers would fit with the existing file. The pre-existing coordinate-system of the layer was a so-called Geographic Coordinate System (GCS) of the type GCS_MAGNA. A GCS is based on longitudes and latitudes to describe the position of a site on the Earth, and thus also a map. The system refers to a 3D ellipsoidal approximation of the Earth (Buckley, 2009).

Since ArcGIS uses panimetric algorithms to conduct geometric calculations, calculating the area of a map with a GCS was not possible. Therefore, to be able to determine the size of the digitalised areas, all maps had to be projected into a Projected Coordinate System (PCS). This is a type of flattening of the GCS, going from a 3D ellipsoidal approximation of the Earth, to a 2D picture of it, allowing areal determination (Buckley, 2009). The maps were projected into the PCS MAGNA Colombia Bogota. One important note is that when projections of this sort are made, displacements can occur. Additionally, since the geometry is slightly altered, areal differences can be evident. Given that the river basin SHP-file already contained known information about the areas of each river basin, these figures could be used to determine if the maps were reasonably projected into PCSs. This was basically done by comparing the area of the river basins in GCS format with the area of the river basins in PCS format. In this case, the areal differences between the GCD and the PCS were less than one percent, which is why the chosen PCS was deemed representative.

I.II Results from digitalisation

This chapter presents the results obtained from the digitalisation, areal determination for the different basins more specifically.

I.III.I Areal determination for the studied years

Table I.1 below shows the area for each catchment in both PCS and GCS, the difference between the two in %, and the area finally used in the study.

Table I.1 The area for each catchment in PCS and GCS respectively, the difference between them in %, and the area finally used in the study

Catchment	Area in PCS [km^2]	Area in GCS[km^2]	Difference [%]	Area used [km^2]
Fucha	148.10	148.23	0.088	148
Salitre	132.52	132.62	0.090	133
Tunjuelo	106.70	106.78	0.075	107
Tintal	34.04	34.07	0.088	34
Jaboque	16.24	16.25	0.062	16
Torca	98.49	98.57	0.081	98
Quebrada Yomasa	15.47	15.48	0.065	15
Quebrada Yerbabuena	9.04	9.05	0.110	9

Table I.2 below shows the urbanised area in km^2 and % urbanised area out of total area for each catchment, each studied year.

Table I.2 The urbanised area in km^2 and % out of total area for each catchment, each studied year

Year	Catchment	Urbanised area [km2]	% Urbanised area out of total area
1797	Fucha	3	2
1848	Fucha	4	2
1923	Fucha	6	4
	Salitre	2	2
1938	Rio Fucha	12	8
	Rio Salitre	11	8
	Rio Tunjuelito	0.1	0.09
1958	Rio Fucha	41	28
	Rio Salitre	28	21
	Rio Tunjuelito	6	6
	Tintal	0.05	0.15
1976	Rio Fucha	75	51
	Rio Salitre	65	49
	Rio Tunjuelito	22	20
	Tintal	4	12
	Jaboque	4	26
	Torca	4	4
1985	Rio Fucha	78	52
	Rio Salitre	68	51
	Rio Tunjuelito	28	26
	Tintal	6	18
	Jaboque	6	37
	Torca	7	7
2013	Fucha	103	69
	Salitre	102	77
	Tunjuelito	70	65
	Tintal	28	81
	Jaboque	13	80
	Torca	43	45

Quebrada Yomasa	4	27
Quebrada Yerbabuena	0.26	3
Tunjuelo incl. Q. Yomasa and Q. Yerbabuena	74.26	57

I.III Pre-urbanisation scenarios

This section includes the figures used for determination of the pre-development scenarios. These include the average basin slope, determined using the Taylor-Schwarz formula, calculated values of CN and S for each basin for the pre-urbanisation scenario, average and effective precipitation over the different catchments, and the pre-development peak flows associated with each sub-basin.

I.III.I Effective precipitation

Table I.3 below shows the effective precipitation over each respective catchment for D=30 min and T_r=3, 5, 10, and 25 years respectively. These are the figures that were used in the study to determine peak flows.

Table I.3 The effective precipitation over each respective catchment for D=30 min, and the recurrence periods T_r = 3, 5, 10, and 25 years respectively

T_r	3	5	10	25
	P_e [mm]	P_e [mm]	P_e [mm]	P_e [mm]
Torca	5.09	6.96	9.94	14.81
Salitre	9.05	9.94	13.78	17.48
Fucha	5.09	6.96	9.05	14.81
Tunjuelo	2.11	2.61	4.08	5.81
Q. Yomasa	3.46	5.09	6.96	12.29
Q. Yerbabuena	2.11	2.11	4.08	5.81
Tintal	2.11	2.61	5.81	7.77
Jaboque	5.09	6.96	11.33	17.48

I.III.II Basin lengths and average basin slopes, after the division into sub-basins

The basin lengths and average basin slopes for each studied sub-basin, determined as explained in chapter 3.4, are compiled in table I.4 below. The basin length is described in m, and the slope in %.

Table I.4 The basin lengths in m and average basin slope in % for each sub-basin after division

	L_W [m]	Y [%]
Torca	9,600	0.37
$Salitre_N$	11,300	0.50
$Salitre_S$	14,200	0.48
Fucha	22,500	0.53
Chiguaza	7,800	6.93
Tunjuelo	21,500	0.13
Tintal	4,500	0.29
Jaboque	7,800	0.12

I.III.III Pre-development Peak Flows (RQ) for each basin

The pre-development peak flows, determined as in chapter 3.5, for each sub-basin for 3-, 5-, 10-, and 25-year recurrence periods are compiled in table I.5. The flows are described in m^3/s.

Table I.5 The pre-development peak flows for each sub-basin for 3-, 5-, 10-, and 25-year recurrence periods in m^3/s

	RQ3	RQ5	RQ10	RQ25
Torca	2.89	3.95	5.64	8.4
$Salitre_N$	8.27	9.09	12.60	15.98

Salitre$_S$	8.04	8.83	12.24	15.53
Fucha	7.38	10.09	13.12	21.48
Chiguaza	2.53	3.13	4.89	6.89
Tunjuelo	1.56	2.33	3.63	5.18
Tintal	2.10	2.59	5.77	7.72
Jaboque	0.99	1.35	2.20	3.40

I.IV BDF-determinations for post-urbanisation scenarios

This chapter gives information about BDF-determination made for the post-development scenarios. Chapter I.IV.I gives a deeper definition of the BDF-parameters, while I.IV.II gives the used BDF-values for each sub-basin and year.

I.IV.I Defining the BDF-parameters

The BDF-parameters and a further detailed description about them are mentioned in this sub-chapter. All information is taken from USGS (1984).

I. Channel improvements

If channel improvements such as straightening, enlarging, deepening, and clearing are prevalent for the main drainage channels and principal tributaries (those that drain directly into the main channel), then a code of 1 is assigned. Any or all of these improvements would qualify for a code of 1. To be considered prevalent, at least 50 percent of the main drainage channels and principal tributaries must be improved to some degree over natural conditions. If channel improvements are not prevalent, then a code of zero is assigned.

II. Channel linings

If more than 50 percent of the length of the main drainage channels and principal tributaries has been lined with an impervious material, such as concrete, then a code of 1 is assigned to this aspect. If less than 50 percent of these channels is lined, then a code of zero is assi6 ned. The presence of channel linings would obviously indicate the presence of channel improvements as well. Therefore, this is an added factor and indicates a more highly developed drainage system.

III. Storm drains, or storm sewers

Storm drains are defined as enclosed drainage structures (usually pipes), frequently used on the secondary tributaries where the drainage is received directly from streets or parking lots. Many of these drains empty into open channels; however, in some basins they empty into channels enclosed as box or pipe culverts. When more than 50 percent of the secondary tributaries within a subarea (third) consists of storm drains, then a code of 1 is assigned to this aspect; if less than 50 percent of the secondary tributaries consists of storm drains, then a code of zero is assigned. It should be noted that if 50 percent or more of the main drainage channels and principal tributaries are enclosed, then the aspects of channel improvements and channel linings would also be assigned a code of 1.

IV. Curb-and-gutter streets

If more than 50 percent of a subarea (third) is urbanized (covered by residential, commercial, and/or industrial development), and if more than 50 percent of the streets and highways in the subarea are constructed with curbs and gutters, then a code of 1 would be assigned to this aspect. Otherwise, it would receive a code of zero. Drainage from curb-and-gutter streets frequently empties into storm drains.

I.IV.II BDF-values

Table I.6 is a compilation of the BDF-values determined for each sub-basin for the years when the BDF-value exceeded 0.

Table I.6 The BDF values for each sub-basin for each year. Only years when BDF exceeds 0 are shown.

	1958	1976	1985	2013
Fucha	3	6	6	8
$Salitre_S$	3	7	8	8
$Salitre_N$		6	6	8
Jaboque		4	4	10
Chiguaza			3	6
Torca			2	12
Tunjuelo			2	6
Tintal				12

I.V Results: Peak discharge development

This chapter includes the results that were not presented in the results section, chapter 4.2

I.V.I Results: figures of peak discharge development

The rest of the figures from chapter 4.2 have been compiled in this section. Figures I.1-I.7 show the increase in peak discharge flow for the Chiguaza, Salitre, Torca, Tintal, Jaboque, and Fucha basins respectively.

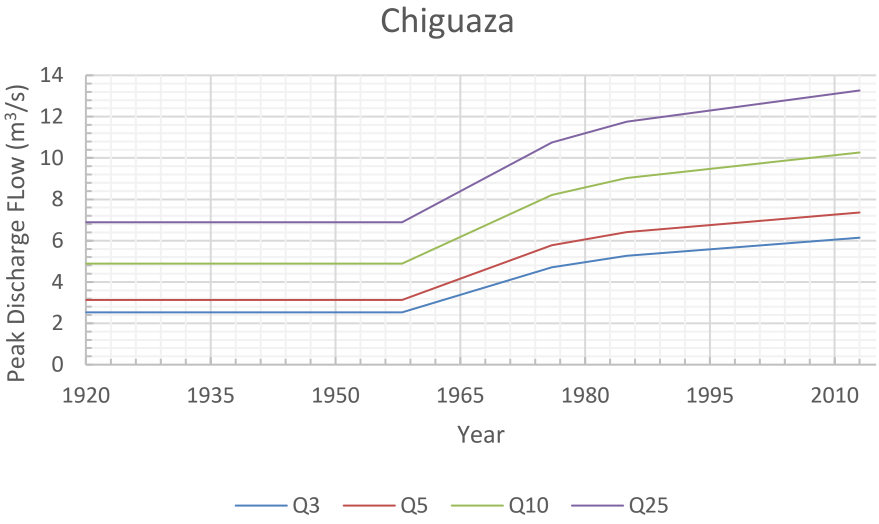

Figure I.1 Increase in runoff peak flows through the studied years for Tr=3, 5, 10, and 25 years for the Chiguaza basin

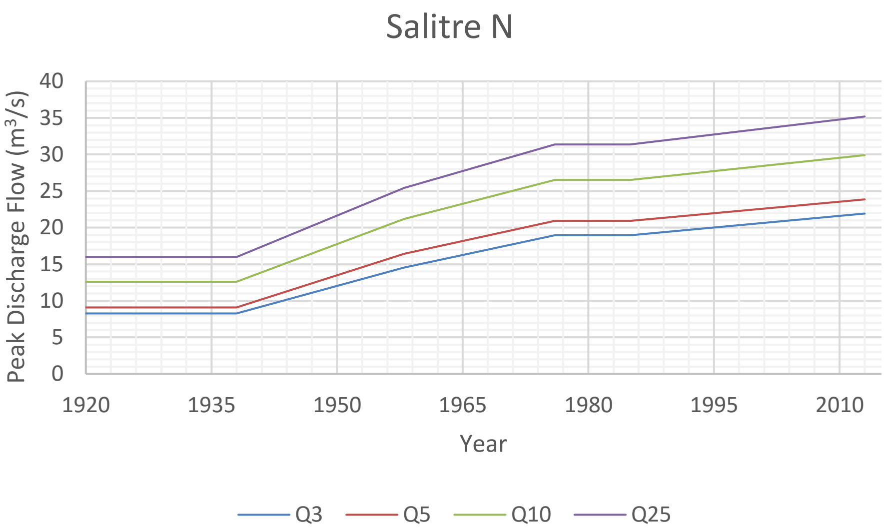

Figure I.2 Increase in runoff peak flows through the studied years for Tr=3, 5, 10, and 25 years for the Salitre$_N$ basin

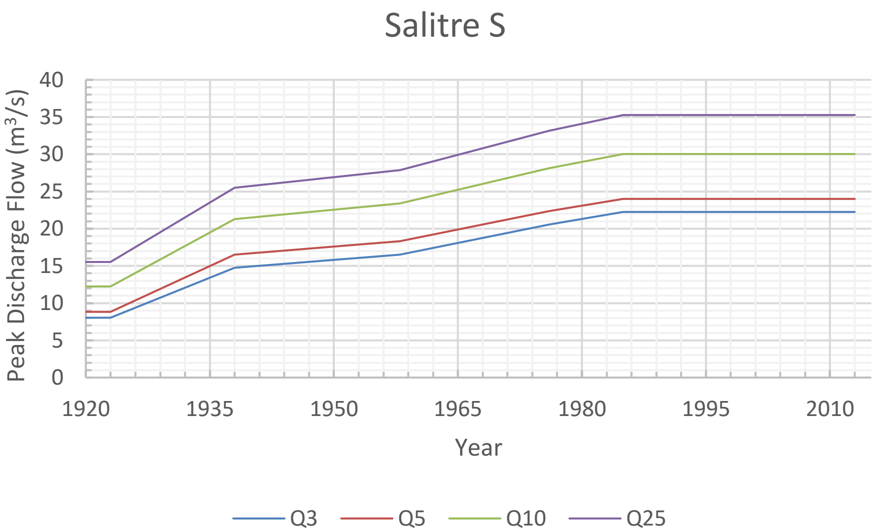

Figure I.3 Increase in runoff peak flows through the studied years for Tr=3, 5, 10, and 25 years for the Salitre$_S$ basin

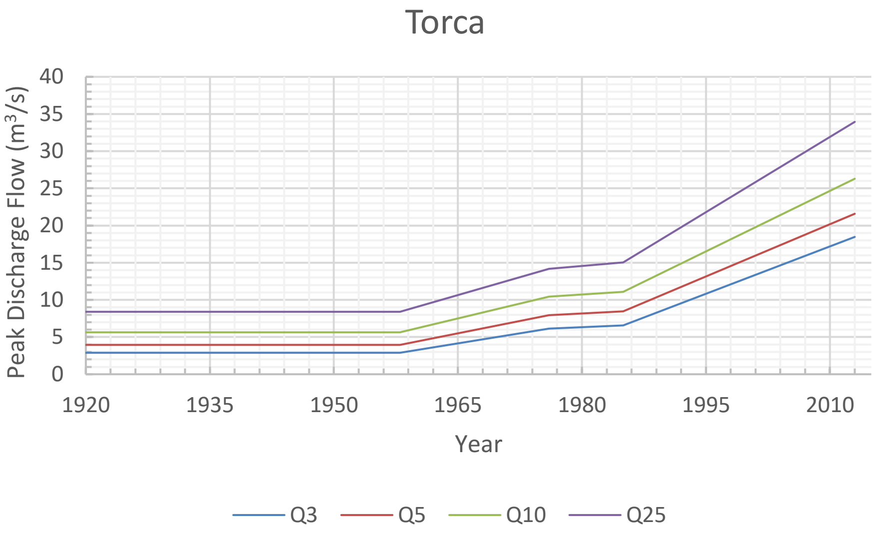

Figure I.4 Increase in runoff peak flows through the studied years for Tr=3, 5, 10, and 25 years for the Torca basin

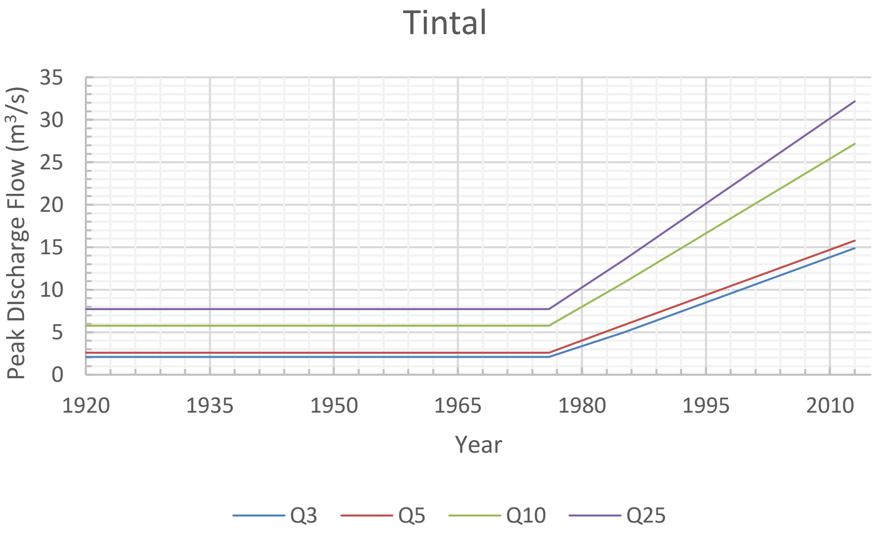

Figure I.5 Increase in runoff peak flows through the studied years for Tr=3, 5, 10, and 25 years for the Tintal basin

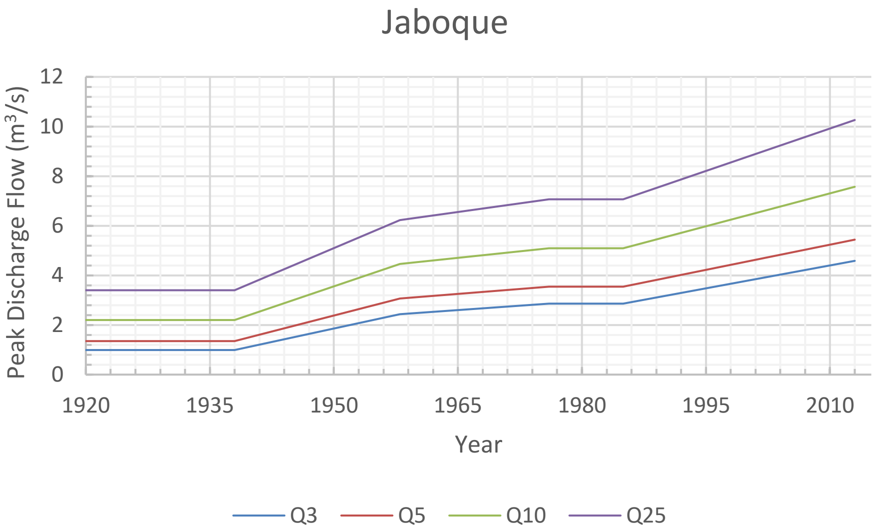

Figure I.6 Increase in runoff peak flows through the studied years for Tr=3, 5, 10, and 25 years for the Jaboque basin

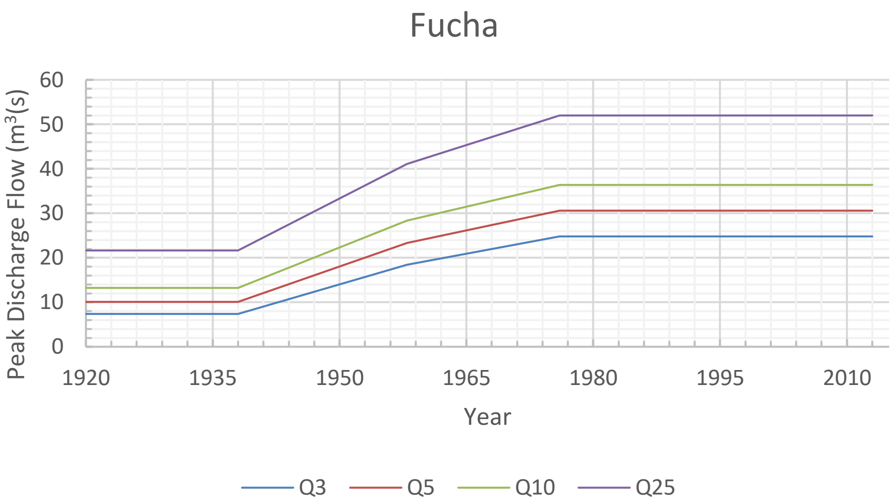

Figure I.7 Increase in runoff peak flows through the studied years for Tr=3, 5, 10, and 25 years for the Fucha basin

I.V.II Results: percental increase in peak discharge

This section includes tables with the percental increase between pre-development and the latest post-development scenario, 2013, for each sub-basin except for the Tunjuelo and Chiguaza basins, since these are presented in the results section, chapter 4.2.

Table I.7 The percental increase in peak flows between the rural scenario and the latest urban scenario (2013) for Salitre$_N$ and Salitre$_S$

	Salitre$_N$			Salitre$_S$		
T_r	**RQ**	**UQ**	**%-increase**	**RQ**	**UQ**	**%-increase**
3	8.27	21.92	165	8.04	22.25	177
5	9.09	23.86	162	8.83	24.00	172
10	12.6	29.91	137	12.24	30.03	145
25	15.98	35.18	120	15.53	35.27	127

Table I.8 The percental increase in peak flows between the rural scenario and the latest urban scenario (2013) for Fucha and Torca

	Fucha			Torca		
T_r	**RQ**	**UQ**	**%-increase**	**RQ**	**UQ**	**%-increase**
3	7.38	24.80	236	2.89	18.47	539
5	10.09	30.59	203	3.95	21.58	446
10	13.23	36.38	175	5.64	26.29	366
25	21.65	51.99	140	8.4	33.94	304

Table I.9 The percental increase in peak flows between the rural scenario and the latest urban scenario (2013) for Tintal and Jaboque

	Tintal			Jaboque		
T_r	**RQ**	**UQ**	**%-increase**	**RQ**	**UQ**	**%-increase**
3	2.10	14.89	611	0.99	4.58	363
5	2.59	15.78	509	1.35	5.44	303
10	5.77	27.17	371	2.2	7.57	244
25	7.72	32.17	317	3.4	10.26	202