# INVESTIGATING THE APPLICABILITY OF WATER SENSITIVE URBAN DESIGN TECHNIQUES FOR GÜZELYURT, NORTHERN CYPRUS


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I certify that this thesis satisfies all the requirements as a thesis for the degree of Master of Science

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This is to certify that we have read this thesis and that in our opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

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# ABSTRACT

INVESTIGATING THE APPLICABILITY OF WATER SENSITIVE URBAN DESIGN TECHNIQUES FOR GÜZELYURT, NORTHERN CYPRUS

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Water Sensitive Urban Design (WSUD) is a method used to mitigate the negative effect of urbanization on the water cycle. Due to a high percentage of impervious surfaces in urban area and reduction in the share of infiltration and evaporation, a considerable amount of rainfall is converted to runoff. This increase in runoff portion in urban areas causes serious problems for public and private properties. Moreover, the impact of climate change on more intensive rainfall events would cause serious problems that need the close attention of different stakeholders. In addition, water scarcity in semiarid regions and groundwater depletion are the issues which can be mitigated by applying WSUD practices. Harvesting rainwater not only reduce the pressure of rainfall runoff on the drainage system but can also be considered as a source of none potable water for irrigation and household usage. In this study, the effect of harvesting water in existing drywells in Güzelyurt city is investigated as to quantify to what extent connecting drywells to the drainage system can reduce the pressure on the system. To achieve this goal, a map of existing drainage inlets is generated in Geographic Information System (GIS) environment in which it includes six parameters (X, Y, Z, inlet picture, inlet width, inlet length) for each of the inlets. Furthermore, a close range observation of urban characteristics results in a comprehensive rainfall runoff model simulated in SWMM software. The whole city is divided into four main subcatchments and the maximum design rainfall event that cause no problem for the drainage system is obtained as 19, 20, 20 and 23 mm/hr for subcatchments A, B, C, and D, respectively. All the results are verified and approved by technical department of the municipality since there is no measured data for verifying the results. In this study, harvesting water in existing drywells considered as a Low Impact Development (LID) controls. Both subcatchments A and B are chosen for applying LID, for Subcatchment B is the most populated region with highest impervious ratio located at the central part of the city and Subcatchment A is the subcatchment with the least impervious area. Results indicate that applying LID controls to Subcatchment B increases drainage system capacity from 20 mm/hr rainfall event to 40 mm/hr rainfall event and for Subcatchment A the increase is from 19 mm/hr to 35 mm/hr. On the other hand, in case of conventional method for improving the drainage capacity, for such an improvement in the drainage system in Subcatchment B, 909 m existing pipelines need to be replaced with 100 cm-concrete pipes which costs approximately 356,000 TL. However, in case of applying LID controls for Subcatchment B, this cost reduces up to approximately 172,000 TL. The economic comparison shows that for improving drainage system capacity in Subcatchment B, by the use of LID controls approximately 182,000 TL would be saved in compare with conventional method.

**Keywords:** Water Sensitive Urban Design, Rainwater harvesting, Rainfall-runoff model, economic assessment, Mediterranean island, Cyprus.

# ÖZ

SUYA DUYARLI ŞEHİR TASARIMI TEKNİKLERİNİN GÜZELYURT, KUZEY KIBRIS'A UYGULANMASININ İNCELENMESİ

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Suya Duyarlı Şehir Tasarımı (SDŞT) şehirleşmenin su döngüsü üzerindeki negatif etkisinin azaltılmasını sağlayan bir yöntemdir. Yerleşim alanlarında geçirimsiz yüzeylerin yüksek oranlarda olmasından ve yeraltına sızma ve buharlaşmanın azalmasından dolayı önemli miktarda yağış miktarı akışa dönmektedir. Yerleşim yerlerinde artan bu akış miktarı kamu ve özel alanlarda ciddi problemlerin yaşanmasına sebep olmaktadır. Buna ek olarak, iklim değişikliği etkisiyle yağış şiddetindeki artışlar farklı paydaşların yakın ilişki içinde olmasını gerektirmektedir. Yarı-kurak bölgelerdeki su kıtlığı ve yeraltı su kaynaklarındaki azalma sorunları SDŞT uygulamalarıyla azaltılabilir. Çatıdan gelen yağmur suyunun depolanması drenaj sistemleri üzerindeki stresin azalmasına yardımcı olmasına ek olarak daha sonra sulama ve evsel kullanım suyu olarak da kullanılabilmektedir. Bu çalışmada, Güzelyurt'ta çatılardan gelen yağmur suyunun öncelikle evlerde bulunan ve şu anda kullanılmayan emici kuyulara yönledirilmesinin yağmur suyu drenaj sistemi üzerindeki stresi ne derece azalttığı incelenmiştir. Bu amaca ulaşabilmek için mevcut drenaj sistemi Coğrafi Bilgi Sistemleri ortamına aktarılmıştır. Ayrıca, bu drenaj sistemi SWMM yazılımı kullanılarak modellenmiştir. Tüm şehir A, B, C ve D olarak dört ana bölgeye ayrılmış ve deneme yanılma metotuyla her bir bölgenin tasarım yağışı sırasıyla 19, 20, 20 ve 23 mm/saat olarak tesbit edilmiştir. Ölçülmüş yağış akış verisi olmadığından sonuçların doğrulanması belediyenin teknik ekibinin tecrübelerine dayanark yapılmıştır. Daha sonra SDŞT yöntemi olarak yağmursuyu depolama uygulaması en yoğun nüfusun ve en çok geçirimsiz yüzeyin olduğu şehir merkezi alanı olarak B-Bölgesine ve en az nüfusun ve geçirimsiz yüzeyin olduğu alan olarak da A-Bölgesine yapılmıştır. Elde edilen sonuçlara göre SDŞT uygulaması B-Bölgesinde tasarım yağışını 20 mm/saat'tan 40 mm/saat'a, A-Bölgesinde is 19 mm/saat'tan 35 mm/saat'a çıkarmıştır. Diğer taraftan A-Bölgesi ve B-Bölgesinin yağmursuyu drenaj sisteminin tasarım yağışında böyle bir artışın olması için SDŞT uygulaması ve klasik boru değiştirme yöntemleri arasındaki ekonomik analiz de yapılmıştır.

**Anahtar Kelimeler:** Suya Duyarlı Şehir Tasarımı, Yağmur suyu depolama, Yağış-akış modeli, ekonomik değerlendirme, Akdeniz adası, Kıbrıs.

# DEDICATION

To *Sajed* for his continuous supports.

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# CHAPTER 1 - INTRODUCTION

## 1.1 Statement of the Problem

Under natural conditions, water cycle includes precipitation, infiltration, surface runoff, and evaporation which can be considered as the sustainable water cycle. However, development of the cities and growing populations significantly change the natural water cycle by mainly interfering in evaporation and infiltration stages; hence higher amount of runoff water would be drained into the conventional water management systems. The changes in the water cycle not only increase the flood risk but also adversely affect the quality and quantity of the available water for different applications. The main reason for having higher runoff is the impermeable surfaces in the urban areas, causing lack of infiltration and rapid discharge into the public drainage system. As compared to the rural district where impervious coverage may only be 1% to 2%, in urban areas these numbers could increase to 10% in low density urban, 50% in multi housing communities, and 90% in dense metropolitan areas (Hoyer et al. 2011). This defected water cycle finally results in negative impacts on ground water recharge, the quality and quantity of water, and urban climate, which is not sustainable anymore. For example water salinity in Guzelyurt aquifer is a serious issue that happened due to excess amount of pumping and lack of recharging processes (Ergil 2000). Also the quantity impact refers to an increase in flood peak and flood volumes while quality impact is  associated with the high pollutant levels in runoff (Goonetilleke et al. 2011).In addition, recent observations on climate change are another challenge for stormwater management in urban areas. As a result of the global warming, unusual extreme events are happening at different locations on the earth and the conventional systems are unable to manage them. The conventional water management systems are not designed for such extraordinary extreme events and this increases the risk of flooding, hence it is clear that the conventional water management systems are neither sustainable nor adaptable to the climate change (Hoyer et al. 2011). Moreover, changing conventional infrastructures in order to meet the demand is too costly.

In the case of Cyprus, water scarcity is a serious problem. As reported by Maden, 2013 in TRNC water scarcity is about 70-75 million cubic meters while the available water in TRNC is 117.5 million cubic meters per year (Maden 2013). In addition to water scarcity that the whole country is suffering from, more frequent flood events with high peak flow is happening which can be explained by global warming. In the case of Guzelyurt due to high population density and lack of impermeable surfaces in urban design many parts of the city face flood situation after not very intensive rainfall event. This causes difficulties in traffic and damages to personal and public properties.

## 1.2 Objective of the Study

The main objective of this study is to propose a method to reduce the negative effects of stormwater runoff in an urban area with minimum necessity of renewing the infrastructure which results in saving capital investments. To this end, applicability of rainwater harvesting as an application of water sensitive urban design is investigated in this study. As mentioned in objective, in order to prevent any modification in the drainage system of the city, the potential for saving runoff water in the drywells which are already existing is assumed to be used as the reservoir. Applying this method, not only reduces the runoff but also saves water for meeting the upcoming needs. The effect of applying rain water harvesting (RWH) in urban area on runoff reduction is quantified for different rainwater intensities and an economic analysis is carried out. The results of this method are compared with the results of conventional method.

## 1.3 Organization of the Thesis

The outlines of this study are as follows. The first chapter starts with an introduction including a statement of the problem and objective of the study. The second chapter comprises of the literature review and the necessity of performing this study. The third chapter describes general characteristics of the city of Guzelyurt which is the case study. In the fourth chapter, the methodology of study and data used for developing the water drainage model for subcatchments of Guzelyurt are presented. The fifth chapter discusses the results of the critical rainfall event for each subcatchment which would cause flood situation considering current drainage system. This chapter also includes the results for different scenarios of expanding the system capacity. Finally, the thesis is concluded by emphasizing on the major findings and recommending the future works.

# CHAPTER 2 : LITERATURE REVIEW

Waters in cities are mainly considered as drinking water, wastewater, stormwater, natural water bodies and artificial water bodies. Having all these types, water plays a significant role in everyday life, however, regardless of the extreme events like floods and droughts people usually are not aware of the functionality of water. In general an urban area is defined as an area with extensive human activity and a large fraction of impervious area with artificial water resources (Zoppou 2001). Under natural conditions water cycle includes precipitation, infiltration, surface runoff and evaporation. Nevertheless, in urban areas this cycle does not work properly due to various reasons. One of these being the impermeable surfaces in the cities, which causes lack of infiltration and rapid discharge to the public drainage system. These processes are so fast in cities that there is no time for infiltration and evaporation which is shown in Figure 1.

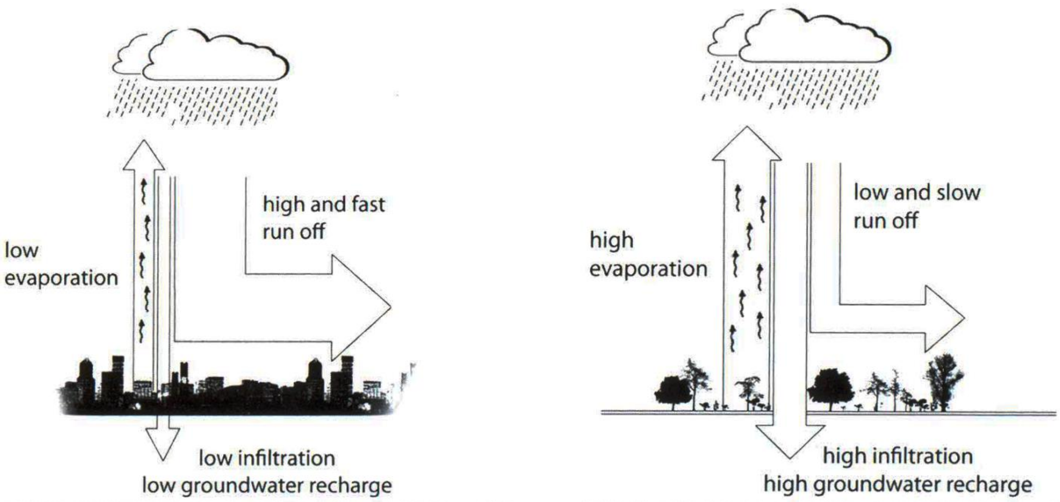



Figure 1. Runoff, infiltration and evaporation rates in urban area (left) in comparison to natural system (right) (Hoyer et al. 2011)

Fletcher et al. (2013) have also highlighted that "urban stream syndrome" is a term that shows the negative impacts of urbanization on both flow peaks and duration. As shown in Figure 2, the flow rates are compared in pre and post development which indicates that in post development situation, peak flow occurs more quickly and intensively. They also stated negative impacts of urbanization on ecology which are; loss of sensitive species, increase in nutrients and toxicants, and loss of organic matter (Fletcher et al. 2013).

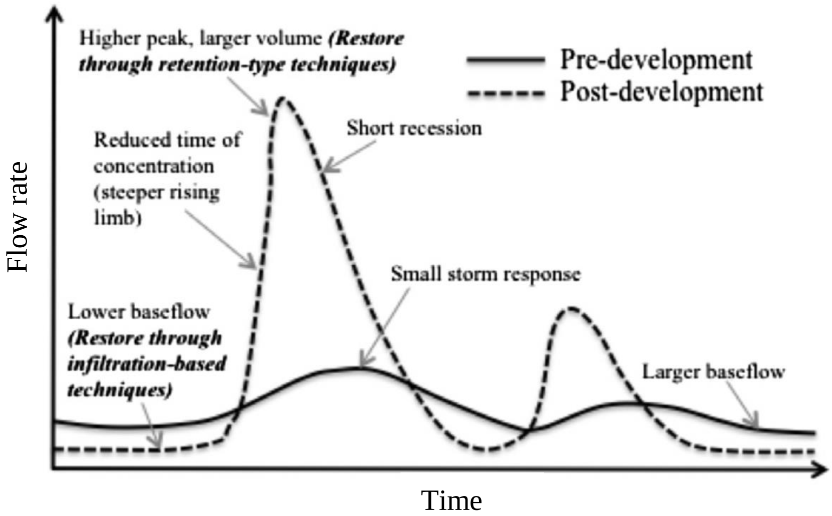



Figure 2. Schematic illustration of the impact of urbanization on hydrology at the catchment scale (Fletcher et al. 2013)

## 2.1 Conventional Storm Water Management

Since the majority of the surfaces in the cities are completely sealed, most of the precipitation quickly turns into runoff. Conventional water system manages it in two ways: a) combined sewerage system and b) separate sewerage system.

In a combined system the runoff water is conducted to waste water and ends up in the waste water treatment plant. It then gets cleaned and discharged into the river. On the other hand, in separate sewerage system the waste water and storm water are collected and treated separately and then discharged to the river or reused (Hoyer et al. 2011).

## 2.2 Problems with Conventional Storm Water Management

Traditionally, water storm has been managed by flushing it out from cities and due to the high speed of this process, ground water infiltration reduces and consequently affects ground water level negatively. Recently, due to global warming some extreme events are happening that the conventional systems are unable to manage them. Since conventional systems are not planned for such events the risk of flooding is increasing so it is clear that the conventional systems are neither sustainable nor adaptable to climate change (Hoyer et al. 2011). As a part of water sensitive urban design (WSUD) projects, the decentralized systems are applied and studied. Various studies have confirmed that decentralized approach is more feasible than the conventional centralized "end-of-pipe" approach for storm water quality treatment (Goonetilleke et al. 2011). In a case study in Australia, total water saving in decentralized system compared with conventional system appeared to be around 60%; moreover the constructional cost for WSUD elements is less than conventional constructions (Coombes et al. 2000). In addition, the conventional systems are mainly underground and invisible so the residents are less aware and responsible towards the water cycle (Goonetilleke et al. 2011). Niemczynowicz (1999) has mentioned that due to changes in surface characteristics, river runoff increases to high peak flow and large runoff volume which results in removal of accumulated sedimentation and pollution transportation from city area to the downstream. Therefore, urban area directly affect the ecological system by changing the whole river system. He has also added that it is generally accepted that storm water should be treated locally and in a small scale, although it encompasses the global environment and sustainable resource management (Niemczynowicz 1999).

Another essential point argued by Niemczynowicz (1999) is eutrophication of river and lake which means "excessive richness of nutrients in a lake or other body of water, frequently due to run-off from the land, which causes a dense growth of plant life". Obviously, the main reason for such problems refer to Increasing number of roads and highways bringing pollution (fertilizers and pesticides) to small, fragile rivers and lakes (Niemczynowicz 1999) .

Furthermore, it is necessary to study the flood situations in water system design. There are two types of flood situations; the first one has been caused by the river and the origin of the flood is being formed at its basin and since the river crosses the city it may cause certain damages. On the other hand, we have another flood situation that its origin is the city itself and it is happening because the drainage system is not capable of conveying water to the treatment plant. Occurrence of these kinds of extreme events which the drainage system is not planned for, are mainly because of global warming.

Commonly used method for designing a stormwater network for an urban area is “rational” method (Akan & Houghtalen 2003). In this method, all the drainage infrastructures are designed based on the peak discharge. However, recently and due to global warming, more extreme events are happening which results in higher rainfall intensities in some regions and consequently higher peak discharge. Therefore, existing drainage systems are not capable of discharging water without facing flood situation. The conventional solution to this problem is extending the capacity of the drainage system by enlarging the infrastructures; however, the alternative solution to this problem is reducing peak discharge by changing runoff coefficient since it is under control. To achieve this goal, there are two possible ways. First, changing surface characteristics by applying different techniques to increase evaporation and infiltration. Second, by means of local water storage which means storing water at the source instead of discharging it directly to the drainage system. All these activities which reduces peak discharge are classified under different terms such as water sensitive urban design (WSUD), low impact development (LID), green infrastructure (GI), sustainable urban drainage system (SUDS), best management practices (BMPs), decentralized rainwater management (DRWM) in different parts of the world (Hoyer et al. 2011).

## 2.3 What Water Sensitive Urban Design Is?

Water sensitive urban design (WSUD) is the interdisciplinary cooperation of water management and urban design and landscape planning. WSUD is considered to manage entire water system like drinking water, storm water runoff, sewerage system and treatment (Hoyer et al. 2011). In other words, water sensitive urban development is a local solution to the global problems created by reliance on conveyance and centralized storage/discharge of water in cities (Coombes et al. 2000).

The objective of WSUD is to make urban water cycle closer to natural cycle with combining the demands of sustainable storm water management with urban planning Figure 3.

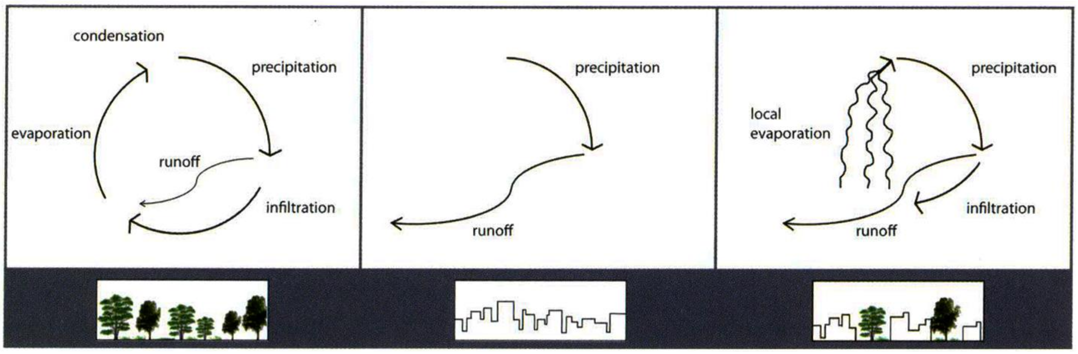



Figure 3. Water cycle in natural system (left); in an urban area without sustainable stormwater management (middle); and in an urban area with sustainable stormwater management (right) (Hoyer et al. 2011).

Sustainable storm water management is a new term added to researches; where its main goal is to reduce storm water runoff by treating the storm water as close to the source as possible. Reducing water runoff needs new technologies to collect storm water and increase the infiltration and evaporation which will end to have a nature oriented water cycle in cities. In this concept treating water does not mean collecting water and discharging it to sewer system which is the usual practice in conventional systems.

Although WSUD covers all parts of the water body, sustainable storm water management play a significant role both as a resource and as the protection of receiving resource. Five main goals of WSUD from storm water management perspective pointed out by Hoyer et al. are:

- Protection of natural water system with urban development.
- Protection of water quality by using filtration and retention quality.
- Reduction of storm water runoff and peak flows by using local detention and measures and minimizing impervious areas.
- Reduction of drainage infrastructure and the related development cost, whilst improving sustainability and amenity of urban areas.
- integration of storm water management into the landscape (Hoyer et al. 2011).

**Technical elements and solution**

There are variety of technical elements and solutions for dealing with sustainable storm water management. Clearly appropriate selection of the method will lead to the success of a system whilst any solution may have its own advantages and disadvantages. The following classification has been done according to their primary function:

**Rainwater use**

Rain water harvesting cistern or water butts are both utilities for storing water which are used underground or aboveground. Cisterns are typically larger than water butts and mainly used for water supply such as toilet or sprinklers but water butts are smaller off-line storage devices used for garden irrigation. Harvesting method can be applied in large complex or individual buildings and play a significant role in architectural design and landscape design like fountains, pools, ponds, etc.

In a study performed by Aladenola and Adeboye (2010), it is stated that by harvesting rainwater in Abeokuta, Nigeria it is possible to meet the monthly demand for flushing and laundry in residential area except in December, January and February. Moreover, it is mentioned that the highest potential for water harvesting is in June and September which is the rainfall peak period in Southwest Nigeria. In another study performed by Petrucci et al. (2012) the effect of rainwater harvesting on runoff is analyzed to investigate the potential of RWH technique for stormwater source control. In an urban catchment with 23 ha area in east of Paris, 1/3 of the private parcels have installed rainwater tanks and the rainfall and runoff were measured before and after tank installation. The results showed that the installed rainwater tanks could affect the runoff for usual rainfall events but are too insufficient to prevent sewer overflows in case of heavy rainfall events(Petrucci et al. 2012). Additionally, in a study that investigated the rainwater utilization in Germany which is performed by Herrmann and Schmida (2000), the objective is mentioned as quantifying the effect of rainwater usage on urban drainage system and the results showed that rainwater usage system can significantly reduce the water consumption and drainage water. In addition, for overflow events it is mentioned that the high specific service water consumption which mainly occurs in multi-story buildings and high population density will lead to reducing or even eliminating overflow runoff(Herrmann & Schmida 2000). Gilory and MacCuen (2009), investigated the effects of location and quantity of cisterns and bioretention pits on stormwater runoff for various return periods and different land uses. They suggested the general trend for locating cistern and bioretention as:

- The importance of efficient volume in controlling peak discharge.
- Locating bioretention in drain pervious surface would be less effective than impervious areas due to partial reduction in runoff rates and volume in grassy areas.
- Effectiveness of cistern and bioretention are highly dependent on the return period of the storm event.
- In large impervious areas with high intensity of rainfall cistern and bioretention should be located in series while for small areas and frequent events it is better to locate them independently.
- Design volume for cistern and bioretention can be based on controlling peak discharge or volume controlling (Gilroy & McCuen 2009).

In another study for investigating the effectiveness of RWH for Northern Cyprus, Okoye et al. (2015), investigated the optimum tank size of a single residential housing unit for rainwater harvesting. They considered a specific rainfall profile, a constant water consumption rate per capita and an assumption of average rooftop area and performed their analyses based on linear programing. The proposed model was applied on the cities in Northern Cyprus and the feasibility of applying RWH as a solution for rehabilitating depleting aquifers has been investigated(Okoye et al. 2015).

**Treatment**

Treatment is the essential process before using stored water in domestic water service or infiltration into the ground water. Coombes et al. 2000 have defined domestic water service as a hot water, toilet flushing, and open space irrigation but have excluded drinking water. There are some particular concerns about treatment issue. For example storm event has the capacity to wash off only a fraction of pollutants and it depends on rainfall intensity, kinetic energy of rainfall, and also pollutants characteristics. Researches prove that high concentration of pollutants in the first flush of all rainfall events occur, so that targeting the initial period of runoff for water quality treatment is highly recommended. Different options for treating water are bioretention, biotopes, and gravel and sand filters (Goonetilleke et al. 2011; Coombes et al. 2000; Hoyer et al. 2011).

Bioretention areas are shallow reservoirs which drain runoff water from below and enhance the quality of runoff water by means of engineered soil, vegetation and filtration and also reduce downstream runoff. (Bioretention systems are different in size or type of vegetation and completely adaptable to urban spaces and making visitors enjoy water retained after rainfall (Hoyer et al. 2011). A sample of applied bioretention is shown in Figure 4.

Figure 4. Bioretention in the city of Ames, IOWA, USA.

Different researches confirm that the water quality is significantly enhanced by use of WSUD method. In the bioretention basin, the outflow was less than 40% of the inflow volume with high attenuation of outflow. Another positive point of the bioretention structure is the reduction of pollutant load of outflow in compare to the inflow as it is shown in Figure 5 (Goonetilleke et al. 2011).

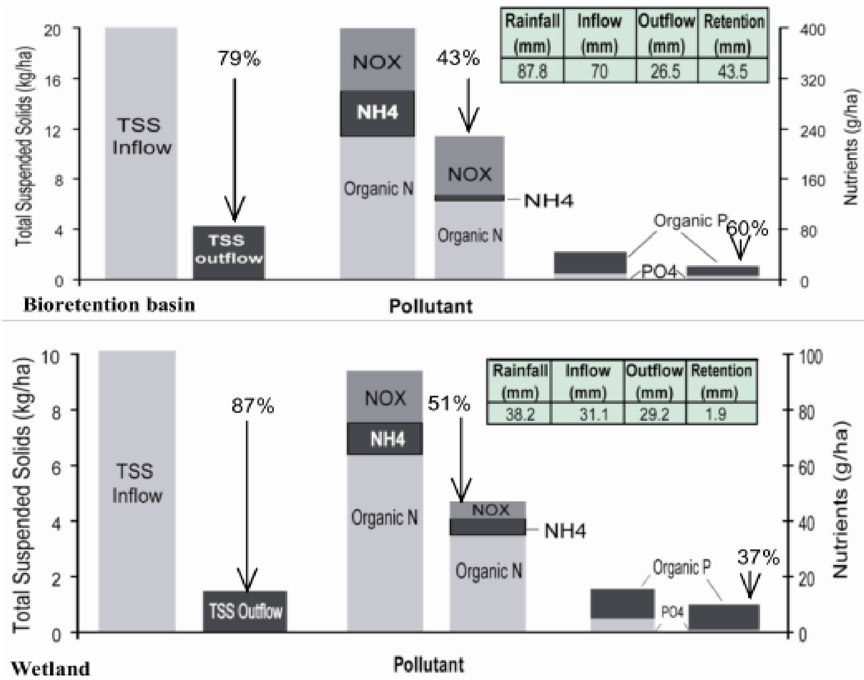



Figure 5. Reduction of pollutant loads in comparison of inflow and outflow of bioretention basin and wetland produced by Goonetilleke et al. 2011.

### Detention and infiltration

The main reason to detain rain water is to reduce the runoff and subsequently reduce stress on storm water, so the flood risk will significantly decrease. Typically, detention systems are a kind of delaying tactic for the peak flows by storing water and gradually infiltrate it into the ground water or conveying water to be infiltrated elsewhere (Hoyer et al. 2011). The followings are two techniques to achieve this goal.

### Rooftop retention

Rooftop retention consists of multilayered structures which are designed in extensive or intensive methods. Extensive roofs are lighter and succulent plants (plants that having thick fleshy leaves or stems adapted to storing water) are the main features for them whereas intensive roofs are thicker and support the deep rooted vegetation. Rooftops affect the appearance of the city and individual buildings as well and easily linked the structured building to the landscape. A sample of vegetated wall is shown in Figure 6 to show the integration of WSUD techniques with urban design. As Hoyer et al. (2011) mentioned in his book "on the city scale green roofs replace lost habitat, repairing urban ecology and biodiversity". In addition, such environments have a direct correlation to human health and psychological issues. Fletcher et al. (2013) mentioned that vegetated roofs have a great benefit over other retention systems due to the coverage of 100% of the catchments and enhance the catchment lag time which may lead to flood mitigation. He also added some technical factors for designing vegetated roofs including depth and type of materials, vegetation cover with low evapotranspiration in order to survival in dry period and slope of the roof and roof position with regard to the wind and sunlight (Fletcher et al. 2013).


Figure 6. A sample of vegetated wall in Florence, Italy.

**Permeable paving**

Permeable paving is a kind of structure that lets water pass through designed sub grade layers such as gravel bed or other porous medium so that water can infiltrate into the ground or drain to the sewer system. This method can be applied especially in cities where space is a commodity. Also, this method is applicable for either pedestrian or vehicle traffic.

**Infiltration zone and techniques**

Infiltration zones are some concentrated planted spaces designed for rapid infiltration, mainly constructed with gravel, sand, and other mineral substructure. Their design criteria is highly dependent on rain water intensity, local soil conditions, and available space. Infiltration zones and trenches are highly adaptable in urban spaces like public and private gardens, parks, road side planters, drive ways, and sidewalks. Clearly these kinds of settings play a significant role in beautifying the urban spaces with highly paved area and consequently improve public health.

**Swales** are linear vegetated drainage feature designed for conveying water with permeable base so that the infiltration during the conveyance is possible. These features can be incorporated into recreation purposes and beautiful landscapes (Hoyer et al. 2011). As an example, Figure 7 shows the grass swale in the city of Ames, IOWA, USA.

Figure 7. A grass swale in the city of Ames, IOWA, USA.

**Detention ponds** are surface storage basins to collect water and they are drained into conveyance system and during this process infiltration is automatically done. During dry seasons ponds can be utilized for recreational use (Hoyer et al. 2011). Figure 8 shows an example of detention pound.

Figure 8. Detention pond in urban area in the city of Ames, IOWA, USA.

**Evapotranspiration**

Evapotranspiration is an important component of the water cycle that affects the temperature, humidity and precipitation to confront with heat island effect ("heat island" describes built up areas that are hotter than nearby rural areas (EPA 2014)). The heat island effect is mainly because of lacking vegetation and water and also heavily paved and massive absorption of heat by means of the materials used for constructions in the cities. Active evapotranspiration is a branch of evapotranspiration which utilizes water directly to influence the temperature and air quality of public spaces by means of rain water walls, fountains, and pools. Passive method refer to vegetated system which is contributed to enhancing the water cycle (Hoyer et al. 2011).

**Principles for successful water sensitive urban design**

For the success of water sensitive urban design these five topics are defined to be checked for any WSUD projects:

**Water sensitivity**

Water sensitivity means solutions and methods which bring urban water management closer to natural water cycle. Main characteristic of natural water cycle is a high evaporation, a high rate of infiltration and low surface runoff. It is important to manage water close to the source to restore small scale water system (Hoyer et al. 2011). Similarly, Niemcznowicz, 1999 stated that the philosophy of this approach is based on control at the source and small and local scale solution. Thus, it should become applicable on a level of a single house, one parking lot, one street or a part of a large highway system (Niemczynowicz 1999).

**Aesthetic benefits**

It is very important to design a visible storm water management system to increase the awareness of the citizens about natural water cycle and make them more sensitive on water resources. WSUD solutions should capture resident attentions by providing an aesthetic benefits and improve the quality of public and private spaces. The second important factor is that WSUD solutions should be adaptable to the design of the surrounding area. Having creative manufactured material and design will lead to a significant contribution towards sustainable approaches (Hoyer et al. 2011).

**Functionality**

For having a successful water sensitive urban design the functionality of the solutions should be checked with these three circumstances (Hoyer et al. 2011):

**Appropriate design**

In the WSUD techniques, consideration of site characteristics such as topography, ground permeability, water table levels, and water quality is highly essential. In addition, planers should be aware of variety of available techniques due to being capable of combining methods and finding the most fitted solution.

**Appropriate maintenance**

Appropriate maintenance is of great importance in water storm management which is often not taken into account. Inadequate maintenance not only causes poor performance but also decreases aesthetic value of the installation.

**Adaptability to uncertain and changing conditions**

Like any other urban infrastructure, solutions should consider the uncertainty of the nature, like climate change or uncertainty of demographic issues or economic change in order to be more flexible. (Beecham & Chowdhury 2012).

**Usability**

Since storm water installation requires a large area and space which are valuable spaces in urban design for other aims such as recreational purposes, this is of great importance to consider all space demands (Hoyer et al. 2011). Niemczynowicz (1999) pointed out some factors that should be considered in WSUD approach including land use policy, city and landscape planning, building construction, development control, economy, legislation, education and public acceptance and local community involvement (Niemczynowicz 1999).

**Public acceptance**

In order to reach the state of public acceptance it is necessary to show the reliability of WSUD solutions. For example collecting rain water could be used for domestic use, and in the case of household use, monitoring the quality of water is highly required to convince people to use this sort of solutions. Temperature, PH, conductivity, turbidity and salinity are characteristics which should be measured. The results obtained from a case study in Australia show no exceedance of guidelines for metal and chemical parameters and also hot water systems in the range of 55'C to 63' C appears to have the capability to eliminate contaminated water from bacteria. Results of the survey in Figtree, Australia, revealed significant acceptance (95%) for rainwater reuse and also 65% reduction of consumption during June to December in 2000. For sure, such kind of activities will raise public awareness of water issues (Coombes et al. 2000). Moreover, WSUD should consider the demands of all stakeholders and involve them in the planning process. Also, solutions should be comparable to the cost of conventional solutions.

WSUD is a relatively new term in water management which refers to solutions and methods to confront with climate change issues. It is a methodology to adapt cities with new emerging problems like increasing flood risk or water scarcity. Clearly, such kind of solutions should be implemented in small scales but more frequent to capture the public acceptance and awareness. In addition, maintenance and affordable cost for such projects are highly important. The main points that can be listed as the conclusions of this literature review are:

- Due to climate change, recently cities are facing extreme events that are not suitable for conventional drainage systems.

- The main issue in conventional system is the adverse impact of urbanization on natural water cycle and ground water level and also ecology of surrounding nature (eutrophication).
- In conventional water management system the whole volume of the rainfall changes to runoff rather than infiltration and evaporation and centralized to drainage system which may cause flood situation in extreme events.
- Since the conventional storm water management systems are mainly underground, urban residents are not aware of the importance of water issues.

Due to all these problems with conventional stormwater management systems, defining a new approach to tackle these issues is highly required. WSUD is an interdisciplinary cooperation of urban planning and water management and its main goals are:

- to make an urban water cycle close to natural water cycle by the means of infiltration and retention,
- reduction of runoff and peak flows by minimizing impervious areas and local detention of stormwater,
- reduction in cost for developing infrastructures,
- an integration between water management and landscape design,
- and managing storm water in a small and local scale.

There are advanced technologies based on infiltration and retention rules such as several kinds of ponds, swale, plant filters, wetlands, green roof, permeable paving and etc. Moreover, there are several rules that should be considered as the principles to insure the success of WSUD projects: water sensitivity, aesthetic benefit, functionality (appropriate design, appropriate maintenance, adaptability to uncertain and changing basic conditions), usability, public acceptance

**WSUD implementation in this study**

As it is noted in literature review, there are various techniques and applications categorized under WSUD concepts. However, the prominent focus of this study is on RWH application. Harvesting rainfall runoff from rooftops in the existing drywells would be considered as a WSUD techniques. It should be noted that in SWMM software that is used for modeling stormwater network, system, the application of cisterns for collecting runoff termed as (LID controls).

# CHAPTER 3 : DESCRIPTION OF THE STUDY AREA

## 3.1 General Characteristics

Cyprus is the third largest island in the Mediterranean Sea which is located south of Turkey and west of Syria. Cyprus is projected on 36th UTM zone, Northern hemisphere in WGS 84 projection system (WGS 84 / UTM zone 36N). This study has been done for a city of Guzelyurt in Northern Cyprus(TRNC).Guzelyurt is the forth main city in Northern Cyprus after Nicosia, Famagusta, and kyrenia with longitude of 32° 59' and latitude of 35° 12'N. Guzelyurt with population of 30037 makes up about 10.4% of the total population in 2011 population census(TRNC 2011). Guzelyurt region is consist of two main district which are Guzelyurt central and Lefke. The population for Guzelyurt central is reported as 18,946 which is 6.6% of the total population in 2011 census. More demographic details are shown in Figure 9. Also, Guzelyurt population from 29,264 in 2006 reached to 30,037 in 2011 which shows 2.6 % growth in population (TRNC 2011).

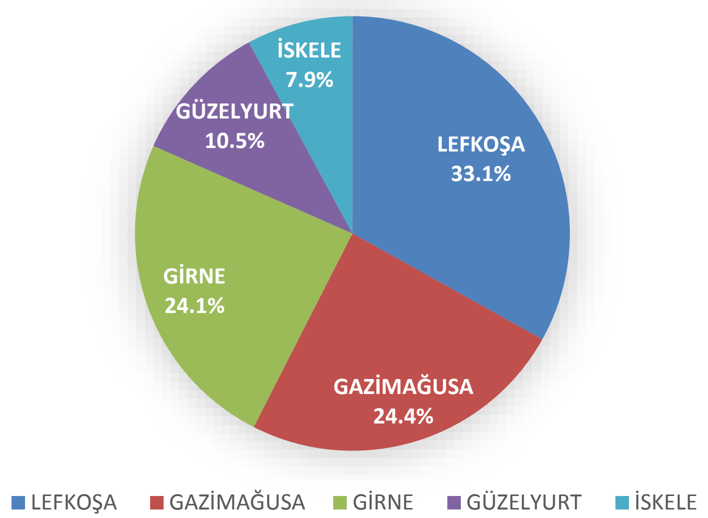



Figure 9. Demographic details of Cyprus.

Guzelyurt city is located on North-West of Cyprus which is about 40 km west of Nicosia and roughly 7 km far from coastline. Figure 10, depicts the location of studied area in different scales.

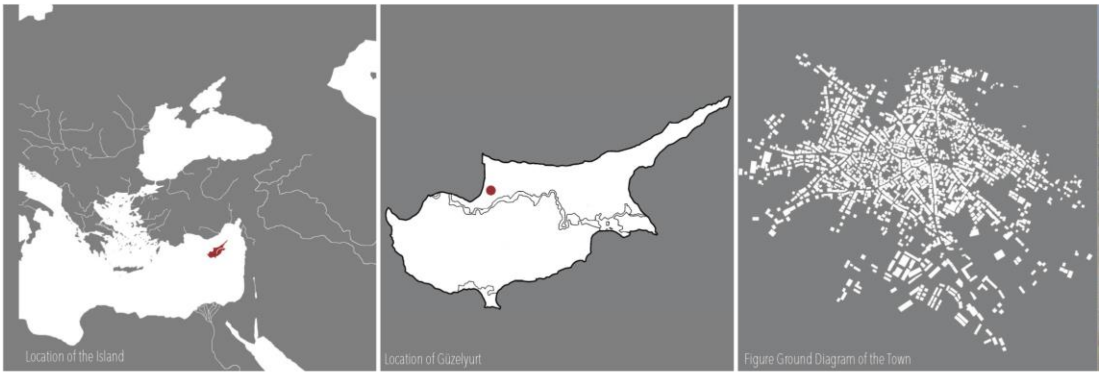



Figure 10. Location of study area in different scale (REAP 2016).

The area for Guzelyurt is reported as 381 square kilometer while the urbanized area is about 3-4 square kilometer which is surrounded by agricultural lands and fruit gardens. The topographic condition is smooth and the ground falls on a fairly regular slope towards the river bed which is located on the north side of the city. Serrahi river bank is always dried due to construction of three dams on upstream side of the river. Therefore, there is no concern about having flood situation on this river bed in Guzelyurt urban area. Also, the largest water table in Northern Cyprus is laid down exactly beneath the Guzelyurt city. This aquifer is one of the main resources for water demand in Northern Cyprus. Actually due to lack of any permanent river in the region, 92% of consumption depends on water aquifers which cause drastic decrease in ground water level specifically for Guzelyurt aquifer. This reduction in ground water level lead to intrusion of sea water to the water body (salinity). In the case of Guzelyurt aquifer, water quality decreased to 5000 ppm in salinity due to over consumption and salinity(Ergil 2000). Guzelyurt climate is both Mediterranean and semi-arid. It experiences hot and dry summer with almost no rainfall during June, July, and August. The urban design is very old with high density of residential area in the central part. The design of the city has not been change since 1970's due to uncertainty in properties ownership. Moreover, due to land's low price in Guzelyurt 67% of dwellings are single family detached houses, 20% semidetached and double storey and only 3% of the overall buildings are apartments and the rest which is 6.7% consist of traced/row housing(REAP 2016). Figure 11 shows some typical dwellings in urban area.

Figure 11. Typical dwellings in Guzelyurt.

Furthermore, in the central part of the city streets are very narrow without proper sidewalks or bicycle lane which makes people to use their private vehicles rather than walking or riding bicycles. Therefore, poor urban infrastructure cause high traffic density in the city during rush hours. Last but not least, there are very few parks and recreational area designed for the city especially in central part which increase impermeable surfaces in urban area and obviously reduce the quality of urbanization.

## 3.2 Existing Pipe System

The current existing pipe system does not cover all the streets in the urban area. Therefore, the streets that do not have any drainage system should convey rainwater runoff through the surface by means of natural slope of the streets to reach the stormwater drain. Hence, in many instances water accumulation and flooded situation would occur in street junctions. Moreover, due to insufficient capacity of the stormwater network system, even during a regular rainfall event the system has faced failure. Having an old sewerage system in the city, there are no available maps or detailed documents about the existing pipe system such as profile map or cross sections. The only way to gather data is contacting with municipality and asking about the locations of pipes and their sizes. All the existing pipes of the whole city with their sizes were determine on the paper map after having three meeting sessions with technical staffs in Guzelyurt municipality. Figure 12 shows the location of the pipes in the city plus their sizes which are determined by the municipality's staffs. As shown in Figure 13, lines with green color are representing pipes with 500 mm diameter which are the most common size used for drainage system in this city, line in purple indicate the pipe size with 400 mm diameter and the yellow ones are the pipes with 300 mm diameter. Also, the one meter diameter pipe size, which are used for conveying the whole runoff to the riverbed, are shown by an orange line in the North part of the map.  On the other hand, the thick red line passing through East to West side of the city is representing an old concrete rectangular channel which were used for irrigating gardens on the West side of the city. However, by using reservoirs on the East side of the city, currently this channel is not used any more for irrigation purpose except few meters for conveying water to another pipe system.

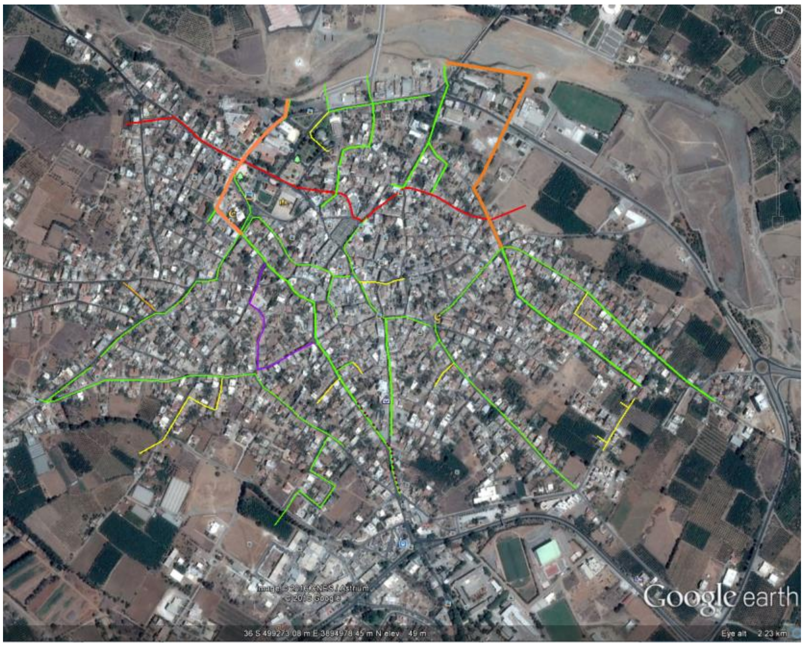



Figure 12. The whole existing pipe system of Guzelyurt generated in Google Earth.

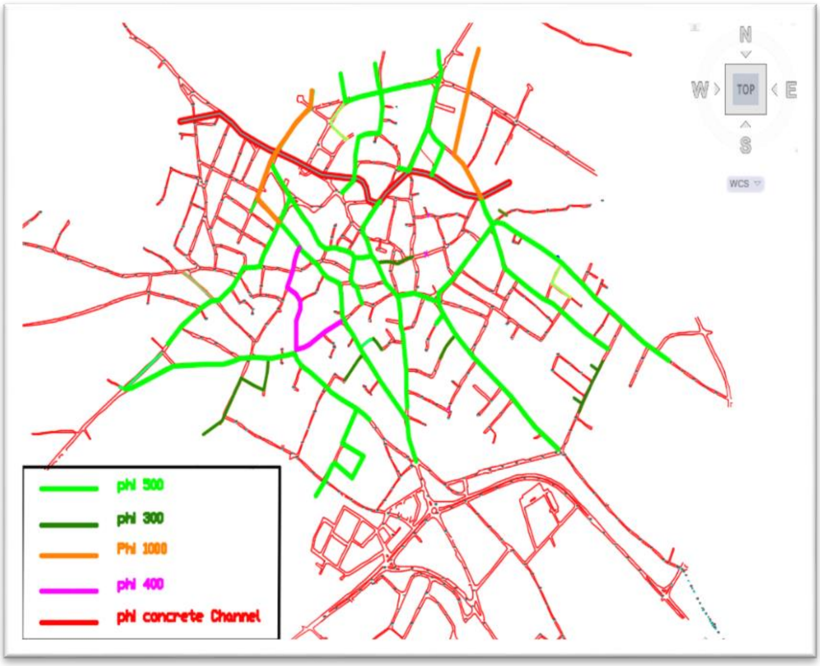



Figure 13. The exisiting pipe system in Guzelyurt.

### 3.3 Slope Condition

As shown in Figure 14, general slope of Guzelyurt is from Southeast to Northwest. The dark blue indicates higher elevation and light blue shows lower elevation. As it can be seen in Figure 14, the whole region has uniform downward slope toward Northwest.

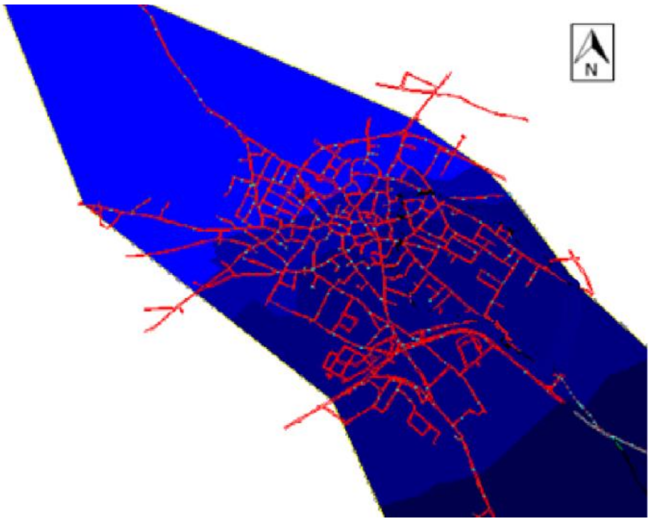



Figure 14. General slope condition of Guzelyurt generated by Auto CAD Civil 3D software.

### 3.4 Land Use

Land use mainly refers to human manipulation of the land and making benefit of its resources by interfering in the ecological processes (Niehoff et al. 2002). Land use changes directly affect the ecosystem by changing water cycle, biodiversity and radiation budget (Riebsame et al. 1994). Depend on different methodology for developing direct runoff model, the influence of land use change on infiltration condition, soil-macroporosity, and dynamic of saturated zone can effectively change the runoff volume. Moreover, land use pattern are highly dynamic and affected by social, economic, and management strategies (Niehoff et al. 2002). Land use and land covers are very crucial for flood control, water-supply planning, and management of available water-resource (Anderson et al. 1964).

Land use in Guzelyurt can be classified into three main types. First, urban area that has high percentage of impervious surface like roofs, streets, pavements, roads, and parking lots. Second, rural area with higher percent of pervious surfaces such as lawns, fields, bare lands, cultivated land, gardens and forest. The third type is the combination of rural and urban area which is called semi urban area.

## 3.5 Drywell

In most of the developing countries, there is no central sewerage system available in urban areas. However, in Northern Cyprus after 2008 the main cities such as Nicosia, Famagusa, Kyrenia, and Guzelyurt got connected to central sewerage system. Since 2008, in Guzelyurt, each house and separate shop was connected to a septic tank and a well located underground. After 2008, all the residential areas got connected to the central sewerage system and since then all of the existing wells got dried and not being used. The characteristics of these wells are shown in Figure 15, adapted from technical report (IMO, 2013). In Figure 15 values that are considered for D, G, and H are 1m, 1.25m and 2.75m, respectively which are adapted from a table developed in a report (IMO, 2013). Therefore, considering the wall thickness of septic tank as 20 cm the volume for septic tank calculated as 2.94 square meter. Also, a cylindrical well with 0.9m diameter and height of 10m is connected to the septic tank which result in the total volume of 9.3 meter cube for both septic tank and well.

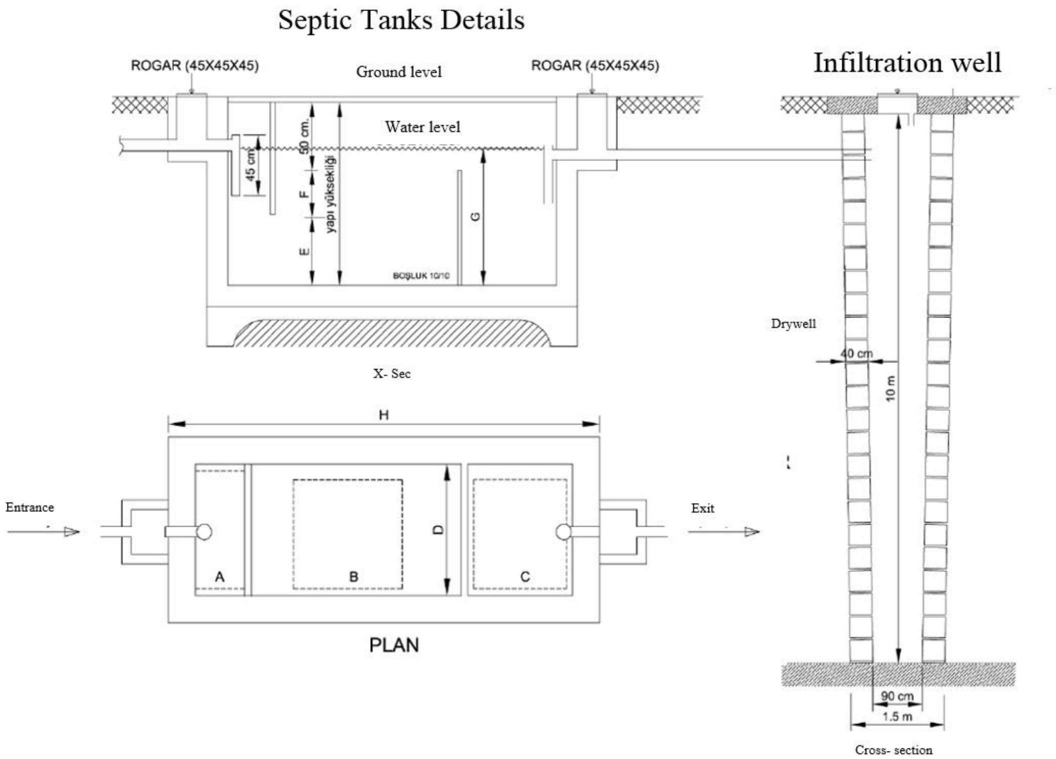



Figure 15. Characteristic information of existing drywell and septic tank (IMO, 2013)

# CHAPTER 4 : DATA AND METHODOLOGY

## 4.1 Field Measurements

Lack of detailed data for existing stormwater management system for developing a model of the system raises the necessity of having a comprehensive database for inlets of the system. The solution that addresses this data shortage is to generate a database in GIS environment for stormwater management system of Guzelyurt. This database consists of six main components. First, location of each inlets with respect to the street map of the Guzelyurt which includes two components (X,Y). Second, surface elevation of each inlet which can represent the elevation difference between the inlets and slope of the surface between inlets. The last three components are attributive parameters which consists of size of each inlet (length and width) and the picture of the inlets. All these features make a complete and comprehensive view of inlets condition and location in a GIS environment. Therefore a field work measurement has been designed in two separate sections. First, a topographic survey for determining the location and elevation of the inlets. Second, a field work to collect the size of the inlets and taking picture of each inlet. The First part of the project which was surveying for the whole inlets of Guzelyurt stormwater management system has been done by two persons in five months. The second part which was collecting attribute data for each inlets such as size and picture of the inlet has been done by one person in one month. One of the deliveries of this study is a comprehensive database for 515 inlets of the stormwater management system of Guzelyurt.

### 4.1.1 Map Coordinate System

The only available map for Guzelyurt is the street map of the city. In this map all the roads and streets are surveyed plus 3D point data of roads elevation. All the 3D points are just inside the street with maximum distance of 50 meter and the minimum distance is mainly dependent on existing condition. The main use of these 3D points is to find the slope of the street since the density of the points are very less and they cannot represent a good estimation of topographic condition of the site.

Due to lack of documentation, the digital map of the Guzelyurt which is provided by municipality does not have specific global coordinate system and specific scale. Therefore, the map needs some modification to be attached to the global coordinate system. By comparing the coordinates of specific points both in the digital maps and Google Earth coordinate system, turns out that the digital map needs a shift to be fitted in global coordinate system (WGS84/UTM). The shift that is applied to the whole map is as follow:

Distance = 1559.7823 m, Delta X = -5.2160m, Delta Y = 1559.7735m,

Delta Z = 0.0000

By applying this shift to the whole map the coordinate system of the digital map change to World Geodetic System 1984 (WGS84) ellipsoid with Universal Transform Mercator (UTM) projection system in zone 39 northern hemisphere (WGS84/ UTM zone 39N). Therefore all the measured data needs to be fitted to the existing map to make a single coherent map.

### 4.1.2 Site Measurements

The equipment that is used for surveying inlets of the city includes Total Station (SET630RK), prism, tripod, rod, EDM (Electronic Distance Measuring) and camera. The method that is used for measuring by Total Station is free station method which is a measuring in a local coordinate. All the inlets for each pipeline is surveyed in a local coordinate system defined by the operator during the measurements for each set up of the Total Station. In site measurement for each set up of the instrument, some reference points such as street corners or waste water management manholes are surveyed in addition to the inlets location. The minimum number of reference points that should be surveyed in the site is three points in order to be capable of justifying the local coordinate with the main coordinate. However, the more number of reference points in the site the better fit to the main map. Figure 16 shows site measurements in Guzelyurt.

Figure 16. Site measurement in Guzelyurt

## 4.2 Processing Data

One of the main procedures after collecting raw data is processing them in a way that can be understood by users and managers. Generating digital map and making GIS geodatabase are the final results of data processing.

### 4.2.1 Converting Measured Data to Digital Map

For processing the measured data all the measurements need to be transferred to the computer. The software used for transferring data from Total Station to the computer is SOKKIA Link. The exported data from Total Station is in a text format. Hence, data processing for converting text file to DWG file is done by using Auto CAD Civil 3D software. At this step, the digital map of the inlets measured in site is generated. Since the measuring coordinate system used in the site is a local coordinate system, an appropriate shift and rotation is needed for every each of data batch collected in each instrument set up to be fitted to the reference map. Finally, using the surveyed reference points and the corresponding points in the map, the inlets location can be fitted to the main map.

### 4.2.2 Arc GIS Database

All the data has imported to Arc GIS environment in order to develop a database consists of geographic information and attribute data. Geographic data consists of Easting, Northing, and Elevation of each inlets. Attribute data are all the information such as length and width of each inlets and the picture captured from each of them. All this information is attached as an attribute information to each of inlets in Arc GIS environment. Therefore, any user would easily have access to all provided data just by a click on specific inlets.

## 4.3 Subcatchments Detection

Subcatchments are territory of land that make surface runoff drain to the single discharge point by means of topographic conditions and drainage system facilities (Rossman, 2015) . Also there is another definition by Akan and Houghtalen 2003, " The land area that contributes flow (runoff) to stormwater structure is usually called Watershed, Catchment, or drainage basin of that structure". For investigating rainfall-runoff analysis, the subareas borders should be defined. By subareas determination, runoff contribution to the specific point can be determined. For defining the border of subareas different parameter should be considered. First, subareas are determined according to the existing pipe system and the location of the inlets. Thus, detailed plan for the location of the inlets is needed. Second, surface flow directions are determined. Due to lack of piping system in each street, runoff flows through the surface to reach to the nearest drain inlets. Therefore, investigating flow direction in each street to determine the border of the subareas is necessary. Third, google earth aerial photos are used to have a comprehensive view of the urban area. Thus, the Google Earth pro software has been used. All these parameters which are necessary for defining subareas are explained in the following chapters in detailed.

### 4.3.1 Existing Inlets

For designing a hydraulic model of the stormwater management system, it is necessary to define a catchment for each inlets in the pipe system. By defining catchments, runoff volume exposed to the specific part of the pipe system, can be calculated. Since the distance between each inlets in current study is maximum 14 m and in many cases around 4 m, it is not necessary to define a catchments for every each of these inlets. In this study, the maximum distance between manholes assume to be less than 100m which is determined by Akan & Houghtalen 2003 to avoid redundancy in subcatchments. Also, minimum distance for designing manholes can be determined according to junctions of pipes, pipe size variation or changing direction in piping system. In designing subcatchments, existing pipe system, topographic conditions, street slops and having manholes at maximum distance of 100 meter are the criteria which are concerned.

### 4.3.2 Flow Direction

Overland flow occurs when the infiltration capacity and depression storage of the surface get full, while the rainfall is continuing with the higher rate compare to the rate of losses. This situation will end to formation of a thin layer of water on the surface which flows by gravity through the greatest slope (Usul, 2001,p 168). Therefore, to investigate the overland flow direction it is necessary to determine the greatest slope in any possible condition to determine a flow direction. Based on the available road map and elevation points which were provided by municipality, the slopes and flow directions between every each of elevation points are determined manually. As it is shown in Figure 17, black arrows are showing the flow directions. Although it is not possible to show the direction of the arrows in such large view, Figure 18 with a closer range view shows some part of the map with flow directions.


Figure 17. The flow direction in each street generated manually regarding to the ground elevation.

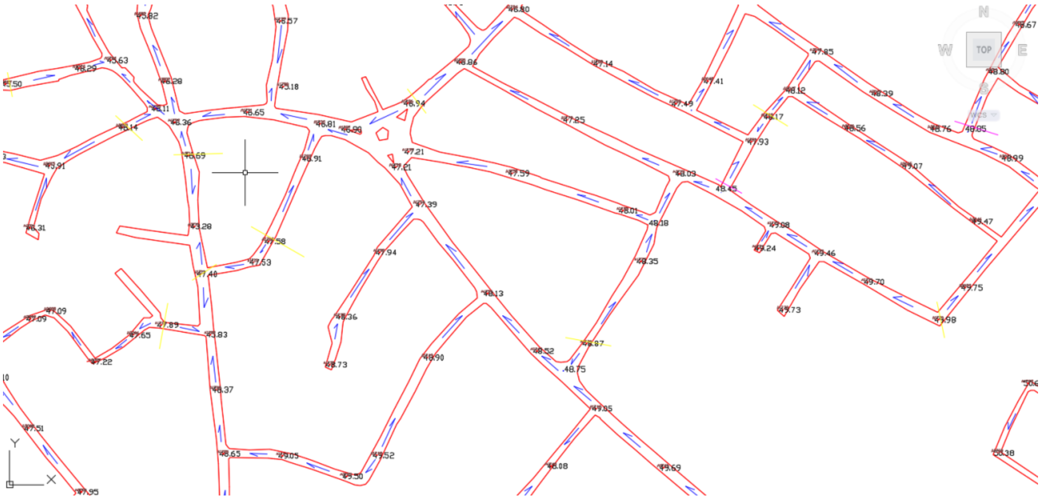


Figure 18. Flow direction in each street generated manually in Auto CAD environment according to road elevation.

## 4.4 Modelling Stormwater Management System

The first computer models for representing water system performance have been developed in mid 1960s. Later on in 1970s models which could simulate the quantity and quality characteristics of water system were developed mainly by US Environment Protection Agency (EPA). All these models cover both simple conceptual models and complicated hydraulic systems (Zoppou 2001). The disadvantages of numerical modeling are that their algorithms which are complicate and not easy to be understood by the users and firm to change the algorithms (Liong et al. 1992).

### 4.4.1 Models Approaches

Models can be classified into different types such as stochastic model, deterministic models, conceptual models, empirical models, distributed or lump models and event or continuous models. Stochastic models are the models which generate different results for each time of calculation having the same inputs. In other words if any of the variables having random characteristics, therefore, the model is considered as stochastics model. On the other hand, when the model generates the same result for the same input each time, it is called deterministic model. The advantage of stochastic model is that the randomness of the variables are considered into the model while for deterministic models there are some reliability techniques that can estimate the uncertainties in the result due to randomness of the inputs. The only restriction for stochastic model is that the variable's distributions should fallow certain probability distributions.

Conceptual models are based on physical laws whereas the empirical models are based on observations experimental results. Since many of the physical laws are based on experiment and defined as empirical laws distinction of this two types is difficult.

Distributed and lumped models are grouped under the term of spatial characteristics and show the sensitivity of the models to the spatial variability. Distributed models are sensitive to the spatial variability while lumped models don't take into account. The majority of urban models for rainfall- runoff is deterministic and distributed models.

Event or continuous models are temporal based models. Event models account for short –term period event or individual events. These models are mainly used for design of hydraulic structures which is called operational models. Additionally, models that deal with long period of time are used for planning models. For example simulating overall water balance for a catchment for a long time can be the basic of the planning models for water resources. Urban storm water models are grouped in operational models.

In general, models can be differentiated by different parameter such as number of input data that is required, the results that is generated by the model, computational complexity and simulation period.

Urban storm water models can be divided into two main parts. First, rainfall-runoff modeling which is related to computation of runoff considering precipitation and losses due to infiltration, evaporation, and transpiration. Second, transport modeling which is associated to routing of flows through the pipe network (Zoppou 2001).

A traditional classification for stormwater modeling which are mainly based on conservation law is either hydrologic or hydraulic models. Using this law, fluid behavior in one dimensional flow is investigated. Various definitions are used for conservation law which is about conservation of volume, continuity, conservation of momentum, and conservation of energy. Hydrological models only deal with continuity equation while the hydraulic models cope with both continuity equation and energy equations (Zoppou 2001).

### 4.4.2 Modeling in SWMM software

There are numerous number of modeling packages which are capable of simulating water quantity and quality available for managers and researchers. They are mainly developed by US agencies such as academic institutions, regulatory authorities, government departments and engineering consultant. All these models offer various types of capabilities with different range of spatial and temporal resolution. In a review paper done by Christopher Zoppou, 2001 eight models which are specifically coping with urban stormwater quantity and quality are reported as: "DR3M–QUAL (Alley and Smith, 1982a,b), HSPF (Bicknell et al., 1993; Johanson et al. 1980, 1984), MIKE–SWMM, QQS (Geiger and Dorsch, 1980), STORM (Hydrologic Engineering Center, 1977), SWMM (Huber and Dickinson, 1988; Huber et al., 1984; Roesner et al., 1988), SWMM Level 1 (Heaney et al., 1976) and the Wallingford Model (Bettess et al., 1978; Price, 1978; Price and Kidd, 1978)". These models can be differentiated with their different capabilities such as modeling of water quality and the types of pollutant that is applied for simulating water quality or differences in simulation method or variation in temporal and spatial resolutions. More detailed about aforementioned models are available in Zoppou, 2001.

In this study an EPA Stormwater management Model (SWMM) version 5.1 is employed. The current version of the software was developed by the Water Supply and Water Resources Division of the US Environmental Protection Agency's National Risk Management Research Laboratory with collaboration of consulting firm of CDM-Smith (L. A. Rossman 2015). The very first version of SWMM was released in 1969-71 and since 2005 it is upgraded and become one of the best known and widely used program. More detailed information about the development history is available in (L. Rossman 2015).

SWMM is a dynamic model which is capable of simulating rainfall runoff procedure both in single event or long term event. The model can be divided into two main parts: first, runoff calculation generated from various subcatchments and second, routing part which accounts for transporting the resultant runoff through pipes, channels, treatment storage, pumps, and regulators(L. A. Rossman 2015).

Various hydrologic processes that is applied in SWMM program are listed in (L. A. Rossman 2015) as below:

- rainfall characteristics
- evaporation of standing surface water
- snow accumulation and melting
- rainfall interception from depression storage
- infiltration of rainfall into unsaturated soil layers
- percolation of infiltrated water into groundwater layers
- interflow between groundwater and the drainage system
- nonlinear reservoir routing of overland flow
- capture and retention of rainfall/runoff with various types of low impact development (LID) practices.

Moreover, in hydraulic section which is simulating the runoff transportation trough the drainage system, modeling capabilities are listed in (L. A. Rossman 2015) as:

- handle networks of unlimited size
- use a wide variety of standard closed and open conduit shapes as well as natural channels
- model special elements such as storage/treatment units, flow dividers, pumps, weirs, and orifices
- apply external flows and water quality inputs from surface runoff, groundwater interflow, rainfall-dependent infiltration/inflow, dry weather sanitary flow, and user-defined inflows

- utilize either kinematic wave or full dynamic wave flow routing methods
- model various flow regimes, such as backwater, surcharging, reverse flow, and surface ponding
- apply user-defined dynamic control rules to simulate the operation of pumps, orifice openings, and weir crest levels.

Since in SWMM program unlimited number of subcatchments can be defined therefore, each catchment can be divided into homogenous subcatchments to fulfill spatial variability in modeling system. Moreover, SWMM can simulate the quantity and quality of runoff in specific period of time (event or long term) with required time step. One of the capabilities of this program is simulation the quality of the runoff generated from subcatchments and calculating the amount of pollution in runoff. Since investigating about the quality of runoff is out of scope of this study therefore detailed data about quality of runoff has not been provided.

The physical elements of a rainfall-runoff system are shown by three main components in SWMM model. Rain gages, subcatchments and conveyance section. SWMM conceptualize all these three components as shown in Figure 19.

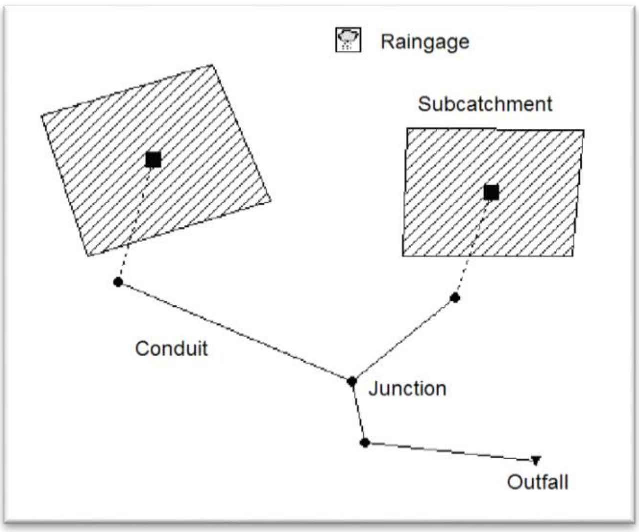



Figure 19. SWMM's conceptual model of storm water drainage system.

As depicted in Figure 19, rain gage consist of precipitation data that occur for all or some of the subcatchments and subcatchments are the hatched parcel which represents a land region that receive the rain gage precipitation and generate runoff which flows into drainage system or another catchment. The drainage system is shown by sets of nodes and links. As shown in Figure 19, nodes represent junctions, dividers, storage units or outfalls while links are the lines which connect the nodes and represent conduits, pipes, channels, and flow regulator such as orifice, weirs, or outlets.

***4.4.2.1 Simulation process overview***

The overview of the steps that is conducted in this study by SWMM model to compute rainfall-runoff process is shown in Figure 20. As it is shown in Figure 20, after applying the precipitation to subcatchment and compute the initial abstraction which is due to infiltration, the surface runoff can be calculated. Whether the LID controls applications are applied or not this step can be considered or not. Next step is related to hydraulics and flow routing in piping system and finally end to the non-pressurized outfall.

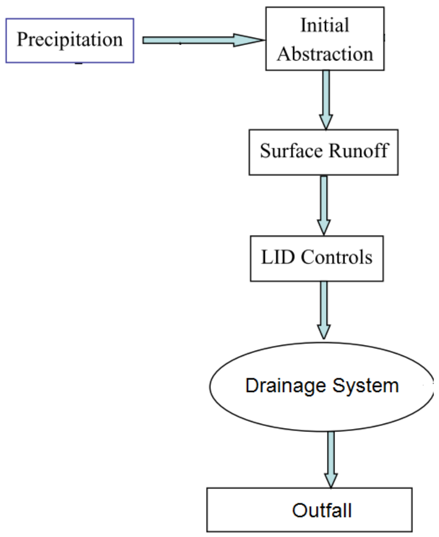


Figure 20. SWMM model rainfall-runoff computation process.

The mathematical representation of the SWMM model in general equations are introduced in reference manual of the SWMM software (L. Rossman 2015) as shown in Equations (1) and (2).

$$X_t = f(X_{t-1}, I_t, P) \tag{1}$$

$$Y_t = g(X_t, P) \tag{2}$$

where :

$X_t =$ a vector of state variables at time t,

$Y_t =$ a vector of output variables at time t,

$I_t =$ a vector of inputs at time t,

$P =$ a vector of constant parameters,

State variables are variables that in each time discrete new values are calculated for them and substituted to the previous values. The initial values for state variables are mainly defined by user or in some cases assume as zero. Some of the state variables in this study are depth of runoff on subcatchment surface, cumulative infiltration volume, depth of water at a node and flow rate in a link. Complete table of state variables are provided in (L. Rossman 2015). The output vector $P$, are the results of the calculations that are reported such as runoff flow rate, infiltration rate and pollution accumulation in runoff or in conveyance system. Input data are information provided by user such as precipitation, air temperature, imposed inflow, and water elevation at specific outfalls. Moreover, constant parameters and coefficients that are used in SWMM can be classified in four groups.

- physical dimensions (land area, invert elevations, pipe diameter),
- field observation (impervious percentage),
- laboratory testing (Soil characteristics), and

- data published in tables (pipe roughness, manning coefficient, depression storage).

Figure 21 shows the simulation process in block diagram adapted from (L. Rossman 2015). The diagram shows that function $f$ gets the input values and state values and with use of constant parameters generats new stat values and output values in first time step. This procesure repeted for each time steps and the outputs and state values get renewed each time.

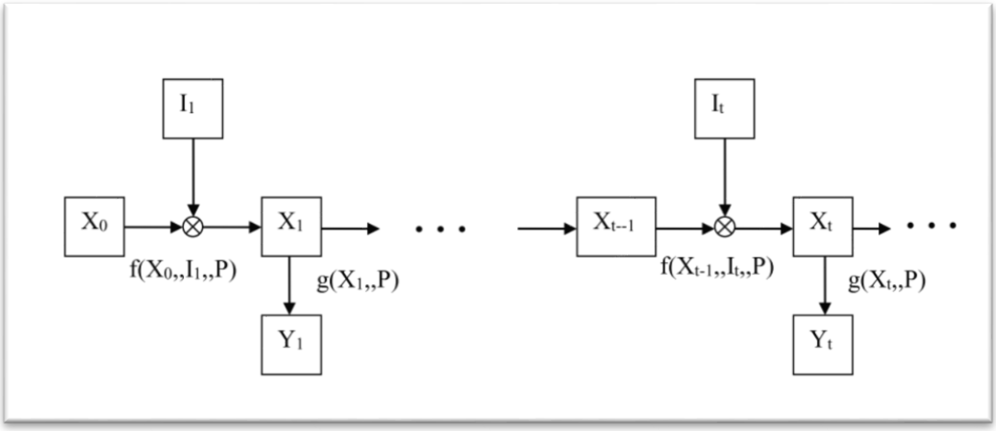



Figure 21. SWMM’s simulation process in block diagram(L. Rossman 2015).

#### *4.4.2.2 Conceptual Elements of SWMM Model*

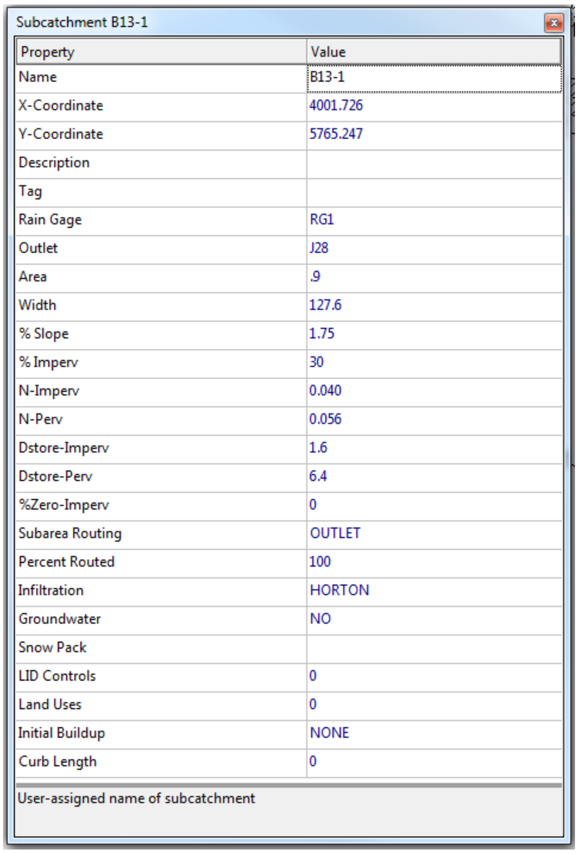
**A. Rain Gages**

Time series are used to show the variation of the specific value during the time. In SWMM different time series can be defined such as temperature data, evaporation data, and precipitation data. Precipitation is the main driving force for rainfall-runoff simulation. The time series may cover a single event with few time steps or a long term event with thousands of time steps that simulate a multiyear model. In SWMM model, the precipitation series are defined in Rain Gage object which is shown by this icon . Different rain gages can be defined in the model so that the spatial variability of the precipitation will be conserved. The time series can be defined as file data or imported manually by the user. Since time steps in SWMM are in the order of minute or less, therefore, daily average precipitations are not useable. In this study, due to lack of measured rainfall in high resolution for Guzelyurt a designed rainfall pattern is used. The rainfall is designed uniformly for one hour with different intensities. Various intensities for rainfall data are defined to check the response of the system. For defining time series, the rainfall intensity is defined for each time interval. The important point is that the time interval introduced in time series should be the same as time interval introduced in the rain gage. Otherwise, each value that is given in time series will assign to the time intervals given in rain gage.

**B. Subcatchments**

Subcatchments are demonstrated in SWMM by hatched parcels. They are consists of pervious and impervious area with different ratio. This ratio is determined by percentage of imperviousness. Moreover, impervious areas themselves are divided into two sub categories which are the one with no depression (zero depression) or the one with depression. All these sub areas are directly connected to the outlet. In SWMM, as shown in Figure 22, in order to define a characteristics of the subcatchment, giving these parameters are necessary:

- Subcatchment area
- Imperviousness
- Imperviousness area w/o depression storage
- Characteristic width of overland flow
- Subcatchment slope
- Manning coefficient for pervious and impervious area
- Depression storage for pervious and impervious area
- Infiltration method



Subcatchment B13-1

| Property | Value |
|---|---|
| Name | B13-1 |
| X-Coordinate | 4001.726 |
| Y-Coordinate | 5765.247 |
| Description | |
| Tag | |
| Rain Gage | RG1 |
| Outlet | J28 |
| Area | .9 |
| Width | 127.6 |
| % Slope | 1.75 |
| % Imperv | 30 |
| N-Imperv | 0.040 |
| N-Perv | 0.056 |
| Dstore-Imperv | 1.6 |
| Dstore-Perv | 6.4 |
| %Zero-Imperv | 0 |
| Subarea Routing | OUTLET |
| Percent Routed | 100 |
| Infiltration | HORTON |
| Groundwater | NO |
| Snow Pack | |
| LID Controls | 0 |
| Land Uses | 0 |
| Initial Buildup | NONE |
| Curb Length | 0 |

User-assigned name of subcatchment

Figure 22. Subcatchment Data window in SWMM.

**Subcatchment Area**

For finding subcatchment's area, three steps are conducted. First, the borders of subcatchments are defined in Google Earth software (GEs) using existing pipe system and flow direction in each street. Second, the KMZ file of subcatchments is exported from GEs and imported to GIS software (Arcmap10) to be converted to AutoCad file to measure the area of each subcatchment in AutoCad environment.

**Imperviousness**

Estimating an impervious area is very important due to high sensitivity of runoff to this parameter. It is recommended to use aerial photo along with topographic maps DiGiano et al. 1977. For finding fraction of imperviousness of each subcatchments, aerial photos of Google Earth is used. By recognizing the roof tops, roads, and parking lots in the google earth, the percentage of impervious surfaces can be obtained. Figure 23 shows the recognition of roof tops in by using GEs aerial photo to have a better estimation of impervious area in subcatchments. In addition to roof tops which is the main components of impervious area, other types of impervious area such as parking lots, roads, and pavements are considered for choosing the impervious percentage.

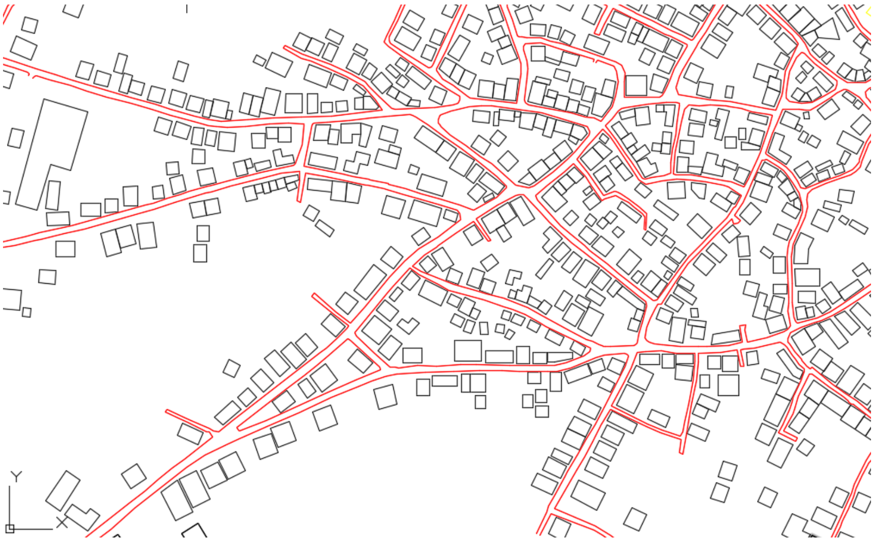


Figure 23. Rooftops recognition using Google Earth images (REAP 2016)

Moreover, two types of impervious areas can be modeled by the user in SWMM program. First impervious areas with depression storage and second, impervious areas without depression storage. More detailed data are available in section "Depression storage for pervious and impervious area" in page 50.

**Characteristic Width of Overland Flow**

Since all the subcatchments are not symmetric rectangular, for finding width of subcatchments different procedures can be applied. Width is an important component for modelling due to a direct influence on hydrograph shape. For example when the subcatchment is narrowed the time of maximum outflow will increase while subcatchment with larger width will quickly reach to its maximum outflow at the outlet point. Different scenario may happen for the shape of the subcatchments. If the subcatchment has a main drainage channel in a center of the rectangular shape then the width will be twice the length of main channel. But if the main channel located on the side of the subcatchment then the width will be equal to length of the channel. DiGiano et al. 1977, generalized this rule for more irregular subcatchments by introducing Equation (3). Where L is the length of the main drainage channel and Z is the ratio of the larger area on the side of the channel ($A_m$) over total area ($A$) of the subcatchment. Z value represent the skewness of the subcatchment from symmetric condition. This equation will limit the width value within two extreme condition of maximum 2L and minimum L DiGiano et al. 1977 and L. Rossman 2015. Also the sensitivity analysis of the 100 percent impervious subcatchment with a drainage system at the center cause 20 percent more runoff than the case with a drainage system located to either side of the subcatchments (DiGiano et al. 1977).

$$W = L + 2L(1 - Z) \tag{3}$$

$$Z = \frac{A_m}{A} \tag{4}$$

Where:

Z= skew factor, $0.5 < Z < 1$

$A_m$= Larger of the two areas on each side of the channel.

$A$ =Total area

$L$ = Length of the main drainage channel

In Figure 24, a subcatchment with a black border is shown. The total area for this subcatchment is 1.7 ha and the thick green line with length of 167 m shows the main drainage system which is not at the center of the subcatchment. The larger area is indicated by yellow line with 1.2 ha area. The Z value calculated for this subcatchment is 0.7 and the width is 267m.

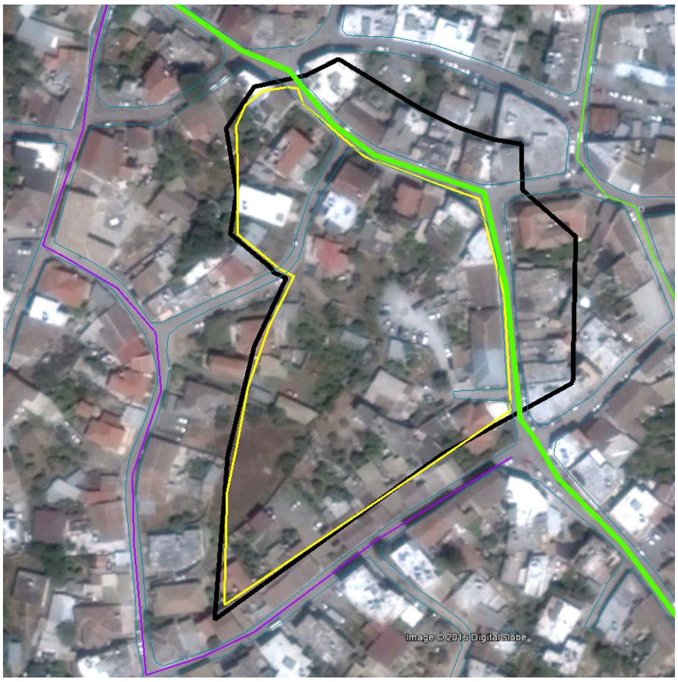



Figure 24. A sample subcatchment with its drainage system

Another good estimation for width is obtaining the average maximum length of overland flow and dividing it by the area. This method is used for the subcatchments that have no main drainage channel inside the subcatchments as shown in Figure 25. In this subcatchment, the maximum flow length is shown in the Figure 25 by black arrow.

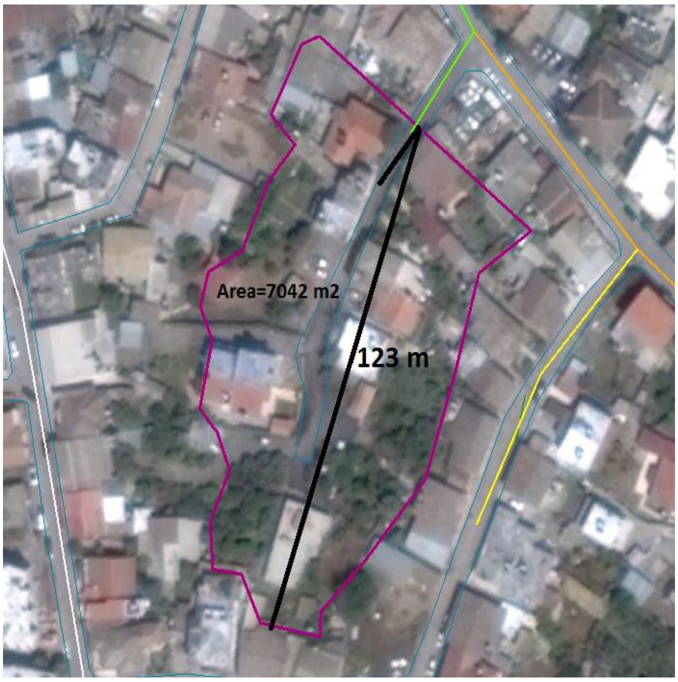



Figure 25. A sample subcatchment w/o any drainage system within the bodrer.

These two methods are mainly used for defining the subcatchment's width for this study.

**Subcatchment Slope**

The subcatchment slope shows the average slops of overland flow through the inlet location. When the shape of the subcatchment is not very complex slope estimation would be more accurate. In developed catchments, slopes should be constructed in a way that the flow convey over ground and uniformly. However in many cases this is not practical and method of area- weighted- average can be applied in case of irregularity. In urban area, overland flow is mainly routing through roads and streets to reach the main drainage system. Therefore, slopes along with streets are calculated in each subcatchment and as recommended in literature (Giannoulis & Haralambopoulos 2011) the area- weighted- average is used for calculating slope of the subcatchment. As shown in Figure 26, three subsections are selected and named as A, B, C, with areas of 2173, 20311, 3519 square meter respectively. Slopes of each subsection from A to C as shown in Figure 26 are -0.42%, -0.54%, -0.48% respectively. Using area- weighted- average method the slope for Subcatchment C17 calculated as 0.52%. It is worth to mention that each subsection should be selected perpendicular to main drainage system which is shown by green line in Figure 26.

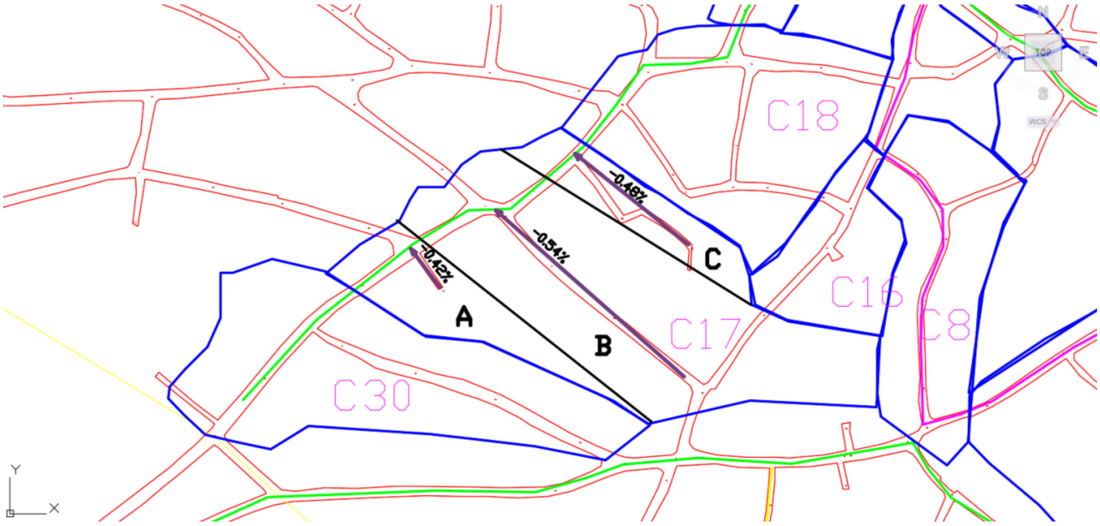



Figure 26. Different slopes calculated for Subcatchment C17 using Auto CAD Civil 3D.

**Manning Coefficient for Pervious and Impervious Area**

In the current study, for finding the volume of runoff from a catchment, manning equation is used. More details are given in section 4.4.2.3. One of the parameters needed for Manning equation is n which accounts for a surface roughness coefficient. Due to high variation in surface characteristics and landscape features, transitions in laminar and turbulent flow, and very small flow depth, n coefficient for overland flow has considerable variation. Different values for n are reported by Crawford and Linsley1966, Engman 1986, Yen 2001, and due to an uncertainty in the value estimation there is no consensus approach between them. In this study, n coefficient for impervious area is assumed to be 0.040 this value is for dense residential land use and for less developed area nominated as suburban residential land use n assumed to be 0.055. Also, pervious area in our case study are mainly consist of bare land, pasture, parks, and lawns and the values that are reported in Yen 2001, for these landscape features are in this range 0.038 - 0.075. According to this range the n coefficient for pervious area for this case study is considered as average value which is equal to 0.056.

**Depression Storage for Pervious and Impervious Area**

Depression storage or retention storage is the volume of rainwater that must be filled on both pervious and impervious area before changing to runoff. This initial abstraction is mainly considered as a loss and usually occur due to surface wetting, surface ponding, interception and evaporation (Viessman & Lewis 2003). The volume of surface depression is related to soil type, land use and slop(ASCE 2006). In pervious area, depression storage closely related to interception on various types of vegetation and stage of growth (ASCE 2006). Different estimation for various type of vegetation are reported in (Linsley et al. 1949)(Maidment & others 1992). For this study depression storage $d_s$ is assumed to be 6.4 mm for pervious and 1.6 mm for impervious area (Tholin & Keifer 1960) which is also very close to the values reported from American Society of Civil Engineering (ASCE 2006).

**Imperviousness Area w/o Depression Storage**

For impervious surfaces depression storage can be considered as zero since it is assumed that immediately after rainfall start, runoff will be generated such as gable roofs. To model impervious surfaces without depression storage in SWWM, there is an option to define the percentage of impervious surfaces without depression storage by the name of *%zero impervious*. Depression storage for impervious surfaces (Dstore-Imperv) will be applied for the rest of the impervious area that has depression storage.

**C. Junction Node**

Junctions in modeling a hydraulic system are representations of manholes or access chambers. They are designed for making convenient access to sewerage system for maintenance and checkup purposes. Also, providing ventilation for sewerage system is another reason for designing manholes. They are mainly designed for the cases where two or more pipes intersect, pipe sizes change, or a change in alignment or grade is needed. Also in long and straight routs, manholes should be designed every 100m to simplified the maintenance (Akan & Houghtalen 2003). For defining junctions in SWMM model, 3 parameters need to be determined. First, the invert elevation of the junction which is the lowest elevation of the manhole structure. For example in this study the depth of the pipe plus the depth of excavation for burying the pipe are subtracted by the ground elevation to calculate the invert elevation. Second parameter is the maximum water depth which is a distance from invert to ground surface. Determining this parameter let the water head increase up to the ground surface and if this value set as zero the water head can be increased to the top of the highest connecting link. The last parameter in this section is the ponded area which is subjected to surface ponding while the water head exceed the maximum water depth. This parameter is important in a case of dynamic wave modelling. In this study, since the model is based on kinematic wave routing the ponded area is set as zero. Figure 27 shows a junction data window in SWMM.

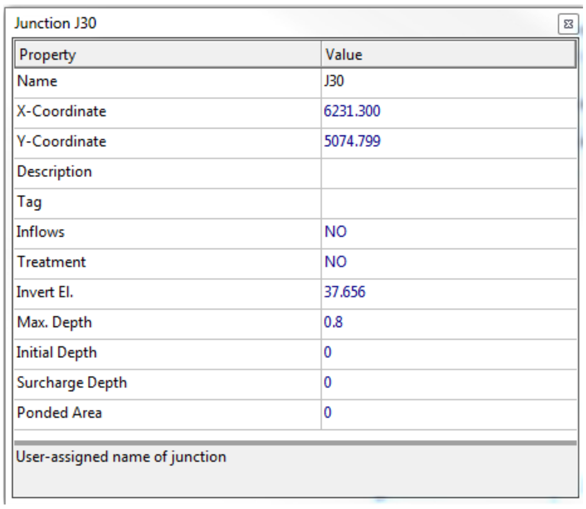
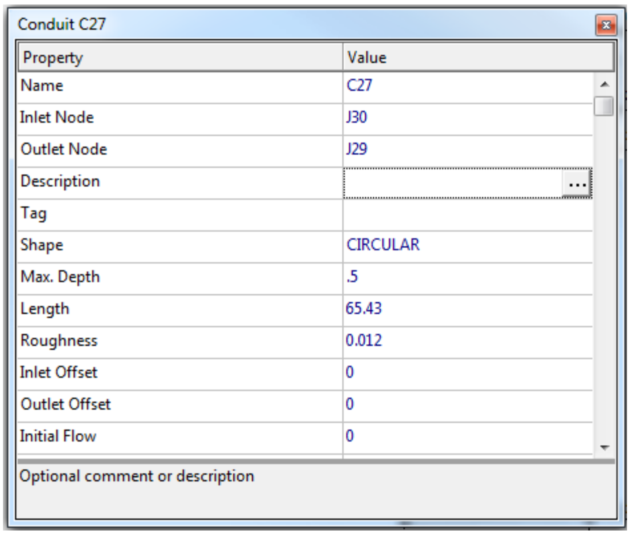


Figure 27.Junction data window in SWMM.

### D. Conduits

For modeling the hydraulic part of the system, SWMM use two main categories which are nodes and links. In the link section different components such as conduits, pumps, orifices, weir, outlets are defined. One of the component for modeling the hydraulic system in this case study is conduits which are schematized by lines that connects two nodes which are supposed to be manholes in real stormwater management system. The principle parameters for defining conduits to a model are inlet node, outlet node, and shape of the conduit, diameter of conduit (Max.depth), length and roughness of the pipe. First of all two nodes should be determined as an inlet node and outlet nod to be connected by conduit. Different shape for conduits can be chosen and depth and bottom width should be determined in case of rectangular shape while for circular shape only diameter of the pipe is enough. Also, the length of the pipe can be obtained from the map. Moreover, according to pipe characteristic, the roughness coefficient of the pipe should be determined. In this case study, two different pipe materials are used for drainage system. The most frequent one is PVC pipe and the other type is concrete channel. The roughness coefficients for PVC pipes are in range of 0.010-0.013 and concrete pipes are in range of 0.012-0.014. Therefore, the roughness coefficient for PVC pipes are assumed as 0.012 and concrete channels as 0.013(ASCE 2006). Furthermore, different pipe sizes such as 300, 500, 400 mm is used in PVC piping system. Also, a rectangular concrete channel with 400 mm height and 600mm bottom width is used for sewerage system.




**E. LID Controls**

Low Impact Developments (LID) are activities that reduce impacts of urbanization by decreasing the resultant runoff from impervious area. LID practices are applying different technics to increase infiltration and evapotranspiration. Different applications of LID controls are Bio-retention Cells, Rain Gardens, Green Roofs, Infiltration Trenches, Permeable pavement, Rooftop Disconnections, Vegetative Swales and Rain Barrels Rossman, 2015. All these WSUD applications can be modelled in SWMM program. In this study Rain barrels are modeled to investigate the effect of applying rainwater harvesting on the runoff. Rain barrels or cisterns are containers which collect roof runoff during the rainfall. This application has two major benefits. First, the rainfall runoff which is collected are considered as source of water that can be used. Second, collecting portion of water during the rainfall can delay the peak time and also reduce the peak runoff.

### *4.4.2.3 Computational Methods*

The basic rules for computational method in SWMM are principles of conservation of mass, energy or momentum. In this study, the processes that are used for runoff computation are surface runoff, infiltration, and application of LID which are explained in following chapters.

**A. Surface Runoff**

In this study, for converting rainfall excess to runoff for each subcatchments, SWMM program is used. In SWMM, subcatchments are conceptualized as a rectangular surface with specific width and a uniform slop which drain water to a single outlet channel as shown in Figure 28.

The concept for finding the runoff is conservation of mass. SWMM considered each subcatchment as a nonlinear reservoir. The inflow to the subcatchments is from precipitation and losses are for evaporation and infiltration as shown in the Figure 28.

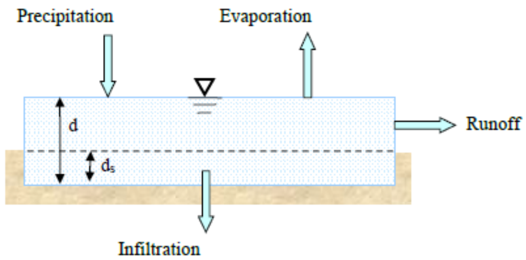



Figure 28. Nonlinear reservoir model of a subcatchment.

The net difference between inflow and losses will be considered as a runoff volume for each subcatchments. So for finding rate of net change in depth of runoff, the difference between inflow and outflow rates for each subcatchment per unit of time is calculated using Equation (5).

$$\frac{\partial d}{\partial t} = i - e - f - q \tag{5}$$

where:

*i = rate of rainfall + snowmelt (m/s)*

*e* = surface evaporation rate (m/s)

*f = infiltration rate (m/s)*

*q* = runoff rate (m/s). Note that the fluxes *i*, *e*, *f*, and *q* are expressed as flow rates per unit area (cms/m$^2$ = m/s).

For finding runoff flow rate per unit of surface area (q), the runoff volume rate need to be calculated and divided by the surface area. To do so, the Manning equation is used assuming a uniform flow routing within a rectangular channel with the height of $d - d_s$, width $W$ (ft) and slop of S.

$$Q = \frac{1.49}{n} S^{1/2} R_x{}^{2/3} A_x \tag{6}$$

where:

*n* is a surface roughness coefficient

*S* the apparent or average slope of the subcatchment (ft/ft),

$A_x$ the area across the subcatchment's width through which the runoff flows (ft$^2$),

$R_x$ is the hydraulic radius associated with this area (ft).

Substituting $A_x = W(d - d_s)$ and $R_x = (d - d_s)$ in Equation (5) and divide it by surface area of the subcatchment, a runoff flow rate per unit of surface $q$ can be obtain.

$$q = 1.49W\, S^{\frac{1}{2}}(d - d_s)^{5/3} \tag{7}$$

Substituting Equation (7) in Equation (5) will give us an ordinary nonlinear differential equation which can be solved by known values for $i, e, f, d_s$ *and* $\alpha$.

$$\frac{\partial d}{\partial t} = i - e - a(d - d_s)^{5/3} \tag{8}$$

where:

$$a = \frac{1.49 W\, S^{1/2}}{A\, n} \tag{9}$$

**B. Infiltration**

Infiltration accounts for the amount of rainwater penetrated to the ground and fills the empty spaces of the soil layers (Akan & Houghtalen 2003). The main loss of rainwater in rainfall/runoff system is related to infiltration. For calculating the amount of infiltrated water to the ground, solving a nonlinear partial differential equation is required which is very complicated and requires relationship between soil permeability, initial soil moisture and pore water tension to be known. Due to this complexity, other methods which are mainly based on empirical observations are developed such as Horton, Modified Horton, Green and Ampt and Curve number. All these infiltration methods are highly dependent on soil type and the initial condition of soil. Therefore, based on infiltration capacity four types of soil groups are classified (A, B, C, and D) by NRCS (Natural Resources Conservation Service, formerly the Soil Conservation Service or SCS). Group A are the one with high infiltration capacity such as sandy soils and group D are the one with poor infiltration capacity like clayey soils.

In SWMM all types of infiltration methods are available for modeling. Horton, Modified Horton, Green and Ampt, Curve number. In current study for finding the infiltration rate, Horton method is chosen since this method is the best known of infiltration equations (L. Rossman 2015). In this method infiltration start from an initial maximum value and exponentially decrease to reach the minimum rate of infiltration. The parameter that shows how fast the exponential curve fall off is decay coefficient. As long as the decay coefficient decreasing the infiltration capacity increasing and the rate of reduction will slow down as shown in Figure 29.

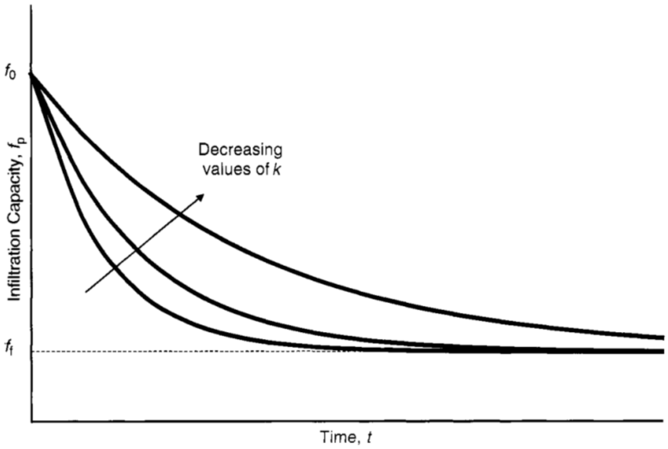



Figure 29. Horton infiltration capacity (Akan & Houghtalen 2003).

The equation for Horton method is an exponential decay function proposed in 1940 by Horton.

$$\boldsymbol{f_p = f_f + (f_0 - f_f)e^{-kt}} \tag{10}$$

where:

$f_p$= Infiltration capacity

$f_f$ = Final Infiltration capacity

$f_0$ = initial Infiltration capacity

$k$ = exponential decay constant and

$t$ = time from beginning of rainfall

In this method three parameter of $f_0$ , $f_f$, $k$ need to be determined according to the soil type. The soil types of the studied region are sandy clay loam soil and based on soil type, values for Horton parameters are extracted from a table presented Akan & Houghtalen, 2003 as:

$f_0 = 5\frac{in}{hr} = 127\frac{mm}{hr}$

$f_f = 0.25\frac{in}{hr} = 6.35mm/hr$

$k = 2\ (1/hr)$

### C. LID Representation

Different LID applications can be modeled in SWMM program. All the LID exercises applied to subcatchments in two different fashions. First, generating a new subcatchments that only represents the specific LID. Second, the LID controls are applied in an existing subcatchment. Therefore, the percentage of non-LID area should be changed according to the portion of LID control that occupied the catchment. In this approach, different LID controls can be applied in one subcatchment and the treated area by each control can be determined separately. Also it is important to mention that by applying LID controls in one subcatchment, the percent of impervious area should be adjusted accordingly. For example, in a subcatchment with the area of A $(m^2)$ and f % of impervious area, an LID control that occupies the area equivalent to $A_{LID}$ is applied. The area that is occupied by LID controls $(A_{LID})$ is calculated by multiplying the area of each control $(a_{lid})$ to the number of controls $(n_{lid})$ that is applied in the subcatchment. Also, the percentage of treated area is considered as $lid_{treated}$%. Therefore, the area that contributes to the runoff and the runoff coefficient are calculated using Equations (11) and (12). Moreover, the resultant inflow by the use of rational method after applying LID is calculated using the values as shown in Equation (13).

$$A_{initial} - A_{LID} = A_{final} \tag{11}$$

$$C_{final} = (f \times lid_{treated}) + \frac{A_{LID}}{A_{final}} \tag{12}$$

$$\text{Total inflow} = (A_{final} \times C_{final} \times i)/(a_{lid} \times n_{lid}) \tag{13}$$

where:

$A_{initial}$ is the total subcatchments area before applying LID control

$A_{LID}$ is the area that the LID control occupies

$C_{final}$ is the final runoff coefficient after applying LID control

$f$ is percent of impervious area

i is rainfall intensity

$a_{lid}$ is area of each LID unit

$n_{lid}$ is number of LID units

Moreover, in this study, the rain barrels do not have underdrain system and they just collect specific amount of water and later on the water will be used in household's activities. The parameters that are considered in rain barrels are shown in Figure 30.

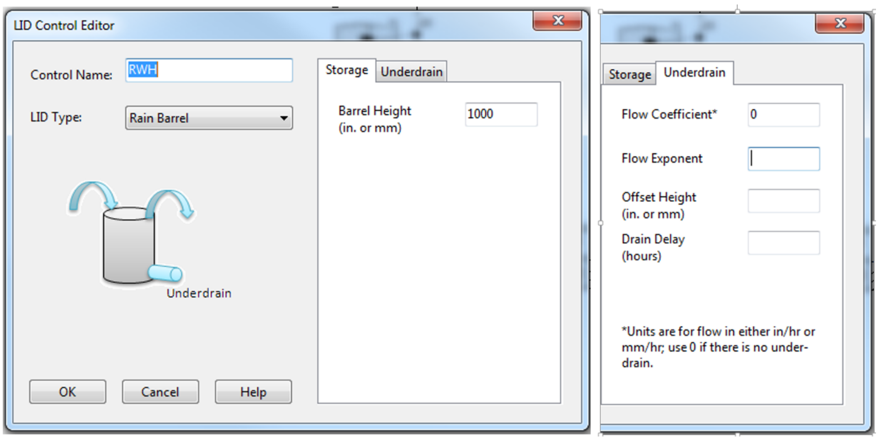



Figure 30. The LID Control Editor window in SWMM program.

As it is shown in Figure 30, the height of each unit of barrels can be determined in Barrel Height section. In under drain tab, the characteristics of under drain can be added to the model. In this study, the existing drywells and septic tanks (detailed information is given in Section 3.5) are considered as a reservoir, therefore no underdrain is considered for the barrels to simulate the wells. Hence, the flow coefficient is set as zero. As the total volume for septic tank and well is important for this study, then the height of the barrel is assumed as 14.7 m to meet the volume of both septic tank and well which is 9.3 cube meter.

Next step for applying LID in the model is defining the LID units for each subareas. To achieve this goal, having the accurate number of buildings and their application is necessary. Therefore, couple of site visits conducted to figure out the exact number of dwellings and the application of each building within Subcatchments A and B. For example all the buildings in Subcatchment B classified to four types. First, administrative offices such as banks, municipality, and communication center. Second, residential section consisted of single houses or two-story houses or apartments. The latter one is very few and uncommon in that region. Third, only commercial such as single shop or restaurants or pharmacies that are apart from residential section. Fourth, the combination of residential and commercial part which is very common in some part of the city. These are some buildings that have shops beneath their residential part. Figure 31 shows different types of buildings in central part of Guzelyurt in Subcatchment B.

Figure 31. Different types of buildings in Subcatchment B in Guzelyurt.

After investigating the number of dwellings in target area, the roof area has investigated to see what percent of impervious area is occupied by the roofs. For finding the total roofs area in each subarea, the AutoCAD file that consists of roof tops in the city has been used. Figure 32 shows the roofs that detected in subareas of B1 and B1-1 using GoogleEarth images. At the end, the total roof area is divided by the impervious area to calculate the treated area by the drywells. It is assumed that all the roofs are connected to the septic tanks and drywells.

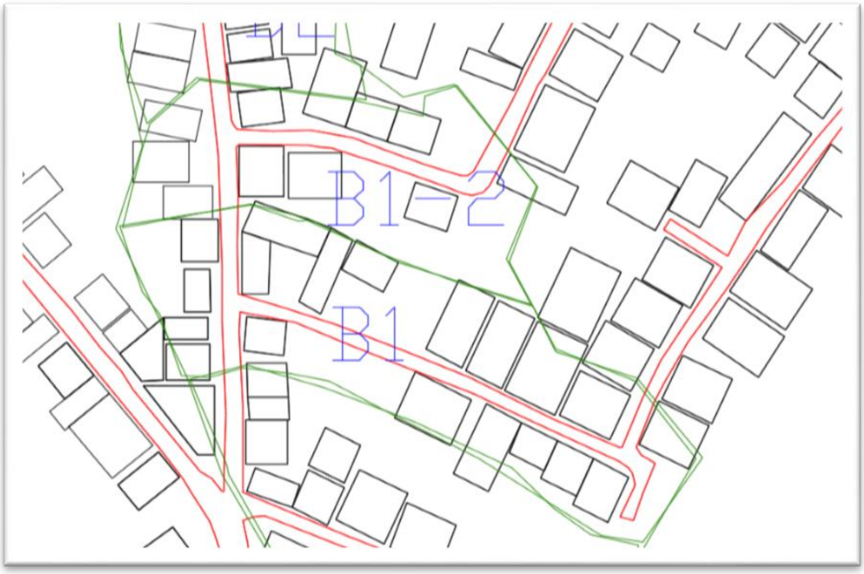



Figure 32. Rooftops area detected from Google Earth.

Both number of dwellings and percent of impervious area are defined in LID Usage Editor as "number of units" and " % of impervious area treated" respectively. Figure 33 shows the parameters that is needed for assigning rain barrels to specific subarea. The area of each unit of LID is considered as 0.63 square meter since the wells are sylindrical with diameter of 0.9 m.

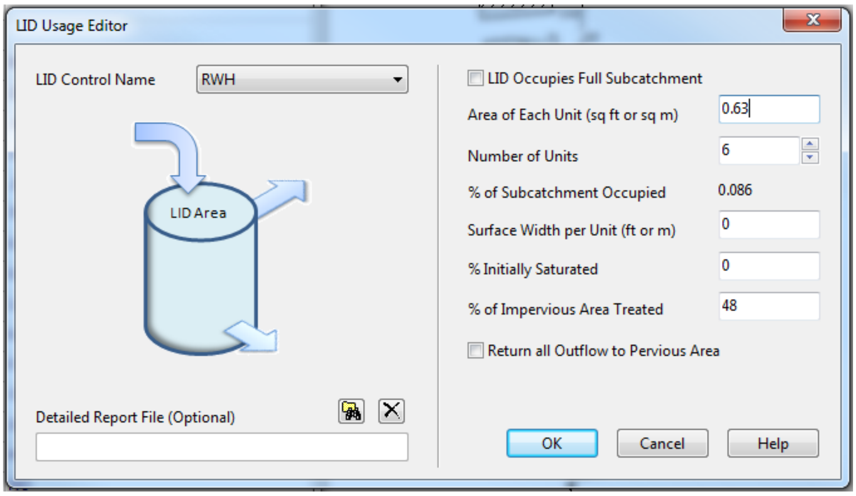



Figure 33. LID Usage Editor Window in SWMM.

Also it worth to mention the term “% of Subcatchment Occupied” which is calculated by the editor, stands for the total LID control area that is applied in subareas divided by the total area. For example if the number of units is 12 and the area for each barrel is 1 square meter then the total area occupied by LID is 12 square meter and if the area for subarea is 750 square meter then the portion would be 1.6%. Also, there is an option to return the outflow from drywells to the pervious area which is not the case in this study because it is assumed that after the drywells get full the outflow will rout to the drainage system.

# CHAPTER 5 : CALCULATIONS, RESULTS AND DISCUSSION

The given methodology is applied for Guzelyurt, to generate necessary data for modeling a rainfall- runoff model. Since the same methodology is applied for all four subcatchments, only the result of Subcatchment B is reported in detail. The results that are reported in this chapter can be classified into two sections. First, the results of generated input data for developing rainfall-runoff model. Second, the results of the generated model for different scenarios.

## 5.1 Inlets GIS Database

515 inlets outspread in whole Guzelyurt has surveyed and the exact location of each has been determined on the map. As it is mentioned in Section 4.2.1, all the coordinates are defined in WGS84/UTM. Moreover, as it is mentioned in Section 4.3.2 a data base consists of geographic data and attribute data is developed in Arc GIS software. In Figure 34, a snapshot of the inlets in GIS environment and a sample of identification table for an inlet are shown. Developing such a complete database for stormwater inlets of the Guzelyurt has not been done since now and it would be very beneficial for making decisions related to stormwater management issues.


Figure 34. A sample information generated for each inlets in Arc GIS software.

## 5.2 Subcatchments division

After determining subareas which contribute to each manhole and merging these subareas, four main subcatchments are defined as shown in Figure 35. The areas for subcatchments A, B, C, and D are 0.54, 0.16, 0.29 and 0.10 square kilometer respectively. The exit point for all the subcatchments are close to Serahis riverbed which is currently dried.

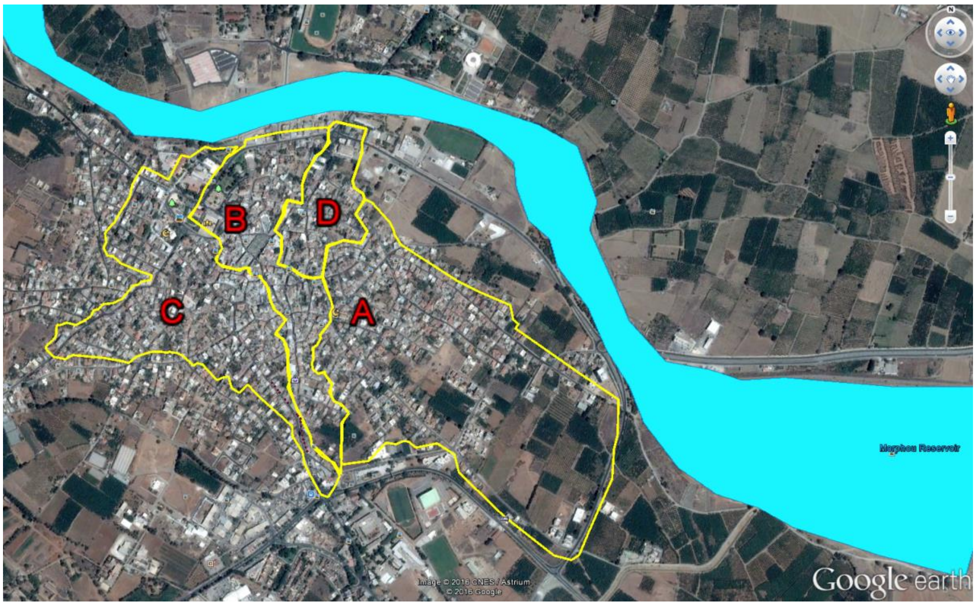


Figure 35. The main four subcatchment in Guzelyurt.

In Table 1, information about total area, number of subareas and percentage of impervious area for each subcatchments is provided. Moreover, Figure 37 shows each subcatchment with its subareas separately.

Table 1. General characteristics of subcatchments in Guzelyurt.

| Subcatchments | Total Area m2 | Subareas | Impervious % |
|---|---|---|---|
| A | 550372 | 35 | 27 |
| B | 149734 | 26 | 64 |
| C | 291374 | 60 | 62 |
| D | 79318 | 9 | 51 |

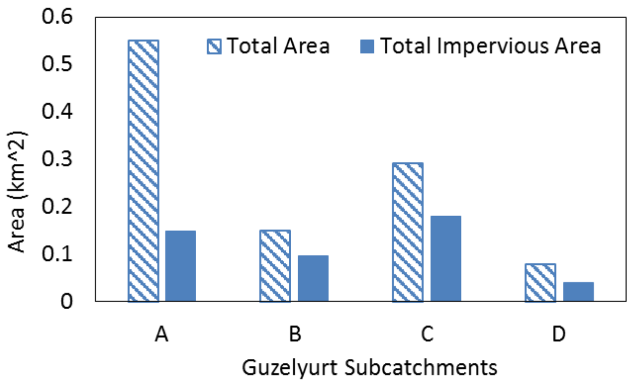



Figure 36. Area related characteristics for subcatchments in Guzelyurt.

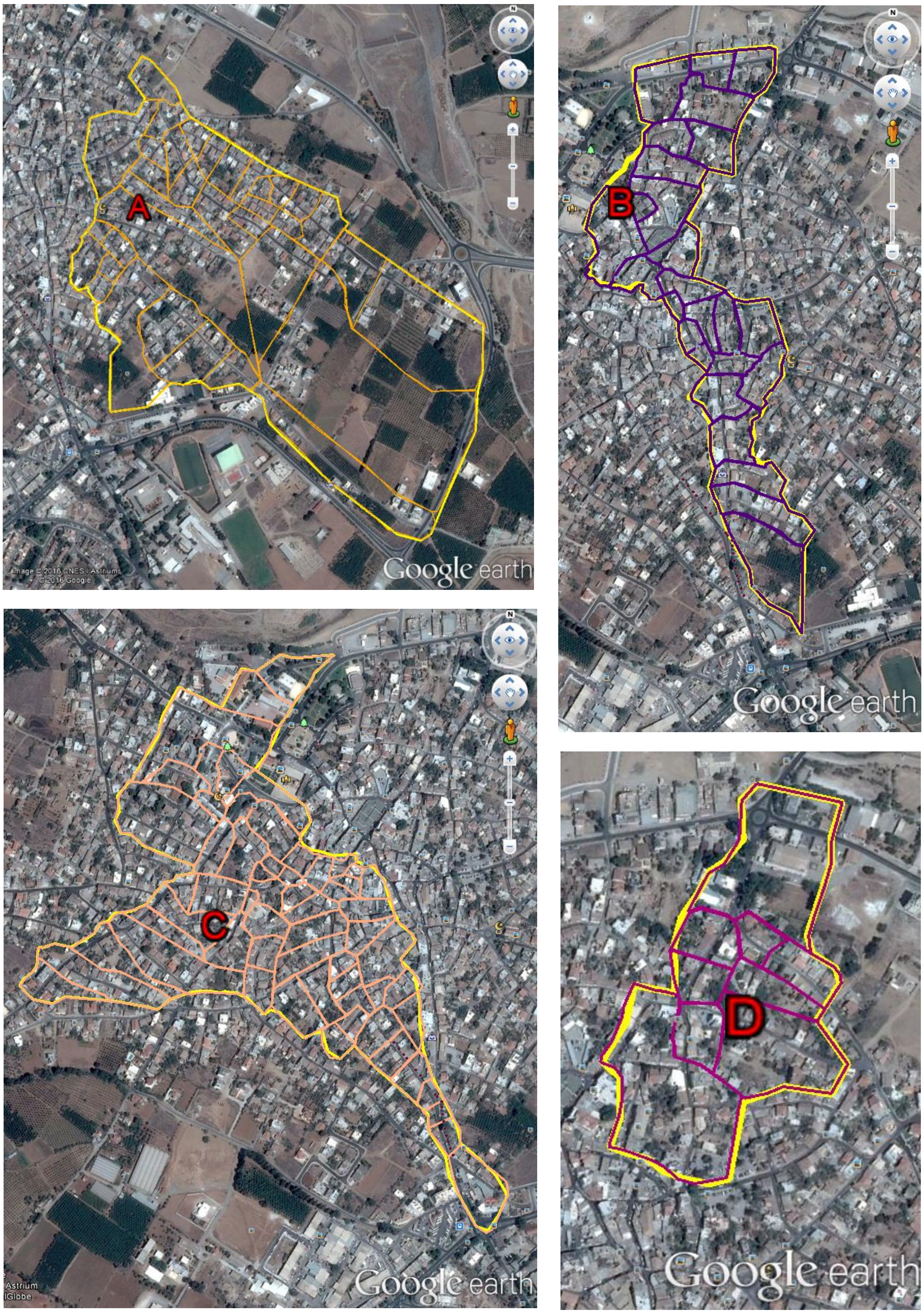



Figure 37.Four main subcatchments in Guzelyurt with their subareas.

## 5.3 Guzelyurt Impervious Area

All the roofs and impervious areas of the studied area are detected from google earth aerial photos and converted to digital map in Auto CAD environment as shown in Figure 38. This map is used to find the impervious portion in each subcatchment to calculate the runoff.

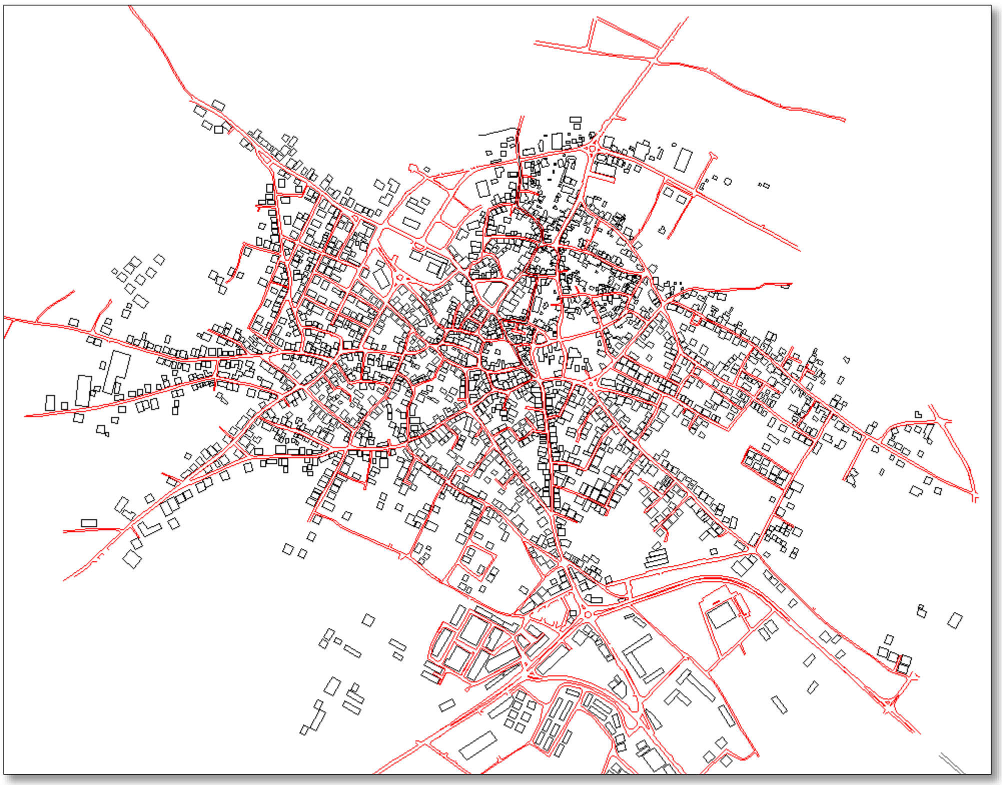


Figure 38. Rooftops area detected from Google Earth images (REAP 2016).

## 5.4 SWMM Modeling for Subcatchments

A separate SWMM model is developed for each subcatchment in Guzelyurt city and the critical rainfall event is investigated for all the subcatchments separately. The In this section detail characteristic data of Subcatchment B which are necessary for developing rainfall-runoff model are reported in detail. Subcatchment B is the most urbanized part of the city with high density population and limited pervious area. For subcatchments A, C, and D the results and characteristics are given in Appendices.

### 5.4.1 Rainfall Scenarios

Due to lack of high resolution measured rainfall data for Guzelyurt, in this study different hourly designed rainfall is used to investigate that which rainfall scenario causes no problem and by which, the model will face flood situation. The time step for rainfall design is considered as 5 min.

Figure 39, shows a 10 mm/hr designed rainfall in SWMM as a sample. In Figure 40, a hyetograph of 10 mm/hr designed rainfall is shown. Vertical axes in hydrograph stands for rainfall intensity in mm/hr and horizontal axes indicate time.

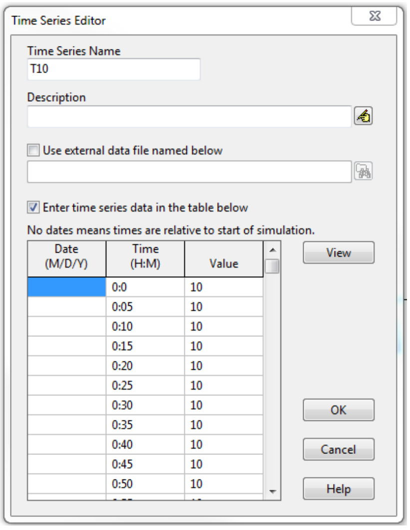


Figure 39. Rainfall time series editor in SWMM.

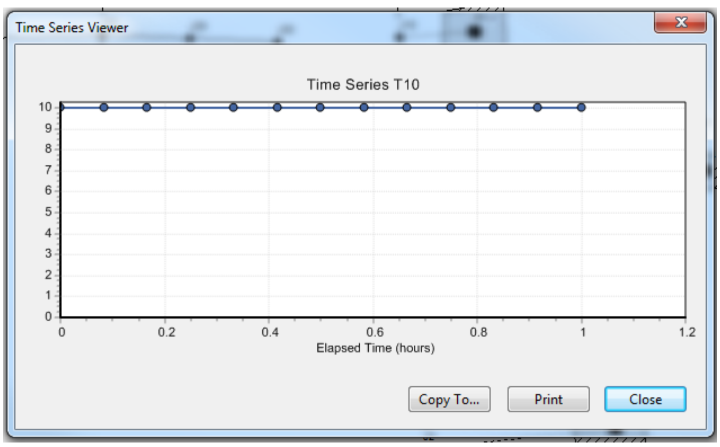


Figure 40. Design rainfall hyetograph viewer in SWMM.

### 5.4.2 Subareas in Subcatchment B

Subcatchment B is divided to 26 subareas based on existing pipelines and flow directions. All the specifications of each subarea that is needed for modeling such as area, impervious percentage, characteristic width and slop for each subarea have been developed and reported in this section. Figure 41 depicts a conceptual model accounts for Subcatchment B.

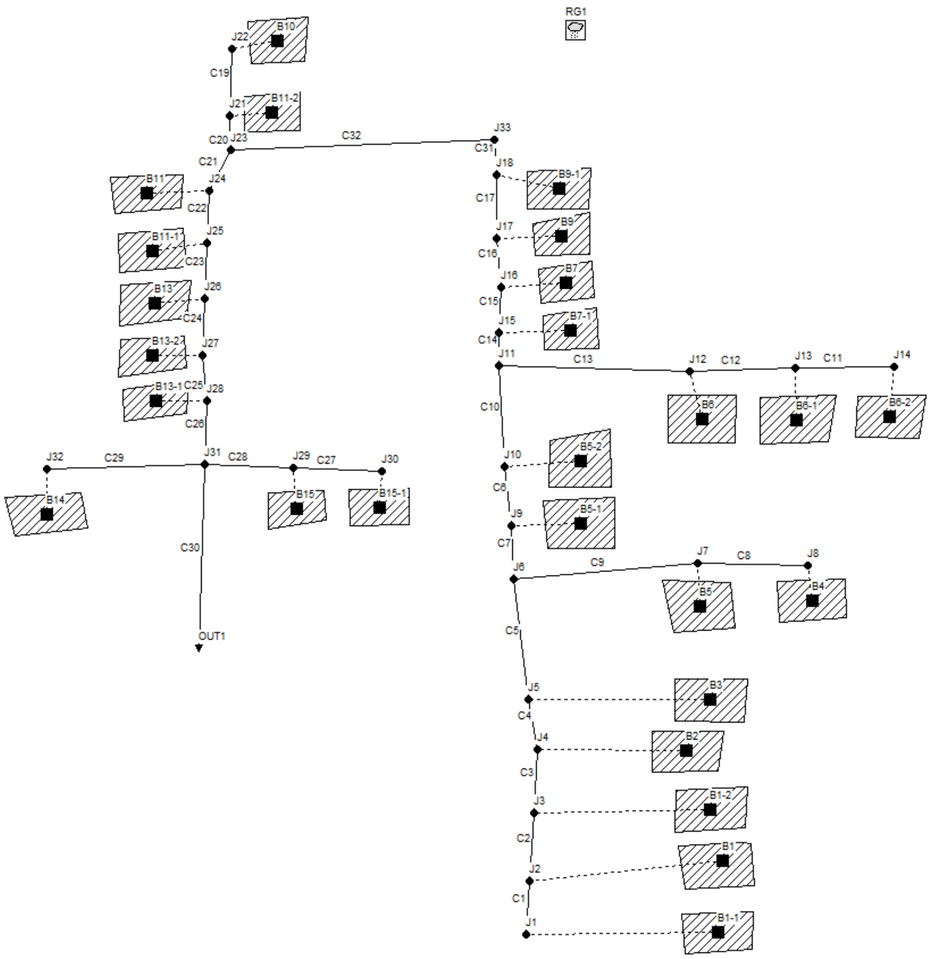


Figure 41. Conceptual model for catchment B in SWMM.

All these parcels stand for a subarea in Subcatchment B which are defined by to the characteristics that is mentioned for each of the subareas in Table 2. The information that is provided in Table 2, is applied for modeling a subarea in SWMM software.

Table 2. Characteristic information for subareas in Subcatchment B.

| Subareas | Outlet | Area (ha) | %Imperv | Width (m) | %Slope |
|---|---|---|---|---|---|
| B6 | J12 | 0.3 | 90 | 95.6 | 1.04 |
| B1-1 | J1 | 1.5 | 15 | 138.48 | 1.1 |
| B1 | J2 | 0.93 | 80 | 60 | 0.98 |
| B1-2 | J3 | 0.59 | 40 | 48.2 | 1.11 |
| B2 | J4 | 0.65 | 60 | 141.2 | 2.5 |
| B3 | J5 | 0.66 | 80 | 150.9 | 1.79 |
| B4 | J8 | 0.56 | 80 | 78.52 | 1 |
| B5 | J7 | 0.3 | 90 | 144.2 | 2 |
| B5-1 | J9 | 0.13 | 90 | 49.5 | 1.32 |
| B5-2 | J10 | 0.21 | 90 | 102.2 | 1.11 |
| B6-1 | J13 | 0.44 | 80 | 64.6 | 1 |
| B6-2 | J14 | 0.67 | 70 | 66.65 | 1.24 |
| B7-1 | J15 | 0.1 | 100 | 107.2 | 0.66 |
| B7 | J16 | 0.43 | 95 | 45.9 | 1.2 |
| B9 | J17 | 0.41 | 90 | 44.77 | 1 |
| B9-1 | J18 | 0.5 | 85 | 119.3 | 0.36 |
| B10 | J22 | 1.1 | 80 | 121.77 | 1.75 |
| B11-2 | J21 | 0.19 | 90 | 68.2 | 0.3 |
| B11 | J24 | 0.7 | 80 | 57.61 | 0.8 |
| B11-1 | J25 | 0.62 | 80 | 79.1 | 1.58 |
| B13 | J26 | 0.45 | 70 | 137.6 | 1.52 |
| B13-2 | J27 | 0.65 | 60 | 70.36 | 3.2 |
| B13-1 | J28 | 0.9 | 30 | 127.6 | 1.75 |
| B14 | J32 | 0.91 | 30 | 161.5 | 0.05 |
| B15 | J29 | 0.44 | 60 | 144 | 0.16 |
| B15-1 | J30 | 0.61 | 60 | 138.7 | 0.71 |

Similar information is provided for subcatchments A, C, D and reported in Appendices.

### 5.4.3 Junctions in Subcatchment B

As shown in Figure 41, all the black nods indicate junctions in the piping system. The invert elevation and maximum water depth for each junction is shown in Table 3.

Table 3. Data provided for modeling junctions in SWMM for Subcatchment B.

| Junctions | Invert Elev | Max. Depth m | Junctions | Invert Elev | Max. Depth m |
|---|---|---|---|---|---|
| J1 | 48.532 | 0.8 | J17 | 43.29 | 1.1 |
| J2 | 48.085 | 0.8 | J18 | 42.8 | 1.1 |
| J3 | 47.719 | 0.8 | J21 | 42.213 | 0.8 |
| J4 | 46.151 | 0.8 | J22 | 42.323 | 0.8 |
| J5 | 45.669 | 0.8 | J23 | 41.91 | 1.1 |
| J6 | 45.594 | 0.8 | J24 | 41.37 | 1.3 |
| J7 | 45.632 | 0.8 | J25 | 40.63 | 1.15 |
| J8 | 45.873 | 0.8 | J26 | 39.13 | 1.2 |
| J9 | 44.948 | 0.8 | J27 | 38.75 | 1 |
| J10 | 44.326 | 0.8 | J28 | 36.98 | 1.3 |
| J11 | 44.06 | 1 | J29 | 37.487 | 0.8 |
| J12 | 44.504 | 0.6 | J30 | 37.656 | 0.8 |
| J13 | 45.002 | 0.6 | J31 | 36.84 | 1.4 |
| J14 | 45.421 | 0.6 | J32 | 37.478 | 0.8 |
| J15 | 43.66 | 1.2 | J33 | 42.74 | 1.1 |
| J16 | 43.41 | 1.2 | | | |

### 5.4.4 Conduits in Subcatchment B

As shown in Figure 41 conduits are depicted as lines that connect two nods which assume as inlet nodes and outlet nodes. In Table 4, all the information necessary for conduits such as inlet node, outlet nod, length of the conduit, pipe roughness, shape of the pipes and dimeter of each pipe is given. Conduit 32 is a concrete rectangular channel with the bottom depth of 0.6 m and maximum depth of 0.4 m.

Table 4. Data provided for modeling conduits in SWMM for Subcatchment B.

| Conduits | From Node | To Node | Length (m) | Pipe size (mm) | Shape | Roughness |
|---|---|---|---|---|---|---|
| C1 | J1 | J2 | 53.25 | 500 | Circular | 0.012 |
| C2 | J2 | J3 | 41.44 | 500 | Circular | 0.012 |
| C3 | J3 | J4 | 79.86 | 500 | Circular | 0.012 |
| C4 | J4 | J5 | 79.74 | 500 | Circular | 0.012 |
| C5 | J5 | J6 | 21.52 | 500 | Circular | 0.012 |
| C6 | J9 | J10 | 57.52 | 500 | Circular | 0.012 |
| C7 | J6 | J9 | 45.84 | 500 | Circular | 0.012 |
| C8 | J8 | J7 | 67.38 | 500 | Circular | 0.012 |
| C9 | J7 | J6 | 5.49 | 500 | Circular | 0.012 |
| C10 | J10 | J11 | 49 | 500 | Circular | 0.012 |
| C11 | J14 | J13 | 42.29 | 300 | Circular | 0.012 |
| C12 | J13 | J12 | 53.72 | 300 | Circular | 0.012 |
| C13 | J12 | J11 | 3.8 | 300 | Circular | 0.012 |
| C14 | J11 | J15 | 48.85 | 500 | Circular | 0.012 |
| C15 | J15 | J16 | 39.7 | 500 | Circular | 0.012 |
| C16 | J16 | J17 | 15.99 | 500 | Circular | 0.012 |
| C17 | J17 | J18 | 76.41 | 500 | Circular | 0.012 |
| C19 | J22 | J21 | 64.38 | 500 | Circular | 0.012 |
| C20 | J21 | J23 | 4.74 | 500 | Circular | 0.012 |
| C21 | J23 | J24 | 30.48 | 500 | Circular | 0.012 |
| C22 | J24 | J25 | 58.64 | 500 | Circular | 0.012 |
| C23 | J25 | J26 | 87.3 | 500 | Circular | 0.012 |
| C24 | J26 | J27 | 19.72 | 500 | Circular | 0.012 |
| C25 | J27 | J28 | 85.04 | 500 | Circular | 0.012 |
| C26 | J28 | J31 | 3.76 | 500 | Circular | 0.012 |
| C27 | J30 | J29 | 65.43 | 500 | Circular | 0.012 |
| C28 | J29 | J31 | 10.88 | 500 | Circular | 0.012 |
| C29 | J32 | J31 | 7.06 | 500 | Circular | 0.012 |
| C30 | J31 | OUT1 | 60 | 500 | Circular | 0.012 |
| C31 | J18 | J33 | 4 | 500 | Circular | 0.012 |
| C32 | J33 | J23 | 107.93 | 400×600 | Rectangular | 0.013 |

All the information that is provided in Section 5.6 are the input values for different components that is necessary for simulating rainfall-runoff model for Subcatchment B.

## 5.1 Sensitivity Analysis

### 5.1.1 Imperviousness of subcatchments

Different parameters affect directly on total runoff from the subcatchment. To investigate the sensitivity of each parameter Subcatchment B is examined to identify how significant each of these parameters can affect the total runoff. One of these parameters is the percentage of imperviousness in a subcatchment. The sensitivity of this parameter on total runoff is investigated by changing the percentage of imperviousness of each subarea in Subcatchment B in the range of +10% and -10%. Since the percentage of imperviousness in some subareas are about 90% the range for testing the effect of imperviousness is limited to 10%. In Table 5, the results of total runoff in mm, total runoff volume in $10^6$ litter and peak runoff in CMS are shown for each subareas in Subcatchment B for both 10% reduction and 10% increase in percentage of impervious area. It can be seen that 10% variation in impervious percent does not affect the peak runoff effectively.

Table 5. Sensitivity analysis for impervious percentage.

| Subcatchment B | Measured Case | | | | 10% reduction in impervious area | | | | 10% increase in impervious area | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Subareas | Measured imperviou % | Total Runoff mm | Total Runoff 10^6 ltr | Peak Runoff CMS | 10% less imperviou % | Total Runoff mm | Total Runoff 10^6 ltr | Peak Runoff CMS | 10% less imperviou % | Total Runoff mm | Total Runoff 10^6 ltr | Peak Runoff CMS |
| B6 | 90 | 8.38 | 0.03 | 0.01 | 80 | 7.7 | 0.03 | 0.01 | 100 | 9.04 | 0.03 | 0.01 |
| B1-1 | 10 | 2.22 | 0.04 | 0.01 | 0 | 1.39 | 0.02 | 0.01 | 20 | 3.04 | 0.05 | 0.02 |
| B1 | 80 | 7.2 | 0.07 | 0.02 | 70 | 6.52 | 0.06 | 0.02 | 90 | 7.85 | 0.07 | 0.02 |
| B1-2 | 40 | 4.4 | 0.03 | 0.01 | 30 | 3.66 | 0.02 | 0.01 | 50 | 5.13 | 0.03 | 0.01 |
| B2 | 80 | 7.46 | 0.05 | 0.02 | 70 | 6.8 | 0.04 | 0.02 | 90 | 8.11 | 0.05 | 0.02 |
| B3 | 90 | 8.1 | 0.05 | 0.02 | 80 | 7.45 | 0.05 | 0.02 | 100 | 8.73 | 0.06 | 0.02 |
| B4 | 90 | 8.03 | 0.04 | 0.01 | 80 | 7.37 | 0.04 | 0.01 | 100 | 8.65 | 0.05 | 0.01 |
| B5 | 90 | 8.29 | 0.02 | 0.01 | 80 | 7.64 | 0.02 | 0.01 | 100 | 8.93 | 0.03 | 0.01 |
| B5-1 | 90 | 8.12 | 0.01 | 0.01 | 80 | 7.48 | 0.01 | 0.01 | 100 | 8.75 | 0.01 | 0.01 |
| B5-2 | 90 | 8.28 | 0.02 | 0.01 | 80 | 7.62 | 0.02 | 0.01 | 100 | 8.92 | 0.02 | 0.01 |
| B6-1 | 80 | 7.5 | 0.03 | 0.01 | 70 | 6.81 | 0.03 | 0.01 | 90 | 8.18 | 0.04 | 0.01 |
| B6-2 | 80 | 7.2 | 0.05 | 0.02 | 70 | 6.54 | 0.04 | 0.02 | 90 | 7.84 | 0.05 | 0.02 |
| B7-1 | 100 | 8.77 | 0.01 | 0.01 | 90 | 8.15 | 0.01 | 0.01 | 110 | 8.77 | 0.01 | 0.01 |
| B7 | 95 | 8.16 | 0.04 | 0.01 | 85 | 7.54 | 0.03 | 0.01 | 105 | 8.46 | 0.04 | 0.01 |
| B9 | 95 | 8.42 | 0.04 | 0.01 | 85 | 7.77 | 0.04 | 0.01 | 105 | 8.73 | 0.04 | 0.01 |
| B9-1 | 85 | 7.69 | 0.04 | 0.01 | 75 | 7.02 | 0.04 | 0.01 | 95 | 8.33 | 0.05 | 0.01 |
| B10 | 80 | 7.23 | 0.09 | 0.03 | 70 | 6.57 | 0.08 | 0.03 | 90 | 7.87 | 0.1 | 0.03 |
| B11-2 | 90 | 8.64 | 0.02 | 0.01 | 80 | 7.92 | 0.02 | 0.01 | 100 | 9.33 | 0.02 | 0.01 |
| B11 | 80 | 7.26 | 0.07 | 0.02 | 70 | 6.56 | 0.07 | 0.02 | 90 | 7.91 | 0.08 | 0.02 |
| B11-1 | 80 | 7.39 | 0.05 | 0.02 | 70 | 6.71 | 0.04 | 0.02 | 90 | 8.04 | 0.05 | 0.02 |
| B13 | 70 | 6.92 | 0.03 | 0.01 | 60 | 6.24 | 0.03 | 0.01 | 80 | 7.6 | 0.03 | 0.01 |
| B13-2 | 60 | 6.15 | 0.04 | 0.02 | 50 | 5.43 | 0.04 | 0.01 | 70 | 6.85 | 0.04 | 0.02 |
| B13-1 | 30 | 3.87 | 0.03 | 0.02 | 20 | 3.15 | 0.03 | 0.01 | 40 | 3.87 | 0.03 | 0.02 |
| B14 | 30 | 3.36 | 0.03 | 0.01 | 20 | 2.59 | 0.02 | 0.01 | 40 | 3.36 | 0.03 | 0.01 |
| B15 | 70 | 6.79 | 0.03 | 0.01 | 60 | 6.08 | 0.03 | 0.01 | 80 | 6.79 | 0.03 | 0.01 |
| B15-1 | 60 | 5.95 | 0.04 | 0.01 | 50 | 5.27 | 0.03 | 0.01 | 70 | 5.95 | 0.04 | 0.01 |

As shown in Figure 42, the dash line represents the total runoff generated from each subcatchment with 10% higher impervious area and dot line indicates the case of 10% below the real portion of impervious area. The rigid line represents the real condition of subareas with regards to impervious percentage.

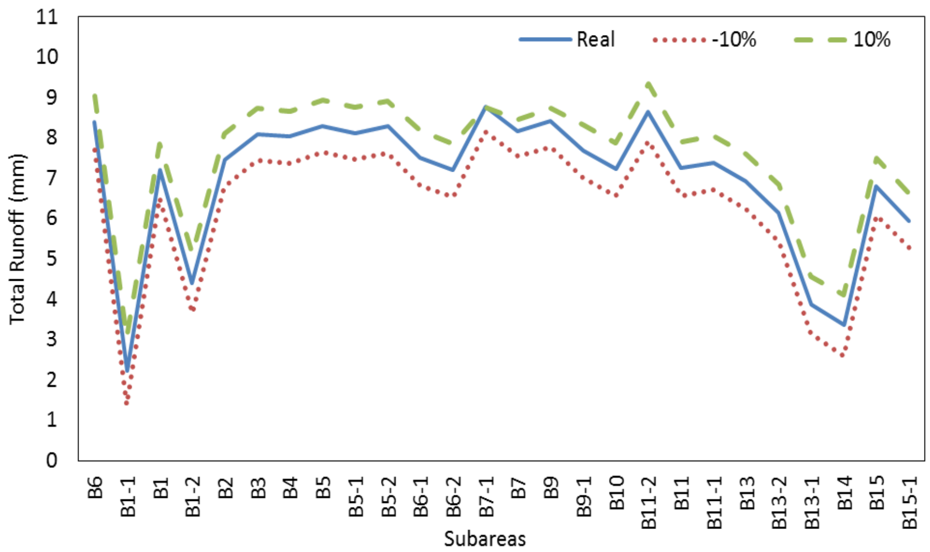



Figure 42. Sensitivity analysis of impervious area for Subcatchment B.

Comparing total runoff (mm) for each scenarios, indicates that by increasing impervious area by 10% the total runoff will increase by 9% and by reducing the impervious percentage by 10% the total runoff will decrease by 10%. The reason that the alteration for total runoff in both scenarios is not the same is due to percentage of impervious area of B7-1 which is 100% and cannot be increased more. Moreover, in first scenario, the 10% increase in impervious area cause no problem in any of the junctions and conduits which is mainly due to insignificant effect of impervious percentage on peak runoff. Also the average area for subareas in Subcatchment B is 0.6 ha and 10% of this area is about 600 square meter which is a big area and the precision of measuring impervious area in each subarea is about few meters.

### 5.1.2 Manning Coefficient of Subcatchments

In this section, manning coefficient of pervious and impervious area is investigated. The maximum and minimum values that are reported in different resources are 0.45 and 0.01(L. Rossman 2015). The maximum roughness is related to landscapes with dense shrubbery and forest litter and Bluegrass sod which is not the case in our studied area. As mentioned in Section 4.4.2.2, in this study, manning coefficient for impervious area with dense residential is used as 0.040 and 0.055 for semi urban area. The sensitivity of total runoff is investigated for both of these values and N=0.030. The results are shown both in Table 6 and Figure 43.

Table 6. Sensitivity analysis for manning coefficient.

| N=0.040 | | | | N=0.055 | | | | N=0.030 | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Total Infil mm | Total Runoff mm | Total Runoff 10^6 ltr | Peak Runoff CMS | Total Infil mm | Total Runoff mm | Total Runoff 10^6 ltr | Peak Runoff CMS | Total Infil mm | Total Runoff mm | Total Runoff 10^6 ltr | Peak Runoff CMS |
| 0.37 | 8.38 | 0.03 | 0.01 | 0.37 | 8.36 | 0.03 | 0.01 | 0.37 | 8.4 | 0.03 | 0.01 |
| 3.3 | 2.22 | 0.04 | 0.01 | 3.3 | 2.22 | 0.04 | 0.01 | 3.3 | 2.22 | 0.04 | 0.01 |
| 0.73 | 7.2 | 0.07 | 0.02 | 0.73 | 7.11 | 0.07 | 0.02 | 0.73 | 7.26 | 0.07 | 0.02 |
| 2.2 | 4.4 | 0.03 | 0.01 | 2.2 | 4.39 | 0.03 | 0.01 | 2.2 | 4.41 | 0.03 | 0.01 |
| 0.73 | 7.46 | 0.05 | 0.02 | 0.73 | 7.45 | 0.05 | 0.02 | 0.73 | 7.47 | 0.05 | 0.02 |
| 0.37 | 8.1 | 0.05 | 0.02 | 0.37 | 8.08 | 0.05 | 0.02 | 0.37 | 8.12 | 0.05 | 0.02 |
| 0.37 | 8.03 | 0.04 | 0.01 | 0.37 | 7.98 | 0.04 | 0.01 | 0.37 | 8.06 | 0.05 | 0.02 |
| 0.37 | 8.29 | 0.02 | 0.01 | 0.37 | 8.28 | 0.02 | 0.01 | 0.37 | 8.3 | 0.02 | 0.01 |
| 0.37 | 8.12 | 0.01 | 0.01 | 0.37 | 8.11 | 0.01 | 0.01 | 0.37 | 8.14 | 0.01 | 0.01 |
| 0.37 | 8.28 | 0.02 | 0.01 | 0.37 | 8.26 | 0.02 | 0.01 | 0.37 | 8.29 | 0.02 | 0.01 |
| 0.73 | 7.5 | 0.03 | 0.01 | 0.73 | 7.47 | 0.03 | 0.01 | 0.73 | 7.53 | 0.03 | 0.01 |
| 0.73 | 7.2 | 0.05 | 0.02 | 0.73 | 7.16 | 0.05 | 0.02 | 0.73 | 7.23 | 0.05 | 0.02 |
| 0.1 | 8.77 | 0.01 | 0.01 | 0.1 | 8.75 | 0.01 | 0.01 | 0.1 | 8.78 | 0.01 | 0.01 |
| 0.18 | 8.16 | 0.04 | 0.01 | 0.18 | 8.1 | 0.03 | 0.01 | 0.18 | 8.2 | 0.04 | 0.01 |
| 0.18 | 8.42 | 0.04 | 0.01 | 0.18 | 8.34 | 0.04 | 0.01 | 0.18 | 8.48 | 0.04 | 0.01 |
| 0.55 | 7.69 | 0.04 | 0.01 | 0.55 | 7.64 | 0.04 | 0.01 | 0.55 | 7.72 | 0.04 | 0.01 |
| 0.73 | 7.23 | 0.09 | 0.03 | 0.73 | 7.19 | 0.09 | 0.03 | 0.73 | 7.26 | 0.09 | 0.03 |
| 0.37 | 8.64 | 0.02 | 0.01 | 0.37 | 8.61 | 0.02 | 0.01 | 0.37 | 8.66 | 0.02 | 0.01 |
| 0.73 | 7.26 | 0.07 | 0.02 | 0.73 | 7.14 | 0.07 | 0.02 | 0.73 | 7.33 | 0.07 | 0.02 |
| 0.73 | 7.39 | 0.05 | 0.02 | 0.73 | 7.36 | 0.05 | 0.02 | 0.73 | 7.41 | 0.05 | 0.02 |
| 1.1 | 6.92 | 0.03 | 0.01 | 1.1 | 6.91 | 0.03 | 0.01 | 1.1 | 6.93 | 0.03 | 0.01 |
| 1.47 | 6.15 | 0.04 | 0.02 | 1.47 | 6.13 | 0.04 | 0.02 | 1.47 | 6.16 | 0.04 | 0.02 |
| 2.57 | 3.87 | 0.03 | 0.02 | 2.57 | 3.86 | 0.03 | 0.02 | 2.57 | 3.87 | 0.03 | 0.02 |
| 2.57 | 3.36 | 0.03 | 0.01 | 2.57 | 3.34 | 0.03 | 0.01 | 2.57 | 3.37 | 0.03 | 0.01 |
| 1.1 | 6.79 | 0.03 | 0.01 | 1.1 | 6.76 | 0.03 | 0.01 | 1.1 | 6.81 | 0.03 | 0.01 |
| 1.47 | 5.95 | 0.04 | 0.01 | 1.47 | 5.94 | 0.04 | 0.01 | 1.47 | 5.96 | 0.04 | 0.01 |

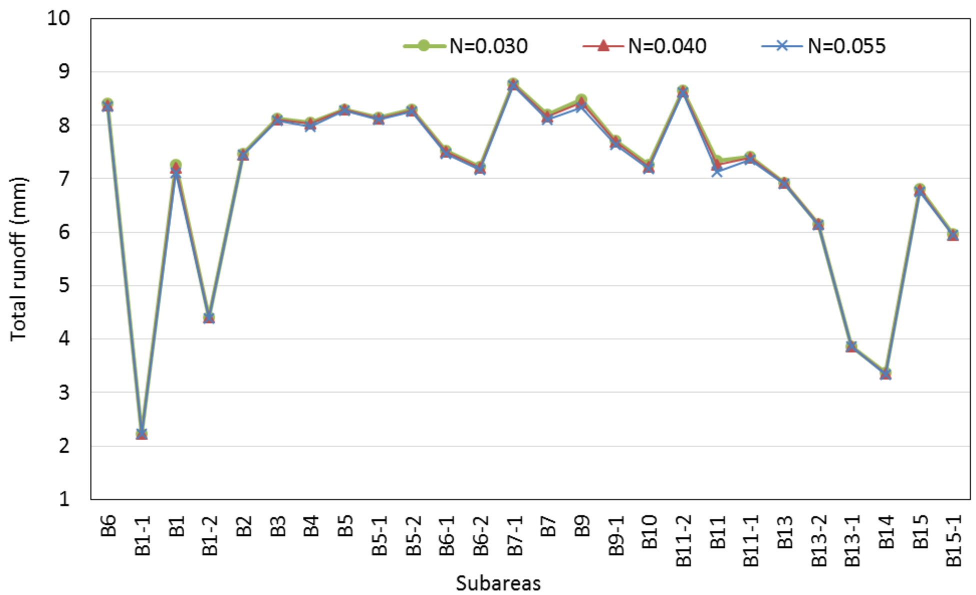



Figure 43. Sensitivity analysis for manning coefficient in Subcatchment B.

As shown in Figure 43, the difference in total runoff for different manning values (N= 0.030, N= 0.040, N= 0.055 ) is not changing considerably which can be concluded that this parameter is not very sensitive and using different values in a given range does not make a significant change in total runoff and peak runoff.

## 5.2 Result of Different Rainfall Scenarios for Existing System in Subcatchment B

Different rainfall intensities are investigated for clarifying the maximum rainfall intensity which causes no flood situation in junctions and conduits. The results of different rainfall intensities are tabulated in Table 7.

Table 7. Result of different rainfall Scenarios for existing system in Subcatchment B.

| Intensity mm/h | Total Precipitation mm | Flooding Junctions | Surcharged Conduits | Outfall Max Flow CMS |
|---|---|---|---|---|
| 20 | 20 | ------- | ------- | 0.79 |
| 21 | 21 | J24, J33 | C22, C32 | 0.82 |
| 23 | 23 | J24, J33, J15, J16 | C22, C32, C15,C16 | 0.84 |
| 25 | 25 | J24, J33, J15, J16, J17, J11, J10 | C22, C32, C15,C16, C17, C10, C14 | 0.86 |
| 30 | 30 | J24, J33, J15, J16, J17, J11, J10, J31, J5 | C22, C32, C15,C16, C17, C10, C14, C30, C5 | 0.91 |

The results indicate that in Subcatchment B the maximum rainfall intensity that cause no flood situation is the rainfall with the intensity of 20 mm per hour. In this scenario there is no problem in any junctions or conduits and maximum flow rate at exit of the system is 0.79 CMS. Therefore the stormwater management system in Subcatchment B is appropriate for the rainfall with maximum 20 mm per hour intensity. The same procedure is done for the rest of subcatchments and results are 19, 20, and 23 mm/hr for subcatchments A, C, and D. The critical rainfall for all subcatchments are shown in Figure 44.

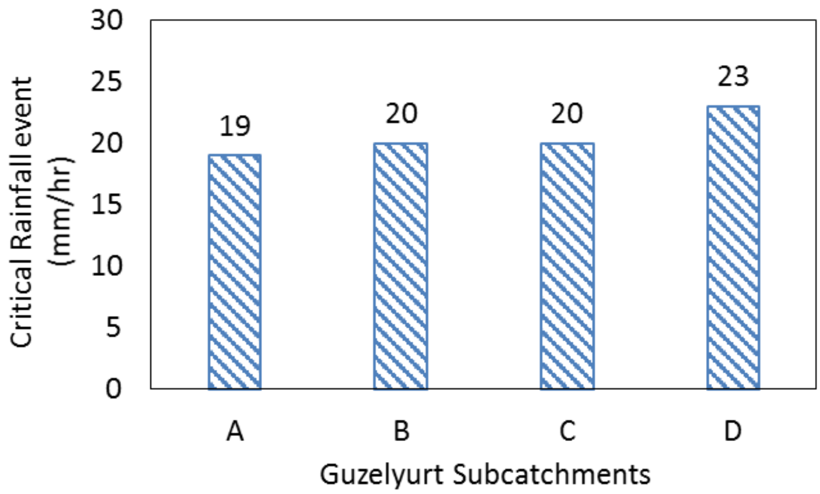


Figure 44. Critical rainfall event for subcatchments in Guzelyurt

In Subcatchment B the minimum intensity that the drainage system faces failure is 21mm/h which causes the failure in junctions 24 and 33 and a surcharged condition for conduits 22 and 32. Different scenarios are examined and the results for problematic junctions and conduits are shown in Table 7. Figure 45 shows, the locations of failures both in model and in the Google Earth image. As shown in Figure 45, the junctions which faced problem in 21mm/hr rainfall are shown by exclamation marks and conduits are shown by thick blue lines. Also, in Figure 46, Figure 47 and Figure 48, rainfalls with different intensities and the problematic locations are shown both in the model and in Google Earth images.

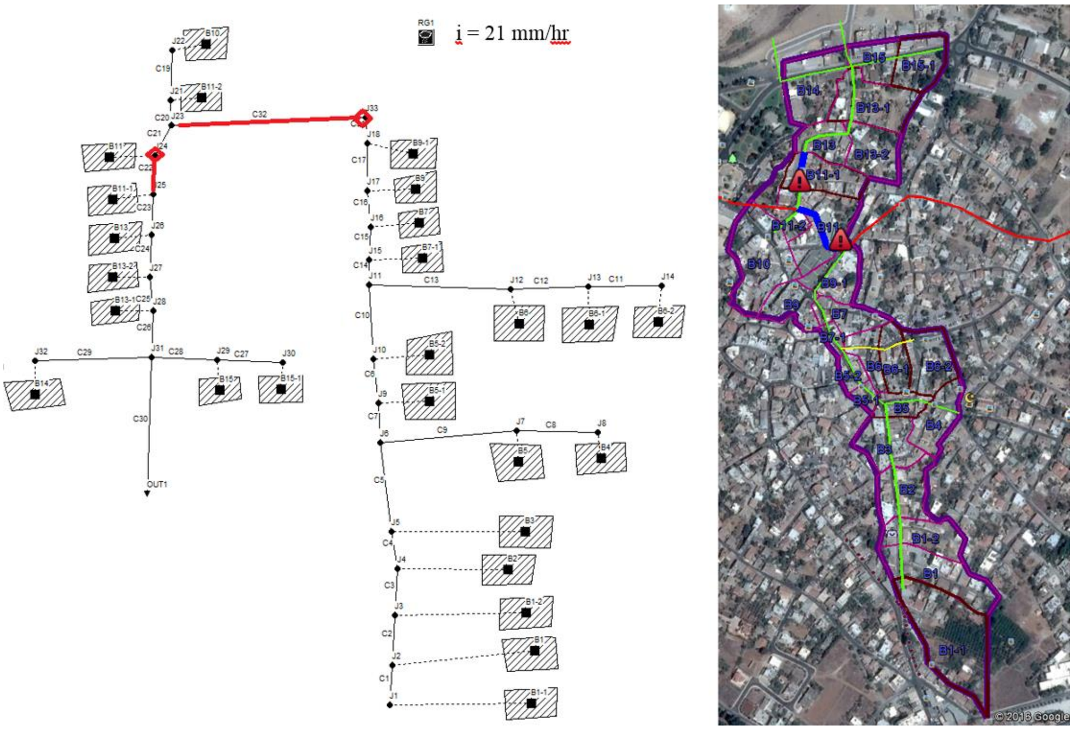


Figure 45. Junctions and conduits in Subcatchment B that face problem in 21 mm/h rainfall event.

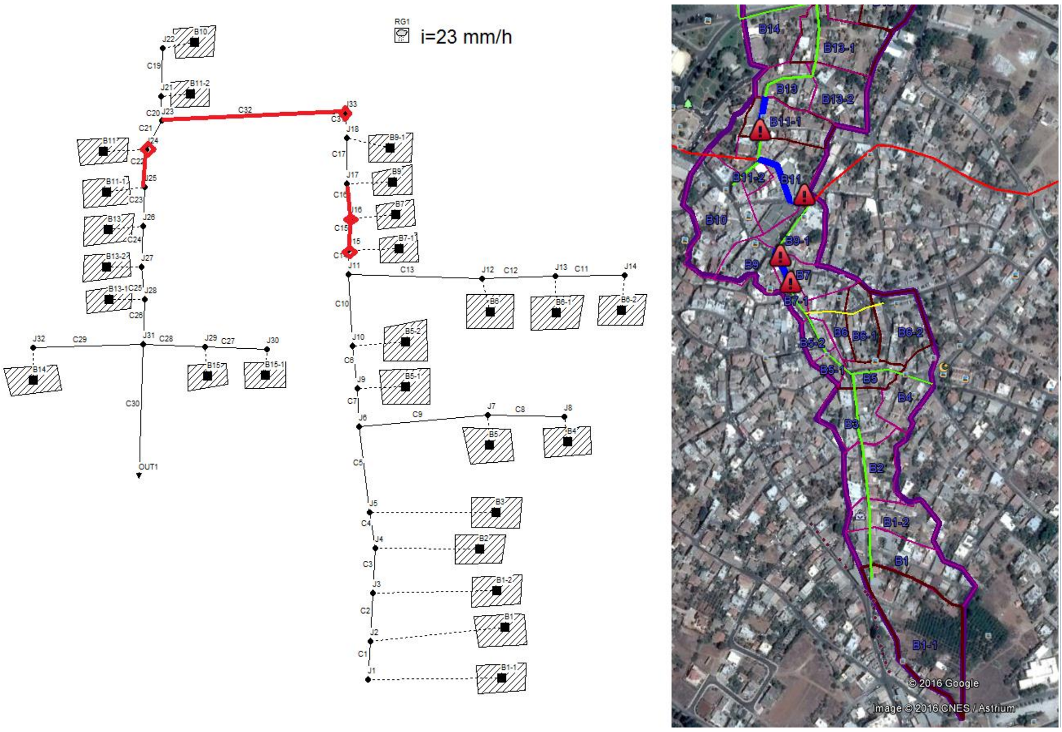


Figure 46. Junctions and conduits in Subcatchment B that face problem in 23 mm/h rainfall event.

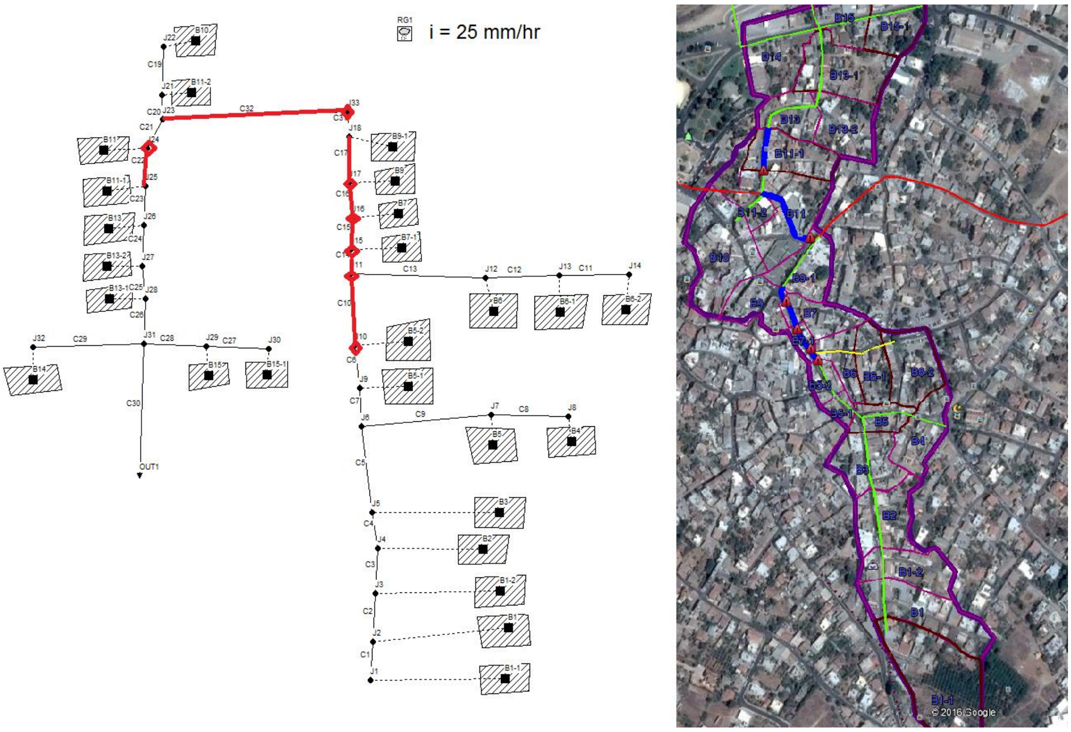



Figure 47. Junctions and conduits in Subcatchment B that face problem in 25 mm/h rainfall event.

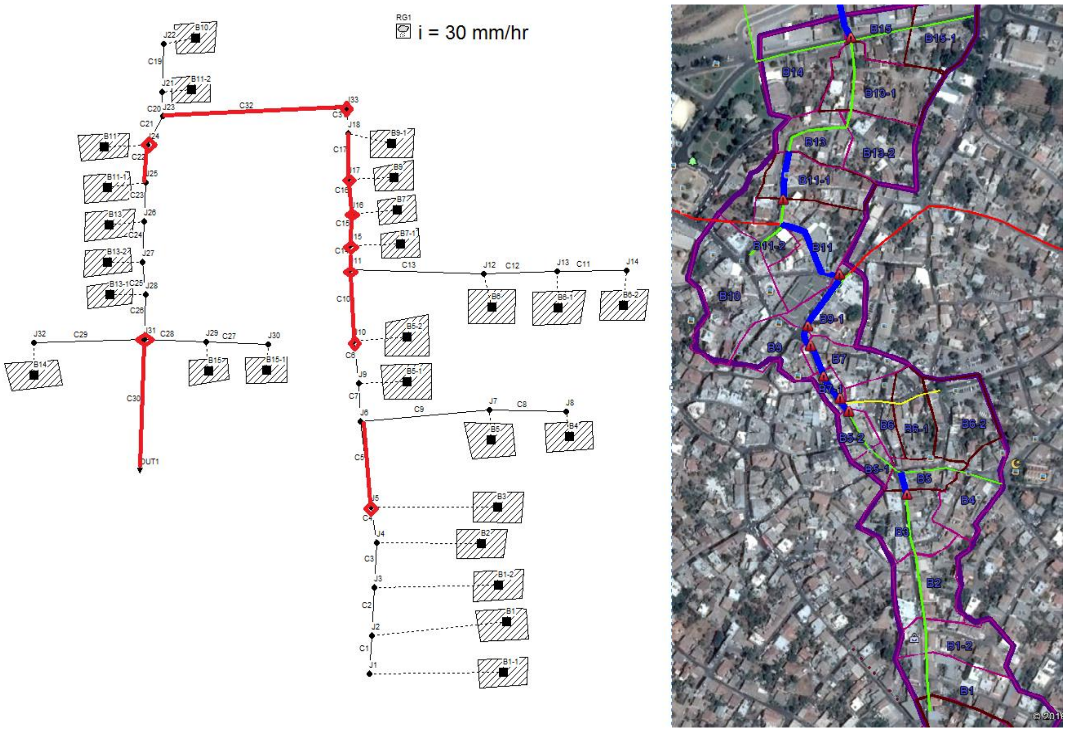



Figure 48. Junctions and conduits in Subcatchment B that face problem in 30 mm/h rainfall event.

### 5.2.1 Detailed Results of Scenario without Flooding

In this section, detailed results for Subcatchment B with rainfall intensity of 20 mm/h is reported, for in this intensity the whole system does not face any flood situation.

#### *5.2.1.1 Subareas*

The runoff that is generated from each subareas is shown in Table 8. This runoff is simulated by applying rainfall pattern of 20 mm rainfall in one hour. The Total precipitation is in (mm), total runoff from subareas (mm), total infiltration (mm), total runoff volume (million liters) and peak runoff that it's unit is the same as flow unit (CMS). Moreover, runoff coefficient is also calculated by dividing total runoff by total precipitation (CMS) which indicates the percentage of runoff that is generated by subcatchment.

Table 8. Runoff results for subareas in Subcatchment B for 20mm/hr rainfall event

| Subarea | Total Precip mm | Total Infil mm | Total Runoff mm | Total Runoff 10^6 ltr | Peak Runoff CMS | Runoff Coeff |
|---|---|---|---|---|---|---|
| B6 | 20 | 2 | 16.64 | 0.05 | 0.02 | 0.832 |
| B1-1 | 20 | 17 | 2.78 | 0.04 | 0.01 | 0.139 |
| B1 | 20 | 4 | 14.58 | 0.14 | 0.04 | 0.729 |
| B1-2 | 20 | 12 | 7.37 | 0.04 | 0.01 | 0.369 |
| B2 | 20 | 8 | 11.11 | 0.07 | 0.02 | 0.556 |
| B3 | 20 | 4 | 14.79 | 0.1 | 0.03 | 0.74 |
| B4 | 20 | 4 | 14.72 | 0.08 | 0.02 | 0.736 |
| B5 | 20 | 2 | 16.69 | 0.05 | 0.02 | 0.835 |
| B5-1 | 20 | 2 | 16.66 | 0.02 | 0.01 | 0.833 |
| B5-2 | 20 | 2 | 16.67 | 0.04 | 0.01 | 0.834 |
| B6-1 | 20 | 4 | 14.73 | 0.06 | 0.02 | 0.736 |
| B6-2 | 20 | 6 | 12.87 | 0.09 | 0.03 | 0.644 |
| B7-1 | 20 | 1 | 18.56 | 0.02 | 0.01 | 0.928 |
| B7 | 20 | 1 | 17.43 | 0.07 | 0.02 | 0.871 |
| B9 | 20 | 2 | 16.51 | 0.07 | 0.02 | 0.825 |
| B9-1 | 20 | 3 | 15.64 | 0.08 | 0.02 | 0.782 |
| B10 | 20 | 4 | 14.73 | 0.16 | 0.05 | 0.736 |
| B11-2 | 20 | 2 | 16.59 | 0.03 | 0.01 | 0.829 |
| B11 | 20 | 4 | 14.62 | 0.1 | 0.03 | 0.731 |
| B11-1 | 20 | 4 | 14.74 | 0.09 | 0.03 | 0.737 |
| B13 | 20 | 6 | 12.96 | 0.06 | 0.02 | 0.648 |
| B13-2 | 20 | 8 | 11.09 | 0.07 | 0.02 | 0.554 |
| B13-1 | 20 | 14 | 5.56 | 0.05 | 0.02 | 0.278 |
| B14 | 20 | 14 | 5.51 | 0.05 | 0.02 | 0.275 |
| B15 | 20 | 8 | 11.06 | 0.05 | 0.01 | 0.553 |
| B15-1 | 20 | 8 | 11.08 | 0.07 | 0.02 | 0.554 |

#### *5.2.1.2 Junctions*

In Table 9, the average and maximum depth of water in junctions, maximum Hydraulic Grade Line (HGL) and maximum total inflow are given for each junctions. Maximum HGL indicate the maximum elevation of the water level in the junctions. This value can be obtained by adding maximum depth of water at the junction to invert elevation of the junction.

Table 9. Junction's depth and inflow summary in Subcatchment B

| Junctions | Average Depth m | Maximum Depth m | Maximum HGL m | Maximum Total Inflow CMS |
|---|---|---|---|---|
| J1 | 0.01 | 0.07 | 48.6 | 0.013 |
| J2 | 0.04 | 0.13 | 48.22 | 0.052 |
| J3 | 0.04 | 0.13 | 47.85 | 0.065 |
| J4 | 0.05 | 0.19 | 46.35 | 0.087 |
| J5 | 0.07 | 0.27 | 45.94 | 0.116 |
| J6 | 0.07 | 0.27 | 45.86 | 0.156 |
| J7 | 0.03 | 0.12 | 45.76 | 0.04 |
| J8 | 0.03 | 0.12 | 45.99 | 0.025 |
| J9 | 0.06 | 0.23 | 45.18 | 0.162 |
| J10 | 0.07 | 0.3 | 44.63 | 0.173 |
| J11 | 0.07 | 0.32 | 44.38 | 0.233 |
| J12 | 0.04 | 0.15 | 44.66 | 0.06 |
| J13 | 0.04 | 0.15 | 45.16 | 0.045 |
| J14 | 0.03 | 0.11 | 45.53 | 0.026 |
| J15 | 0.08 | 0.34 | 44.00 | 0.239 |
| J16 | 0.08 | 0.35 | 43.76 | 0.261 |
| J17 | 0.08 | 0.35 | 43.64 | 0.281 |
| J18 | 0.08 | 0.34 | 43.14 | 0.305 |
| J21 | 0.05 | 0.2 | 42.41 | 0.058 |
| J22 | 0.05 | 0.2 | 42.52 | 0.049 |
| J23 | 0.16 | 0.39 | 42.3 | 0.363 |
| J24 | 0.09 | 0.41 | 41.78 | 0.393 |
| J25 | 0.09 | 0.41 | 41.04 | 0.42 |
| J26 | 0.09 | 0.38 | 39.51 | 0.437 |
| J27 | 0.09 | 0.37 | 39.12 | 0.459 |
| J28 | 0.09 | 0.37 | 37.35 | 0.474 |
| J29 | 0.03 | 0.11 | 37.6 | 0.035 |
| J30 | 0.03 | 0.11 | 37.77 | 0.02 |
| J31 | 0.08 | 0.35 | 37.19 | 0.524 |
| J32 | 0.01 | 0.04 | 37.52 | 0.015 |
| J33 | 0.16 | 0.39 | 43.13 | 0.305 |
| OUT1 | 0.08 | 0.35 | 35.25 | 0.524 |

#### *5.2.1.3 Conduits*

The maximum flow rates in each conduit that happened after a rainfall with the intensity of 20 mm/h are shown in Table 10. In this rain pattern, none of the conduits are surcharged. The highest flow rate occurs in a Conduit 30 with 0.524 $m^3/s$ which is located at the exit of the drainage system as shown in Figure 41.

Table 10. Conduit flow summary in Subcatchment B

| Conduit | Maximum Flow CMS | Max Veloc m/sec | Conduit | Maximum Flow CMS | Max Veloc m/sec |
|---|---|---|---|---|---|
| C1 | 0.013 | 0.81 | C17 | 0.281 | 1.97 |
| C2 | 0.052 | 1.23 | C19 | 0.049 | 0.68 |
| C3 | 0.065 | 1.75 | C20 | 0.058 | 2.56 |
| C4 | 0.087 | 1.25 | C21 | 0.363 | 2.63 |
| C5 | 0.116 | 1.09 | C22 | 0.393 | 2.3 |
| C6 | 0.162 | 1.8 | C23 | 0.42 | 2.66 |
| C7 | 0.156 | 1.97 | C24 | 0.437 | 2.8 |
| C8 | 0.025 | 0.73 | C25 | 0.459 | 2.92 |
| C9 | 0.04 | 1.04 | C26 | 0.474 | 3.73 |
| C10 | 0.173 | 1.41 | C27 | 0.02 | 0.62 |
| C11 | 0.026 | 1.11 | C28 | 0.035 | 2.14 |
| C12 | 0.045 | 1.25 | C29 | 0.015 | 1.92 |
| C13 | 0.06 | 3.38 | C30 | 0.524 | 3.59 |
| C14 | 0.233 | 1.77 | C31 | 0.305 | 2.37 |
| C15 | 0.239 | 1.68 | C32 | 0.305 | 1.76 |
| C16 | 0.261 | 1.79 | | | |

### 5.2.2 Conventional Retrofitting Method

In this scenario, some of the problematic conduits that are introduced in Table 7 assumed to be enlarged to increase the capacity of the system. The modification that is applied to the model is reported in this section and it is clarified to what extent this modification could increase the capacity of the system to convey a runoff generated by more intensive rainfall. In Section 5.7, it is stated that the system can convey the runoff generated from 20 mm/hr rainfall event without any problem, however in the intensity of 21 mm/hr the system starts facing problems in C22 and C32 conduits and J24 and J33 junctions. As a first step for improving the system the two conduits that faced problem in 21mm/hr rainfall have changed from 0.5m diameter to 0.6 m diameter pipe and their junctions have changed accordingly. In this case, the model has run for 21 mm/hr rainfall without any problem. However, for 22 mm/hr event the system faces flooding in conduit C23. By changing the size of conduit C23 from 0.5m to 0.6m diameter, no improvement has happened in the system and only the next conduit C24 faces surcharged condition. This procedure is continued till all the conduits C32, C21, C22, C23, C24, C25, C26 and C30 are changed to 0.6 m diameter pipe size although the rainfall intensity stays at 22mm/hr. At this stage we need to change the size of C15 and C16 to see what would the effect of this change be on system capacity. The result of this changes indicates that by increasing the diameter for C15 and C16 no improvement occurred for rainfall intensity and it seems that the surcharged condition transferred to the adjacent conduit C17. By changing C17 from 0.5 m to 0.6 m diameter the intensity has changed to 23 mm/hr. This procedure has been continued up to the point that the system can convey the runoff from 30 mm/hr rainfall without any problem and the results are reported in Table 11. Also, the conduits that need to be enlarged up to 0.6 m to meet the specific intensity are specified. After some changes in the system, the rainfall intensity has increased more than one step like the last modification which increased the rainfall intensity up to 30 mm/hr. Also, the total length of conduits which is needed to be replaced by 0.6 m pipe size are calculated and reported in the Table 11. As a result, we need to replace 780 m of the existing conduits with a 0.6 m size to meet the capacity for conveying runoff generated from 30mm/hr rainfall.

Table 11. Conduit replacement from 50 cm to 60 cm pipe size for different rainfall in Subcatchment B.

| Rainfall intensity | Conduits replacement | Total length |
|---|---|---|
| 20 | ------ | ------ |
| 21 | C22,C32 | 167 |
| 22, 23 | C21 C23 C24 C25 C26 C30 C15 C16 C17 | 386 |
| 24 | C14 | 49 |
| 25 | C10 C32 | 157 |
| 26, 27, 28,30 | C5 | 22 |
| | **Total Conduits length:** | 780 |

Considering financial parameters in the retrofitting design, forms some constrains that should be taken into account. The main issue is that importing different size of PVC pipes to the island is very costly. Therefore, it is more cost effective to use the concrete pipes which are produced in Northern Cyprus. Although the concrete pipes are produced in limited sizes such as 50 cm and 100 cm, it is economically beneficial to use local productions even with a larger size than an imported design size. Therefore, considering this constrain for retrofitting design, all the 50 cm pipes which are needed to be changed should be replaced by a concrete -100 cm pipe with circular shape and manning coefficient of 0.013 results in the following outcomes. The pipes that needed to be replaced by 100 cm -concrete pipes to convey a higher rainfall runoff are shown in Table 12. Also, the total length of pipes are given for each step. In total, to improve the system capacity from 20mm/hr to 40mm/hr, 909 m of old pipes should be replaced with 100 cm -concrete pipes. The economic analysis of the replacements is given in Section 5.7.4 in detail.

Table 12. Conduit replacement from 50cm to 100cm pipe size for different rainfall.

| Rainfall intensity mm/hr | Conduits replacement | Total length m |
|---|---|---|
| 20 | ------ | ------ |
| 21 | C22,C32 | 167 |
| 22 | C23 C24 C25 C26 C30 | 256 |
| 23 | C15 C16 | 56 |
| 24 | C14 C17 | 125 |
| 25 | C10 C21 | 79 |
| 26 | C5 | 22 |
| 35 | C31 | 50 |
| 40 | C4, C5, C12 | 155 |
| | **Total Conduits length:** | 909 |

### 5.2.3 Applying Drywells

As it is mentioned in Section 4.4.2.3, in order to consider existing infiltration wells as drywells to the model, the number of dwellings and the roof top areas in each subarea are necessary. So, by conducting couple of site visits, the number of buildings and their applications has been investigated. Moreover, the roof top areas are calculated using the Auto CAD file generated from Google Earth. Results for each subarea in Catchment B is tabulated in Table 13. The letters that are used in land use columns are h, s, hs, b, ad which stand for independent house, independent shop, house and shop together, bank and administrative office, respectively. In general, total area for Subcatchment B is 149,734 square meter which 64% of total area is impervious and 57% of impervious area is roofs. As it is reported in Table 13, the percentage of roof areas in impervious areas for all the subareas is above 40%. This percentage increases up to 85% which indicates that the subarea has less road or parking lot and most of the impervious area is consist of buildings and houses like subarea B1-2.

Table 13. Land use characteristic for subareas in Subcatchment B.

| Subareas | Area (m2) | % Impervious | Number of Buildings | Building's Application | Total Roof area (m2) | Impervious area (m2) | % of Roofs in Impervious Area |
|---|---|---|---|---|---|---|---|
| B1 | 9275.6 | 80 | 19 | 17h+2s | 4736 | 7420.5 | 64% |
| B1-1 | 15063.6 | 15 | 11 | 11h | 1792 | 2259.5 | 79% |
| B1-2 | 5891.2 | 40 | 12 | 11h+1hs | 2001 | 2356.5 | 85% |
| B2 | 6486.9 | 60 | 11 | 8h+4hs | 3240 | 3892.1 | 83% |
| B3 | 6563.3 | 80 | 20 | 6h+14hs | 3430 | 5250.7 | 65% |
| B4 | 5630.6 | 80 | 14 | 12h+2s | 2208 | 4504.5 | 49% |
| B5 | 2994.8 | 90 | 14 | 11h+3hs | 1578 | 2695.4 | 59% |
| B5-1 | 1344.7 | 90 | 10 | 3h+7s | 731 | 1210.2 | 60% |
| B5-2 | 2074.6 | 90 | 13 | 13s | 1157 | 1867.1 | 62% |
| B6 | 3047.0 | 90 | 22 | 6h+1b+15s | 1806 | 2742.3 | 66% |
| B6-1 | 4357.8 | 80 | 10 | 4h+6s | 2054 | 3486.3 | 59% |
| B6-2 | 6731.9 | 70 | 13 | 10h+3s | 2356 | 4712.3 | 50% |
| B7 | 4275.2 | 95 | 22 | 6h+8hs+8s | 1860 | 4061.5 | 46% |
| B7-1 | 1337.7 | 100 | 15 | 15s | 810 | 1337.7 | 61% |
| B9 | 4142.7 | 90 | 23 | 7h+6hs+8s+2b | 2094 | 3728.4 | 56% |
| B9-1 | 5020.0 | 85 | 14 | 6s+3hs+3h+2 admin | 2330 | 4267.0 | 55% |
| B10 | 11030.5 | 80 | 25 | 11h+8hs+6s | 5008 | 8824.4 | 57% |
| B11 | 7023.9 | 80 | 10 | 9h+1hs | 3524 | 5619.1 | 63% |
| B11-1 | 6242.6 | 80 | 18 | 18h | 2554 | 4994.1 | 51% |
| B11-2 | 1934.8 | 90 | 6 | 6h | 947 | 1741.3 | 54% |
| B13 | 4537.6 | 70 | 14 | 14h | 1925 | 3176.3 | 61% |
| B13-1 | 8991.3 | 30 | 14 | 14h | 2265 | 2697.4 | 84% |
| B13-2 | 6451.6 | 60 | 16 | 15h+1s | 2187 | 3871.0 | 56% |
| B14 | 9127.4 | 30 | 6 | 5h+1admin | 1646 | 2738.2 | 60% |
| B15 | 4364.2 | 60 | 6 | 2hs+4h | 1469 | 2618.5 | 56% |
| B15-1 | 6104.2 | 60 | 9 | 2h+military+4hs | 1940 | 3662.5 | 53% |

Note: In Building's application column h,s, hs, b, and admin stand for house, shop, a hous which has a shop in first floor, bank, and administrative office.

The characteristic information for drywells which are applied as a LID control in the model is given in Section 3.5. Therefore, each unit of LID control is considered as rain barrel with the area of 0.63 square meter and height of 14.7 m without any underdrain. Applying all the above information to the model results in the following outcomes. As shown in Table 14, after applying drywell to each dwelling, the designed hyetograph that is applied to the model rises up to 40 mm/hr. However, this value before applying drywells was 20 mm/hr. Total precipitation, total infiltration, total runoff are given for each of the subcatchments in mm and the values for peak runoff are given in cubic meter per second.

Table 14. Runoff results for subareas in Subcatchment B after applying LID controls for 40 mm/hr rainfall event

| Subareas | Total Precip mm | Total Infil mm | Total Runoff mm | Peak Runoff CMS | Runoff Coeff |
|---|---|---|---|---|---|
| B6 | 40 | 3.97 | 14.87 | 0.01 | 0.372 |
| B1-1 | 40 | 33.98 | 1.46 | 0.01 | 0.036 |
| B1 | 40 | 7.99 | 11.44 | 0.03 | 0.286 |
| B1-2 | 40 | 23.97 | 1.23 | 0.01 | 0.031 |
| B2 | 40 | 15.98 | 7.41 | 0.04 | 0.185 |
| B3 | 40 | 7.98 | 10.83 | 0.02 | 0.271 |
| B4 | 40 | 7.99 | 15.71 | 0.03 | 0.393 |
| B5 | 40 | 3.99 | 14.33 | 0.01 | 0.358 |
| B5-1 | 40 | 3.98 | 13.9 | 0.01 | 0.348 |
| B5-2 | 40 | 3.98 | 13.23 | 0.01 | 0.331 |
| B6-1 | 40 | 7.99 | 12.63 | 0.02 | 0.316 |
| B6-2 | 40 | 7.99 | 15.38 | 0.03 | 0.384 |
| B7-1 | 40 | 1.98 | 15.06 | 0.01 | 0.377 |
| B7 | 40 | 1.99 | 19.67 | 0.02 | 0.492 |
| B9 | 40 | 3.99 | 17.93 | 0.02 | 0.448 |
| B9-1 | 40 | 7.99 | 15.7 | 0.02 | 0.393 |
| B10 | 40 | 7.99 | 15.1 | 0.05 | 0.377 |
| B11-2 | 40 | 5.99 | 15.07 | 0.01 | 0.377 |
| B11 | 40 | 7.99 | 17.26 | 0.03 | 0.432 |
| B11-1 | 40 | 7.99 | 15.1 | 0.03 | 0.377 |
| B13 | 40 | 11.98 | 10.58 | 0.01 | 0.265 |
| B13-2 | 40 | 15.97 | 10.2 | 0.02 | 0.255 |
| B13-1 | 40 | 27.97 | 1.86 | 0.01 | 0.047 |
| B14 | 40 | 27.99 | 5.35 | 0.01 | 0.134 |
| B15 | 40 | 15.99 | 12.04 | 0.02 | 0.301 |
| B15-1 | 40 | 15.98 | 10.9 | 0.02 | 0.273 |

Moreover, the runoff coefficient is calculated by dividing total runoff by total precipitation. This value shows that what percentage of rainfall changes to runoff. The higher runoff coefficient the more impervious area. Comparing the runoff coefficient before applying LID from Table 8, with runoff coefficient after applying LID from Table 14 indicates that by applying LID controls to the model, the runoff coefficients are decreased for 56% in average. The minimum reduction rate is associated to Subarea B6-2 with 40% reduction in runoff coefficient and the maximum reduction rate is for Subarea B1-2 with 92% reduction. Figure 49 shows the runoff coefficient comparison, before and after applying LID controls. As shown in Figure 49, the reduction rate is not the constant for all the subareas. The reduction rate varies regarding the number of LID controls that are applied in each subarea and also impervious percentage of the subarea.

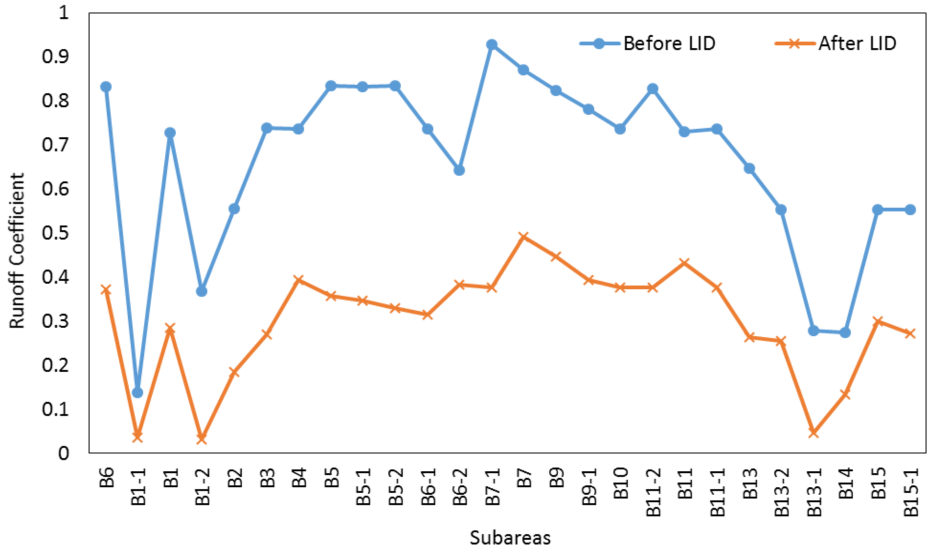



Figure 49. Runoff coefficient comparison, before and after applying LID controls.

LID performance summary is given in Table 15. The total inflow to the drywell in each subarea indicates the total runoff generated from roofs and the final storage shows the amount of water that is stored in drywells and surface outflow indicates that the capacity of the drywells are not enough for the generated runoff and the extra amount of water rerouted to impervious area. In most of the subareas the runoff from roofs are collected in the wells and still there is enough capacity to save more water, and there is no surface outflow reported for many of the subareas. The ultimate capacity for each well is 14700 mm which is equal to the height of the well.

Table 15. LID performance summary for 40mm/hr design rainfall

| Subareas | Total Inflow mm | Surface outflow mm | Final Storage mm |
|---|---|---|---|
| B6 | 2471 | 0 | 2471 |
| B1-1 | 9358 | 0 | 9358 |
| B1 | 15023 | 323 | 14700 |
| B1-2 | 10940 | 0 | 10939 |
| B2 | 17885 | 3185 | 14700 |
| B3 | 10408 | 0 | 10407 |
| B4 | 9471 | 0 | 9471 |
| B5 | 6943 | 0 | 6943 |
| B5-1 | 4276 | 0 | 4276 |
| B5-2 | 5490 | 0 | 5490 |
| B6-1 | 12537 | 0 | 12536 |
| B6-2 | 12424 | 0 | 12424 |
| B7-1 | 2494 | 0 | 2494 |
| B7 | 5156 | 0 | 5156 |
| B9 | 4651 | 0 | 4651 |
| B9-1 | 8460 | 0 | 8460 |
| B10 | 10842 | 0 | 10842 |
| B11-2 | 8794 | 0 | 8793 |
| B11 | 14798 | 98 | 14700 |
| B11-1 | 8498 | 0 | 8497 |
| B13 | 8352 | 0 | 8352 |
| B13-2 | 8281 | 0 | 8281 |
| B13-1 | 9872 | 0 | 9872 |
| B14 | 16448 | 1748 | 14700 |
| B15 | 12769 | 0 | 12769 |
| B15-1 | 13060 | 0 | 13060 |

Junction summary for each of the junctions are given in Table 16. Average depth, maximum depth and HGL values are given in meter. Maximum HGL indicate the maximum elevation of the water level in the junctions. This value can be obtained by adding maximum depth of water at the junction to invert elevation of the junction.

Table 16. Junction's depth and inflow summary in Subcatchment B after applying LID controls for 40 mm/hr design rainfall.

| Junction | Average Depth m | Maximum Depth m | Maximum HGL m | Maximum Total Inflow CMS |
|---|---|---|---|---|
| J1 | 0.01 | 0.05 | 48.58 | 0.006 |
| J2 | 0.03 | 0.11 | 48.2 | 0.036 |
| J3 | 0.03 | 0.11 | 47.83 | 0.038 |
| J4 | 0.04 | 0.19 | 46.35 | 0.081 |
| J5 | 0.06 | 0.25 | 45.92 | 0.099 |
| J6 | 0.06 | 0.25 | 45.85 | 0.136 |
| J7 | 0.03 | 0.13 | 45.76 | 0.038 |
| J8 | 0.03 | 0.12 | 45.99 | 0.025 |
| J9 | 0.05 | 0.22 | 45.17 | 0.14 |
| J10 | 0.06 | 0.28 | 44.61 | 0.146 |
| J11 | 0.07 | 0.3 | 44.36 | 0.201 |
| J12 | 0.04 | 0.16 | 44.67 | 0.059 |
| J13 | 0.04 | 0.16 | 45.16 | 0.046 |
| J14 | 0.03 | 0.12 | 45.54 | 0.03 |
| J15 | 0.07 | 0.32 | 43.98 | 0.205 |
| J16 | 0.07 | 0.33 | 43.74 | 0.227 |
| J17 | 0.07 | 0.33 | 43.62 | 0.247 |
| J18 | 0.07 | 0.33 | 43.13 | 0.269 |
| J21 | 0.05 | 0.21 | 42.42 | 0.056 |
| J22 | 0.05 | 0.21 | 42.53 | 0.048 |
| J23 | 0.15 | 0.38 | 42.29 | 0.323 |
| J24 | 0.09 | 0.4 | 41.77 | 0.355 |
| J25 | 0.09 | 0.4 | 41.03 | 0.38 |
| J26 | 0.08 | 0.37 | 39.5 | 0.392 |
| J27 | 0.08 | 0.36 | 39.11 | 0.41 |
| J28 | 0.08 | 0.36 | 37.34 | 0.414 |
| J29 | 0.03 | 0.11 | 37.6 | 0.034 |
| J30 | 0.03 | 0.11 | 37.77 | 0.019 |
| J31 | 0.08 | 0.33 | 37.17 | 0.459 |
| J32 | 0.01 | 0.04 | 37.52 | 0.012 |
| J33 | 0.15 | 0.38 | 43.12 | 0.268 |
| OUT1 | 0.08 | 0.33 | 35.23 | 0.459 |

The maximum flow rates in each conduit that happened after a rainfall with the intensity of 40 mm/h are shown in Table 17. Also, the maximum velocity for each conduit is given in m/sec.

Table 17. Conduit flow summary in Subcatchment B for 40 mm/hr design rainfall

| Conduit | Maximum \|Flow\| CMS | Max Veloc m/sec | Conduit | Maximum \|Flow\| CMS | Max Veloc m/sec |
|---|---|---|---|---|---|
| 1 | 0.006 | 0.64 | 17 | 0.247 | 1.82 |
| 2 | 0.036 | 1.05 | 19 | 0.048 | 0.65 |
| 3 | 0.038 | 1.43 | 20 | 0.056 | 2.41 |
| 4 | 0.08 | 1.17 | 21 | 0.323 | 2.43 |
| 5 | 0.099 | 0.99 | 22 | 0.355 | 2.14 |
| 6 | 0.139 | 1.65 | 23 | 0.38 | 2.48 |
| 7 | 0.135 | 1.81 | 24 | 0.392 | 2.6 |
| 8 | 0.026 | 0.71 | 25 | 0.41 | 2.71 |
| 9 | 0.038 | 0.97 | 26 | 0.414 | 3.43 |
| 10 | 0.146 | 1.29 | 27 | 0.019 | 0.59 |
| 11 | 0.03 | 1.09 | 28 | 0.034 | 2.03 |
| 12 | 0.046 | 1.19 | 29 | 0.012 | 1.72 |
| 13 | 0.059 | 3.19 | 30 | 0.459 | 3.32 |
| 14 | 0.201 | 1.63 | 31 | 0.268 | 2.19 |
| 15 | 0.205 | 1.54 | 32 | 0.268 | 1.62 |
| 16 | 0.227 | 1.65 | | | |

The same study is applied for Subcatchment A and the result shows that applying drywells to the stormwater management system would increase the critical rainfall from 19 mm/hr to 35 mm/hr.

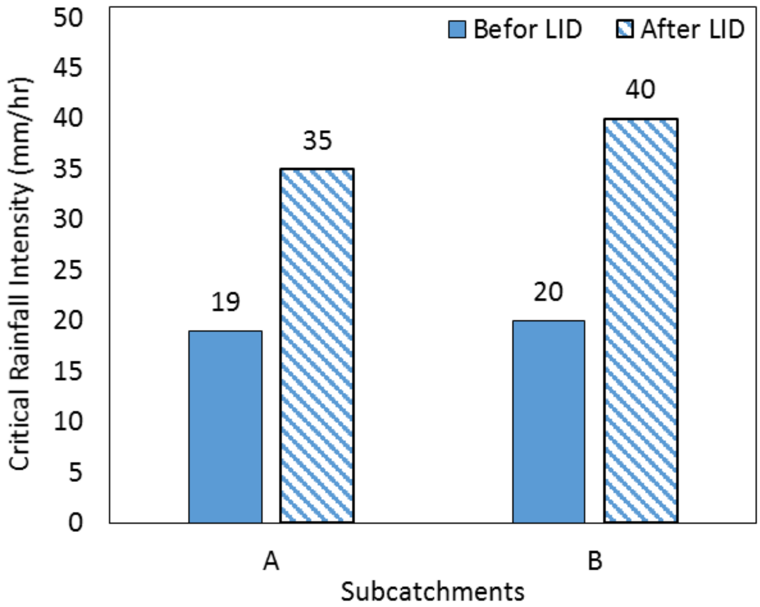



Figure 50. The effect of applying LID controls (drywells) on critical rainfall event for subcatchments A and B.

As shown in Figure 50, applying drywells in Subcatchment B raises the critical rainfall event for 20 mm/hr (50%, from 20 mm/hr to 40 mm/hr), however, for Subcatchment A this increase in critical rainfall event is 16 mm/hr (46%, from 19 mm/hr to 35 mm/hr) which is mainly due to less population density and consequently less impervious area that is occurred in Subcatchment A. As it is mentioned before the total impervious area in Subcatchment B is 64% and 60% of the impervious area measured as a roof tops which is treated by LID controls. However, in Subcatchment A the total impervious area is 29% and 54% of it is identified as rooftops.

### 5.2.4 Economics Comparison

**Economic Analysis for Conventional Method**

As it is already noted, in Subcatchment B, in conventional retrofitting design for increasing the system capacity from 20 mm/hr to 40 mm/hr, 909 m length of existing pipelines need to be replaced by 100 cm-concrete pipe lines. The replacement processes can be divided into two parts. First, excavation, replacement, and filling with concrete part and second paving with asphalt. The costs varies according to pipe sizes. The costs and detail information are provided and reported by technical office of municipality. As it is shown in Figure 51, the total cost details are reported in two parts. First, cost of asphalt and second, excavating and installation of new pipes. For cost1, the provided data from municipality is the whole price including material and constructions which is 150 TL per ton of asphalt with 2.4 $ton/m^3$ density.

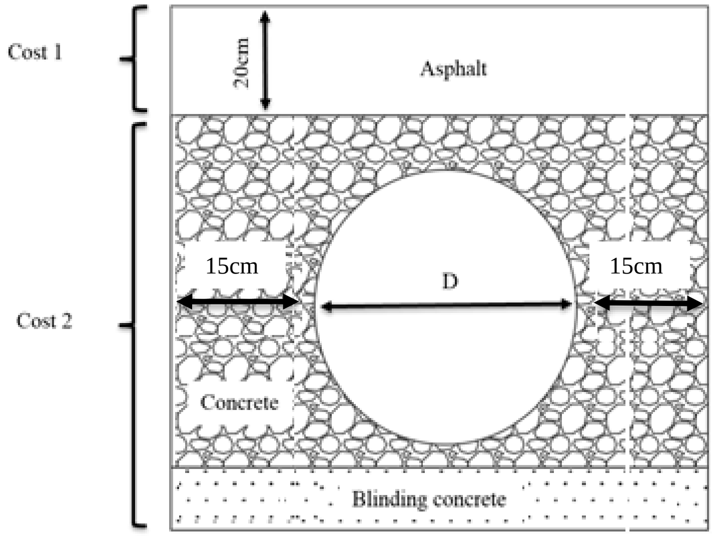


Figure 51. Detailed characteristics of piping.

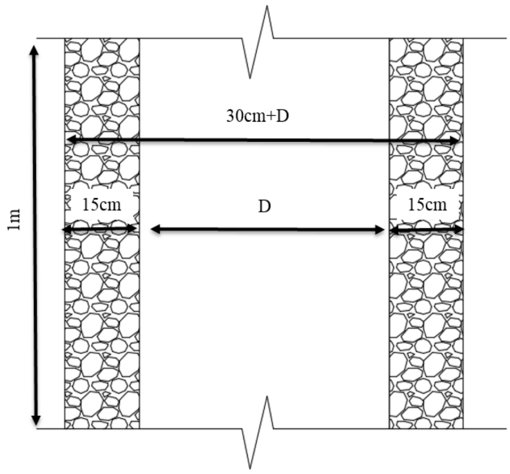


Figure 52. Schematic plan of piping.

Thus, the price for paving 1m length as shown in Figure 52 can be obtained as

For $D = 50cm$

$Volume\ of\ asphalt/m = (0.3 + 0.5) \times 1 \times 0.2 = 0.16\ m^3/m$

$Weight\ of\ asphalt/m = 0.16 \times 2.4 = 0.384\ ton/m$

$Cost1 = 0.384 \times 150 = 57.6 \sim 60\ TL/m$

For $D = 100cm$

$Volume\ of\ asphalt/m = (0.3 + 1) \times 1 \times 0.2 = 0.26\ m^3/m$

$Weight\ of\ asphalt/m = 0.26 \times 2.4 = 0.624\ ton/m$

$Cost1 = 0.624 \times 150 = 93.6 \sim 95\ TL/m$

Also, the cost for part 2 which is indicated in Figure 51, is 120 TL per meter for replacing the old pipe with 50cm-concrete pipe and 210 TL for replacing with 100cm concrete pipe. These costs include all the expenses for excavating, the price for the pipe and refilling processes per meter of length. Therefore, the total cost for replacing existing pipes with 50 cm concrete pipe would be 180 TL and with 100 cm concrete pipe 305Tl. Table 18 shows the detail cost for replacing different pipe size.

Table 18. Detail cost analysis for pipe replacement in different sizes.

| **Cost Detail** | **Pipe Size** | |
|---|---|---|
| | 50 cm | 100 cm |
| Cost 1 | 60 TL | 95 TL |
| Cost2 | 120 TL | 210 TL |
| Total | 180 TL | 305 TL |

These prices are for January 2016 with exchange rate of 3.05 TL to USD. Therefore, the price for replacing existing pipes with 100cm concrete pipe would be 100 USD per meter. As mentioned before, for increasing the system capacity from 20 mm/hr to 40 mm/hr, 909 m existing pipelines should be replaced with 100cm-concrete pipes in Subcatchment B. Thus, the total cost for system improvement in Subcatchment B is 90,900 USD. Considering the current exchange rate for USD to TL which is 3.9 the price would be 354,510 TL. However, using the existing potential in the city for improving the system capacity would eliminate all these expenses. In the next chapter, applying existing drywells as a reservoir for collecting rainfall runoff from rooftops is investigated and since these wells are already available in each house, the cost for connecting rooftops runoff is investigated.

**Economic Analysis for LID Application**

Since the septic tanks and drywells are already existed the only cost for applying drywells to drainage system is the cost for connecting the roofs to the septic tanks which are mainly located at the back side of each buildings. Estimating the price for connecting roofs to septic tanks is not very accurate, for the area for each house and the location of septic tanks are not known. Therefore, it is assumed that the average area for roof tops are 100 $m^2$ and the total pipe length that is needed for connecting the rooftops to the septic tank would be 35 m as shown in Figure 53.

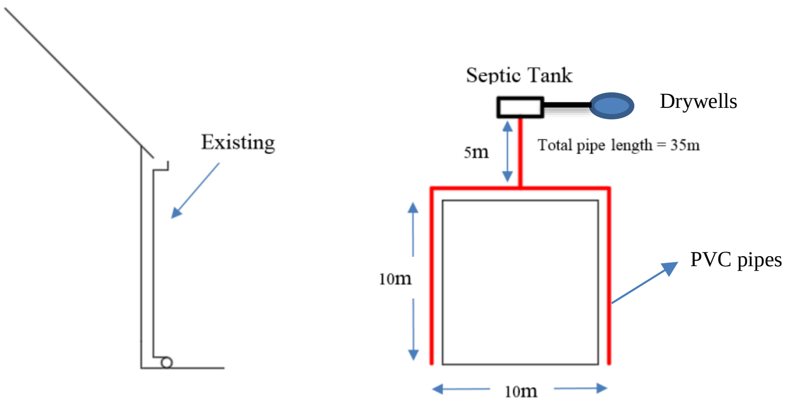



Figure 53. Schematic plan for connecting rooftops runoff to septic tanks.

The pipe size that is considered for this application is 3 in diameter PVC pipe with the price of 28 TL per 6m length and the labor that is needed is 2 persons for one day which make the total cost for labor as 300 TL. Considering 6 units of pipe and labor cost together will result in $(6 \times 28TL) + 300TL = 468{\sim}470TL$ for connecting a rooftop to the septic tank. In Subcatchment B, 367 buildings is existed which makes the total cost as $367 \times 470TL = 172490\ TL$.

**Economic Comparison of Scenarios**

As it is discussed previously, there are two scenarios for improving the capacity of drainage system. First, conventional method which stands for replacing existing pipes with larger pipes. In this scenario, for improving system capacity from 20 mm/hr to 40 mm/hr, 909 m of 100 cm-concrete pipe is required. The total cost is estimated as 354,510 TL or 90,900 USD. This cost does not include the administrative cost. Second scenario is applying LID controls which in this study corresponds to applying drywells as a cistern for harvesting rainfall. In this scenario, the system capacity of Subcatchment B is improved from 20 mm/hr to 40 mm/hr. The cost that is estimated for connecting 367 buildings' rooftops to existing septic tank is 172,490 TL or 44,228 USD. As shown in Figure 54, by applying LID controls to drainage system, 182,020 TL or 46,672 USD would be saved. It should be noted that this cost benefit is only a direct advantage of applying rainwater harvesting. However, if the difficulties that are externalized to the public and private sectors during constructional activities for replacing pipes could be evaluated in monetary values, a better comparison would be achieved. These difficulties would be listed as: drastic problems in traffic flow, dust and sound emissions, and difficulty for using heavy duty constructional machines for narrow streets in Guzelyurt.

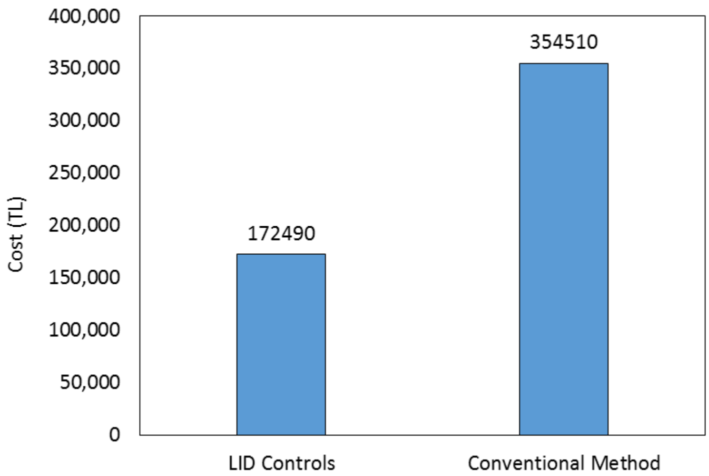


Figure 54. Economic analysis for different Scenarios to improve the drainage capacity from 20 mm/hr to 40 mm/hr.

## 5.3 Calibration and Verification

As it is stated by Akan and Houghtalen, 2003 none of the models can perfectly simulate real applications however calibration and validation can improve the validity of the results. Moreover, calibration results in validation and for both of these processes the measured rainfall and runoff data from the studied area is necessary. Different parameters of the model such as flow length, n values, infiltration parameters, etc. can be adjusted by having measured rainfall and runoff data. The calibrated model can be verified by new rainfall event to examine how closely can predict the runoff. The minimum measured rainfall–runoff data that is necessary for calibration and verification is reported as at least 6 or more (Akan & Houghtalen 2003). This can be an expensive proposition and most of the modeling is done without calibration and verification. Verification and calibration are not very critical for models which employed physically based algorithm such as kinematic wave modelling which is the case that is used in this study(Akan & Houghtalen 2003). However, all the results and deliveries of this study have been discussed in different meetings with municipality engineers who are the most experienced with respect to water management system in Guzelyurt. All the rainfall events and problematic locations are confirmed by the municipality engineers which is considered as a kind of verification.

# CHAPTER 6 : CONCLUSION

Water sensitive urban design is promising approach to mitigate the defeated water cycle. Different application of this method is introduced in this study, however, rainwater harvesting is the prominent focus of this study. Both improving groundwater resources and harvesting water for none potable usage such as irrigating and household usage are the main benefits of rain water harvesting. This study investigated the potential of applying drywells to the existing stormwater network system of Guzelyurt to quantify the positive effect on reducing pressure on drainage system during critical rainfall. Connecting drywells would mitigate the problems that occurs during critical rainfall events. The problems such as damages to private properties and difficulties in transportation cause a huge inconvenience for residents. Also, water shortage, groundwater depletion and salinity are the cases that would be improved by applying drywells. Moreover, applying drywells to the current drainage system has no significant cost since the drywells are already existed in each of dwellings. The drywells would be beneficial irrespective of whether the harvested water is to be pumped out for irrigation and household usage, or left in drywells to feed the depleting ground water.

The whole Guzelyurt city was divided to 4 main subcatchments and the rainfall-runoff model were developed for each of the subcatchments in SWMM program. The critical rainfall in which the system faces problem has investigated for each of the subcatchments and the results were 19, 20, 20, and 23 mm/hr for Subcatchment A, B, C, and D respectively. Subcatchment A and B were chosen for detailed study on applying LID controls since B was the most populated subcatchment with the highest impervious percentage in Guzelyurt city and A has the lowest impervious area relative to other subcatchments. Different scenarios were investigated for Subcatchment B.

First, applying conventional method and improving existing infrastructure by replacing the pipes with larger size pipes. Results for this scenario showed that to increase the system capacity from 20 mm/hr to 40 mm/hr rainfall, 909 m pipelines with the 50cm diameter should be replaced by 100cm diameter pipe size. However, this pipe size increase from 50 cm to 100 cm is due to lack of PVC pipe production in Northern Cyprus and high cost of importing pipes from foreign countries. Therefore, it is more economical to use local concrete pipes that are made in Cyprus but in limited size of 50cm and 100cm. In this case, the total capacity of the drainage system would be increased up to 40 mm/hr rainfall. Second scenario is adding existing drywells as a reservoir to the current drainage system. The results from both Subcatchments A and B indicate an improvement of the system capacity with 46% and 50% respectively. This improvement for Subcatchments A is from 19 mm/hr to 35mm/hr and for Subcatchments B is from 20 mm/hr to 40 mm/hr. Moreover, economic analysis is done for these two scenarios in Subcatchment B. The cost for conventional method is calculated as approximately 355,000 TL considering materials and construction costs. However, the cost for second scenario, applying drywells, is approximately 172,000 TL. The economic analysis indicates that for increasing drainage system capacity from 20 to 40 mm/hr in Subcatchment B, applying Drywells would decrease the expenses by 51% in comparison with conventional method. This would only be the direct reduction in construction costs. However, if the problems associated with having a large construction project inside the city of Guzelyurt are also considered, other benefits of applying LID would also be appreciated. The mentioned problems due to a large construction project would be long-term interruption in traffic flow, interruption in local businesses and other similar problems.

For future study, calibration of the models could be done in order to examine how these models related to actual rainfall-runoff data and this would be possible only by installing flowmeter at the exit point of each subcatchment and having rainfall measurement with high resolution. In this study it is assumed that the whole runoff from rooftops would convey to the septic tanks regardless of the possible losses which can be investigated in more details in future studies. Moreover, the whole capacity of the septic tanks and drywells are  are considered to be available for water storage in this study; however, the real available capacity of the septic tanks should be studied as future works considering the remained sludges and sedimentation in the septic tanks and the resulting volume reduction. Studying runoff contamination and the effect of that on ground water could be a potential future study. More detailed study on cost-benefit analysis should be conducted to make sure all the advantages and disadvantages are taken into account. Investigating the rainfall-runoff model for continues rainfall events and considering a household consumption in the model to make a more realistic model. Also, investigating other techniques of WSUD such as permeable parking lots and bioretention systems could be a possible solution for decreasing runoff in Guzelyurt and comparing the effectiveness of each of method with the proposed method in this study.

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# APPENDICES

In this section, input values for each subcatchment model reported separately which is discussed in chapter 5.6 in detail.

### Appendix A: Subcatchment A Input Characteristics for SWMM Model

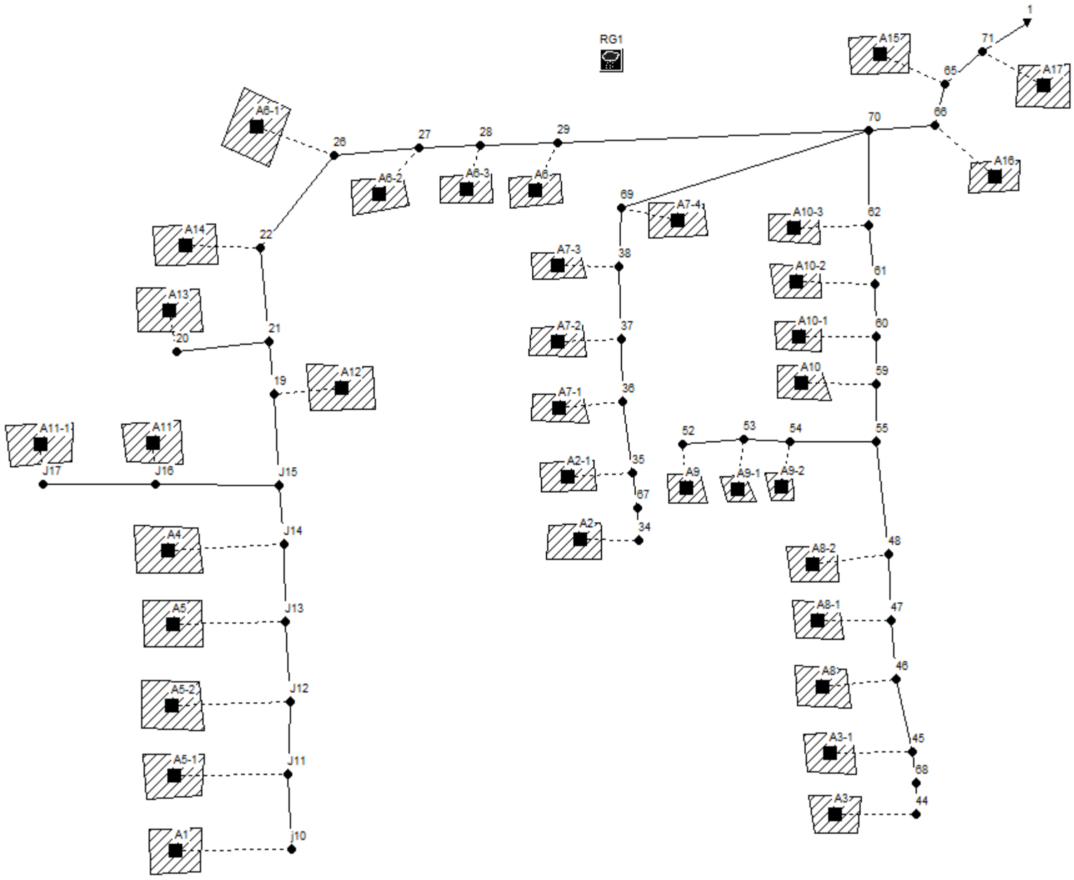


Figure A.1. Conceptual model for catchment A in SWMM.

Table A.1. Characteristic information for subareas in Subcatchment A.

| Subareas | Outlet | Area ha | %Imperv | Width m | %Slope | Subareas | Outlet | Area ha | %Imperv | Width m | %Slope |
|---|---|---|---|---|---|---|---|---|---|---|---|
| A5 | J13 | 1.66 | 40 | 154 | 1.5 | A7-3 | 38 | 0.59 | 90 | 114.4 | 1.1 |
| A5-1 | J11 | 0.75 | 5 | 168.5 | 0.4 | A3 | 44 | 7.14 | 3 | 250.4 | 0.67 |
| A1 | j10 | 3.52 | 3 | 69.39 | 1.6 | A3-1 | 45 | 1.53 | 7 | 176.7 | 0.85 |
| A4 | J14 | 3.24 | 30 | 126 | 0.9 | A8 | 46 | 1 | 50 | 162.4 | 0.6 |
| A5-2 | J12 | 1.5 | 17 | 136.8 | 0.7 | A8-1 | 47 | 0.48 | 45 | 102.5 | 0.3 |
| A11-1 | J17 | 0.3 | 85 | 59.37 | 0.7 | A8-2 | 48 | 0.35 | 47 | 120.8 | 0.2 |
| A11 | J16 | 0.24 | 90 | 146.8 | 0.2 | A9 | 52 | 0.31 | 35 | 46.21 | 0.92 |
| A12 | 19 | 0.4 | 90 | 159.8 | 1.5 | A9-1 | 53 | 0.3 | 85 | 108.5 | 0.18 |
| A13 | 20 | 0.7 | 60 | 60.19 | 1.03 | A9-2 | 54 | 0.21 | 90 | 101.8 | 0.65 |
| A14 | 22 | 0.19 | 85 | 73.6 | 0.9 | A10 | 59 | 0.43 | 85 | 62.7 | 0.21 |
| A6-1 | 26 | 2.7 | 30 | 102 | 0.97 | A10-1 | 60 | 0.72 | 35 | 88.3 | 0.5 |
| A6-2 | 27 | 0.92 | 80 | 39.8 | 0.71 | A10-2 | 61 | 0.44 | 60 | 75.5 | 0.7 |
| A6-3 | 28 | 0.46 | 70 | 70.1 | 0.73 | A10-3 | 62 | 0.39 | 80 | 114.1 | 1.4 |
| A6 | 29 | 0.34 | 91 | 94.7 | 0.72 | A7-4 | 69 | 0.57 | 88 | 148.1 | 1.5 |
| A2 | 34 | 13.03 | 5 | 234.38 | 1.12 | A15 | 65 | 2.2 | 80 | 90.82 | 0.92 |
| A2-1 | 35 | 3.05 | 20 | 166.6 | 0.8 | A16 | 66 | 0.51 | 70 | 133.4 | 1.15 |
| A7-1 | 36 | 0.34 | 80 | 178 | 0.98 | A17 | 71 | 0.7 | 10 | 159.6 | 0.15 |
| A7-2 | 37 | 1.18 | 83 | 123.9 | 0.8 | | | | | | |

Table A. 2. Data provided for modeling junctions in Subcatchment A in SWMM.

| Junctions | Invert Elev m | Max. Depth m | Junctions | Invert Elev m | Max. Depth m |
|---|---|---|---|---|---|
| j10 | 51.9 | 0.8 | 44 | 51.4 | 0.8 |
| J11 | 50.7 | 0.8 | 45 | 49.5 | 0.8 |
| J12 | 49.8 | 0.8 | 46 | 48.9 | 0.8 |
| J13 | 48.8 | 0.8 | 47 | 48.5 | 0.8 |
| J14 | 47.6 | 0.8 | 48 | 47.9 | 0.8 |
| J15 | 47.3 | 1.8 | 52 | 48.5 | 0.6 |
| J16 | 47.8 | 0.6 | 53 | 48.2 | 0.6 |
| J17 | 48.4 | 0.6 | 54 | 48 | 0.6 |
| 19 | 46.7 | 0.8 | 55 | 47.6 | 0.8 |
| 20 | 47 | 0.8 | 59 | 47.4 | 0.8 |
| 21 | 46.6 | 0.8 | 60 | 46.8 | 0.8 |
| 22 | 46.4 | 0.8 | 61 | 46 | 0.8 |
| 26 | 45.7 | 1.1 | 62 | 44.8 | 0.8 |
| 27 | 45.3 | 1.3 | 65 | 42.8 | 0.8 |
| 28 | 44.7 | 1.2 | 66 | 42.9 | 1.8 |
| 29 | 44.1 | 1.2 | 67 | 49.2 | 0.8 |
| 34 | 49.9 | 0.8 | 68 | 50.4 | 0.8 |
| 35 | 48.5 | 0.8 | 69 | 44.7 | 0.8 |
| 36 | 47.9 | 0.8 | 70 | 43.2 | 2 |
| 37 | 46.7 | 0.8 | 71 | 41.95 | 1.5 |
| 38 | 45.5 | 0.8 | | | |

Table A. 3. Data provided for modeling conduits in Subcatchment A in SWMM.

| Conduits | From Node | To Node | Length m | Pipe size mm | Shape | Roughness |
|---|---|---|---|---|---|---|
| 1 | j10 | J11 | 95.95 | 0.5 | CIRCULAR | 0.012 |
| 2 | J11 | J12 | 50.52 | 0.5 | CIRCULAR | 0.012 |
| 3 | J12 | J13 | 108.66 | 0.5 | CIRCULAR | 0.012 |
| 4 | J13 | J14 | 109.76 | 0.5 | CIRCULAR | 0.012 |
| 5 | J14 | J15 | 27.6 | 0.5 | CIRCULAR | 0.012 |
| 6 | J16 | J15 | 17.2 | 0.3 | CIRCULAR | 0.012 |
| 7 | J17 | J16 | 46.39 | 0.3 | CIRCULAR | 0.012 |
| 8 | J15 | 19 | 58.87 | 0.5 | CIRCULAR | 0.012 |
| 9 | 19 | 21 | 8.56 | 0.5 | CIRCULAR | 0.012 |
| 10 | 20 | 21 | 27.24 | 0.5 | CIRCULAR | 0.012 |
| 11 | 21 | 22 | 26.48 | 0.5 | CIRCULAR | 0.012 |
| 12 | 22 | 26 | 97.04 | 0.5 | CIRCULAR | 0.012 |
| 13 | 26 | 27 | 41.97 | 0.5 | CIRCULAR | 0.012 |
| 14 | 27 | 28 | 49.45 | 0.5 | CIRCULAR | 0.012 |
| 15 | 28 | 29 | 51.88 | 0.5 | CIRCULAR | 0.012 |
| 17 | 35 | 36 | 89.89 | 0.5 | CIRCULAR | 0.012 |
| 18 | 36 | 37 | 88.06 | 0.5 | CIRCULAR | 0.012 |
| 19 | 37 | 38 | 69.32 | 0.5 | CIRCULAR | 0.012 |
| 22 | 52 | 53 | 71.45 | 0.3 | CIRCULAR | 0.012 |
| 23 | 53 | 54 | 54.6 | 0.3 | CIRCULAR | 0.012 |
| 24 | 54 | 55 | 2.33 | 0.3 | CIRCULAR | 0.012 |
| 26 | 45 | 46 | 80.52 | 0.5 | CIRCULAR | 0.012 |
| 27 | 46 | 47 | 61.87 | 0.5 | CIRCULAR | 0.012 |
| 28 | 47 | 48 | 62.79 | 0.5 | CIRCULAR | 0.012 |
| 29 | 48 | 55 | 12.64 | 0.5 | CIRCULAR | 0.012 |
| 30 | 55 | 59 | 48.06 | 0.5 | CIRCULAR | 0.012 |
| 31 | 59 | 60 | 64.02 | 0.5 | CIRCULAR | 0.012 |
| 32 | 60 | 61 | 48.5 | 0.5 | CIRCULAR | 0.012 |
| 33 | 61 | 62 | 62.99 | 0.5 | CIRCULAR | 0.012 |
| 41 | 34 | 67 | 70.28 | 0.5 | CIRCULAR | 0.012 |
| 42 | 67 | 35 | 70.63 | 0.5 | CIRCULAR | 0.012 |
| 43 | 44 | 68 | 58.28 | 0.5 | CIRCULAR | 0.012 |
| 44 | 68 | 45 | 66.7 | 0.5 | CIRCULAR | 0.012 |
| 45 | 38 | 69 | 66 | 0.5 | CIRCULAR | 0.012 |
| 46 | 29 | 70 | 29 | 0.5 | CIRCULAR | 0.012 |
| 47 | 69 | 70 | 14.5 | 0.5 | CIRCULAR | 0.012 |
| 48 | 62 | 70 | 22.5 | 0.5 | CIRCULAR | 0.012 |
| 49 | 70 | 66 | 78 | 1 | CIRCULAR | 0.012 |
| 50 | 66 | 65 | 8 | 1 | CIRCULAR | 0.012 |
| 51 | 65 | 71 | 93 | 1 | CIRCULAR | 0.012 |
| 52 | 71 | 1 | 70 | 1 | CIRCULAR | 0.012 |

**Appendix B: Subcatchment B Input Characteristics for SWMM Model.**

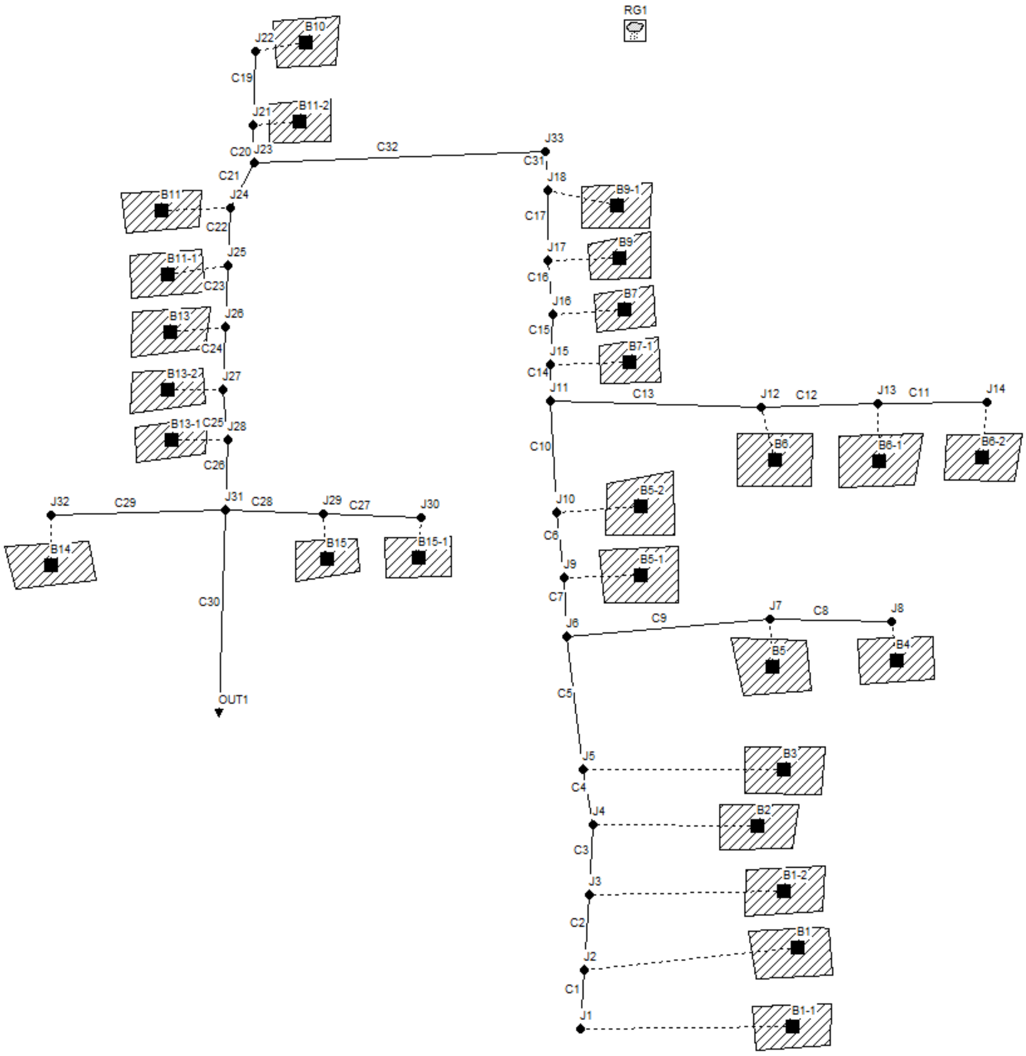


Figure B.1. Conceptual model for catchment B in SWMM.

Table B.1. Characteristic information for subareas in Subcatchment B.

| Subareas | Outlet | Area (ha) | %Imperv | Width (m) | %Slope |
|---|---|---|---|---|---|
| B6 | J12 | 0.3 | 90 | 95.6 | 1.04 |
| B1-1 | J1 | 1.5 | 15 | 138.48 | 1.1 |
| B1 | J2 | 0.93 | 80 | 60 | 0.98 |
| B1-2 | J3 | 0.59 | 40 | 48.2 | 1.11 |
| B2 | J4 | 0.65 | 60 | 141.2 | 2.5 |
| B3 | J5 | 0.66 | 80 | 150.9 | 1.79 |
| B4 | J8 | 0.56 | 80 | 78.52 | 1 |
| B5 | J7 | 0.3 | 90 | 144.2 | 2 |
| B5-1 | J9 | 0.13 | 90 | 49.5 | 1.32 |
| B5-2 | J10 | 0.21 | 90 | 102.2 | 1.11 |
| B6-1 | J13 | 0.44 | 80 | 64.6 | 1 |
| B6-2 | J14 | 0.67 | 70 | 66.65 | 1.24 |
| B7-1 | J15 | 0.1 | 100 | 107.2 | 0.66 |
| B7 | J16 | 0.43 | 95 | 45.9 | 1.2 |
| B9 | J17 | 0.41 | 90 | 44.77 | 1 |
| B9-1 | J18 | 0.5 | 85 | 119.3 | 0.36 |
| B10 | J22 | 1.1 | 80 | 121.77 | 1.75 |
| B11-2 | J21 | 0.19 | 90 | 68.2 | 0.3 |
| B11 | J24 | 0.7 | 80 | 57.61 | 0.8 |
| B11-1 | J25 | 0.62 | 80 | 79.1 | 1.58 |
| B13 | J26 | 0.45 | 70 | 137.6 | 1.52 |
| B13-2 | J27 | 0.65 | 60 | 70.36 | 3.2 |
| B13-1 | J28 | 0.9 | 30 | 127.6 | 1.75 |
| B14 | J32 | 0.91 | 30 | 161.5 | 0.05 |
| B15 | J29 | 0.44 | 60 | 144 | 0.16 |
| B15-1 | J30 | 0.61 | 60 | 138.7 | 0.71 |

Table B.2. Data provided for modeling junctions in SWMM for Subcatchment B.

| Junctions | Invert Elev | Max. Depth m | Junctions | Invert Elev | Max. Depth m |
|---|---|---|---|---|---|
| J1 | 48.532 | 0.8 | J17 | 43.29 | 1.1 |
| J2 | 48.085 | 0.8 | J18 | 42.8 | 1.1 |
| J3 | 47.719 | 0.8 | J21 | 42.213 | 0.8 |
| J4 | 46.151 | 0.8 | J22 | 42.323 | 0.8 |
| J5 | 45.669 | 0.8 | J23 | 41.91 | 1.1 |
| J6 | 45.594 | 0.8 | J24 | 41.37 | 1.3 |
| J7 | 45.632 | 0.8 | J25 | 40.63 | 1.15 |
| J8 | 45.873 | 0.8 | J26 | 39.13 | 1.2 |
| J9 | 44.948 | 0.8 | J27 | 38.75 | 1 |
| J10 | 44.326 | 0.8 | J28 | 36.98 | 1.3 |
| J11 | 44.06 | 1 | J29 | 37.487 | 0.8 |
| J12 | 44.504 | 0.6 | J30 | 37.656 | 0.8 |
| J13 | 45.002 | 0.6 | J31 | 36.84 | 1.4 |
| J14 | 45.421 | 0.6 | J32 | 37.478 | 0.8 |
| J15 | 43.66 | 1.2 | J33 | 42.74 | 1.1 |
| J16 | 43.41 | 1.2 | | | |

Table B.3. Data provided for modeling conduits in SWMM for Subcatchment B.

| Conduits | From Node | To Node | Length (m) | Pipe size (mm) | Shape | Roughness |
|---|---|---|---|---|---|---|
| C1 | J1 | J2 | 53.25 | 500 | Circular | 0.012 |
| C2 | J2 | J3 | 41.44 | 500 | Circular | 0.012 |
| C3 | J3 | J4 | 79.86 | 500 | Circular | 0.012 |
| C4 | J4 | J5 | 79.74 | 500 | Circular | 0.012 |
| C5 | J5 | J6 | 21.52 | 500 | Circular | 0.012 |
| C6 | J9 | J10 | 57.52 | 500 | Circular | 0.012 |
| C7 | J6 | J9 | 45.84 | 500 | Circular | 0.012 |
| C8 | J8 | J7 | 67.38 | 500 | Circular | 0.012 |
| C9 | J7 | J6 | 5.49 | 500 | Circular | 0.012 |
| C10 | J10 | J11 | 49 | 500 | Circular | 0.012 |
| C11 | J14 | J13 | 42.29 | 300 | Circular | 0.012 |
| C12 | J13 | J12 | 53.72 | 300 | Circular | 0.012 |
| C13 | J12 | J11 | 3.8 | 300 | Circular | 0.012 |
| C14 | J11 | J15 | 48.85 | 500 | Circular | 0.012 |
| C15 | J15 | J16 | 39.7 | 500 | Circular | 0.012 |
| C16 | J16 | J17 | 15.99 | 500 | Circular | 0.012 |
| C17 | J17 | J18 | 76.41 | 500 | Circular | 0.012 |
| C19 | J22 | J21 | 64.38 | 500 | Circular | 0.012 |
| C20 | J21 | J23 | 4.74 | 500 | Circular | 0.012 |
| C21 | J23 | J24 | 30.48 | 500 | Circular | 0.012 |
| C22 | J24 | J25 | 58.64 | 500 | Circular | 0.012 |
| C23 | J25 | J26 | 87.3 | 500 | Circular | 0.012 |
| C24 | J26 | J27 | 19.72 | 500 | Circular | 0.012 |
| C25 | J27 | J28 | 85.04 | 500 | Circular | 0.012 |
| C26 | J28 | J31 | 3.76 | 500 | Circular | 0.012 |
| C27 | J30 | J29 | 65.43 | 500 | Circular | 0.012 |
| C28 | J29 | J31 | 10.88 | 500 | Circular | 0.012 |
| C29 | J32 | J31 | 7.06 | 500 | Circular | 0.012 |
| C30 | J31 | OUT1 | 60 | 500 | Circular | 0.012 |
| C31 | J18 | J33 | 4 | 500 | Circular | 0.012 |
| C32 | J33 | J23 | 107.93 | 400×600 | Rectangular | 0.013 |

## Appendix C: Subcatchment C Input Characteristics for SWMM Model.

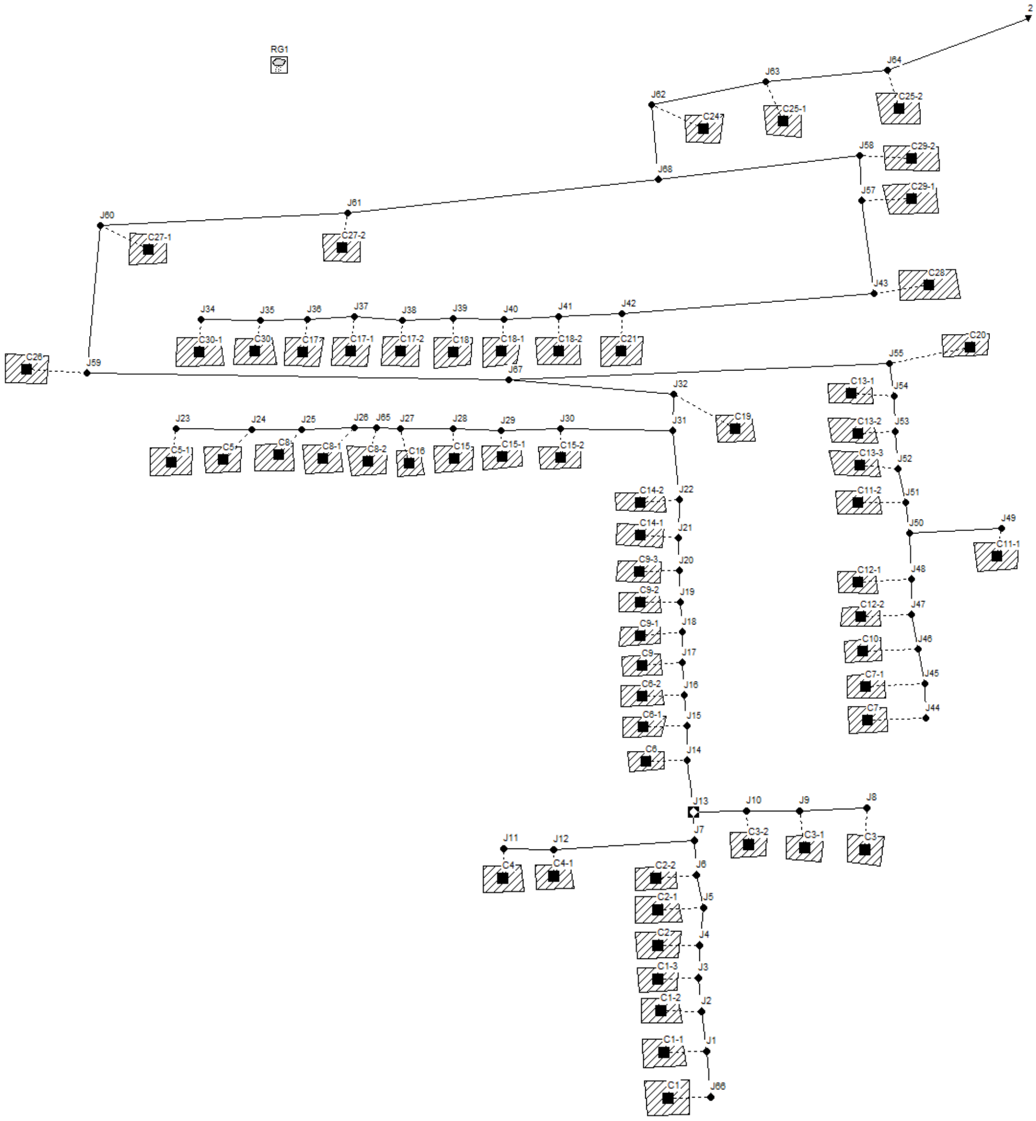

Figure C.1.Conceptual model for catchment C in SWMM.

Table C.1. Characteristic information for subareas in Subcatchment C part 1.

| Subareas | Outlet | Area (ha) | %Imperv | Width | %Slope |
|---|---|---|---|---|---|
| C1-1 | J1 | 0.26 | 80 | 124.4 | 1.35 |
| C1-2 | J2 | 0.16 | 70 | 61.7 | 1.5 |
| C1-3 | J3 | 0.17 | 75 | 83.2 | 0.93 |
| C2 | J4 | 0.39 | 85 | 87.5 | 0.91 |
| C2-1 | J5 | 0.44 | 80 | 90.7 | 1.66 |
| C2-2 | J6 | 0.51 | 60 | 106.7 | 1.1 |
| C4-1 | J12 | 0.22 | 80 | 94.8 | 1.07 |
| C4 | J11 | 0.27 | 38 | 50 | 0.43 |
| C3-2 | J10 | 0.17 | 88 | 72.2 | 0.31 |
| C3-1 | J9 | 0.31 | 85 | 54.2 | 0.07 |
| C3 | J8 | 0.34 | 35 | 36.3 | 1.38 |
| C6 | J14 | 0.23 | 60 | 74.6 | 1.7 |
| C6-1 | J15 | 0.25 | 60 | 52.1 | 1.32 |
| C6-2 | J16 | 0.18 | 95 | 65.7 | 1.5 |
| C9 | J17 | 0.61 | 75 | 47.8 | 0.12 |
| C9-1 | J18 | 0.53 | 45 | 68.1 | 0.05 |
| C9-2 | J19 | 0.32 | 70 | 80.3 | 0.33 |
| C9-3 | J20 | 0.22 | 68 | 52.3 | 0.06 |
| C14-1 | J21 | 0.12 | 70 | 58.2 | 0.56 |
| C14-2 | J22 | 0.11 | 75 | 78.4 | 0.1 |
| C15-2 | J30 | 0.1 | 30 | 54.9 | 0.04 |
| C15-1 | J29 | 0.29 | 35 | 23.5 | 0.11 |
| C15 | J28 | 0.08 | 95 | 89.2 | 0.08 |
| C8-2 | J65 | 0.4 | 90 | 88.6 | 0.42 |
| C8-1 | J26 | 0.34 | 60 | 115.1 | 1.1 |
| C8 | J25 | 0.4 | 85 | 173.1 | 0.7 |
| C5 | J24 | 0.67 | 30 | 70.7 | 1.04 |
| C5-1 | J23 | 0.51 | 33 | 76.1 | 1.21 |
| C30-1 | J34 | 0.63 | 65 | 112.5 | 0.2 |
| C30 | J35 | 1.19 | 70 | 75.7 | 0.49 |

Table C.2. Characteristic information for subareas in Subcatchment C part 2.

| Subareas | Outlet | Area (ha) | %Imperv | Width | %Slope |
|---|---|---|---|---|---|
| C17 | J36 | 0.8 | 20 | 91.1 | 0.45 |
| C17-1 | J37 | 1.7 | 35 | 59.8 | 0.57 |
| C17-2 | J38 | 0.47 | 70 | 42.6 | 0.5 |
| C18 | J39 | 0.79 | 70 | 55.6 | 0.66 |
| C18-1 | J40 | 0.25 | 80 | 87.9 | 0.79 |
| C18-2 | J41 | 0.65 | 75 | 64.62 | 1.23 |
| C19 | J32 | 0.65 | 30 | 118.4 | 0.55 |
| C21 | J42 | 0.45 | 85 | 206.3 | 0.41 |
| C28 | J43 | 0.23 | 10 | 121.9 | 0.2 |
| C7 | J44 | 0.79 | 80 | 120.86 | 0.52 |
| C7-1 | J45 | 0.36 | 90 | 79.7 | 2.1 |
| C10 | J46 | 0.39 | 95 | 76.8 | 2.1 |
| C12-2 | J47 | 0.16 | 90 | 12.3 | 0.82 |
| C12-1 | J48 | 0.13 | 85 | 45.9 | 0.89 |
| C11-1 | J49 | 0.27 | 50 | 487 | 0.11 |
| C11-2 | J51 | 0.07 | 90 | 126.4 | 1.76 |
| C13-3 | J52 | 0.13 | 70 | 58.8 | 0.41 |
| C13-2 | J53 | 0.23 | 70 | 42 | 0.62 |
| C13-1 | J54 | 0.24 | 85 | 81.8 | 0.01 |
| C20 | J55 | 0.64 | 70 | 188.2 | 0.37 |
| C29-1 | J57 | 0.65 | 30 | 65.1 | 0.64 |
| C29-2 | J58 | 0.36 | 65 | 90.3 | 0.33 |
| C26 | J59 | 0.19 | 55 | 134.6 | 0.8 |
| C27-1 | J60 | 0.46 | 65 | 48 | 0.47 |
| C27-2 | J61 | 0.31 | 70 | 124.6 | 0.65 |
| C24 | J62 | 2.3 | 70 | 122.1 | 0.96 |
| C25-1 | J63 | 0.8 | 70 | 114.2 | 1.27 |
| C25-2 | J64 | 0.73 | 25 | 109.2 | 2.45 |
| C16 | J27 | 0.6 | 35 | 114.3 | 52.65 |
| C1 | J66 | 1.05 | 70 | 66.63 | 1.26 |

Table C.3. Data provided for modeling junctions in Subcatchment C in SWMM.

| Junctions | Invert Elev | Max. Depth | Junctions | Invert Elev | Max. Depth |
|---|---|---|---|---|---|
| J1 | 49.1 | 0.8 | J40 | 43.13 | 0.9 |
| J10 | 46.2 | 0.8 | J41 | 43.01 | 0.9 |
| J11 | 47.3 | 0.8 | J42 | 42.65 | 0.8 |
| J12 | 46.5 | 0.8 | J43 | 42.28 | 0.8 |
| J13 | 46.1 | 0.8 | J44 | 45.01 | 0.8 |
| J14 | 45.9 | 0.8 | J45 | 44.64 | 0.8 |
| J15 | 45.7 | 0.8 | J46 | 44.18 | 0.7 |
| J16 | 45.3 | 0.8 | J47 | 44.15 | 0.8 |
| J17 | 44.9 | 0.8 | J48 | 44.08 | 0.9 |
| J18 | 44.5 | 0.8 | J49 | 44.34 | 0.7 |
| J19 | 43.9 | 0.8 | J5 | 47.1 | 0.8 |
| J2 | 48.8 | 0.8 | J50 | 44.07 | 0.8 |
| J20 | 43.65 | 1.4 | J51 | 43.93 | 1.1 |
| J21 | 43.3 | 0.8 | J52 | 43.68 | 1.7 |
| J22 | 43 | 0.8 | J53 | 43.43 | 1.1 |
| J23 | 45.9 | 0.7 | J54 | 43.05 | 0.9 |
| J24 | 45.8 | 0.7 | J55 | 42.29 | 0.8 |
| J25 | 45 | 0.7 | J57 | 41.97 | 0.9 |
| J26 | 44.5 | 0.7 | J58 | 41.43 | 1 |
| J27 | 43.93 | 0.8 | J59 | 41.21 | 1.6 |
| J28 | 43.7 | 0.7 | J6 | 46.5 | 0.8 |
| J29 | 43.5 | 0.7 | J60 | 41.06 | 1.52 |
| J3 | 48.3 | 0.8 | J61 | 40.77 | 1.5 |
| J30 | 43.2 | 0.7 | J62 | 40 | 1.5 |
| J31 | 42.8 | 1 | J63 | 37.91 | 1.3 |
| J32 | 42.32 | 1.1 | J64 | 36.69 | 1.3 |
| J34 | 43.82 | 0.8 | J65 | 44 | 0.7 |
| J35 | 43.72 | 0.8 | J66 | 49.7 | 0.8 |
| J36 | 43.6 | 0.9 | J67 | 41.5 | 2.1 |
| J37 | 43.52 | 0.9 | J68 | 40.27 | 1.8 |
| J38 | 43.42 | 0.9 | J7 | 46.3 | 0.8 |
| J39 | 43.28 | 0.9 | J8 | 46.4 | 0.8 |
| J4 | 47.8 | 0.8 | J9 | 46.3 | 0.8 |

Table C.4.Data provided for modeling conduits in Subcatchment C in SWMM part1.

| Conduits | From Node | To Node | Length (m) | Pipe size (mm) | Shape | Roughness |
|---|---|---|---|---|---|---|
| 1 | J1 | J2 | 35.98 | 0.5 | CIRCULAR | 0.012 |
| 2 | J2 | J3 | 46.6 | 0.5 | CIRCULAR | 0.012 |
| 3 | J3 | J4 | 49 | 0.5 | CIRCULAR | 0.012 |
| 4 | J4 | J5 | 52.83 | 0.5 | CIRCULAR | 0.012 |
| 5 | J5 | J6 | 61.39 | 0.5 | CIRCULAR | 0.012 |
| 6 | J6 | J7 | 10.99 | 0.5 | CIRCULAR | 0.012 |
| 7 | J7 | J13 | 20.37 | 0.5 | CIRCULAR | 0.012 |
| 8 | J11 | J12 | 56.66 | 0.5 | CIRCULAR | 0.012 |
| 9 | J12 | J7 | 8.88 | 0.5 | CIRCULAR | 0.012 |
| 10 | J8 | J9 | 38.7 | 0.5 | CIRCULAR | 0.012 |
| 11 | J9 | J10 | 37 | 0.5 | CIRCULAR | 0.012 |
| 12 | J10 | J13 | 12 | 0.5 | CIRCULAR | 0.012 |
| 13 | J23 | J24 | 60.5 | 0.4 | CIRCULAR | 0.012 |
| 14 | J24 | J25 | 62 | 0.4 | CIRCULAR | 0.012 |
| 15 | J25 | J26 | 60.5 | 0.4 | CIRCULAR | 0.012 |
| 17 | J27 | J28 | 45 | 0.4 | CIRCULAR | 0.012 |
| 18 | J28 | J29 | 21 | 0.4 | CIRCULAR | 0.012 |
| 19 | J29 | J30 | 32.5 | 0.4 | CIRCULAR | 0.012 |
| 20 | J30 | J31 | 14 | 0.4 | CIRCULAR | 0.012 |
| 21 | J13 | J14 | 27.6 | 0.5 | CIRCULAR | 0.012 |
| 22 | J14 | J15 | 37 | 0.5 | CIRCULAR | 0.012 |
| 23 | J15 | J16 | 37.8 | 0.5 | CIRCULAR | 0.012 |
| 24 | J16 | J17 | 38 | 0.5 | CIRCULAR | 0.012 |
| 25 | J17 | J18 | 46.3 | 0.5 | CIRCULAR | 0.012 |
| 26 | J18 | J19 | 51.5 | 0.5 | CIRCULAR | 0.012 |
| 27 | J19 | J20 | 33.25 | 0.5 | CIRCULAR | 0.012 |
| 28 | J20 | J21 | 36.8 | 0.5 | CIRCULAR | 0.012 |
| 29 | J21 | J22 | 35.3 | 0.5 | CIRCULAR | 0.012 |
| 30 | J22 | J31 | 6.5 | 0.5 | CIRCULAR | 0.012 |
| 31 | J34 | J35 | 54.8 | 0.5 | CIRCULAR | 0.012 |
| 32 | J35 | J36 | 78.5 | 0.5 | CIRCULAR | 0.012 |
| 33 | J36 | J37 | 51.44 | 0.5 | CIRCULAR | 0.012 |
| 34 | J37 | J38 | 36.8 | 0.5 | CIRCULAR | 0.012 |

Table C.5.Data provided for modeling conduits in Subcatchment C in SWMM part2.

| Conduits | From Node | To Node | Length (m) | Pipe size (mm) | Shape | Roughness |
|---|---|---|---|---|---|---|
| 35 | J38 | J39 | 53.2 | 0.5 | CIRCULAR | 0.012 |
| 36 | J39 | J40 | 49 | 0.5 | CIRCULAR | 0.012 |
| 37 | J40 | J41 | 22.3 | 0.5 | CIRCULAR | 0.012 |
| 38 | J41 | J42 | 90.5 | 0.5 | CIRCULAR | 0.012 |
| 40 | J31 | J32 | 40 | 0.5 | CIRCULAR | 0.012 |
| 43 | J44 | J45 | 44 | 0.5 | CIRCULAR | 0.012 |
| 44 | J45 | J46 | 45.5 | 0.5 | CIRCULAR | 0.012 |
| 45 | J46 | J47 | 12.3 | 0.5 | CIRCULAR | 0.012 |
| 46 | J47 | J48 | 25.5 | 0.5 | CIRCULAR | 0.012 |
| 47 | J48 | J50 | 10.3 | 0.5 | CIRCULAR | 0.012 |
| 48 | J49 | J50 | 9.7 | 0.5 | CIRCULAR | 0.012 |
| 49 | J50 | J51 | 20.3 | 0.5 | CIRCULAR | 0.012 |
| 50 | J51 | J52 | 30.4 | 0.5 | CIRCULAR | 0.012 |
| 51 | J52 | J53 | 35.4 | 0.5 | CIRCULAR | 0.012 |
| 52 | J53 | J54 | 47 | 0.5 | CIRCULAR | 0.012 |
| 53 | J54 | J55 | 95 | 0.5 | CIRCULAR | 0.012 |
| 57 | J57 | J58 | 61 | 0.5 | CIRCULAR | 0.012 |
| 59 | J59 | J60 | 28.5 | 1 | CIRCULAR | 0.012 |
| 60 | J60 | J61 | 66.5 | 1 | CIRCULAR | 0.012 |
| 62 | J62 | J63 | 76.2 | 1 | CIRCULAR | 0.012 |
| 63 | J63 | J64 | 67 | 1 | CIRCULAR | 0.012 |
| 64 | J64 | 2 | 55 | 1 | CIRCULAR | 0.012 |
| 65 | J26 | J65 | 46 | 0.4 | CIRCULAR | 0.012 |
| 66 | J65 | J27 | 15.5 | 0.4 | CIRCULAR | 0.012 |
| 67 | J66 | J1 | 62.51 | 0.5 | CIRCULAR | 0.012 |
| 69 | J42 | J43 | 74 | 0.5 | CIRCULAR | 0.012 |
| 70 | J55 | J67 | 67.3 | 0.5 | CIRCULAR | 0.012 |
| 72 | J67 | J59 | 81.3 | 1 | CIRCULAR | 0.012 |
| 73 | J61 | J68 | 33 | 1 | CIRCULAR | 0.012 |
| 74 | J58 | J68 | 39.3 | 0.5 | CIRCULAR | 0.012 |
| 75 | J68 | J62 | 42 | 1 | CIRCULAR | 0.012 |
| 76 | J43 | J57 | 35.2 | 0.5 | CIRCULAR | 0.012 |
| 77 | J32 | J67 | 20 | 0.5 | CIRCULAR | 0.012 |

## Appendix D: Subcatchment D Input Characteristics for SWMM Model.

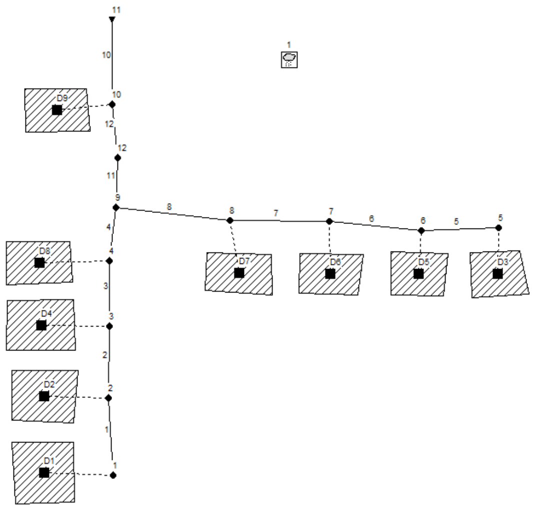

Figure D.1. Conceptual model for catchment D in SWMM.

Table D.1. Characteristic information for subareas in Subcatchment D.

| Subareas | Outlet | Area (ha) | %Imperv | Width | %Slope |
|---|---|---|---|---|---|
| D1 | 1 | 2.3 | 80 | 110.7 | 1.3 |
| D2 | 2 | 0.4 | 85 | 50.2 | 1.5 |
| D4 | 3 | 0.33 | 40 | 106 | 1 |
| D8 | 4 | 0.4 | 50 | 88 | 2.3 |
| D3 | 5 | 1.37 | 60 | 104.2 | 0.4 |
| D5 | 6 | 0.39 | 10 | 66.6 | 1.1 |
| D6 | 7 | 0.4 | 70 | 48.4 | 0.7 |
| D7 | 8 | 0.23 | 80 | 97.2 | 0.5 |
| D9 | 10 | 2.05 | 10 | 219.8 | 1.2 |

Table D.2. Data provided for modeling junctions in Subcatchment D in SWMM.

| Junctions | Invert Elev | Max. Depth |
|---|---|---|
| 1 | 43.3 | 0.8 |
| 2 | 42.6 | 1.1 |
| 3 | 41.6 | 0.8 |
| 4 | 40.3 | 0.8 |
| 5 | 42.5 | 0.8 |
| 6 | 42.4 | 1 |
| 7 | 42.3 | 1 |
| 8 | 40.6 | 0.8 |
| 9 | 40.1 | 0.8 |
| 10 | 38.6 | 0.9 |
| 12 | 39.2 | 0.8 |

Table D.3. Data provided for modeling conduits in Subcatchment C in SWMM part2.

| Conduits | From Node | To Node | Length m | Pipe size mm | Shape | Roughness |
|---|---|---|---|---|---|---|
| 1 | 1 | 2 | 49 | 0.5 | CIRCULAR | 0.012 |
| 2 | 2 | 3 | 55 | 0.5 | CIRCULAR | 0.012 |
| 3 | 3 | 4 | 59 | 0.5 | CIRCULAR | 0.012 |
| 4 | 4 | 9 | 10 | 0.5 | CIRCULAR | 0.012 |
| 5 | 5 | 6 | 39.5 | 0.5 | CIRCULAR | 0.012 |
| 6 | 6 | 7 | 28 | 0.5 | CIRCULAR | 0.012 |
| 7 | 7 | 8 | 48 | 0.5 | CIRCULAR | 0.012 |
| 8 | 8 | 9 | 11 | 0.5 | CIRCULAR | 0.012 |
| 10 | 10 | 11 | 60 | 0.5 | CIRCULAR | 0.012 |
| 11 | 9 | 12 | 60 | 0.5 | CIRCULAR | 0.012 |
| 12 | 12 | 10 | 79 | 0.5 | CIRCULAR | 0.012 |