POLITECNICO DI TORINO

Master's Degree Course in

Building Engineering

Hydrological Assessment and Building-Scale SuDs Evaluation for Urban Runoff Reduction: A Case Study of Pinerolo

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To Nazi and Teymoor

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Abstract

Urban stormwater management is increasingly challenged by urbanization, climate change, and the growth of impervious surfaces. This thesis analyzes a residential catchment in Pinerolo, Italy, using the Rational Method for a 10-year return period storm, and evaluates the capacity of the existing drainage network.

The results show that the estimated peak discharge, 1.944 m^3/s, exceeds the pipe capacity of about 1.613 m^3/s, indicating a hydraulic deficit. To address this issue, two building-scale SuDs solutions were assessed: rainwater harvesting tanks and rain gardens.

The standard tank scenario provides 237 m^3 of storage and reduces about 24% of total runoff volume, while rain gardens provide about 467.6 m^3 of temporary storage and 8.1 m^3 of infiltration capacity, reducing nearly 48% of total runoff. Overall, the study confirms that decentralized SuDs can reduce runoff, relieve pressure on the drainage network, and improve urban flood resilience.

Keywords

Urban stormwater management; Rational Method; SuDs; rainwater harvesting; rain gardens; runoff reduction; Pinerolo.

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Chapter 1 – Introduction

Urban stormwater management has become one of the most pressing challenges in contemporary urban planning and hydraulic engineering. Rapid urbanization has transformed natural landscapes into built environments characterized by buildings, roads, parking areas, and other impervious surfaces. This transition has disrupted the natural hydrological cycle by reducing infiltration capacity, decreasing evapotranspiration, and increasing both the volume and velocity of surface runoff. As a result, urban catchments have become increasingly vulnerable to flooding, drainage system overload, and water quality degradation.

At the same time, climate change has intensified these challenges by increasing the frequency, intensity, and unpredictability of short-duration extreme rainfall events. Traditional drainage systems were generally designed based on historical rainfall patterns and the principle of rapid runoff conveyance through centralized sewer networks. Under current climatic conditions, these systems are often unable to adequately manage increased hydraulic loads, resulting in localized flooding, sewer surcharge, infrastructure failure, and the transfer of flood risk to downstream areas (Caldwell et al., 2012; Fereshtehpour & Najafi, 2025).

In response to these limitations, there has been a global shift toward more sustainable and integrated approaches to stormwater management. Green Infrastructure (GI) and Sustainable Urban Drainage Systems (SuDs) have emerged as effective alternatives that aim to restore natural hydrological processes within urban environments. Rather than rapidly removing stormwater from the site, these systems manage water at its source through infiltration, detention, storage, evapotranspiration, reuse, and delayed discharge. Solutions such as rain gardens and rainwater harvesting systems can reduce runoff volumes by a measurable margin, attenuate peak flows, improve water quality, and enhance overall urban resilience (YING ET AL., 2022).

Despite the recognized benefits of SuDs, a research gap remains regarding their quantitative performance at the building scale within real residential urban catchments. Most previous studies have focused on large metropolitan areas, regional hydrological models, or theoretical simulations, while limited attention has been given to medium-sized urban contexts where spatial constraints and existing drainage conditions strongly influence implementation feasibility. In particular, the cumulative effect of decentralized building-scale interventions on overall catchment hydrology is still not fully understood.

The city of Pinerolo, located in the Piedmont region of northern Italy, represents a suitable case study for investigating these challenges. The selected study area, located in the northern part of the city, includes residential buildings, internal roads, and green spaces identified in local planning and flood risk management maps. Considering the hydraulic assessment of the existing stormwater drainage network presented in Section 4.3, which shows that the current conventional system is likely insufficient under critical rainfall conditions, this finding provides a strong technical justification for implementing decentralized source-control interventions.

Furthermore, the inclusion of economic comparisons among different tank scenarios and the distributed costs of rain gardens further supports practical decision-making for real urban implementation.

By comparing baseline conditions with improved SuDs scenarios, this study aims to quantify the contribution of decentralized interventions to flood risk reduction and enhanced drainage system resilience.

Chapter 2 – Literature Review

2.1 Urban Hydrology and the Limitations of Conventional Drainage Systems

Urbanization has fundamentally altered the hydrological response of catchments by replacing permeable natural surfaces with impervious materials such as roofs, asphalt roads, pavements, and parking areas. Under natural conditions, rainfall is distributed among interception, infiltration, evapotranspiration, soil moisture storage, and surface runoff. In urban environments, however, infiltration opportunities are sharply reduced, and a larger proportion of precipitation is rapidly converted into direct runoff. This results in higher runoff volumes, shorter concentration times, increased peak discharges, and a greater probability of localized flooding (Paul & Meyer, 2001) .

In addition to runoff quantity, urban stormwater also presents major water quality concerns. Surface runoff commonly transports suspended solids, nutrients, hydrocarbons, heavy metals, and pathogens from roads and urban surfaces into receiving water bodies, thereby creating both hydraulic and environmental risks. As explained by Butler and Davies (2011), stormwater management should therefore be understood not only as a drainage problem but also as an issue of environmental protection, water quality control, and urban resilience.

Climate change has further intensified these challenges. Extreme short-duration rainfall events are becoming more frequent and more intense, particularly in temperate European climates, increasing the hydraulic stress imposed on existing drainage systems. CORDUAN AND KÜHN (2024) highlight that climate projections for Central Europe indicate a growing concentration of rainfall within fewer but more intense events, resulting in higher flood risk and reduced system reliability. This trend is most acute in heavily urbanized areas with high imperviousness, where drainage systems were originally designed using historical rainfall patterns that no longer adequately represent present conditions.

2.2 Sustainable Urban Drainage Systems (SuDs) and Nature-Based Solutions

Sustainable Urban Drainage Systems (SuDs), together with Low Impact Development (LID), Green Infrastructure (GI), and broader Nature-Based Solutions (NBS), represent the most significant evolution in urban stormwater management during recent decades. These approaches seek to reduce runoff generation at the source by promoting infiltration, detention, storage, evapotranspiration, reuse, and delayed discharge rather than immediate conveyance.

FERRANS ET AL. (2022), through a review of more than 120 studies on SuDs modelling and decision-support systems, demonstrated that the objective of SuDs extends far beyond simple runoff reduction. These systems are intended to simultaneously provide hydraulic, environmental, ecological, social, and economic benefits, including flood mitigation, water quality improvement, biodiversity enhancement, urban cooling, and improved landscape quality. Their study also showed that decision-support tools increasingly focus on identifying which SuDs type should be selected, where it should be implemented, and at what scale it provides maximum efficiency.

Similarly, MEDINA RIVERA ET AL. (2025) emphasized that Blue-Green Infrastructure plays a central role in improving urban resilience under uncertain climatic conditions, particularly in areas where hydrological data are limited. Their research showed that decentralized systems can significantly improve stormwater performance while maintaining flexibility in design and adaptation to local constraints.

Among the wide range of SuDs solutions, building-scale interventions are particularly important because they directly address runoff generation at its source. Roofs, courtyards, access roads, and private paved surfaces represent the first stage of runoff production, and interventions at this level can substantially reduce the cumulative hydraulic pressure imposed on the larger drainage network.

The following sections examine rainwater harvesting systems and rain gardens in greater detail, focusing on their design principles and documented performance

2.3 Rainwater Harvesting Systems

Rainwater harvesting (RWH) systems are among the most established and technically reliable building-scale SuDs solutions. These systems generally consist of a collection surface, conveyance elements, filtration devices, a storage tank, and, where necessary, a pumping system for reuse. Their importance lies in their dual hydraulic function: reducing stormwater runoff at the source while simultaneously providing an alternative water supply for non-potable domestic uses.

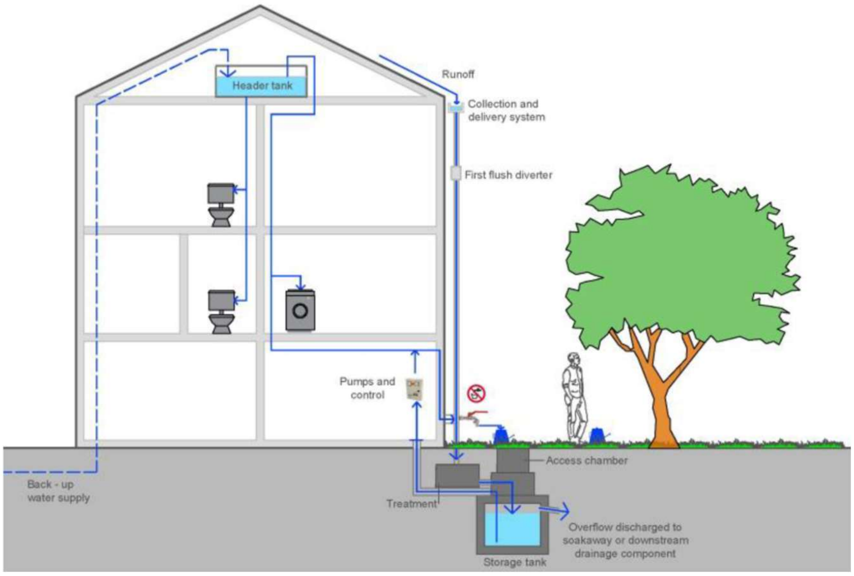


Figure 2.1 *Schematic of a building-scale rainwater harvesting system showing roof catchment, conveyance system, first-flush diversion, storage tank, and non-potable water distribution.*

(Source: MDPI Hydrology, 2020, https://www.mdpi.com/2306-5338/10/3/59)

In residential environments, a significant proportion of potable water consumption is associated with activities that do not require drinking water quality, such as toilet flushing, irrigation, outdoor cleaning, and service uses. By replacing potable water with harvested rainwater for these purposes, RWH systems reduce both municipal water demand and stormwater discharge.

WARD ET AL. demonstrated that a monitored rainwater harvesting system in a UK office building was able to supply approximately 79% of toilet flushing demand, showing that substantial reductions in potable water use are achievable under realistic operating conditions. Similarly, JAMALI ET AL. reported that widespread use of rainwater storage tanks at catchment scale could reduce annual flood damage by approximately 30%, highlighting the flood mitigation value of these systems.

The effectiveness of RWH systems depends strongly on proper tank sizing. An undersized tank leads to frequent overflow and limited water-saving performance, while an oversized tank produces unnecessary construction costs and inefficient use of space.

2.4 Rain Gardens and Bioretention Systems

Rain gardens, also referred to as bioretention systems, bioswales, or stormwater biofilters, are engineered vegetated depressions designed to temporarily store stormwater, promote infiltration, support evapotranspiration, and improve water quality before runoff reaches the conventional drainage network. They are considered one of the most multifunctional and widely applied forms of green stormwater infrastructure.

CORDUAN AND KÜHN (2024) describe bioretention systems as a central component of decentralized stormwater management and highlight their growing importance in European climate adaptation strategies. Their systematic review shows that bioretention systems are particularly effective in temperate climates where increasing rainfall extremes and urban heat island effects require multifunctional blue-green infrastructure solutions.

The typical structure of a rain garden includes a surface ponding zone, a filter media layer, a drainage or storage layer, vegetation, and in some cases an underdrain system. Water entering the system is temporarily detained, filtered through the engineered soil media, and gradually removed through infiltration, evapotranspiration, drainage, or overflow.

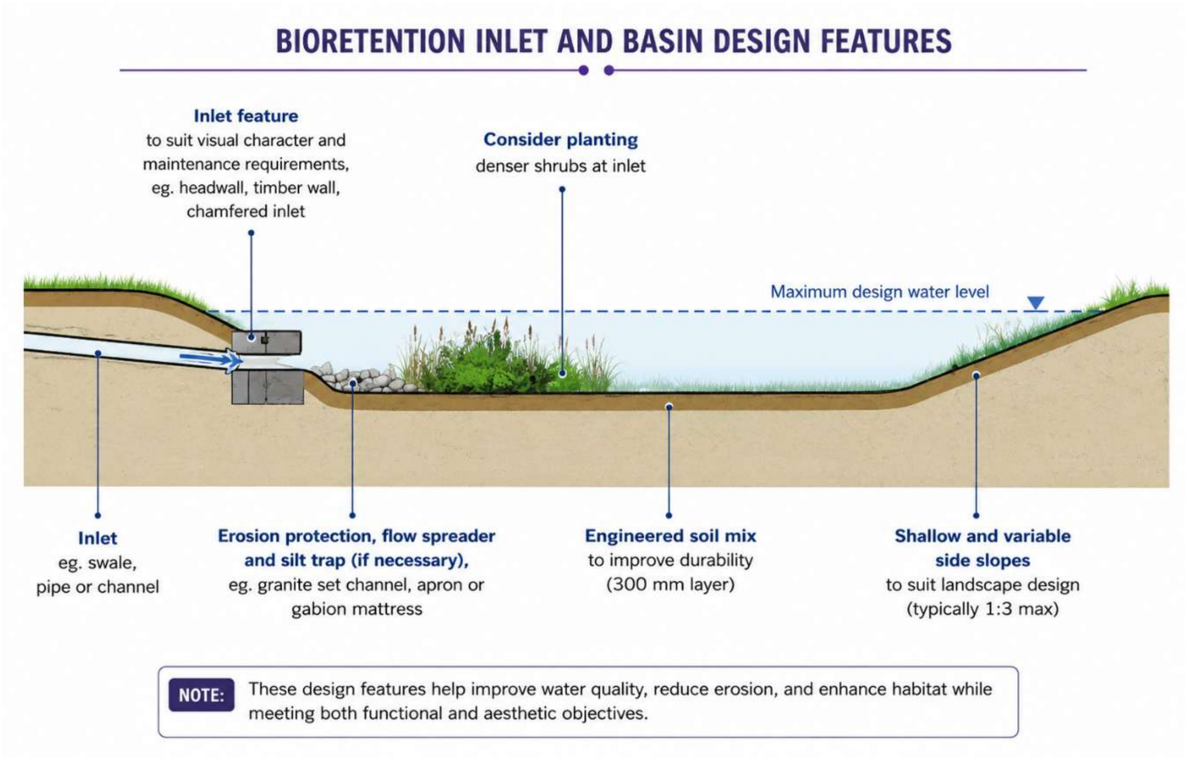


Figure 2.2 *Schematic section of a bioretention basin showing the inlet arrangement, temporary ponding area, engineered soil layer, vegetation, erosion protection, and maximum design water level.*

source: Adapted from CIRIA guidance on sustainable drainage systems (SuDs).

WANG ET AL. (2024) demonstrated through bibliometric analysis that research on rain gardens has expanded rapidly in recent years, especially after 2015, with dominant themes including runoff reduction, urban hydrology, green infrastructure, and resilience planning. This confirms that rain gardens are no longer treated as purely landscape elements, but as central hydraulic infrastructure for urban flood mitigation.

Their flexibility of scale and compatibility with residential landscapes make rain gardens well-suited to medium-sized urban catchments such as Pinerolo.

Chapter 3 – Methodology

3.1 Research Framework

The hydrological analysis was conducted using the Rational Method to assess existing runoff conditions and to quantify the performance of the selected SuDs interventions. Among the various available solutions, rainwater harvesting tanks and rain gardens were identified as the most feasible and hydraulically effective source-control measures for the characteristics of the study area.

Rainwater harvesting tanks and rain gardens were consequently selected as the principal SuDs interventions. Their hydraulic performance was evaluated by comparing the baseline condition of the study area with improved scenarios involving individual implementation. This approach allowed the quantitative assessment of runoff reduction, peak flow attenuation, water-saving efficiency, and overall drainage system resilience.

The methodological framework is structured in three main phases. The first phase consists of hydrological characterization through land-use analysis, identification of permeable and impermeable surfaces, and evaluation of drainage pathways. The second phase involves the estimation of baseline peak discharge using the Rational Method under existing conditions, without any SuDs intervention. The third phase consists of a comparative assessment between baseline and improved scenarios to quantify hydraulic improvement and validate the engineering effectiveness of the proposed decentralized stormwater management strategy.

The three phases are described in detail in Sections 3.2 through 3.5.

3.2 Hydrological Analysis Using the Rational Method

The Rational Method was selected for peak discharge estimation given its suitability for small urban catchments and its alignment with Italian regional design practice. The method estimates maximum peak discharge as:

$$Q = \frac{10 \cdot \mathrm{C} \cdot \mathrm{a} \cdot \mathrm{A}}{{t_c}^{(1-n)}}$$

where:

- Q is the peak discharge during the rainfall event (m^3/h)
- C is the runoff coefficient
- a is the rainfall intensity parameter derived from IDF curves.
- A is the drainage area (ha)
- t_c is the time of concentration (h)
- n is the empirical exponent related to rainfall distribution and basin characteristics

The fundamental assumption of the Rational Method is that the maximum runoff occurs when the rainfall duration becomes equal to the time of concentration of the basin. In other words, peak discharge is reached when the entire drainage area contributes simultaneously to the outflow.

Accurate estimation of concentration time and runoff coefficient is therefore required for reliable hydraulic design.

3.2.1 Runoff Coefficient Determination

The runoff coefficient represents the fraction of rainfall that becomes direct surface runoff instead of infiltrating into the soil, evaporating, or being temporarily retained. Its value depends on land use, soil permeability, slope, vegetation cover, and the degree of urbanization.

Since the study area contains both permeable and impermeable surfaces, a weighted runoff coefficient was calculated using the proportional contribution of each land-use category:

$$C = \frac{\sum c_i \cdot A_i}{A_{tot}}$$

where:

- C_i is the runoff coefficient of each surface type.
- A_i is the corresponding area.
- A_{tot} is the total drainage area.

The runoff coefficient values adopted for different land-use categories are summarized in Table 3.1, based on standard hydrological references.

Surface Type	φ
Paved surfaces	0.7 – 0.9
Unpaved roads (dirt roads)	0.4 – 0.6
Grassy surfaces	0.1 – 0.7
Residential areas	0.3 – 0.7
Forests	0.1 – 0.3
Cultivated land	0.2 – 0.6

Table 3.1 *Recommended runoff coefficient (φ) ranges for various urban and natural surface types adopted for peak discharge estimation using the Rational Method. Source: Modified after Butler & Davies (2011), Chow et al. (1988), and FHWA (2024)*

3.2.2 Rainfall Intensity and IDF Curve Parameters

Rainfall intensity was determined using the Intensity–Duration–Frequency (IDF) curves provided by the ARPA Piemonte "Atlante delle Piogge Intense," which represents the standard hydrological reference for rainfall design in the Piedmont region.

The general expression for rainfall intensity is:

$$i = a.t^{n-1}$$

where:

- i is rainfall intensity

- a is the climatic coefficient
- t is rainfall duration
- n is the empirical exponent

A 10-year return period was selected because it provides an appropriate balance between hydraulic safety and economic feasibility for residential drainage systems.

3.2.3 Time of Concentration

The time of concentration represents the time required for runoff generated at the most hydraulically distant point of the catchment to reach the outlet of the drainage system. It was calculated as the sum of two components:

$$t_c=t_a+t_r$$

where:

- t_a is the access time, representing the surface runoff travel time before entering the drainage network.
- t_r is the travel time inside the drainage network.

The access time (t_a) was estimated using an empirical formula derived from kinematic runoff models:

$$t_a = \left(\frac{3600^{\frac{n-1}{4}} \cdot 0,5 \cdot l_i}{{s_i}^{0,375} \cdot (a \cdot C_i \cdot A_i)^{0,25}} \right)^{\frac{4}{n+3}}$$

where:

- l_i is the surface or hydraulic flow length (m);
- s_i is the average slope.
- C_i is the runoff coefficient.
- A_i is the drainage area (ha);
- **a** and **n** are the IDF curve parameters describing rainfall intensity as a function of duration and return period.

The adopted equation provides a practical and widely accepted approach for estimating access time in small urban catchments and is consistent with the objectives of preliminary stormwater drainage design. Nevertheless, its application is subject to certain limitations, as its accuracy may decrease in large basins, areas with very steep or very flat slopes, situations characterized by concentrated channel flow, or when the adopted IDF parameters are not fully representative of local rainfall conditions. Considering the relatively small size and urban characteristics of the study area, the use of this equation is considered appropriate and sufficiently reliable for the purposes of this research.

The travel time within the drainage network (tr) was calculated using the hydraulic relation for stormwater pipe systems:

$$t_r = \left(\frac{23{,}6 \cdot \left(\frac{l_r}{K_s}\right)^{0{,}6}}{3600^{0{,}4\cdot(1-n)} \cdot \left(\frac{a}{1000}\right)^{0{,}4} \cdot p^{0{,}3}} \right)^{\frac{1}{0{,}6+0{,}4\cdot n}}$$

where:

- l_r is the maximum length of the drainage network (m).
- K_s is the Gauckler–Strickler coefficient, depending on channel or pipe roughness.
- **p** is the average slope of the hydraulic path;
- **a** and **n** are the IDF curve parameters describing rainfall intensity as a function of duration and return period.

The adopted formulation is intended to represent the average flow propagation through the drainage network under typical design conditions. As with any empirical approach, the reliability of the results depends on the assumption of relatively uniform hydraulic characteristics along the flow path. Therefore, the equation may be less representative in drainage systems affected by significant hydraulic interactions, abrupt changes in slope or roughness, and complex flow regimes. Given the characteristics of the investigated area and the level of detail required for this study, the method is considered suitable for estimating network travel time.

3.3. Discharge Capacity in Circular Pipes

To evaluate the hydraulic performance of the drainage network and determine the discharge capacity of circular pipes, the uniform flow approach for partially filled conduits was applied using the Kutter formula. This method is based on the geometric characteristics of the pipe section, wall roughness, and hydraulic slope, allowing the estimation of discharge under different flow depths.

Initially, the geometric parameters of the pipe section, including pipe diameter d, radius r, flow depth h, longitudinal slope p, and Kutter roughness coefficient m, were defined. Subsequently, the hydraulic properties of the partially filled circular section were calculated, including the central angle α, wetted perimeter P_b, flow area A, and hydraulic radius R_i.

The central angle of the wetted section was calculated using:

$$\alpha=2\cos^{-1}\left(\frac{r-h}{r}\right)$$

The wetted perimeter was determined from:

$$P_b=2\pi r\left(\frac{\alpha}{360^\circ}\right)$$

The flow area of the partially filled pipe section was calculated as:

$$A = \frac{r^2}{2}\,(\frac{\pi\alpha}{180°} - \sin\alpha\,)$$

The hydraulic radius was then obtained from the ratio between the flow area and the wetted perimeter:

$$R_i = \frac{A}{P_b}$$

To evaluate the hydraulic resistance, the Kutter friction coefficient was calculated using:

$$c = \frac{100\sqrt{R_i}}{m + \sqrt{R_i}}$$

The mean flow velocity was subsequently determined according to the uniform flow equation:

$$V = c\sqrt{Ri \cdot p}$$

Finally, the discharge capacity of the pipe was calculated using the continuity equation:

$$Q = A \cdot V$$

3.4 Rainwater Harvesting Tank Design Using the Simplified Method

For each residential parcel, the annual collectible rainwater volume was first calculated using:

$$Q = h \cdot A \cdot C$$

where:

Q is the annual collectible rainwater volume

h is the average annual rainfall.

A is the roof catchment area.

C is the runoff coefficient of the roof surface.

The annual non-potable water demand was estimated using:

$$D = N \cdot d$$

where:

D is the annual non-potable water demand

N is the number of residents

d is the average annual non-potable demand per person which is considered (0.4)

The useful tank volume was then determined according to the simplified UNI method:

$$V_{useful} = \min(Q, D) \times 0.06$$

where the coefficient 0.06 accounts for dry periods without rainfall and represents the minimum effective storage required.

To obtain the practical installed volume, the following correction was applied:

$$V_{optimal}=V_{useful}\times 1.5$$

This adjustment considers installation practicality, commercial tank availability, and minimum construction constraints.

3.5 Rain Garden Design Method Based on Runoff Volume, Storage Capacity, and Infiltration Performance

In addition to rainwater harvesting tanks, rain gardens were selected as a complementary building-scale Sustainable Urban Drainage System (SuDs) for reducing stormwater runoff and improving local infiltration within the study area. Unlike underground storage tanks, rain gardens provide both hydraulic and environmental benefits by combining temporary detention, infiltration, evapotranspiration, pollutant removal, and landscape improvement within a single decentralized system.

The design approach adopted in this thesis is based on a simplified hydrological balance between the runoff volume generated by the contributing impervious surfaces and the total effective capacity of the rain garden system. This method allows verification of whether the proposed rain garden area is hydraulically sufficient to retain and infiltrate the design storm event without generating overflow toward the conventional drainage network.

The first step consists of estimating the runoff volume produced by the design rainfall event using the Rational Method principles. The runoff volume is calculated as:

$$V_{runoff} = C \cdot A_c \cdot h / 1000$$

where:

- V_{runoff} is the runoff volume generated during the rainfall event (m^3)
- C is the runoff coefficient of the contributing surface
- A_c is the contributing catchment area (m^2)
- h is the rainfall depth (mm)

Since the study area is mainly composed of roof surfaces, paved residential courtyards, and road networks, relatively high runoff coefficients were adopted based on standard hydrological reference tables. Roof surfaces and paved areas were considered the primary contributors to direct runoff generation.

The second step involves calculating the temporary storage volume provided by the rain garden system. This storage depends on the total area of all rain gardens and the equivalent water depth available for ponding and effective soil storage:

$$V_{stor} = A_{rg} \cdot d_{eq}$$

where:

- V_{stor} is the available temporary storage volume (m³)
- A_{rg} is the total surface area of all rain gardens (m²)
- d_{eq} is the equivalent storage depth (m), including both surface ponding and effective soil void storage

The equivalent water depth represents the combined detention capacity of the surface depression and the engineered filter media, and it is one of the most important design parameters affecting system performance.

In addition to temporary storage, infiltration into the underlying soil significantly contributes to runoff reduction during the rainfall event. The infiltrated volume is estimated using:

$$V_{inf}=A_{rg}\cdot f\cdot t$$

where:

- V_{inf} is the infiltrated water volume (m³)
- A_{rg} is the total rain garden area
- f is the infiltration rate of the soil (m/h)
- t is the rainfall duration (h)

The infiltration rate depends on the characteristics of the native soil and the engineered filter media used in the rain garden structure. Higher infiltration rates improve hydraulic performance and reduce the required storage area, while poorly draining soils require larger rain garden surfaces to achieve the same runoff control capacity.

The total effective hydraulic capacity of the rain garden system is therefore calculated by combining temporary storage and infiltration:

$$V_{garden} = V_{stor} + V_{inf}$$

This methodology corresponds to a storage–infiltration balance approach, commonly used for preliminary rain garden design in residential applications where rapid verification of hydraulic feasibility is required. Unlike highly complex numerical models, this method provides a clear and defendable engineering approach for comparing runoff generation and local detention capacity.

The distributed arrangement of multiple rain gardens within the available courtyard and landscaped areas was also considered during the design phase. Instead of using one centralized basin, several smaller rain gardens were strategically positioned near runoff generation zones to maximize interception efficiency and maintain architectural functionality. This decentralized configuration improves hydraulic efficiency by reducing flow concentration and promoting localized infiltration close to the source of runoff generation.

The results of this comparison are presented in Section 5.4.

Chapter 4 – Case Study

4.1 Study Area and Site Analysis

4.1.1. Introduction

The case study is in the municipality of Pinerolo, in the Piedmont region of northern Italy. The city is a medium-sized urban center situated near the foothills of the Alps and within the metropolitan influence of Turin. Its geographical position gives it a hybrid urban–environmental character, where residential areas coexist with open spaces, agricultural land, and sloped terrain.

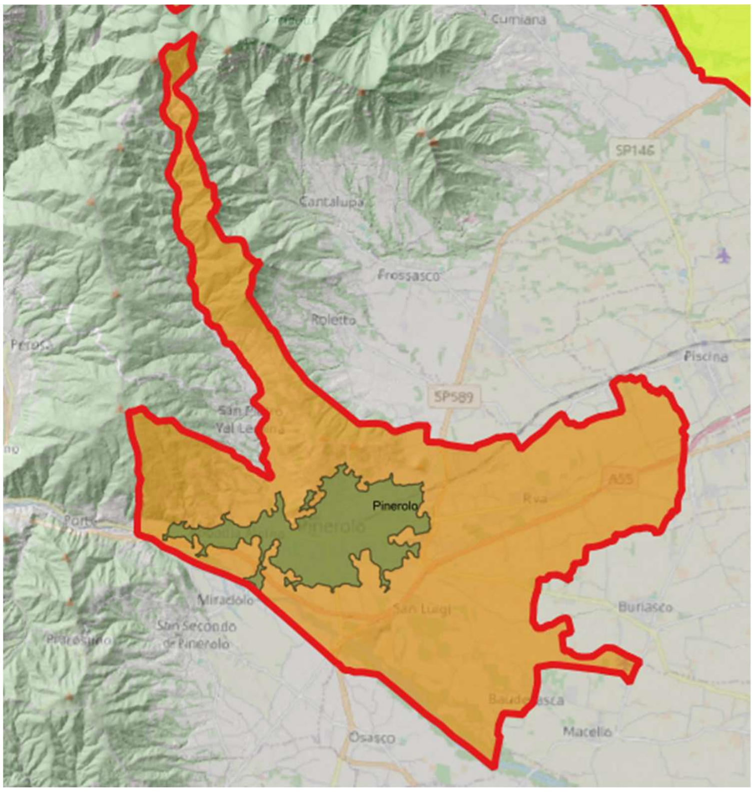


Figure 4.1 *Boundary of the Pinerolo study area within the regional watershed context.*

(Source: Author's elaboration based on Regione Piemonte GIS data)

Urban expansion within the Besucco district has increased impervious cover to approximately 50% of the total area, creating hydraulic conditions representative of medium-density Italian residential development.

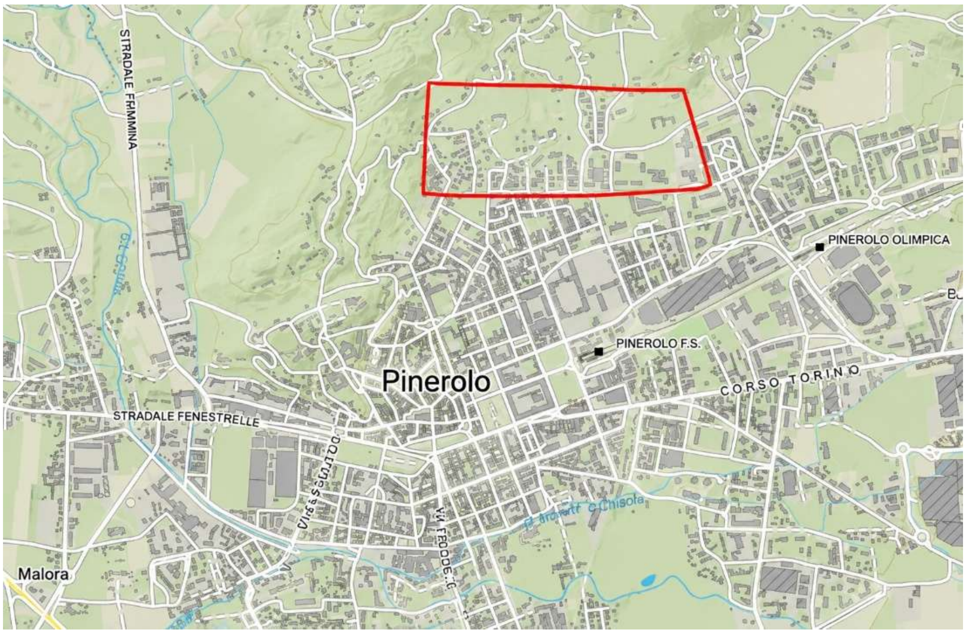


Figure 4.2 *Location of the study area in Pinerolo highlighted on the urban map.*

(Source: OpenStreetMap / Regione Piemonte GIS)

The selected case study is in the Besucco area, in the northern part of the municipality. The total study area covers approximately 92,000 m² (9.2 hectares), representing a medium-scale urban catchment suitable for Rational Method application, which is generally recommended for drainage basins smaller than 50 hectares.

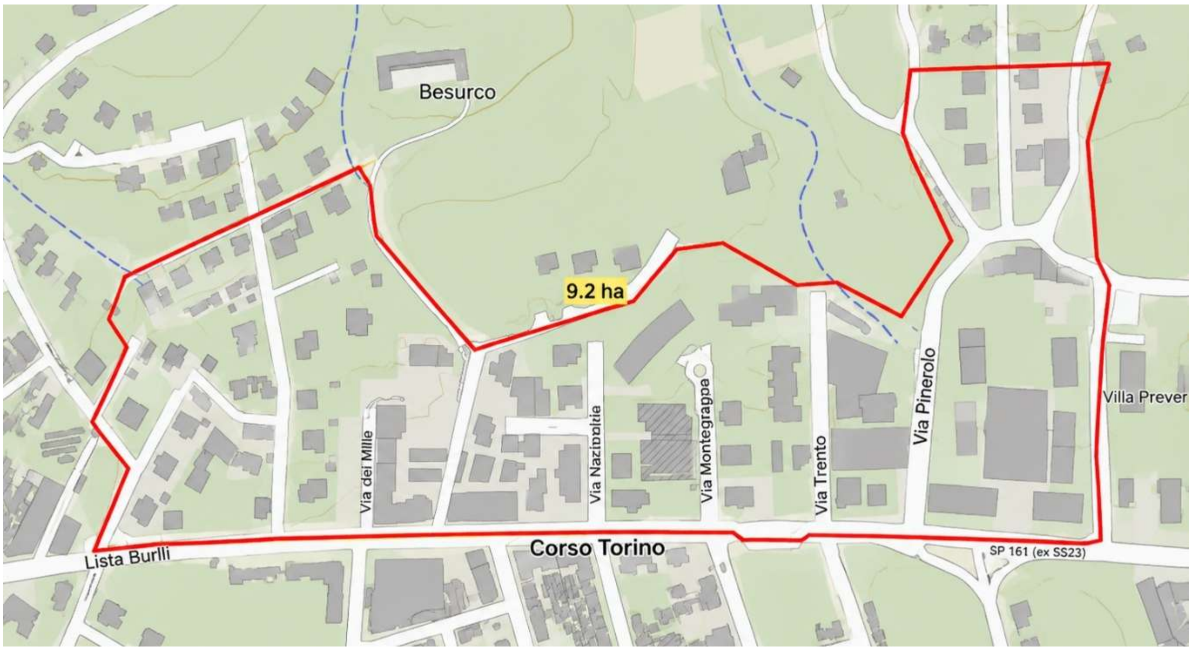


Figure 4.3 *Delineated study catchment area (9.2 ha) used for hydrological runoff analysis in Pinerolo.*

(Source: Author's elaboration based on GIS mapping)

The site includes residential buildings, internal roads, paved surfaces, gardens, agricultural land, and open green areas. This composition creates a transitional zone between dense residential development and semi-natural spaces. Such conditions are particularly relevant for stormwater analysis, as runoff generation is strongly influenced by the coexistence of permeable and impermeable surfaces. The land-use composition of the site is described in detail in Section 4.1.4.

***Figure 4.4** Aerial view of the delineated study catchment area in Pinerolo used for land-use and runoff analysis.*

(Source: Author's elaboration based on satellite imagery)

According to PRG planning maps, PGRA flood risk maps, and PAI hydrogeological classifications, parts of the study area fall within Class II and Class III sensitivity zones, indicating potential environmental constraints such as surface runoff accumulation, localized flooding, and insufficient drainage capacity. These conditions support the need for runoff reduction and source-control strategies through Sustainable Urban Drainage Systems implementation.

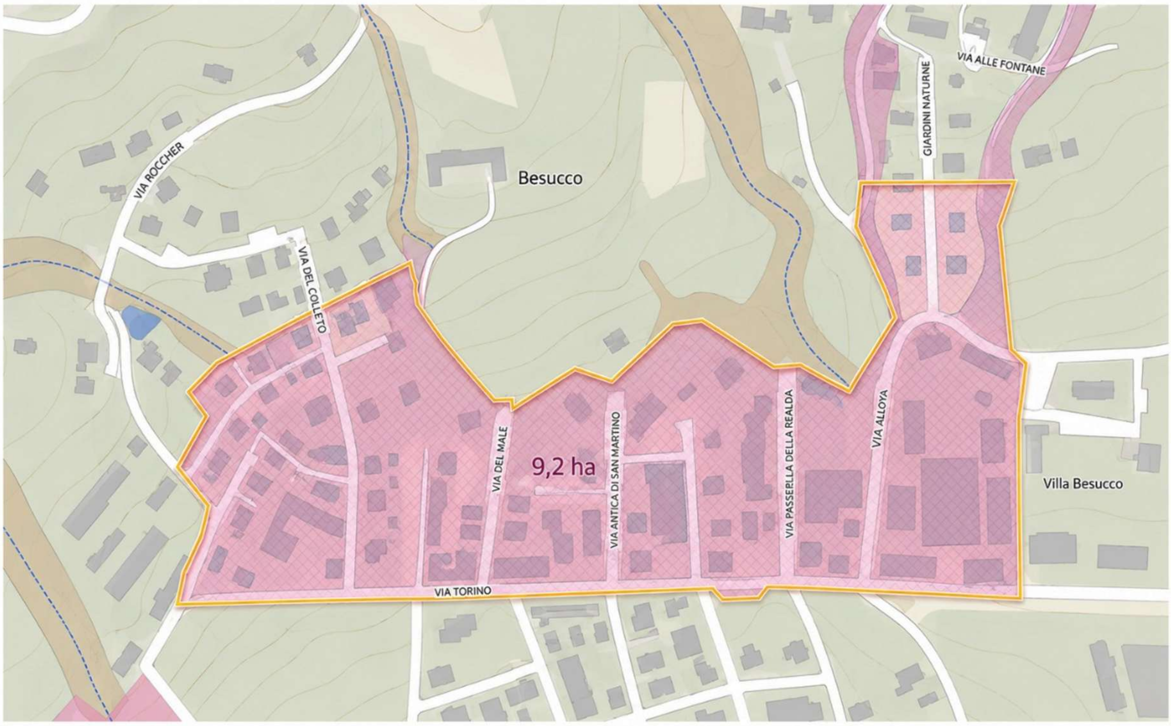


Figure 4.5 *Overlay of the delineated study catchment area (9.2 ha) with hydrogeological constraint zones from the PRG synthesis map.*

(Source: Author's elaboration based on ARPA Piemonte WebGIS data)

Based on these characteristics, the area was selected because it combines several key elements relevant to this thesis, including residential development, road infrastructure, green spaces, sloped terrain, and mapped hydrological sensitivity zones. These features allow for the evaluation of both existing runoff processes and the potential effectiveness of building-scale SuDs solutions, such as rainwater harvesting tanks and rain gardens.

4.1.2 Site Boundaries and Physical Delimitation

The physical boundaries of the site are clearly defined by a network of local and collector roads that strongly influence both urban accessibility and stormwater drainage patterns.

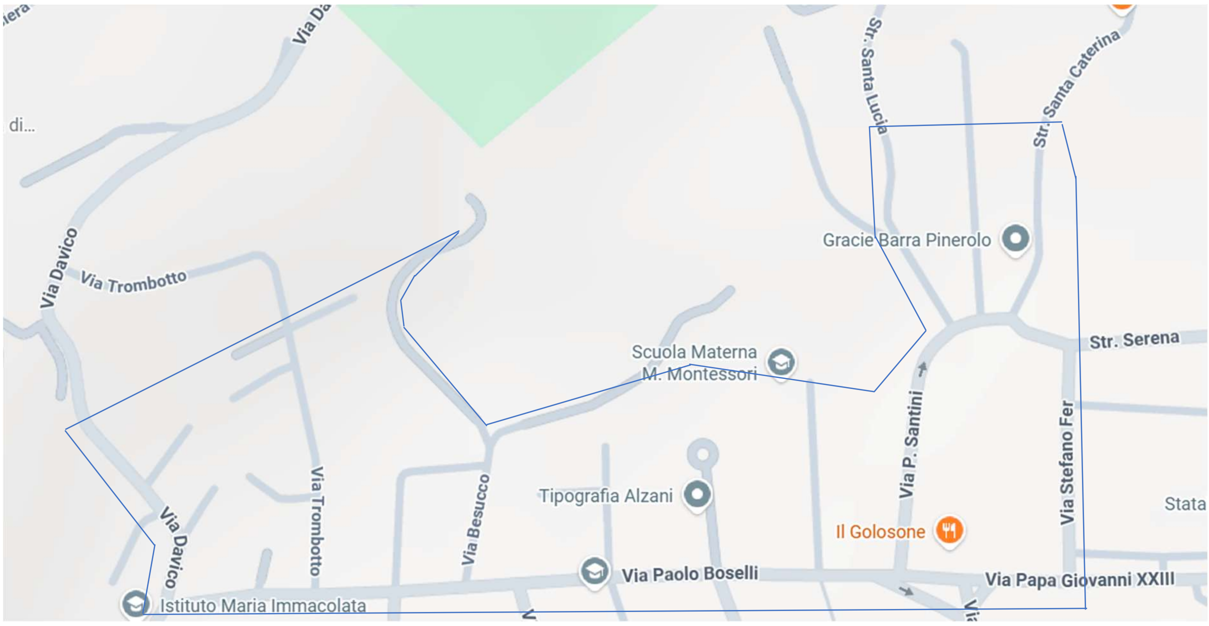


***Figure 4.6** Detailed map of the study area in Pinerolo showing the local road network and spatial layout of the neighborhood.(Source: OpenStreetMap)*

To the south, the area is bordered by Via Paolo Boselli, which acts as one of the main access roads and an important urban connection route within the residential fabric. This road represents a significant impervious surface and plays an important role in runoff conveyance toward the lower parts of the catchment.

To the west, the boundary is defined by Via Davico together with portions of Via Besucco. These roads separate the study area from adjacent residential blocks and contribute to the western hydraulic limits of the basin. Via Besucco also functions as an important internal axis, structuring part of the urban layout and influencing local surface flow directions.

The northern limit of the site is represented by Strada Santa Lucia, located near slightly elevated terrain and transitional open spaces. This upper boundary is particularly relevant from a hydrological perspective because runoff generated from higher elevations tends to move toward the central and southern parts of the study area.

To the east, the site is enclosed by a road system composed of Via P. Santini and Via Stefano Fer. These roads form the eastern hydraulic boundary and help define the final surface runoff pathways toward the drainage collection zones.

The delimitation of the study area was not based only on administrative boundaries, but primarily on hydrological behavior and drainage logic. The selected perimeter includes the main contributing surfaces that directly affect runoff generation, including residential roofs, roads, paved courtyards, green areas, and open permeable land. This allows a more reliable estimation of runoff coefficients, concentration time, and peak discharge.

The road network surrounding the site also contributes significantly to the concentration and redirection of stormwater flows. As impervious linear elements, these roads accelerate runoff generation and influence the location of potential flood-prone areas, particularly in the central and southern sections of the catchment where flow accumulation is more likely to occur.

4.1.3 Topography and Slope Analysis

The study area in the Besucco district presents a gently sloping topography characterized by a transition from the higher northern section toward the lower central and southern parts of the site. This natural gradient is particularly important because it controls the general direction of surface runoff during rainfall events. Water generated from the upper portions of the basin tends to move progressively toward the lower residential areas, where runoff concentration and temporary ponding are more likely to occur.

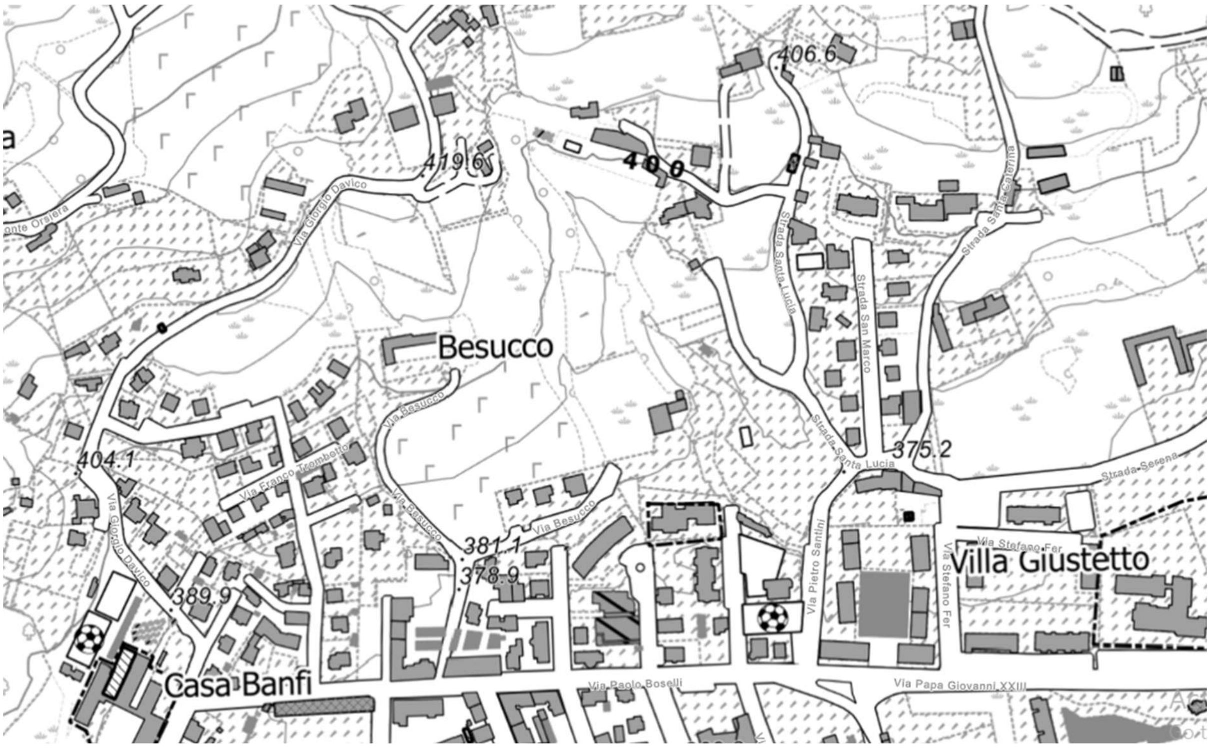


Figure 4.7 Topographic base map of the study area in Pinerolo obtained from the ARPA Piemonte WebGIS platform, showing elevation references, road network, building distribution, and terrain morphology used for watershed delineation and hydrological analysis.

The northern boundary near Strada Santa Lucia is located at a relatively higher elevation and is adjacent to more open and partially permeable land. This zone contributes upstream runoff toward the internal urbanized sections of the catchment. As the terrain descends particularly in the flood-prone zones identified in Section 4.1.5, the percentage of impervious surfaces increases and the capacity for natural infiltration decreases, creating more critical hydraulic conditions.

The average ground slope considered for the hydrological calculations was approximately 6.67%, based on site morphology and drainage path analysis. This value was used in the estimation of access time (t_a) within the Rational Method, as slope directly affects the speed of overland flow before runoff enters the drainage network.

The topographic data therefore confirms that the study area presents a natural hydraulic tendency toward runoff accumulation in its lower sections, justifying the need for decentralized source-control measures capable of intercepting stormwater before it reaches these critical zones.

Rain garden locations were accordingly selected in the flatter northern and central zones, as shown in Figure 4.8.

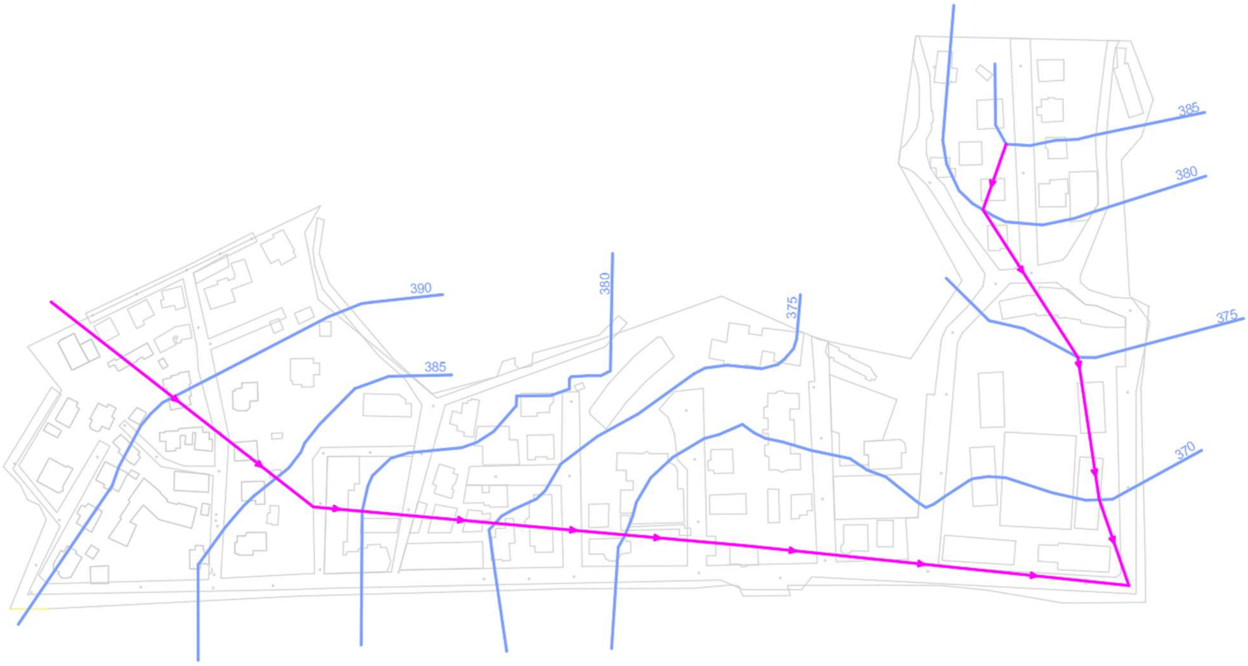


Figure 4.8 *Contour map of the study area showing elevation variations and critical low points influencing surface runoff patterns.*

(Source: Author's elaboration based on topographic data)

4.1.4 Land Use and Surface Classification

Surface classification of the 9.2 ha catchment identified three hydrological categories — roof surfaces, paved impervious areas, and permeable green spaces — each assigned a distinct runoff coefficient for the weighted average calculation.

Residential roof surfaces represent approximately 25% of the total area, corresponding to nearly 23,397 m². These surfaces are considered almost completely impermeable and generate direct runoff immediately after rainfall begins. Since rooftops are smooth and have negligible infiltration capacity, according to the values presented in Table 3.1, a runoff coefficient of 0.90 was adopted for this category. In addition, these surfaces were identified as the primary collection areas for the rainwater harvesting systems analyzed in this thesis.

The road network and paved impervious surfaces also represent approximately 25% of the total area, corresponding to around 24,000 m². This category includes asphalt roads, internal access routes, parking areas, paved courtyards, and pedestrian surfaces. Because these areas have very limited infiltration capacity and rapid runoff response, a runoff coefficient of 0.85 was adopted. These surfaces play a major role in peak flow generation and significantly contribute to hydraulic pressure on the drainage network.

Public green spaces, gardens, agricultural land, and open permeable areas constitute approximately 50% of the total area, corresponding to about 45,200 m². These areas are mainly located in the central and northern sections of the site and include vegetated zones with higher infiltration potential. Although these surfaces contribute less to direct runoff, local compaction and partial urban disturbance reduce their natural permeability. For this reason, a runoff coefficient of 0.30 was adopted for this category.

This distribution highlights an important spatial contrast within the catchment: the southern and southwestern parts are characterized by denser residential development and a higher percentage of impermeable surfaces, while the northern and central sections maintain larger permeable and semi-natural areas. This spatial imbalance strongly influences runoff concentration and explains why flood-sensitive zones are mainly located in the lower and more urbanized parts of the study area.

The land-use classification also provided the spatial basis for selecting the most suitable locations for Sustainable Urban Drainage Systems (SuDs). Roof surfaces were used for rainwater harvesting tank design, while permeable open spaces and landscaped courtyard zones were identified as potential locations for rain gardens and infiltration-based solutions.

The resulting surface classification provided the spatial basis for the SuDs design presented in Chapter 5.

4.1.5 Hydrological Risk and Flood-Prone Areas

The assessment was based on three main sources of territorial information: the PRG (Piano Regolatore Generale) urban planning maps, the PGRA (Piano di Gestione del Rischio Alluvioni – Flood Risk Management Plan), and the PAI (Piano per l'Assetto Idrogeologico) hydrogeological classification layers for the municipality of Pinerolo.

The PAI hydrogeological classification indicates that significant portions of the site fall within Class II and Class III sensitivity zones. These classes identify areas subject to environmental constraints and potential hydrogeological problems such as surface runoff accumulation, localized flooding, limited drainage efficiency, and in some cases minor soil instability. These conditions are particularly evident in the central and southern sectors of the catchment, where topographic depression and higher urban density coincide.

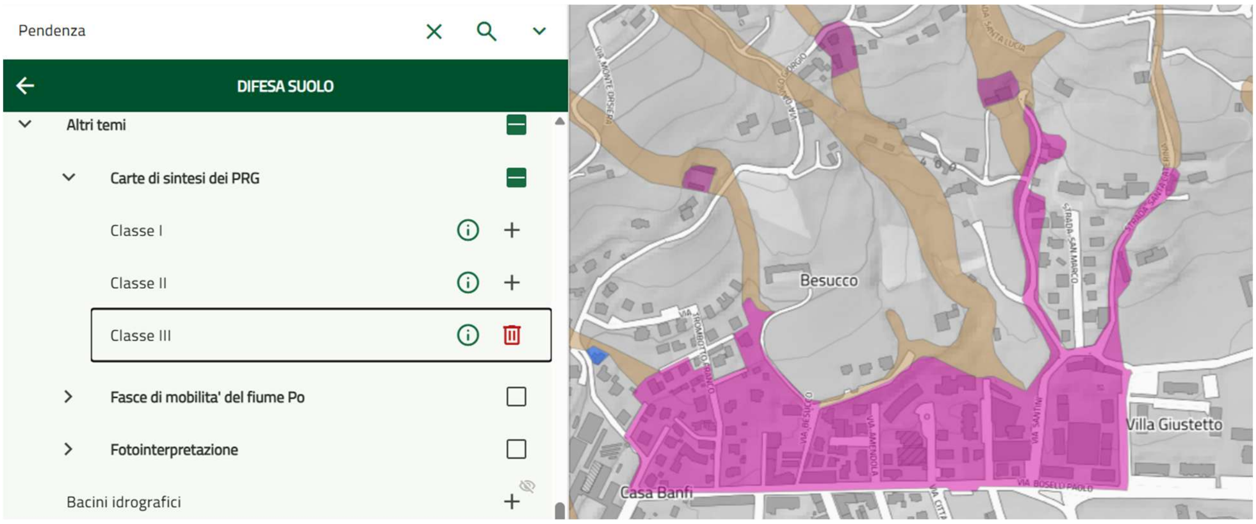


Figure 4.9 *PRG land suitability map showing soil classification (Class III) and areas with geomorphological and hydrogeological constraints within the study area.*

(Source: ARPA Piemonte WebGIS – PRG Synthesis Map)

The PGRA flood risk maps further confirm the presence of flood-prone zones within the study area. These areas, generally represented by hatched sections in the planning maps, are mainly concentrated in the central portion of the site and correspond to locations where runoff accumulation is most likely to occur during intense rainfall events. Their spatial alignment with the main surface flow paths suggests that stormwater generated from higher elevations is progressively directed toward these lower zones, increasing the probability of temporary flooding and hydraulic surcharge.

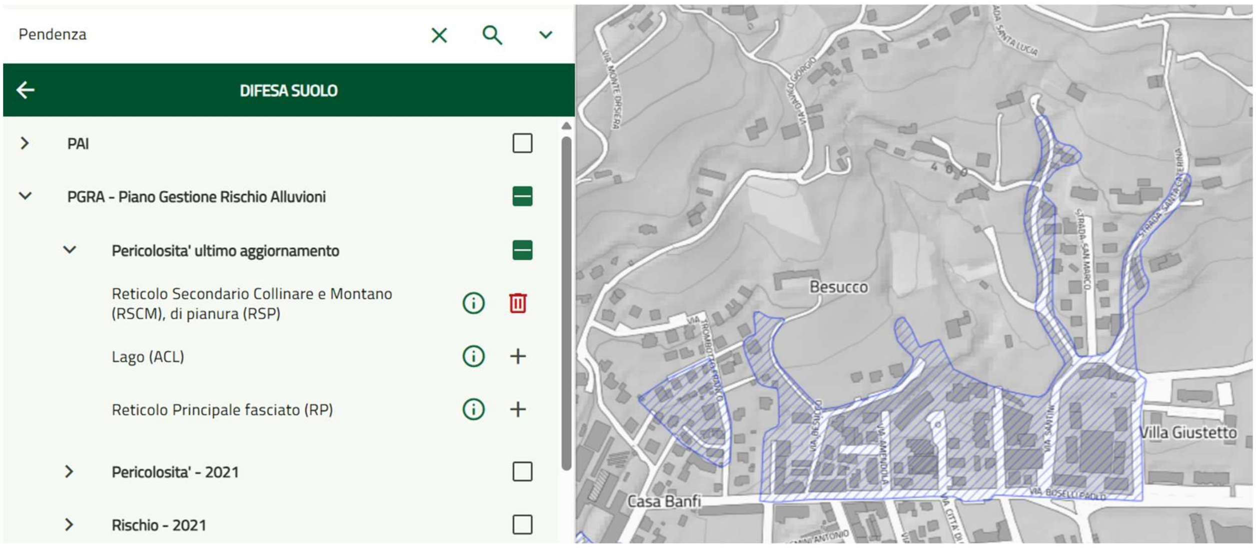


Figure 4.10 *Flood hazard map (PGRA) showing areas exposed to hydraulic risk within the study area.*

(Source: ARPA Piemonte WebGIS – PGRA)

The proximity of these flood-sensitive areas to residential buildings, access roads, and local infrastructure significantly increases the vulnerability of the site. During short-duration high-intensity storms, even moderate runoff concentration may generate operational problems such as road ponding, temporary inaccessibility, drainage inlet surcharge, and localized flooding of low-elevation urban spaces.

This vulnerability is further intensified by the fact that the existing conventional drainage network likely operates primarily through rapid runoff conveyance rather than local detention and infiltration. As a result, the system lacks sufficient hydraulic flexibility to manage sudden peak discharges generated by upstream impervious surfaces.

The identification of these hydrological risk zones provides strong technical justification for the implementation of decentralized source-control strategies.

Rather than increasing only the downstream drainage capacity, building-scale interventions such as rainwater harvesting tanks and rain gardens allow runoff to be reduced directly at the source, decreasing the hydraulic load before water reaches the critical flood-prone zones.

4.1.6 Residential Water Demand Data

For the design of rainwater harvesting tanks using the Simplified Method, an estimate of non-potable water demand was required. Demand values were derived from the number of residents, household characteristics, and standard references for domestic water consumption, allowing the balance between potential rainwater supply and water demand to be assessed.

Non-potable water demand included toilet flushing, irrigation, outdoor cleaning, and general service uses. The average demand per person was assumed according to European residential water reuse standards, 50 liters per person per day.

These data were combined with roof catchment area and rainfall input to simulate tank performance and determine the optimal storage capacity.

Regione	Piemonte
Città metrop.	Torino (**TO**)
Codice Istat	001191
Cod.catastale	**G674**
Popolazione	35.439 ab. (01/01/2025 - Istat)
Superficie	49,70 km^2
Densità	713,07 ab./km^2
Prefisso	0121
CAP	**10064**

Figure 4.11 *General administrative and demographic information of the municipality of Pinerolo, including regional location, population, total surface area, and urban density, used for the territorial characterization of the study area.*

(Source: Official municipal and ISTAT territorial database)

Tipo dato	abitazioni occupate al 31 dicembre			
Seleziona periodo	2019			
Tipo di possesso	proprietà	affitto	altro titolo diverso da proprietà, affitto	tutte le voci
Territorio				
Pessinetto	232	55	32	319
Pianezza	4 909	823	655	6 387
Pinasca	1 450	461	147	2 058
Pinerolo	**11 213**	**4 932**	**771**	**16 917**
Pino Torinese	2 861	547	299	3 707
Piobesi Torinese	1 299	241	39	1 579
Pisassco	6 074	1 277	349	7 701
Piscina	1 104	311	30	1 445
Piverone	471	40	70	581
Poirino	2 828	1 022	362	4 212
Pomaretto	314	129	41	484

Figure 4.12 *Housing occupancy data for the municipality of Pinerolo obtained from the ISTAT GIS Portal, showing the number of occupied residential units classified by ownership type (ownership, rental, and other tenure conditions), used for estimating residential water demand and non-potable consumption patterns.*

(Source: ISTAT GIS Portal – IstatViewer, Italian National Institute of Statistics)

COMUNE	CAP	VIA	FAMIGLIE
PINEROLO	10064	STRADA SAN MARCO	31
PINEROLO	10064	STRADA SANTA CATERINA	5
PINEROLO	10064	STRADA SANTA LUCIA	10
PINEROLO	10064	VIA ACHILLE GRANDI	27
PINEROLO	10064	VIA BEATA MARGHERITA D'ACAJA	12
PINEROLO	10064	VIA RESUCCO	16
PINEROLO	10064	VIA FRANCO TROMBOTTO	41
PINEROLO	10064	VIA GIOVANNI AMENDOLA	20
PINEROLO	10064	VIA PAOLO BOSELLI	53
PINEROLO	10064	VIA PIETRO SANTINI	47
PINEROLO	10064	VIA SAN GIOVANNI	59
PINEROLO	10064	VIA STEFANO FER	203

NUMBER OF FAMILY	524
NUMBER OF PEOPLE	1 100.4
NUMBER OF BUILDING	90
AREA OF BUILDING (m^2)	23 426

Data di riferimento 31 dicembre 2019	Comune PINEROLO (TO)	CAP 10064

Table 4.1 *Residential building and household distribution data for the selected study area in Pinerolo, including the number of families, estimated population, number of buildings, and total building area, used for domestic water demand estimation and rainwater harvesting system sizing.*

(Source: Regione Piemonte Open Data Portal – Territorial and demographic database, author's elaboration)

4.1.7. Climatic and Rainfall Data

Rainfall data for Pinerolo were obtained from the ARPA Piemonte Atlante delle Piogge Intense, which provides the official IDF relationships for the Piedmont region.

The municipality of Pinerolo is in the Piedmont region of northern Italy and is characterized by a temperate climate with moderate annual precipitation and seasonal variability. According to regional climatic records and the ARPA Piemonte hydrological database, the average annual rainfall in the study area is approximately 720 mm/year. Rainfall events are generally concentrated in spring and autumn, when short-duration high-intensity storms are more frequent and urban flooding risk becomes more significant.

For the hydrological design developed in this thesis, rainfall data were obtained from the “Atlante delle Piogge Intense in Piemonte” published by ARPA Piemonte, which represents the standard regional reference for rainfall probability analysis and urban drainage design. This atlas provides the Intensity–Duration–Frequency (IDF) relationships necessary for determining design rainfall intensity based on the selected return period and rainfall duration.

The rainfall intensity is expressed through the general IDF relationship:

$$i = a \cdot t^{n-1}$$

where:

- i is the rainfall intensity (mm/h)
- a is the climatic coefficient ($mm \cdot h^{-n}$)
- t is the rainfall duration (h)
- n is the empirical exponent related to rainfall distribution

For the municipality of Pinerolo, the regional rainfall probability curve parameters were identified as:

- a = 26.2
- n = 0.39

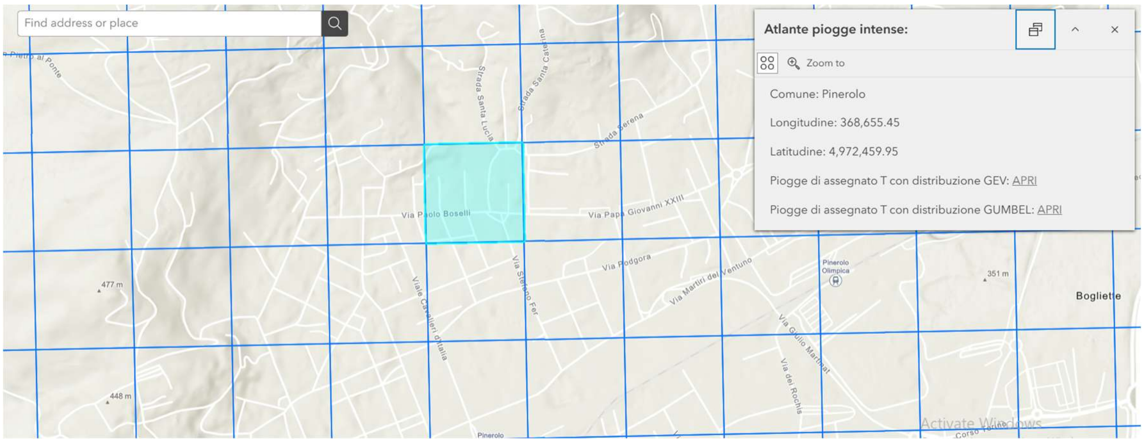


__Figure 4.13__ Location of the selected rainfall grid cell from the Intense Rainfall Atlas (Atlante Piogge Intense) for the study area in Pinerolo, used for the extraction of design rainfall parameters for hydrological calculations.

(Source: Regional Rainfall Atlas of Piedmont Region, Italy)

Atlante piogge intense in Piemonte (GUMBEL)

Comune di **Pinerolo** (lat: 4972459.95 , lon: 368655.45)

Parametri della curva di probabilità pluviometrica. **a: 26.2 n: 0.39**

CSV Excel

Fattore di crescita KT

K2	K5	K10	K20	K50	K100	K200
0.94	1.265	1.48	1.686	1.954	2.154	2.354

Piogge di assegnato tempo di ritorno per durate da 10 minuti a 24 ore (mm)

CSV Excel

Durata	Tempo di ritorno in anni						
	2	5	10	20	50	100	200
10 minuti	12.2	16.4	19.2	21.8	25.3	27.9	30.5
20 minuti	16.1	21.6	25.3	28.8	33.4	36.8	40.2
30 minuti	18.8	25.3	29.6	33.8	39.1	43.2	47.2
1 ora	24.6	33.1	38.8	44.2	51.2	56.4	61.7
3 ore	37.6	50.6	59.2	67.5	78.2	86.2	94.2
6 ore	49.1	66.1	77.4	88.1	102.1	112.6	123
12 ore	64.1	86.3	101	115.1	133.4	147.1	160.7
24 ore	83.8	112.8	132	150.4	174.2	192.1	209.9

__Figure 4.14__ Intense Rainfall Atlas of Piedmont Region (Gumbel distribution) for the municipality of Pinerolo, showing the pluviometric probability curve parameters (a = 26.2, n = 0.39) and the design rainfall depths

associated with different return periods and storm durations used for hydrological calculations.

(Source: ARPA Piemonte – Atlante delle Piogge Intense, Piedmont Region, Italy)

These values describe the reference rainfall behavior for the area and must be adjusted according to the selected return period using the appropriate growth factor K_t.

In this study, a 10-year return period was adopted as the design criterion for the Rational Method analysis and the baseline drainage assessment. According to the ARPA Piemonte atlas, the growth factor corresponding to a 10-year return period is:

- $K_{10} = 1.48$

Therefore, the adjusted rainfall coefficient becomes:

$$a_{10} = a \cdot K_{10} = 26.2 \cdot 1.48 = 38.776 \ (\mathrm{mm} \cdot \mathrm{h}^{-n})$$

This corrected value was used in all peak discharge calculations performed with the Rational Method.

These parameters are applied directly in the Rational Method calculations of Section 5.2.

4.2. Hydrological studies

The objective of this section is to determine the maximum peak discharge generated by the critical storm event with a 10-year return period in the site area. This value will subsequently be used as the reference basis for assessing both the existing condition and the proposed interventions.

The reliability of the hydrological analysis depends strongly on the correct identification and justification of the main modeling parameters used in the Rational Method and in the design of Sustainable Urban Drainage Systems (SuDs). Parameters such as runoff coefficient, concentration time, rainfall intensity, flow path length, average slope, and hydraulic roughness directly influence the estimation of peak discharge and the evaluation of runoff reduction strategies.

The first and most important parameter is the runoff coefficient (C), which represents the fraction of rainfall converted into direct surface runoff. Since the study area contains both permeable and impermeable surfaces, a weighted runoff coefficient was calculated using the proportional contribution of roofs, roads, and green areas.

The final weighted runoff coefficient, calculated based on the equation presented in Section 3.2.1 and the values adopted for each land-use category defined in Section 4.1.4, was obtained as follows:

$$C = \frac{(23397 * 0.9) + (46756 * 0.3) + (24000 * 0.85)}{23397 + 46756 + 24000} = 0.59$$

This value reflects the interaction between permeable and impermeable surfaces and was used as the reference runoff coefficient for the Rational Method calculations.

The second key parameter is the drainage area (A), which includes all surfaces contributing runoff toward the drainage network. This value is fully compatible with the applicability limits of the Rational Method, which is generally recommended for urban catchments smaller than 50 hectares.

The time of concentration (t_c) was determined as the sum of access time (t_a) and travel time (t_r):

$$tc = t_a + t_r$$

The access time depends primarily on surface slope, overland flow distance, and surface characteristics. Based on topographic analysis, the following values were adopted:

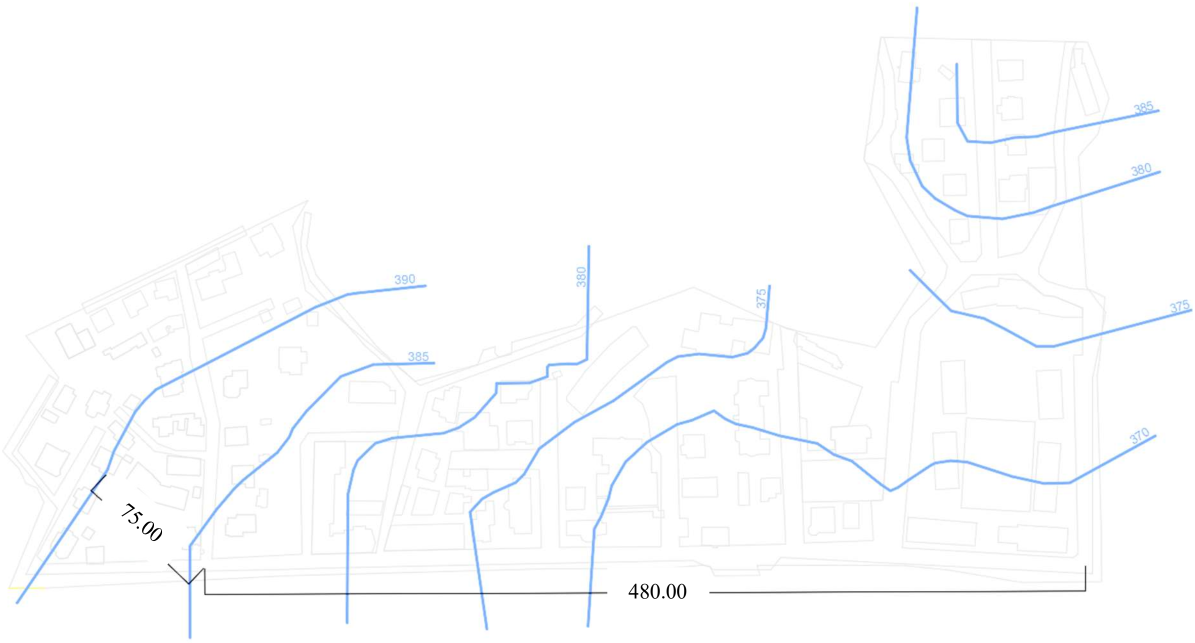


Figure 4.15 *Surface flow length and drainage network flow length used for concentration time (tc) calculation in the study area.(Source: Author's elaboration)*

- Surface flow length (l_i): 75 m
- Average basin slope (s_i): 6.67%

These values represent one of the longest runoff paths within the catchment and provide a conservative estimate of runoff concentration. Based on the above values and the equations presented in Section 3.2.2, the following results were obtained:

$$t_a = \left(\frac{3600^{\frac{0.39-1}{4}} \cdot 0{,}5 \cdot 75}{0.067^{0{,}375} \cdot (38.776 \cdot 0.59 \cdot 9.2)^{0{,}25}} \right)^{\frac{4}{0.39+3}} = 11.3\ S = 0.2\ min$$

The travel time inside the drainage network depends on the length of the hydraulic path, network slope, and internal roughness of the drainage system. The adopted parameters were:

- Maximum drainage network length (l_r): 480 m
- Average hydraulic slope (p): 4%
- Gauckler–Strickler roughness coefficient (K_s): 70

The selected value of K_s corresponds to typical concrete or plastic drainage pipes and reflects the existing urban drainage characteristics of the site.

Using these parameters, the final concentration time was calculated as:

$$t_r = \left(\frac{23,6 \cdot \left(\frac{480}{70}\right)^{0,6}}{3600^{0,4 \cdot (1-0.39)} \cdot \left(\frac{38.776}{1000}\right)^{0,4} \cdot 0.04^{0,3}} \right)^{\frac{1}{0,6+0,4 \cdot 0.39}} = 496.1\ S = 8.3\ min$$

Therefore, the final concentration time for the study area was:

$$t_c = 507.4\ s = 8.5\ min = 0.14\ h$$

The final rainfall intensity associated with the concentration time of the basin was calculated using the equation presented in Section 3.2.2 and the values obtained for parameters a and n in Section 4.1.7, as follows:

$$i = 38.776.(\frac{8.5}{60})^{0.39-1}$$

$$i = 127 \text{ mm/h}$$

which corresponds to a rainfall depth of:

$$h = 17.8 \text{ mm}$$

The complete set of hydrological parameters used in this thesis is summarized in Table 4.2 This table provides the engineering basis for all calculations developed in the following sections and ensures transparency and reproducibility of the methodological approach.

Parameter	Symbol	Value	Unit	Source / Justification
Total drainage area	A	9.2	ha	GIS land-use analysis
Weighted runoff coefficient	C	0.59	-	Weighted land-use calculation
Climatic coefficient	a	38.776	-	ARPA Piemonte
Rainfall exponent	n	0.39	-	ARPA Piemonte
Return period	T	10	years	Residential drainage design
Rainfall intensity	i	127	mm/h	IDF calculation
Rainfall depth	h	17.8	mm	$i \times t_c$
Access flow length	l_i	75	m	Site geometry
Network flow length	l_r	480	m	Drainage path
Average surface slope	s_i	6.67	%	Topographic analysis
Network slope	p	4	%	Drainage network
Roughness coefficient	K_s	70	-	Standard pipe value
Concentration time	t_c	8.5	min	Hydraulic calculation

Table 4.2 *Summary of hydrological and hydraulic modeling parameters used for peak discharge estimation with the Rational Method, including drainage area, runoff coefficient, rainfall intensity parameters, and time of concentration.*

Source: Author's elaboration based on GIS analysis and ARPA Piemonte data

Based on the above calculations and considering the equation presented in Section 3.2, the maximum peak discharge generated by runoff during the critical storm event in the study area was calculated as follows:

$$Q = \frac{10 \cdot 0.59 \cdot 38.776 \cdot 9.2}{(\frac{8.5}{60})^{(1-0.39)}} = 6999.6\frac{m^3}{h} = 1.944\ \frac{m^3}{s}$$

4.3. Existing Stormwater Drainage Network

Figure 4.16 *Estimated locations of stormwater manholes identified from aerial imagery and urban layout analysis within the study area.*

(Source: Author's elaboration based on aerial imagery)

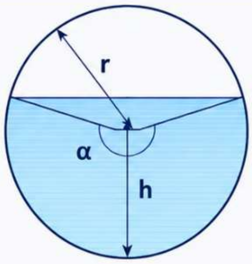
The assessment of the existing stormwater drainage network was carried out based on the location of the manholes and by considering a 630 mm diameter stormwater conveyance pipe located at the downstream end of the network. Since no official documentation regarding the pipe diameter was available, the adopted downstream pipe diameter of 630 mm should be considered an engineering assumption based on the observed geometric characteristics of the drainage system. Based on this assumed diameter, the existing drainage network appears to be insufficient under the 10-year design storm. Therefore, the result should be interpreted as an engineering assessment rather than a definitive cadastral verification.

The roughness coefficient values were selected based on standard hydraulic references for concrete pipes. Furthermore, the discharge capacity was evaluated for different water depths within the pipe, allowing the development of a discharge–depth relationship for assessing the hydraulic performance of the drainage system under various runoff conditions. Based on the equations presented in Section 3.3, the maximum conveyance discharge capacity of the pipe was calculated as follows:

SECTION CHARACTERISTICS

KNOWN DATA (to be entered)

Symbol		Value	Unit	Description
d	⇨	0.60	m	Diameter
r	⇨	0.30	m	Radius
h	⇨	0.48	m	Water depth
p	⇨	4.0	%	Slope
m	⇨	0.2	-	Kutter roughness coefficient



RESULTING DATA

Parameter	Symbol	Formula		Result	Unit
Central angle	α	-	⇨	254.1	°
Wetted perimeter	P_b	$P_b = 2\pi\left(\frac{\alpha}{360^o}\,r\right)$	⇨	1.328	m
Flow area	A	$A = \frac{1}{2}\,r^2\left(\frac{\pi\alpha}{180^o} - \sin\alpha\right)$	⇨	0.2421	m²
Hydraulic radius	R_i	$R_i = \frac{A}{P_b}$	⇨	0.182	m

DISCHARGE CAPACITY FOR A WATER DEPTH h = 0.48 m

FORMULAS (uniform flow)

Parameter	Formula	Where
Discharge	$Q = AV$	A = Flow area V = Flow velocity
Flow velocity	$V = c\sqrt{R_i\,p}$	c = friction coefficient R_i = hydraulic radius p = slope
Friction coefficient	$c = \frac{100\sqrt{R_i}}{m + \sqrt{R_i}}$	m = Kutter roughness coefficient

RESULTS

Symbol		Value	Unit
c	⇨	68.10	-
V	⇨	5.81	m/sec
Q	⇨	1.407	m³/sec

By comparing the runoff discharge generated by the rainfall event, calculated in Section 4.2, with the conveyance discharge capacity of the pipe calculated in this section, it can be concluded that the existing drainage network does not have sufficient capacity to convey the total runoff generated during the critical storm event with a 10-year return period. Therefore, the study area requires intervention measures to compensate for this hydraulic deficiency.

Chapter 5 – Results and Discussion

5.1 General Results Framework and Building-Scale Assessment

Since runoff generation originates primarily from roofs, private courtyards, paved access areas, and small impermeable surfaces, the study adopted a decentralized approach in which each residential parcel was analyzed separately. For every individual property, roof area, yard area, runoff volume, rainwater harvesting tank capacity, rain garden feasibility, overflow volume, and implementation cost were calculated independently.

This building-scale methodology allowed a more realistic evaluation of source-control strategies compared to simplified catchment-wide assumptions. Instead of proposing one centralized detention structure, the study investigated how multiple small-scale decentralized interventions distributed across individual houses could collectively reduce hydraulic pressure on the conventional drainage network.

More than 90 residential parcels were included in the analysis. For each parcel, rainwater harvesting tanks and rain gardens were assessed as separate intervention scenarios, allowing their individual hydraulic and economic performance to be evaluated independently.

This building-by-building approach made it possible to evaluate hydraulic performance, economic feasibility, and implementation practicality for each house individually, while also measuring the cumulative hydraulic improvement at the urban catchment scale.

5.2 Baseline Hydrological Condition

The baseline hydrological assessment of the study area was carried out using the Rational Method to evaluate runoff conditions before the implementation of any Sustainable Urban Drainage Systems (SuDs).

Within each parcel, roof surfaces and yard areas were separated from one another, and the runoff generated from each surface was calculated according to their respective runoff coefficients. The following table presents part of the calculations performed for each individual parcel, as well as the cumulative results for all parcels combined. It should be noted that the complete table is available in the appendix of this study.

Parcel number	Parcel area (total) (m²)	Roof area (m²)	Yard area (m²)	V runoff roof (m³)	V runoff yard (m³)	V runoff total for 10 years return period (m³)
180	39	15	24	0.2	0.1	**0.4**
221	168	28	140	0.4	0.7	**1.2**
227	931	33	898	0.5	4.8	**5.3**
343	100	34	66	0.5	0.4	**0.9**
210	2,023	1,058	965	16.9	5.2	**22.1**
299	1,838	1,060	778	17.0	4.2	**21.1**
850	1,830	1,830	0	29.3	0.0	**29.3**
TOTAL	**69,015**	**23,397**	**46,756**	**373.6**	**249.7**	**623.3**
	m²	m²	m²	m³	m³	m³

Table 5.1 *Parcel-level of roof area, yard area, and total stormwater runoff volume for the 10-year return period design storm across all residential parcels in the study area. Source: Author's elaboration based on GIS land-use analysis and Rational Method calculations*

The total runoff volume generated during the design storm event was calculated as approximately 623.3 m³, of which:

- Roof runoff contributed approximately 373.6 m³
- Yard and paved surface runoff contributed approximately 249.7 m³

These results clearly demonstrate that roof surfaces represent the dominant source of stormwater generation within the residential catchment and therefore provide the most effective starting point for source-control interventions.

5.3 Performance and Cost Evaluation of Rainwater Harvesting Systems

Two tank sizing approaches were applied and compared: a roof-area-based method aligned with commercially available tank sizes, and a demand–supply method based on annual rainfall and household non-potable water consumption.

The objective of using different approaches was not only to calculate the required tank volume, but also to understand how the design changes when different criteria are adopted. In fact, rainwater harvesting systems can be designed from different perspectives: practical installation feasibility, flood control, or water demand satisfaction. For this reason, the comparison between the two methods provides a more complete basis for selecting the most appropriate solution.

5.3.1 Roof-Area-Based Tank Sizing Scenario

In the first approach, tank volumes were assigned to each residential parcel according to the available roof surface. This method represents a practical and implementation-oriented sizing strategy. The resulting storage capacities were then evaluated under the critical design storm event considered in this study to assess their overall hydrological performance.

A simplified classification was adopted based on commercially available residential tank sizes. For buildings with roof areas between 0 and 200 m², a tank volume of 2000 L was assigned. For roof areas between 200 and 400 m², a tank volume of 3000 L was selected. For buildings with roof areas greater than 400 m², a tank volume of 5000 L was adopted.

The table below presents a representative excerpt of the calculations performed for the individual parcels included in the study. It should be noted that the runoff volumes reported in this section refer exclusively to runoff generated from roof surfaces, which were considered the collection areas for the rainwater harvesting systems. The complete dataset and calculation results are provided in the Appendix.

Particell number	V runoff roof (m^3)	V runoff yard (m^3)	V runoff totall for 10 years return priod (m^3)	tank size based on 10 years runoff (m^3)	over flow (m^3)	cost (tank)
180	0.2	0.1	**0.4**	2.0	0	1,300 €
221	0.4	0.7	**1.2**	2.0	0	1,300 €
227	0.5	4.8	**5.3**	2.0	3	1,300 €
343	0.5	0.4	**0.9**	2.0	0	1,300 €
195	0.6	0.9	**1.5**	2.0	0	1,300 €
1084	11.8	12.5	**24.3**	5.0	19	2,000 €
197	12.4	8.7	**21.1**	5.0	16	2,000 €
377	16.2	0.0	**16.2**	5.0	11	2,000 €
330	16.7	2.3	**19.1**	5.0	14	2,000 €
210	16.9	5.2	**22.1**	5.0	17	2,000 €
299	17.0	4.2	**21.1**	5.0	16	2,000 €
850	29.3	0.0	**29.3**	5.0	24	2,000 €
TOTAL	**373.6**	**249.7**	**623.3**	**237.0**	**394**	**131,300 €**
	m^3	m^3	m^3	m^3	m^3	€

Table 5.2 *Roof-area-based rainwater harvesting tank sizing: assigned tank volumes, overflow volumes, and installation costs per residential parcel for the standard storage configuration (total capacity: 237 m³).*

Source: Author's elaboration

This procedure resulted in a distributed rainwater harvesting configuration, whereby each residential parcel was assigned an individual storage tank sized according to its estimated roof runoff volume. The total storage capacity for the study area was obtained by aggregating the capacities of all individual tanks, resulting in an overall storage volume of approximately 237 m^3.

The total investment cost associated with this configuration is approximately €131,300, providing a reasonable balance between storage performance and implementation cost. It should be noted that this estimate includes the purchase and installation costs of the storage tanks only. Additional expenses related to excavation works, plumbing modifications, and the connection of the system to the internal water distribution network may affect the final implementation cost.

5.3.2 Extended Capacity Analysis of the Roof-Area-Based Scenario

To further investigate the effect of increased storage capacity during the critical design storm event, an additional scenario was analyzed in which the volume of each storage tank was approximately doubled. This resulted in a total storage capacity of approximately 474 m^3 across the study area.

Parcel number	V runoff roof (m³)	V runoff yard (m³)	V runoff total for 10 years return period (m³)	tank size based on 10 years runoff (m³)²	over flow (m³)²	cost (tank)²
180	0.2	0.1	**0.4**	4	0	3,200 €
221	0.4	0.7	**1.2**	4	0	3,200 €
227	0.5	4.8	**5.3**	4	1	3,200 €
343	0.5	0.4	**0.9**	4	0	3,200 €
330	16.7	2.3	**19.1**	10	9	4,900 €
210	16.9	5.2	**22.1**	10	12	4,900 €
299	17.0	4.2	**21.1**	10	11	4,900 €
850	29.3	0.0	**29.3**	10	19	4,900 €
TOTAL	**373.6**	**249.7**	**623.3**	**474**	**149**	**323,800 €**
	m³	m³	m³	m³	m³	€

Table 5.3 *Roof-area-based rainwater harvesting tank sizing: assigned tank volumes, overflow volumes, and installation costs per residential parcel for the increased-capacity configuration (total capacity: 474 m³).*

(Source: Author's elaboration based on UNI/TS 11445)

The results indicate a clear improvement in hydraulic performance under the critical design storm event considered in this study. In particular, the total overflow volume decreases significantly, from approximately 394 m³ in the initial configuration to about 149 m³ in the increased-capacity scenario. These results suggest that larger storage capacities improve the system's ability to retain runoff and reduce overflow volumes during the critical rainfall event.

However, this improvement comes at a considerable cost. The total investment increases from approximately €131,300 to €323,800, corresponding to an increase of about 2.5 times.

This comparison highlights a key aspect of the performance of the rainwater harvesting system: the relationship between storage volume, performance, and cost is not linear. While increasing storage capacity does improve performance, the magnitude of this improvement becomes progressively smaller relative to the additional investment required. In other words, the marginal hydraulic benefit decreases as storage volume increases.

5.3.3 Demand–Supply-Based Tank Sizing Method (Simplified Method)

In addition to the roof-area-based approach, a method was adopted to estimate tank volumes based on the balance between annual rainfall supply and domestic non-potable water demand. Compared to the previous method, the annual rainfall volume collected from roof surfaces was compared with the corresponding annual non-potable water demand. Two key storage parameters were defined: the useful tank volume and the real tank volume. The useful volume represents the fraction of storage that effectively contributes to water reuse, whereas the real tank volume reflects the actual storage capacity required under practical operating conditions, considering system inefficiencies and temporal mismatches between supply and demand.

The corresponding equations used for the calculations of this method were presented in Section 3.4. The estimation of the number of residents for each parcel was obtained through relatively detailed analyses using official statistical data described in Section 4.1.6. Ultimately, a population coefficient of 0.05 persons per meter square was adopted.

The following table presents part of the calculations together with the overall results. It should be noted that the complete table for all parcels will be available in the appendix.

Parcel number	Parcel area (m^2)	Roof area (m^2)	Yard area (m^2)	Number of person	D: Domestic usage (annually) (m^3)	Q: Annual rainfall (roof) (m^3)	V useful tank (m^3)	V real tank (m^3)
180	39	15	24	1	5.5	9.7	0.3	0.5
221	168	28	140	1	10.2	18.2	0.6	0.9
227	931	33	898	2	12.0	21.4	0.7	1.1
343	100	34	66	2	12.4	22.1	0.7	1.1
195	209	37	172	2	13.5	24.0	0.8	1.2
100	190	44	146	2	16.1	28.6	1.0	1.4
330	1,481	1,045	436	52	381.4	678.3	22.9	34.3
210	2,023	1,058	965	53	386.2	686.7	23.2	34.8
299	1,838	1,060	778	53	386.9	688.0	23.2	34.8
850	1,830	1,830	0	92	668.0	1,187.8	40.1	60.1
TOTAL	**69,015**	**23,397**	**46,756**	**1,169.9**	**8,539.9**	**15,185.7**	**512.4**	**768.6**
	m^2	m^2	m^2	–	m^3	m^3	m^3	m^3

Table 5.4 *Demand–supply-based rainwater harvesting tank sizing*

(Source: Author's elaboration based on UNI/TS 11445)

For the study area, the total annual domestic water demand was estimated at approximately 8,539.9 m^3, while the total annual rainfall volume collected from roof surfaces reached approximately 15,185.7 m^3. The analysis, considering a conversion coefficient of 0.06 to account for the number of dry days throughout the year, resulted in a total useful storage volume of approximately 512.4 m^3, while the corresponding required tank storage capacity was estimated at approximately 768.6 m^3.

Overall, this method provides a more comprehensive understanding of the actual water reuse potential of the system. While it confirms that larger storage volumes can improve performance, it also clearly illustrates the risk of oversizing, particularly when the additional storage cannot be effectively exploited.

5.3.4 Comparison of the two Methods

The comparison between the two sizing approaches shows that each method addresses a different design objective.

The roof-area-based method is simple, practical, and compatible with commercially available tank sizes. It is suitable for real implementation at the building scale and provides a cost-effective solution. Its main limitation is that it does not explicitly account for household demand or extreme runoff events.

The demand–supply-based method (Simplified Method) provides a more realistic evaluation of the system's water reuse potential. It considers both rainfall availability and domestic demand, making it more suitable for estimating the storage volume required to improve non-potable water saving. However, the results also show that larger storage volumes may not always be efficiently used, especially when demand is limited.

Overall, the analysis confirms that no single method can fully represent all aspects of rainwater harvesting system design. The two methods should therefore be considered complementary rather than alternative. The roof-area-based method defines a feasible starting configuration and the demand–supply method evaluates the real water reuse potential.

5.3.5 Final Evaluation

Based on the combined evaluation of hydraulic performance, storage efficiency, and investment cost, the configuration with a total storage capacity of 237 m^3 represents the most balanced solution among the scenarios considered. While larger storage capacities provide additional runoff reduction, the associated benefits become progressively smaller relative to the increase in storage volume and implementation cost.

The simplified method indicates that larger storage capacities, approximately in the range of 512–769 m^3, could theoretically improve water reuse performance. Similarly, the runoff-based analysis shows that higher storage volumes would be required to control runoff under a 10-year return period rainfall event. However, both approaches lead to significantly larger storage requirements.

The doubled-capacity scenario confirmed this point: increasing the total storage volume to 474 m^3 reduced overflow significantly, but the total cost increased to

€323,800. Therefore, while larger tanks provide better hydraulic performance, the improvement is not proportional to the increase in investment cost.

For this reason, the selected 237 m³ configuration can be considered an appropriate compromise between hydraulic efficiency, water saving potential, implementation feasibility, and economic sustainability. The analysis also confirms an important design principle: oversizing rainwater harvesting tanks does not necessarily lead to proportional improvements in system efficiency.

Therefore, the final design should not be based solely on maximizing storage capacity, but rather on a balanced evaluation of hydraulic performance, investment cost, water demand, available roof area, and project objectives. Based on the analyses carried out in this study, the roof-area-based configuration represents the most realistic and practically implementable solution. The increased-capacity scenario and the Simplified Method provide useful complementary references, illustrating both the potential benefits of additional storage and the limitations associated with alternative sizing approaches.

5.4 Hydraulic and Economic Performance of Rain Gardens

Rain garden sizing was based on allocating 5% of each available yard area to decentralized gardens, positioned to intercept runoff near its source rather than concentrating flow in a single basin.

The design strategy adopted in this study considered the allocation of approximately 5% of each available yard area for decentralized rain garden implementation. This allowed the distributed placement of multiple small rain gardens across individual parcels rather than relying on one centralized detention basin.

In this section, based on the equations presented in Section 3.5 and considering an equivalent storage depth of 20 cm and an infiltration rate of 0.025 m/h, the corresponding calculations were performed for each individual parcel under the critical design storm event considered in this study. Part of these calculations, together with the overall results, is presented in the following table. It should be noted that the complete table of calculations for all parcels is available in the appendix.

Parcel number	V runoff roof (m^3)	V runoff yard (m^3)	V runoff total for 10 years return period (m^3)	5% of yard rain garden (m^2)	V store (m^3)	V infiltration (m^3)	cost (rain garden)
180	0.2	0.1	0.4	0.0	0.0	0.0	0 €
221	0.4	0.7	1.2	0.0	0.0	0.0	0 €
227	0.5	4.8	5.3	44.9	9.0	0.2	4,490 €
343	0.5	0.4	0.9	0.0	0.0	0.0	0 €
195	0.6	0.9	1.5	0.0	0.0	0.0	0 €
377	16.2	0.0	16.2	0.0	0.0	0.0	0 €
330	16.7	2.3	19.1	21.8	4.4	0.1	2,180 €
210	16.9	5.2	22.1	48.3	9.7	0.2	4,825 €
299	17.0	4.2	21.1	38.9	7.8	0.1	3,890 €
850	29.3	0.0	29.3	0.0	0.0	0.0	0 €
TOTAL	**373.6**	**249.7**	**623.3**	**2,337.8**	**467.6**	**8.1**	**233,780 €**
	m^3	m^3	m^3	m^2	m^3	m^3	€

Table 5.5 *Rain Garden hydraulic performance and construction cost per residential parcel: allocated garden area (5% of yard), temporary storage volume, infiltration volume, and implementation cost.*

(Source: Author's elaboration based on CIRIA SuDs Manual C753 and site data)

The total effective rain garden area reached approximately 2,337.8 m^2, producing:

- Temporary storage capacity of 467.6 m^3
- Additional infiltration volume of 8.1 m^3
- The construction cost was estimated using unit costs reported in the literature for rain garden implementation, ranging between 20 and 80 €/m^2. Based on the total rain garden area required within the study area, the overall implementation cost was estimated at approximately €233,780.

This corresponds to approximately 48% of the total runoff volume associated with the 10-year return period design storm, demonstrating the significant role of rain gardens in runoff attenuation and local stormwater retention.

The distributed configuration improved runoff interception efficiency and reduced flow concentration close to the source of runoff generation. The average drainage emptying time was approximately 8 hours, confirming that the system can restore storage capacity within a relatively short period after the storm event and remain effective for subsequent rainfall events.

Although rain gardens require a higher initial investment compared to standard rainwater harvesting tanks, they provide broader environmental and hydraulic benefits, including water quality improvement, landscape enhancement, biodiversity support, and reduction of urban heat island effects.

From a long-term engineering perspective, the cost-performance ratio of rain gardens is highly favorable because the hydraulic benefits extend beyond simple runoff reduction and contribute to overall urban resilience.

5.5 Methodological Consistency and Hydro-Hydraulic Baseline

The reliability of the hydrological and hydraulic results obtained in this study was assessed through engineering validation based on standard design references, comparison with literature values, and consistency checks with regional technical guidelines. Although long-term monitored field data were not available for direct calibration, the adopted methodology follows conventional engineering practice widely applied in urban stormwater management.

The baseline runoff calculations were performed using the Rational Method, which is a well-established approach for peak discharge estimation in small to medium urban catchments. Rainfall parameters were derived from the official *Atlante delle Piogge Intense* published by ARPA Piemonte, ensuring consistency with regional hydrological standards for the municipality of Pinerolo. The selected return period of 10 years is also aligned with common design criteria for residential urban drainage systems.

The weighted runoff coefficient calculated for the study area (C = 0.59) falls within the typical range reported in literature for mixed residential catchments characterized by roof surfaces, paved roads, and partially permeable green areas.

Similarly, the estimated concentration time and rainfall intensity values are consistent with expected hydraulic behavior for urban catchments of comparable size and morphology.

The hydraulic performance of rain gardens was evaluated against established design recommendations from the CIRIA SuDs Manual (C753), particularly in terms of storage depth, infiltration assumptions, and drainage emptying time. The calculated emptying time of approximately 8 hours confirms that the system operates within acceptable limits and is capable of restoring storage capacity between storm events.

An additional level of validation was provided through the hydraulic verification of the existing drainage network. The comparison between the estimated peak discharge ($Q = 1.944\ m^3/s$) and the pipe capacity ($Q = 1.613\ m^3/s$) demonstrates that the conventional system is hydraulically undersized under critical rainfall conditions. This result is consistent with the flood-prone characteristics identified in regional hydrogeological planning frameworks such as the PGRA and PAI maps.

From an engineering perspective, the strong consistency between hydrological modeling, hydraulic verification, land-use characterization, and SuDs performance confirms that the adopted methodology is technically reliable and hydraulically coherent. The agreement with established literature values further supports the validity of the results for practical design applications, even in the absence of long-term monitored data.

The methodology is therefore considered reliable for practical design application in comparable residential catchments.

5.6 Discussion and Engineering Interpretation

The following table presents the overall results of the calculations performed and the comparison between the investigated scenarios.

RUNOFF VOLUME SUMMARY	
Description	**Value**
Total area of parcels	69,015 m^2
Total area of roof	23,397 m^2
Total area of open area	46,756 m^2
Total runoff volume of roofs	373.6 m^3
Total runoff volume of open area	249.7 m^3
Total runoff volume of building (roof + yards)	**623.3 m^3**
total runoff volume of roads	363.1 m^3
total runoff volume of zone	**986.4 m^3**
saving runoff by using tank	**237.0** m^3
percentage of saving runoff by using tank	**24%**
volume of runoff storage by rain garden	**475.6** m^3
V garden / V runoff	**48%**
Time of evacuation	**8** HOUR

Table 5.6 *Summary of runoff volumes, rainwater harvesting performance, and rain garden storage capacity for the study area under the selected design storm scenario.*

(Source: Author's calculations based on GIS analysis and hydrological assessment)

The findings of this research confirm that building-scale Sustainable Urban Drainage Systems can play a major role in improving urban stormwater management in medium-sized residential catchments such as Pinerolo.

The most important engineering result is not only the numerical reduction in runoff volume, but the redistribution of hydraulic stress throughout the drainage network. By reducing runoff generation directly at the source, the hydraulic pressure on downstream infrastructure is significantly lowered, improving the reliability of the entire system during short-duration extreme rainfall events.

The parcel-by-parcel analysis demonstrated that each individual building contributes only a limited runoff volume; however, when these contributions are aggregated across the catchment, they create substantial hydraulic pressure. This confirms that flood mitigation should not rely exclusively on large centralized infrastructure, but should strongly incorporate decentralized source-control measures at the building scale.

On the other hand, the 24% reduction in runoff volume achieved in the rainwater harvesting tank scenario and the 48% reduction obtained in the rain garden scenario demonstrated that, despite the common tendency to apply green infrastructure approaches mainly at large urban scales, these strategies can also be effectively implemented at the building scale while still achieving significant runoff reduction benefits.

Compared to conventional detention systems, the proposed SuDs strategy offers:

- better compatibility with existing residential morphology
- possibility of phased and independent implementation
- lower construction complexity
- reduced maintenance concentration
- improved water resource efficiency
- stronger environmental co-benefits

The economic analysis also showed that optimal design should not focus only on maximizing storage volume, but rather on identifying the most efficient balance between hydraulic benefit, investment cost, available space, and practical feasibility.

Although larger tanks provide better hydraulic performance, the marginal improvement is often not proportional to the increase in construction cost. Similarly, rain gardens require higher initial investment but offer significant long-term environmental and hydraulic advantages.

Furthermore, the decentralized nature of these approaches creates the possibility of shifting investment from public or governmental funding sources toward private and household-level financial participation. This aspect may provide an effective strategy for improving urban drainage systems during periods when public financial resources are more limited, by supporting stormwater management interventions through private investment.

Some limitations should be acknowledged. The study is based on design rainfall events and standard hydraulic assumptions, while long-term monitored field data were not available for complete operational validation. In addition, household water demand patterns and soil infiltration rates may vary depending on local conditions.

This thesis therefore demonstrates that decentralized building-scale SuDs are not only hydraulically efficient, but also represent a realistic and sustainable engineering solution for improving urban drainage resilience under current and future climatic conditions.

5.7 Comparison with Previous Studies

The results obtained in this study are consistent with findings reported in previous research on decentralized Sustainable Urban Drainage Systems (SuDs), particularly regarding the hydraulic effectiveness of rainwater harvesting systems and rain gardens in residential urban catchments.

Several studies have demonstrated that rainwater harvesting tanks can significantly reduce roof-generated runoff while simultaneously improving non-potable water supply efficiency. CAMPISANO AND MODICA (2016) highlighted that tank-based rainwater harvesting systems used as source-control measures can effectively reduce roof runoff peaks before stormwater reaches downstream drainage networks, especially in free-standing residential buildings. This supports the results of the present study, which indicate that the standard tank scenario achieved a runoff reduction of approximately 24% of the total runoff volume under the 10-year return period design storm. This finding suggests that the direct interception and storage of roof runoff can make a significant contribution to decentralized stormwater management at the parcel scale.

SIMILARLY, VAN DIJK ET AL. (2020) demonstrated that strategically implemented rooftop rainwater harvesting systems can prevent between 35% and 56% of rooftop runoff from entering the drainage network on a monthly basis, with even higher reductions observed during individual storm events. Although the present study focuses on a medium-sized residential catchment rather than a large metropolitan system, the obtained results are generally consistent with this range.

For rain gardens and infiltration-based systems, the CIRIA SUDS MANUAL (C753) identifies bioretention systems and rain gardens as highly effective source-control measures capable of significantly reducing runoff volume, improving infiltration, and restoring local drainage capacity. In this thesis, the proposed rain garden system provided a temporary storage capacity corresponding to approximately 48% of the total runoff volume, demonstrating strong agreement with standard

SuDs design expectations and confirming the effectiveness of decentralized infiltration-based solutions in residential landscapes.

The economic analysis of this study is also consistent with findings reported in the literature. VAN DIJK ET AL. (2020) emphasized that increasing storage capacity does not always produce proportional economic benefits and that strategic tank sizing is essential for cost-effective implementation. This directly supports the results obtained in Pinerolo, where doubling the tank storage capacity significantly increased investment costs while providing limited additional economic efficiency.

An important distinction of the present study is the parcel-by-parcel evaluation of each residential unit rather than the use of simplified catchment-wide assumptions. This approach provides a more realistic representation of decentralized implementation feasibility by considering each house individually in terms of roof area, yard area, runoff generation, overflow volume, and implementation cost.

The parcel-scale approach adopted here extends these findings to individual building level, offering a more transferable framework for residential catchment design.

Chapter 6 – Conclusion and Future Recommendations

6.1 Conclusion

This thesis investigated the effectiveness of building-scale Sustainable Urban Drainage Systems (SuDs) for reducing stormwater runoff volume and peak discharge within a residential urban catchment located in the northern part of Pinerolo. The study was developed in response to the increasing hydraulic pressure imposed on conventional drainage systems due to urbanization, expansion of impervious surfaces, and the growing intensity of short-duration extreme rainfall events.

The hydrological baseline analysis, carried out using the Rational Method, showed that stormwater runoff is strongly influenced by impermeable surfaces, with roof runoff representing a dominant contribution. The total runoff volume and peak discharge highlight the significant hydraulic stress imposed on the drainage system during critical rainfall events.

The hydraulic verification of the existing drainage network demonstrated that the system is insufficient under design conditions. Although the main pipe (diameter 0.63 m) provides a theoretical capacity of approximately 1.613 m^3/s, the estimated peak discharge reaches 1.944 m^3/s, resulting in a deficit of 0.331 m^3/s. This corresponds to a required reduction of approximately 17% in peak flow to restore safe operating conditions.

This mismatch confirms that, during intense rainfall events, the drainage system operates under surcharge conditions, leading to temporary ponding, inlet overflow, and localized flooding, particularly in the lower central and southern areas of the catchment. Therefore, the implementation of SuDs is not optional, but a necessary engineering solution to compensate for this hydraulic deficit.

To address these limitations, two main building-scale SuDS interventions were evaluated: rainwater harvesting tanks and rain gardens. Rainwater harvesting systems were designed using a roof-area-based practical sizing approach and a demand–supply-based simplified method, while rain gardens were designed through a storage–infiltration balance approach. The two interventions were assessed independently at the parcel scale under the 10-year return period design storm, allowing their cumulative contribution to runoff reduction to be quantified at the catchment level.

The results showed that rainwater harvesting tanks significantly reduce roof runoff while improving non-potable water reuse. However, increasing tank volume does not always result in proportional economic efficiency, highlighting the importance of optimal sizing based on performance–cost balance.

Rain gardens provided a greater hydraulic contribution through temporary detention and localized infiltration. Their distributed configuration allows effective runoff interception near the source, contributing significantly to peak flow attenuation.

One of the key findings of this research is that flood mitigation in residential areas should not rely solely on centralized infrastructure. Instead, decentralized parcel-scale interventions provide a more effective and flexible solution by addressing runoff at its source.

From an engineering perspective, the proposed SuDs strategy offers multiple advantages, including adaptability, compatibility with existing urban morphology, improved water resource efficiency, and enhanced long-term resilience under climate change conditions.

Therefore, this thesis demonstrates that the integration of rainwater harvesting tanks and rain gardens represents a practical, technically effective, and sustainable approach to urban flood mitigation. The proposed methodology establishes a clear link between catchment-scale hydrological processes and building-scale interventions, providing a replicable framework for resilient urban stormwater management.

6.2 Limitations of the Study

Although the results of this research provide strong technical evidence supporting the effectiveness of decentralized Sustainable Urban Drainage Systems (SuDs), several limitations of this study should be recognized.

First, the hydrological analysis for runoff estimation was based on design rainfall events and Intensity–Duration–Frequency (IDF) curves derived from ARPA

Piemonte, which are widely used for peak discharge calculations in urban drainage design. In parallel, approximately 25 years of historical rainfall data were used for the sizing and performance assessment of rainwater harvesting tanks and rain gardens, particularly to evaluate storage efficiency and long-term system behavior. While this combined approach is consistent with standard engineering practice, the absence of direct field monitoring of runoff and system performance limits the possibility of full calibration and validation under real operating conditions.

Second, several key design parameters—such as non-potable household water demand, soil infiltration rates, and rain garden storage performance—were defined based on standard assumptions and literature values. In practice, these parameters may vary significantly due to user behavior, soil heterogeneity, maintenance conditions, and seasonal variability, which may influence the actual performance of the implemented systems.

Third, the economic analysis focused primarily on initial construction costs and did not include a comprehensive life-cycle cost assessment. Operational costs, maintenance requirements, system durability, and long-term economic benefits—such as reduced flood damage and water savings—were not explicitly quantified.

Finally, the study was conducted for a specific residential catchment in Pinerolo. Although the proposed methodology is transferable, its application to other urban contexts may require local calibration to account for differences in climatic conditions, topography, land use, and infrastructure characteristics.

Despite these limitations, the adopted methodology integrates design-based hydrological modeling with long-term rainfall analysis and hydraulic verification of the existing drainage system. The strong consistency between these components ensures that the results remain technically robust and sufficiently reliable to support practical engineering decision-making in decentralized urban stormwater management.

6.3 Future Recommendations

Future research could enhance the findings of this study by integrating long-term monitored rainfall and runoff data to enable direct calibration and validation of SuDs performance under real operating conditions. The application of dynamic hydrological–hydraulic simulation tools, such as SWMM or similar urban drainage models, would allow a more detailed analysis of time-dependent system behavior, including responses to consecutive storm events and varying antecedent moisture conditions.

A more comprehensive economic evaluation could be achieved through a detailed life-cycle cost analysis, incorporating operation and maintenance costs, system durability, replacement cycles, flood damage reduction, and long-term potable water savings. Such an approach would provide a more complete assessment of the economic viability of decentralized SuDs.

Future studies could also explore the integration of additional SuDs components—such as permeable pavements, green roofs, bioswales, and infiltration trenches—to evaluate the combined performance of a more extensive blue-green infrastructure network and its cumulative impact on runoff reduction and urban resilience.

At the urban planning scale, policy-oriented strategies should be developed to support the widespread implementation of decentralized SuDs in residential areas. Municipal authorities—such as those in Pinerolo—could introduce financial incentives, technical guidelines, and streamlined permitting procedures to facilitate adoption at the building scale.

Finally, the parcel-scale methodology developed in this thesis could be extended to larger urban districts or entire municipalities, providing a scalable framework for strategic flood risk management and climate adaptation planning in medium-sized cities.

Overall, the increasing challenges posed by urban flooding and climate uncertainty highlight the need for a transition from conventional drainage systems toward more resilient, adaptive, and sustainable stormwater management approaches. In this context, building-scale SuDs represent a practical and effective pathway for achieving this transition.

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Appendix A → Pipe hydraulic calculation

KUTTER ROUGHNESS COEFFICIENT (m) SELECTION GUIDE

No.	Surface Description	γ	m
1	Smooth cement surfaces, perfectly finished, or planed wooden boards, or asbestos-cement pipes	0.06	0.12
2	Smooth cement surfaces, or planed wooden boards, or steel pipes without welding	0.10	0.15
3	Ordinary plaster, ceramic bricks, thin metal sheets with small rivets, new cast iron	0.16	0.20
4	Smooth cement pipes with diameter > 0.40 m, or metal pipes with many rivets	0.18	—
5	Finished concrete, cement pipes with frequent joints, cast iron in normal service	0.23	0.25
6	Cement surfaces not well finished, or rough wooden boards, or well-built masonry, or very regular earth, or old cast iron pipes, or metal pipes with many rivets	0.36	—
7	Poorly finished cement, or ordinary stone masonry	0.46	0.55
8	Irregular earth, rough or old concrete, slag cement, old cast iron	0.85	0.75
9	Earth channels with slight sand deposits, or masonry in poor condition, or metal/steel-lined surfaces with ordinary riveting	1.00	1.25
10	Irregular earth with protruding vegetation, natural streams with regular beds	1.30	1.75
11	Earth channels in poor condition, vegetation on bed and banks, irregular deposits of stones and gravel	1.75	2.50
12	Abandoned earth channels, almost obstructed by vegetation, or natural streams with gravel beds	2.30	3.00

Notes:

γ = Kutter coefficient

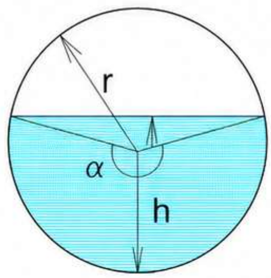
m = Kutter roughness coefficient used in hydraulic calculations

Recommended Values for Common Pipe Materials		
PVC Pipe (New)	⇨	m = 0.12 – 0.15
Concrete Pipe (New)	⇨	m = 0.18 – 0.25

DISCHARGE CAPACITY – CIRCULAR HYDRAULIC SECTION FOR DIFFERENT WATER DEPTHS

SECTION CHARACTERISTICS		
d	0.60	Diameter [m]
p	4.0%	Slope
m	0.2	Kutter roughness coefficient

h [m]	Q [m³/sec]
0.03	0.006
0.06	0.028
0.09	0.068
0.12	0.124
0.15	0.195
0.18	0.280
0.21	0.377
0.24	0.484
0.27	0.599
0.30	0.719
0.33	0.842
0.36	0.966
0.39	1.088
0.42	1.204
0.45	1.311
0.48	1.405
0.51	1.481
0.54	1.532
0.57	1.545
0.60	1.438



h = water depth

Q = discharge corresponding to that water depth

DISCHARGE vs. WATER DEPTH CURVE

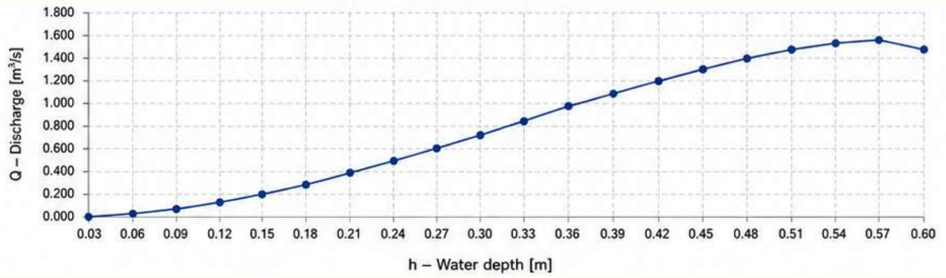


Table A-1 *- Roughness coefficients (γ and m) for different channel and pipe surface conditions based on Kutter formulation*

Appendix B → Parcel-level estimation of roof area, yard area, and total stormwater runoff volume in the study area.

PARCEL RUNOFF VOLUME SUMMARY

PART 1 OF 2

Parcel No.	Parcel area (total) (m²)	Roof area (m²)	Yard area (m²)	V runoff roof (m³)	V runoff yard (m³)	V runoff total 10-year return period (m³)
180	39	15	24	0.2	0.1	0.4
221	168	28	140	0.4	0.7	1.2
227	931	33	898	0.5	4.8	5.3
343	100	34	66	0.5	0.4	0.9
195	209	37	172	0.6	0.9	1.5
100	190	44	146	0.7	0.8	1.5
459	275	45	230	0.7	1.2	1.9
186	230	52	178	0.8	1.0	1.8
458	250	53	197	0.8	1.1	1.9
231	172	66	106	1.1	0.6	1.6
772	114	77	37	1.2	0.2	1.4
228	258	85	173	1.4	0.9	2.3
57	463	93	370	1.5	2.0	3.5
232	297	98	199	1.6	1.1	2.6
233	570	102	468	1.6	2.5	4.1
267	105	104	1	1.7	0.0	1.7
220	350	108	242	1.7	1.3	3.0
240	410	109	301	1.7	1.6	3.4
773	159	112	47	1.8	0.3	2.0
241	364	112	252	1.8	1.3	3.1
226	414	112	302	1.8	1.6	3.4
223	680	114	566	1.8	3.0	4.8
560	142	116	26	1.9	0.1	2.0
242	428	117	311	1.9	1.7	3.5
691	201	120	81	1.9	0.4	2.4
295	373	120	253	1.9	1.4	3.3
175	748	121	627	1.9	3.3	5.3
593	398	125	273	2.0	1.5	3.5
182	369	127	242	2.0	1.3	3.3
185	463	129	334	2.1	1.8	3.9
284	497	130	367	2.1	2.0	4.0
200	571	135	436	2.2	2.3	4.5
265	377	140	237	2.2	1.3	3.5
230	635	143	492	2.3	2.6	4.9
293	1457	143	1314	2.3	7.0	9.3
224	730	145	585	2.3	3.1	5.4
188	370	146	224	2.3	1.2	3.5
239	317	150	167	2.4	0.9	3.3
258	400	150	250	2.4	1.3	3.7
366	759	150	609	2.4	3.3	5.7
164	423	151	272	2.4	1.5	3.9
117	449	153	296	2.5	1.6	4.0
313	1871	153	1718	2.5	9.2	11.6
1099	578	155	423	2.5	2.3	4.7
151	443	157	286	2.5	1.5	4.0

PARCEL RUNOFF VOLUME SUMMARY						PART 2 OF 2
Parcel No.	Parcel area (total) (m²)	Roof area (m²)	Yard area (m²)	V runoff roof (m³)	V runoff yard (m³)	V runoff total 10-year return period (m³)
392	968	157	811	2.5	4.3	6.8
771	248	163	85	2.6	0.5	3.1
229	262	163	99	2.6	0.5	3.1
64	897	163	734	2.6	3.9	6.5
238	993	165	828	2.6	4.4	7.1
543	384	169	215	2.7	1.1	3.9
113	480	170	310	2.7	1.7	4.4
212	1142	179	963	2.9	5.1	8.0
350	393	184	209	2.9	1.1	4.1
62	591	185	406	3.0	2.2	5.1
143	1946	188	1758	3.0	9.4	12.4
194	736	190	546	3.0	2.9	6.0
589	430	194	236	3.1	1.3	4.4
590	414	195	219	3.1	1.2	4.3
247	890	195	695	3.1	3.7	6.8
243	669	196	473	3.1	2.5	5.7
222	450	200	250	3.2	1.3	4.5
229	643	205	438	3.3	2.3	5.6
493	1187	214	973	3.4	5.2	8.6
644	373	217	156	3.5	0.8	4.3
237	613	229	384	3.7	2.1	5.7
187	846	255	591	4.1	3.2	7.2
429	1709	271	1438	4.3	7.7	12.0
460	1202	278	924	4.5	4.9	9.4
326	619	284	335	4.5	1.8	6.3
691	1132	347	785	5.6	4.2	9.8
235	852	350	502	5.6	2.7	8.3
1082	1095	355	740	5.7	4.0	9.6
965	1333	365	968	5.8	5.2	11.0
541	561	377	184	6.0	1.0	7.0
806	1374	391	983	6.3	5.2	11.5
692	922	454	468	7.3	2.5	9.8
848	1157	461	696	7.4	3.7	11.1
67	1019	481	538	7.7	2.9	10.6
60	763	570	193	9.1	1.0	10.2
146	1777	593	1184	9.5	6.3	15.8
329	2438	605	1833	9.7	9.8	19.5
69	3230	715	2515	11.5	13.4	24.9
1084	3079	735	2344	11.8	12.5	24.3
197	2405	775	1630	12.4	8.7	21.1
377	1012	1012	0	16.2	0.0	16.2
330	1481	1045	436	16.7	2.3	19.1
210	2023	1058	965	16.9	5.2	22.1
299	1838	1060	778	17.0	4.2	21.1
850	1830	1830	0	29.3	0.0	29.3
TOTAL	**69,015**	**23,397**	**46,756**	**373.6**	**249.7**	**623.3**
	m2	m2	m2	m3	m3	m3

Note: Runoff volumes are calculated for the 10-year return period and grouped by parcel surfaces.

Appendix C → Roof-area-based rainwater harvesting tank sizing - Scenario 1

RAINWATER TANK RUNOFF REDUCTION SUMMARY

PART 1 OF 2

Parcel No.	V runoff roof (m³)	V runoff yard (m³)	V runoff total 10-year return period (m³)	Tank size based on 10-year runoff (m³)	Overflow (m³)	Cost (tank)
180	0.2	0.1	0.4	2.0	0	1,300 €
221	0.4	0.7	1.2	2.0	0	1,300 €
227	0.5	4.8	5.3	2.0	3	1,300 €
343	0.5	0.4	0.9	2.0	0	1,300 €
195	0.6	0.9	1.5	2.0	0	1,300 €
100	0.7	0.8	1.5	2.0	0	1,300 €
459	0.7	1.2	1.9	2.0	0	1,300 €
186	0.8	1.0	1.8	2.0	0	1,300 €
458	0.8	1.1	1.9	2.0	0	1,300 €
231	1.1	0.6	1.6	2.0	0	1,300 €
772	1.2	0.2	1.4	2.0	0	1,300 €
228	1.4	0.9	2.3	2.0	0	1,300 €
57	1.5	2.0	3.5	2.0	1	1,300 €
232	1.6	1.1	2.6	2.0	1	1,300 €
233	1.6	2.5	4.1	2.0	2	1,300 €
267	1.7	0.0	1.7	2.0	0	1,300 €
220	1.7	1.3	3.0	2.0	1	1,300 €
240	1.7	1.6	3.4	2.0	1	1,300 €
773	1.8	0.3	2.0	2.0	0	1,300 €
241	1.8	1.3	3.1	2.0	1	1,300 €
226	1.8	1.6	3.4	2.0	1	1,300 €
223	1.8	3.0	4.8	2.0	3	1,300 €
560	1.9	0.1	2.0	2.0	0	1,300 €
242	1.9	1.7	3.5	2.0	2	1,300 €
691	1.9	0.4	2.4	2.0	0	1,300 €
295	1.9	1.4	3.3	2.0	1	1,300 €
175	1.9	3.3	5.3	2.0	3	1,300 €
593	2.0	1.5	3.5	2.0	1	1,300 €
182	2.0	1.3	3.3	2.0	1	1,300 €
185	2.1	1.8	3.9	2.0	2	1,300 €
284	2.1	2.0	4.0	2.0	2	1,300 €
200	2.2	2.3	4.5	2.0	2	1,300 €
265	2.2	1.3	3.5	2.0	2	1,300 €
230	2.3	2.6	4.9	2.0	3	1,300 €
293	2.3	7.0	9.3	2.0	7	1,300 €
224	2.3	3.1	5.4	2.0	3	1,300 €
188	2.3	1.2	3.5	2.0	2	1,300 €
239	2.4	0.9	3.3	2.0	1	1,300 €
258	2.4	1.3	3.7	2.0	2	1,300 €
366	2.4	3.3	5.7	2.0	4	1,300 €
164	2.4	1.5	3.9	2.0	2	1,300 €
117	2.5	1.6	4.0	2.0	2	1,300 €
313	2.5	9.2	11.6	2.0	10	1,300 €
1099	2.5	2.3	4.7	2.0	3	1,300 €
151	2.5	1.5	4.0	2.0	2	1,300 €

RAINWATER TANK RUNOFF REDUCTION SUMMARY						PART 2 OF 2
Parcel No.	V runoff roof (m³)	V runoff yard (m³)	V runoff total 10-year return period (m³)	Tank size based on 10-year runoff (m³)	Overflow (m³)	Cost (tank)
392	2.5	4.3	6.8	2.0	5	1,300 €
771	2.6	0.5	3.1	2.0	1	1,300 €
229	2.6	0.5	3.1	2.0	1	1,300 €
64	2.6	3.9	6.5	2.0	5	1,300 €
238	2.6	4.4	7.1	2.0	5	1,300 €
543	2.7	1.1	3.9	2.0	2	1,300 €
113	2.7	1.7	4.4	2.0	2	1,300 €
212	2.9	5.1	8.0	2.0	6	1,300 €
350	2.9	1.1	4.1	2.0	2	1,300 €
62	3.0	2.2	5.1	2.0	3	1,300 €
143	3.0	9.4	12.4	2.0	10	1,300 €
194	3.0	2.9	6.0	2.0	4	1,300 €
589	3.1	1.3	4.4	2.0	2	1,300 €
590	3.1	1.2	4.3	2.0	2	1,300 €
247	3.1	3.7	6.8	2.0	5	1,300 €
243	3.1	2.5	5.7	2.0	4	1,300 €
222	3.2	1.3	4.5	3.0	2	1,600 €
229	3.3	2.3	5.6	3.0	3	1,600 €
493	3.4	5.2	8.6	3.0	6	1,600 €
644	3.5	0.8	4.3	3.0	1	1,600 €
237	3.7	2.1	5.7	3.0	3	1,600 €
187	4.1	3.2	7.2	3.0	4	1,600 €
429	4.3	7.7	12.0	3.0	9	1,600 €
460	4.5	4.9	9.4	3.0	6	1,600 €
326	4.5	1.8	6.3	3.0	3	1,600 €
691	5.6	4.2	9.8	3.0	7	1,600 €
235	5.6	2.7	8.3	3.0	5	1,600 €
1082	5.7	4.0	9.6	3.0	7	1,600 €
965	5.8	5.2	11.0	3.0	8	1,600 €
541	6.0	1.0	7.0	3.0	4	1,600 €
806	6.3	5.2	11.5	3.0	9	1,600 €
692	7.3	2.5	9.8	5.0	5	2,000 €
848	7.4	3.7	11.1	5.0	6	2,000 €
67	7.7	2.9	10.6	5.0	6	2,000 €
60	9.1	1.0	10.2	5.0	5	2,000 €
146	9.5	6.3	15.8	5.0	11	2,000 €
329	9.7	9.8	19.5	5.0	14	2,000 €
69	11.5	13.4	24.9	5.0	20	2,000 €
1084	11.8	12.5	24.3	5.0	19	2,000 €
197	12.4	8.7	21.1	5.0	16	2,000 €
377	16.2	0.0	16.2	5.0	11	2,000 €
330	16.7	2.3	19.1	5.0	14	2,000 €
210	16.9	5.2	22.1	5.0	17	2,000 €
299	17.0	4.2	21.1	5.0	16	2,000 €
850	29.3	0.0	29.3	5.0	24	2,000 €
TOTAL	**373.6**	**249.7**	**623.3**	**237.0**	**394**	**131,300 €**
	m3	**m3**	**m3**	**m3**	**m3**	

Note: Tank size and overflow are reported for the selected runoff reduction scenario.

Appendix D → Roof-area-based rainwater harvesting tank sizing - Scenario 2

RUNOFF VOLUME AND TANK SIZE SUMMARY (10 YEARS RETURN PERIOD)						PART 1 OF 2
Parcel Number	V runoff roof (m³)	V runoff yard (m³)	V runoff total for 10 years return period (m³)	Tank size based on 10 years runoff (m³)	Overflow (m³)	Cost (tank)
180	0.2	0.1	0.4	4	0	3,200 €
221	0.4	0.7	1.2	4	0	3,200 €
227	0.5	4.8	5.3	4	1	3,200 €
343	0.5	0.4	0.9	4	0	3,200 €
195	0.6	0.9	1.5	4	0	3,200 €
100	0.7	0.8	1.5	4	0	3,200 €
459	0.7	1.2	1.9	4	0	3,200 €
186	0.8	1.0	1.8	4	0	3,200 €
458	0.8	1.1	1.9	4	0	3,200 €
231	1.1	0.6	1.6	4	0	3,200 €
772	1.2	0.2	1.4	4	0	3,200 €
228	1.4	0.9	2.3	4	0	3,200 €
57	1.5	2.0	3.5	4	0	3,200 €
232	1.6	1.1	2.6	4	0	3,200 €
233	1.6	2.5	4.1	4	0	3,200 €
267	1.7	0.0	1.7	4	0	3,200 €
220	1.7	1.3	3.0	4	0	3,200 €
240	1.7	1.6	3.4	4	0	3,200 €
773	1.8	0.3	2.0	4	0	3,200 €
241	1.8	1.3	3.1	4	0	3,200 €
226	1.8	1.6	3.4	4	0	3,200 €
223	1.8	3.0	4.8	4	1	3,200 €
560	1.9	0.1	2.0	4	0	3,200 €
242	1.9	1.7	3.5	4	0	3,200 €
691	1.9	0.4	2.4	4	0	3,200 €
295	1.9	1.4	3.3	4	0	3,200 €
175	1.9	3.3	5.3	4	1	3,200 €
593	2.0	1.5	3.5	4	0	3,200 €
182	2.0	1.3	3.3	4	0	3,200 €
185	2.1	1.8	3.9	4	0	3,200 €
284	2.1	2.0	4.0	4	0	3,200 €
200	2.2	2.3	4.5	4	0	3,200 €
265	2.2	1.3	3.5	4	0	3,200 €
230	2.3	2.6	4.9	4	1	3,200 €
293	2.3	7.0	9.3	4	5	3,200 €
224	2.3	3.1	5.4	4	1	3,200 €
188	2.3	1.2	3.5	4	0	3,200 €
239	2.4	0.9	3.3	4	0	3,200 €
258	2.4	1.3	3.7	4	0	3,200 €
366	2.4	3.3	5.7	4	2	3,200 €
164	2.4	1.5	3.9	4	0	3,200 €
117	2.5	1.6	4.0	4	0	3,200 €
313	2.5	9.2	11.6	4	8	3,200 €
1099	2.5	2.3	4.7	4	1	3,200 €
151	2.5	1.5	4.0	4	0	3,200 €

RUNOFF VOLUME AND TANK SIZE SUMMARY (10 YEARS RETURN PERIOD)						PART 2 OF 2
Parcel Number	V runoff roof (m³)	V runoff yard (m³)	V runoff total for 10 years return period (m³)	Tank size based on 10 years runoff (m³)	Overflow (m³)	Cost (tank)
392	2.5	4.3	6.8	4	3	3,200 €
771	2.6	0.5	3.1	4	0	3,200 €
229	2.6	0.5	3.1	4	0	3,200 €
64	2.6	3.9	6.5	4	3	3,200 €
238	2.6	4.4	7.1	4	3	3,200 €
543	2.7	1.1	3.9	4	0	3,200 €
113	2.7	1.7	4.4	4	0	3,200 €
212	2.9	5.1	8.0	4	4	3,200 €
350	2.9	1.1	4.1	4	0	3,200 €
62	3.0	2.2	5.1	4	1	3,200 €
143	3.0	9.4	12.4	4	8	3,200 €
194	3.0	2.9	6.0	4	2	3,200 €
589	3.1	1.3	4.4	4	0	3,200 €
590	3.1	1.2	4.3	4	0	3,200 €
247	3.1	3.7	6.8	4	3	3,200 €
243	3.1	2.5	5.7	4	2	3,200 €
222	3.2	1.3	4.5	6	0	4,000 €
229	3.3	2.3	5.6	6	0	4,000 €
493	3.4	5.2	8.6	6	3	4,000 €
644	3.5	0.8	4.3	6	0	4,000 €
237	3.7	2.1	5.7	6	0	4,000 €
187	4.1	3.2	7.2	6	1	4,000 €
429	4.3	7.7	12.0	6	6	4,000 €
460	4.5	4.9	9.4	6	3	4,000 €
326	4.5	1.8	6.3	6	0	4,000 €
691	5.6	4.2	9.8	6	4	4,000 €
235	5.6	2.7	8.3	6	2	4,000 €
1082	5.7	4.0	9.6	6	4	4,000 €
965	5.8	5.2	11.0	6	5	4,000 €
541	6.0	1.0	7.0	6	1	4,000 €
806	6.3	5.2	11.5	6	6	4,000 €
692	7.3	2.5	9.8	10	0	4,900 €
848	7.4	3.7	11.1	10	1	4,900 €
67	7.7	2.9	10.6	10	1	4,900 €
60	9.1	1.0	10.2	10	0	4,900 €
146	9.5	6.3	15.8	10	6	4,900 €
329	9.7	9.8	19.5	10	9	4,900 €
69	11.5	13.4	24.9	10	15	4,900 €
1084	11.8	12.5	24.3	10	14	4,900 €
197	12.4	8.7	21.1	10	11	4,900 €
377	16.2	0.0	16.2	10	6	4,900 €
330	16.7	2.3	19.1	10	9	4,900 €
210	16.9	5.2	22.1	10	12	4,900 €
299	17.0	4.2	21.1	10	11	4,900 €
850	29.3	0.0	29.3	10	19	4,900 €
TOTAL	**373.6**	**249.7**	**623.3**	**474**	**149**	**323,800 €**
	m3	m3	m3	m3	m3	

Note: Values are based on the 10-year return period runoff calculation. Overflow = runoff volume exceeding the selected tank size.

Appendix E → Simplified sizing method

RAINWATER HARVESTING TANK SIZING BY PARCEL — PART 1 OF 2

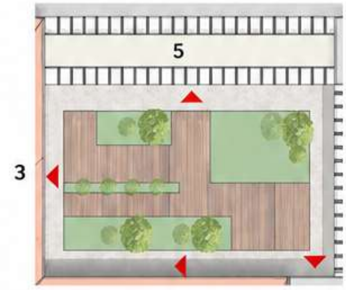
Parcel No.	Parcel area (m^2)	Roof area (m^2)	Yard area (m^2)	No. of persons	Domestic usage annually (m^3)	Annual rainfall roof (m^3)	Useful tank (m^3)	Real tank (m^3)
180	39	15	24	1	5.5	9.7	0.3	0.5
221	168	28	140	1	10.2	18.2	0.6	0.9
227	931	33	898	2	12.0	21.4	0.7	1.1
343	100	34	66	2	12.4	22.1	0.7	1.1
195	209	37	172	2	13.5	24.0	0.8	1.2
100	190	44	146	2	16.1	28.6	1.0	1.4
459	275	45	230	2	16.4	29.2	1.0	1.5
186	230	52	178	3	19.0	33.8	1.1	1.7
458	250	53	197	3	19.3	34.4	1.2	1.7
231	172	66	106	3	24.1	42.8	1.4	2.2
772	114	77	37	4	28.1	50.0	1.7	2.5
228	258	85	173	4	31.0	55.2	1.9	2.8
57	463	93	370	5	33.9	60.4	2.0	3.1
232	297	98	199	5	35.8	63.6	2.1	3.2
233	570	102	468	5	37.2	66.2	2.2	3.4
267	105	104	1	5	38.0	67.5	2.3	3.4
220	350	108	242	5	39.4	70.1	2.4	3.5
240	410	109	301	5	39.8	70.7	2.4	3.6
773	159	112	47	6	40.9	72.7	2.5	3.7
241	364	112	252	6	40.9	72.7	2.5	3.7
226	414	112	302	6	40.9	72.7	2.5	3.7
223	680	114	566	6	41.6	74.0	2.5	3.7
560	142	116	26	6	42.3	75.3	2.5	3.8
242	428	117	311	6	42.7	75.9	2.6	3.8
691	201	120	81	6	43.8	77.9	2.6	3.9
295	373	120	253	6	43.8	77.9	2.6	3.9
175	748	121	627	6	44.2	78.5	2.6	4.0
593	398	125	273	6	45.6	81.1	2.7	4.1
182	369	127	242	6	46.4	82.4	2.8	4.2
185	463	129	334	6	47.1	83.7	2.8	4.2
284	497	130	367	7	47.5	84.4	2.8	4.3
200	571	135	436	7	49.3	87.6	3.0	4.4
265	377	140	237	7	51.1	90.9	3.1	4.6
230	635	143	492	7	52.2	92.8	3.1	4.7
293	1457	143	1314	7	52.2	92.8	3.1	4.7
224	730	145	585	7	52.9	94.1	3.2	4.8
188	370	146	224	7	53.3	94.8	3.2	4.8
239	317	150	167	8	54.8	97.4	3.3	4.9
258	400	150	250	8	54.8	97.4	3.3	4.9
366	759	150	609	8	54.8	97.4	3.3	4.9
164	423	151	272	8	55.1	98.0	3.3	5.0
117	449	153	296	8	55.8	99.3	3.4	5.0
313	1871	153	1718	8	55.8	99.3	3.4	5.0
1099	578	155	423	8	56.6	100.6	3.4	5.1
151	443	157	286	8	57.3	101.9	3.4	5.2

RAINWATER HARVESTING TANK SIZING BY PARCEL								PART 2 OF 2
Parcel No.	Parcel area (m^2)	Roof area (m^2)	Yard area (m^2)	No. of persons	Domestic usage annually (m^3)	Annual rainfall roof (m^3)	Useful tank (m^3)	Real tank (m^3)
392	968	157	811	8	57.3	101.9	3.4	5.2
771	248	163	85	8	59.5	105.8	3.6	5.4
229	262	163	99	8	59.5	105.8	3.6	5.4
64	897	163	734	8	59.5	105.8	3.6	5.4
238	993	165	828	8	60.2	107.1	3.6	5.4
543	384	169	215	8	61.7	109.7	3.7	5.6
113	480	170	310	9	62.1	110.3	3.7	5.6
212	1142	179	963	9	65.3	116.2	3.9	5.9
350	393	184	209	9	67.2	119.4	4.0	6.0
62	591	185	406	9	67.5	120.1	4.1	6.1
143	1946	188	1758	9	68.6	122.0	4.1	6.2
194	736	190	546	10	69.4	123.3	4.2	6.2
589	430	194	236	10	70.8	125.9	4.2	6.4
590	414	195	219	10	71.2	126.6	4.3	6.4
247	890	195	695	10	71.2	126.6	4.3	6.4
243	669	196	473	10	71.5	127.2	4.3	6.4
222	450	200	250	10	73.0	129.8	4.4	6.6
229	643	205	438	10	74.8	133.1	4.5	6.7
493	1187	214	973	11	78.1	138.9	4.7	7.0
644	373	217	156	11	79.2	140.8	4.8	7.1
237	613	229	384	11	83.6	148.6	5.0	7.5
187	846	255	591	13	93.1	165.5	5.6	8.4
429	1709	271	1438	14	98.9	175.9	5.9	8.9
460	1202	278	924	14	101.5	180.4	6.1	9.1
326	619	284	335	14	103.7	184.3	6.2	9.3
691	1132	347	785	17	126.7	225.2	7.6	11.4
235	852	350	502	18	127.8	227.2	7.7	11.5
1082	1095	355	740	18	129.6	230.4	7.8	11.7
965	1333	365	968	18	133.2	236.9	8.0	12.0
541	561	377	184	19	137.6	244.7	8.3	12.4
806	1374	391	983	20	142.7	253.8	8.6	12.8
692	922	454	468	23	165.7	294.7	9.9	14.9
848	1157	461	696	23	168.3	299.2	10.1	15.1
67	1019	481	538	24	175.6	312.2	10.5	15.8
60	763	570	193	29	208.1	370.0	12.5	18.7
146	1777	593	1184	30	216.4	384.9	13.0	19.5
329	2438	605	1833	30	220.8	392.7	13.2	19.9
69	3230	715	2515	36	261.0	464.1	15.7	23.5
1084	3079	735	2344	37	268.3	477.0	16.1	24.1
197	2405	775	1630	39	282.9	503.0	17.0	25.5
377	1012	1012	0	51	369.4	656.8	22.2	33.2
330	1481	1045	436	52	381.4	678.3	22.9	34.3
210	2023	1058	965	53	386.2	686.7	23.2	34.8
299	1838	1060	778	53	386.9	688.0	23.2	34.8
850	1830	1830	0	92	668.0	1187.8	40.1	60.1
TOTAL	**69,015**	**23,397**	**46,756**	**1169.9**	**8539.9**	**15185.7**	**512.4**	**768.6**
	m2	m2	m2		m3	m3	m3	m3

Note: Domestic demand and annual roof rainfall are used to estimate useful and real rainwater harvesting tank volumes.

Appendix F → Rain Garden table

RUNOFF VOLUME CALCULATION (10 YEARS RETURN PERIOD)



V runoff = C * Ac * h / 1000
in case of turin from table
return period = 10 years
rainfall duration = 8.3 minutes
h = 17.8

Vstor = (Area of all rain gardens) · (Equivalent water depth)
Vinf = (Area of all rain gardens) · (infiltration rate) · (rainfall duration)
Vgarden = Vstor +Vinf

runoff coefficient from table

Surface Type	Ac (m^2)	Runoff Coefficient (C)	h (min)	Runoff (m^3)
Roofs	23,397	0.90	17.8	375
Pavement	24,000	0.85	17.8	363
Sandy Soil	46,756	0.30	17.8	250
SUM	**94,153**	—	**17.8**	**988**

Area of All Rain Gardens (m^2)	Equivalent Water Depth (m)	Vstor (m^3)
2,338	**0.20**	**468**

Area of All Rain Gardens (m^2)	Infiltration Rate (m/min)	Rainfall Duration (min)	Vinf (m^3)
2,338	**0.025**	**8.3**	**8**

Vgarden (m^3)	Vstor (m^3)	Vinf (m^3)
476	**468**	**8**

Vgarden / Vrunoff
0.481608661

Notes:

- Rainfall duration = 8.3 minutes corresponding to return period of 10 years.
- Runoff coefficients (C) are taken from the table.
- h = rainfall intensity duration (min).
- All areas are in square meters (m^2) and volumes in cubic meters (m^3).

Appendix G → Rain gardens

RAIN GARDEN STORAGE AND INFILTRATION SUMMARY							PART 1 OF 2
Parcel No.	V runoff roof (m³)	V runoff yard (m³)	V runoff total 10-year return period (m³)	5% of yard rain garden (m²)	V store (m³)	V infiltration (m³)	Cost (rain garden)
180	0.2	0.1	0.4	0.0	0.0	0.0	0 €
221	0.4	0.7	1.2	0.0	0.0	0.0	0 €
227	0.5	4.8	5.3	44.9	9.0	0.2	4,490 €
343	0.5	0.4	0.9	0.0	0.0	0.0	0 €
195	0.6	0.9	1.5	0.0	0.0	0.0	0 €
100	0.7	0.8	1.5	0.0	0.0	0.0	0 €
459	0.7	1.2	1.9	11.5	2.3	0.0	1,150 €
186	0.8	1.0	1.8	0.0	0.0	0.0	0 €
458	0.8	1.1	1.9	0.0	0.0	0.0	0 €
231	1.1	0.6	1.6	0.0	0.0	0.0	0 €
772	1.2	0.2	1.4	0.0	0.0	0.0	0 €
228	1.4	0.9	2.3	0.0	0.0	0.0	0 €
57	1.5	2.0	3.5	18.5	3.7	0.1	1,850 €
232	1.6	1.1	2.6	0.0	0.0	0.0	0 €
233	1.6	2.5	4.1	23.4	4.7	0.1	2,340 €
267	1.7	0.0	1.7	0.0	0.0	0.0	0 €
220	1.7	1.3	3.0	12.1	2.4	0.0	1,210 €
240	1.7	1.6	3.4	15.1	3.0	0.1	1,505 €
773	1.8	0.3	2.0	0.0	0.0	0.0	0 €
241	1.8	1.3	3.1	12.6	2.5	0.0	1,260 €
226	1.8	1.6	3.4	15.1	3.0	0.1	1,510 €
223	1.8	3.0	4.8	28.3	5.7	0.1	2,830 €
560	1.9	0.1	2.0	0.0	0.0	0.0	0 €
242	1.9	1.7	3.5	15.6	3.1	0.1	1,555 €
691	1.9	0.4	2.4	0.0	0.0	0.0	0 €
295	1.9	1.4	3.3	12.7	2.5	0.0	1,265 €
175	1.9	3.3	5.3	31.4	6.3	0.1	3,135 €
593	2.0	1.5	3.5	13.7	2.7	0.0	1,365 €
182	2.0	1.3	3.3	12.1	2.4	0.0	1,210 €
185	2.1	1.8	3.9	16.7	3.3	0.1	1,670 €
284	2.1	2.0	4.0	18.4	3.7	0.1	1,835 €
200	2.2	2.3	4.5	21.8	4.4	0.1	2,180 €
265	2.2	1.3	3.5	11.9	2.4	0.0	1,185 €
230	2.3	2.6	4.9	24.6	4.9	0.1	2,460 €
293	2.3	7.0	9.3	65.7	13.1	0.2	6,570 €
224	2.3	3.1	5.4	29.3	5.9	0.1	2,925 €
188	2.3	1.2	3.5	11.2	2.2	0.0	1,120 €
239	2.4	0.9	3.3	0.0	0.0	0.0	0 €
258	2.4	1.3	3.7	12.5	2.5	0.0	1,250 €
366	2.4	3.3	5.7	30.5	6.1	0.1	3,045 €
164	2.4	1.5	3.9	13.6	2.7	0.0	1,360 €
117	2.5	1.6	4.0	14.8	3.0	0.1	1,480 €
313	2.5	9.2	11.6	85.9	17.2	0.3	8,590 €
1099	2.5	2.3	4.7	21.2	4.2	0.1	2,115 €
151	2.5	1.5	4.0	14.3	2.9	0.0	1,430 €

RAIN GARDEN STORAGE AND INFILTRATION SUMMARY

PART 2 OF 2

Parcel No.	V runoff roof (m³)	V runoff yard (m³)	V runoff total 10-year return period (m³)	5% of yard rain garden (m²)	V store (m³)	V infiltration (m³)	Cost (rain garden)
392	2.5	4.3	6.8	40.6	8.1	0.1	4,055 €
771	2.6	0.5	3.1	0.0	0.0	0.0	0 €
229	2.6	0.5	3.1	0.0	0.0	0.0	0 €
64	2.6	3.9	6.5	36.7	7.3	0.1	3,670 €
238	2.6	4.4	7.1	41.4	8.3	0.1	4,140 €
543	2.7	1.1	3.9	10.8	2.2	0.0	1,075 €
113	2.7	1.7	4.4	15.5	3.1	0.1	1,550 €
212	2.9	5.1	8.0	48.2	9.6	0.2	4,815 €
350	2.9	1.1	4.1	10.5	2.1	0.0	1,045 €
62	3.0	2.2	5.1	20.3	4.1	0.1	2,030 €
143	3.0	9.4	12.4	87.9	17.6	0.3	8,790 €
194	3.0	2.9	6.0	27.3	5.5	0.1	2,730 €
589	3.1	1.3	4.4	11.8	2.4	0.0	1,180 €
590	3.1	1.2	4.3	11.0	2.2	0.0	1,095 €
247	3.1	3.7	6.8	34.8	7.0	0.1	3,475 €
243	3.1	2.5	5.7	23.7	4.7	0.1	2,365 €
222	3.2	1.3	4.5	12.5	2.5	0.0	1,250 €
229	3.3	2.3	5.6	21.9	4.4	0.1	2,190 €
493	3.4	5.2	8.6	48.7	9.7	0.2	4,865 €
644	3.5	0.8	4.3	0.0	0.0	0.0	0 €
237	3.7	2.1	5.7	19.2	3.8	0.1	1,920 €
187	4.1	3.2	7.2	29.6	5.9	0.1	2,955 €
429	4.3	7.7	12.0	71.9	14.4	0.2	7,190 €
460	4.5	4.9	9.4	46.2	9.2	0.2	4,620 €
326	4.5	1.8	6.3	16.8	3.4	0.1	1,675 €
691	5.6	4.2	9.8	39.3	7.9	0.1	3,925 €
235	5.6	2.7	8.3	25.1	5.0	0.1	2,510 €
1082	5.7	4.0	9.6	37.0	7.4	0.1	3,700 €
965	5.8	5.2	11.0	48.4	9.7	0.2	4,840 €
541	6.0	1.0	7.0	0.0	0.0	0.0	0 €
806	6.3	5.2	11.5	49.2	9.8	0.2	4,915 €
692	7.3	2.5	9.8	23.4	4.7	0.1	2,340 €
848	7.4	3.7	11.1	34.8	7.0	0.1	3,480 €
67	7.7	2.9	10.6	26.9	5.4	0.1	2,690 €
60	9.1	1.0	10.2	0.0	0.0	0.0	0 €
146	9.5	6.3	15.8	59.2	11.8	0.2	5,920 €
329	9.7	9.8	19.5	91.7	18.3	0.3	9,165 €
69	11.5	13.4	24.9	125.8	25.2	0.4	12,575 €
1084	11.8	12.5	24.3	117.2	23.4	0.4	11,720 €
197	12.4	8.7	21.1	81.5	16.3	0.3	8,150 €
377	16.2	0.0	16.2	0.0	0.0	0.0	0 €
330	16.7	2.3	19.1	21.8	4.4	0.1	2,180 €
210	16.9	5.2	22.1	48.3	9.7	0.2	4,825 €
299	17.0	4.2	21.1	38.9	7.8	0.1	3,890 €
850	29.3	0.0	29.3	0.0	0.0	0.0	0 €
TOTAL	**373.6**	**249.7**	**623.3**	**2,337.8**	**467.6**	**8.1**	**233,780 €**
	m3	m3	m3	m2	m3	m3	

Note: Rain garden area is estimated as 5% of yard area. Storage, infiltration, and costs are reported for the selected parcels.