AN INTEGRATED APPROACH TO WATER MANAGEMENT IN KAYSERI: RAIN WATER COLLECTION AND STORAGE DESIGN FOR KAYSERI HARIKALAR DIYARI WATER-SKI PARK

by

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ABSTRACT

Water management in metropolitan areas in developing countries is a major environmental challenge of the future. Turkey's position for increased regional leadership on water issues is gaining acceptance through developing regions of Central Asia, Middle East and Africa. Concepts and technologies about integration and vision must be implemented to gain broader acceptance and relevance. Within this study; current water and wastewater management systems are investigated for Kayseri, with the purpose of bringing water into the center of city life by a reformed proposed pilot project.

In this study, ways of utilizing water as a central feature of Kayseri urban design within the frame of the IWA Cities of the Future Project was investigated. A water-ski park was designed, located in a recreational area called the Kayseri Harikalar Diyarı. Water in the park was rain and storm water collected in tanks and circulated continuously. In order to both assess the quantity and dynamics of water flow through the Kayseri Harikalar Diyarı Water-Ski Park, Environmental Protection Agency's (EPA) Storm Water Management Model (SWMM) was used. The evaporation rates, temperature and snow melt data were introduced to the model as input parameters. The model simulated the water quantity and response to the events that will occur continuously for a 3-year rainfall record.

Simulations were conducted in the EPA SWMM 5.0 to assess a proposed triangular open channel with a length of 2147 m and a slope of 0.3%. Two underground storage tanks, each with a volume of 50m^3 were designed for storm/rain water collection.

The obtained results demonstrated that after three years, 38,300 m^3 rain water could be stored in the two proposed underground storage tanks. 43% of the water-ski lake with a volume of 90,000m^3, could be recharged with the stored rain water. According to World Health Organization (WHO) guidelines for water reuse for different purposes of water and EU Directive for Bathing Water (2006/7/EC), the rain water analyses conducted in KASKI laboratories showed that the rainwater quality objectives met requirements for use for both irrigation and bathing water purposes.

ÖZET

Gelişmekte olan ülkelerde, büyük şehirlerin su yönetim stratejileri, şüphesiz ki geleceğin en büyük sorunudur. Türkiye'nin su sorunları karşısında ortaya koyduğu lider tutum, Orta Asya, Orta Doğu ve Afrika'da gelişmekte olan bölgeler tarafından kabul görmektedir. Kavram ve teknolojilerinin geliştirilmesi ve uygulanması geniş bir bakış açısı kazanılması ve bütüncül yaklaşımların kabul görmesi açısından zorunludur. Bu çalışma ile birlikte, Kayseri şehrinin ilgili su ve atıksu problemleri ortaya konularak, pilot bir proje geliştirilmiş ve suyun bütüncül bir yaklaşımla, yaşamın merkezine getirilmesi sağlanmıştır.

Bu çalışmada, IWA Geleceğin Şehirleri Projesi çerçevesinde suyun faydalı kullanımının Kayseri'nin kentsel tasarımının ana parçası olabilmesi için çözümler araştırılmıştır. Çalışma kapsamında, Kayseri Harikalar Diyarı Su-kay Parkı'nın son üç yıllık yağmur suyu miktarı benzetimi yapılmıştır. Kayseri Harikalar Diyarı Su-kay Park'ta akışa geçecek olan suyun miktarının ve dinamik özelliklerinin değerlendirilmesi EPA SWMM 5.0 kullanılarak sağlanmıştır. Buharlaşma miktarları, sıcaklık ve akışa geçen eriyen kar suyu miktarı modele tanıtılmıştır. Sonraki adımda modelde, üç yıllık yağmur suyu kayıtlarına göre oluşacak yağış olaylarında toplanacak net su miktarı belirlenmiştir.

EPA SWMM 5.0 benzetim sonuçlarıyla 2147 m uzunluğunda ve 0.3% eğime sahip olan üçgen kesitli açık kanal değerlendirilmiştir. Bu benzetimle iki adet yeraltı su deposunda toplanacak yağmur suyu miktarı belirlenmiştir.

Üç yılın sonunda, iki adet yeraltı su deposunda elde edilen yağmur suyu miktarı 38300 m^3'tür. Bu su ile 90000 m^3 hacme sahip olan Su-kay Parkı'nın 48%'i doldurulabilir durumdadır. Dünya Su Örgütü'nün belirlediği farklı amaçlar için suyun tekrar kullanımı kılavuzuna ve Avrupa Birliği yüzme suyu direktifine göre (2006/7EC), KASKI'de gerçekleştirilen yağmur suyu analiz sonuçlarıyla beraber, yağmur suyunun belirtilen kalite amaçları doğrultusunda sulama ve yüzme suyu kullanımı için uygun olduğu görülmüştür.

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LIST OF SYMBOLS/ABBREVIATIONS

Symbols	**Explanation**	**Unit Used**
DSI	State Hydraulic Works	-
DMI	State Meteorological Works	-
EPA SWMM	Environmental Protection Agency	-
	Storm Water Management	
KASKI	Kayseri Metropolitan Municipality Water and Sewerage	-
	Administration General Directorate	-
IMS	Integrated Management Strategy	-
IWRM	Integrated Water Resources Management	-
MoEF	Ministry of Environment and Forestry	-
A	Cross-Sectional Area	m^2
b	Open Channel Bottom Width	m
Es	Water Energy Line	m
F	Infiltration Rate	mm
Fr	Froud's Number	-
i	Rainfall Intensity	mm/h
K_T	Trapezoidal Open Channel Conveyance Factor	-
n	Manning's Roughness Coefficient	-
P	Wetted Perimeter	m
Q	Runoff Rate	m^3/s
R	Hydraulic Radius	m
S	Slope	m/m
H	Hydraulic Head	m

1. INTRODUCTION

Global climate change and increase in human population have affected water resources and fresh water ecosystems throughout the world. Water management worldwide is facing great challenges, due to continual and growing pressure on water systems. Areas which show increased precipitation variability are going to face seasonal runoff shifts on water supply, water quality and runoff risks. In this context, planning, developing, distributing and managing the optimum use of resources come into prominence. An integrated approach has to be developed in order to control the management problem which tends to be magnified every year (Braga et al., 2006).

Global water challenges will pose more severe risks in especially metropolitan areas. Many cities today face the dual challenge of securing access to safe water for their urban peoples; cutting down on wasteful and illegal usage. Most major water based problems that humans will directly experience from climate change are increased flooding, increased drought, and increased sea levels (Yu, 2008).

In recent years, researches have been focused on the integrated management strategies in order to develop sustainable feasible models for water management. Integration is the general term used to indicate all of the methods that are used for management of water strategies. Integrated Water Resources Management (IWRM) approach which upholds the implementation of environmental and economic principles at the water catchment level is imposed by the evolution of water problems in the industrialized world. The IWRM approach includes a combined, holistic management of ground and surface waters, paying due attention to water and land resources use in a specific catchment area (Billib et al., 2009).

Turkey, being the accession country to the European Union (EU), undertakes various obligations in the adaptation process to the Union. Since the late 1990s, the EU has embarked on the implementation of the IWRM within its political geography, taking into account the available water amounts (Table 1.1) and current management systems.

Table 1.1. Available water amounts for Turkey (DMI, 2011).

Annual available surface water amount	98 billion m^3
Annual available groundwater amount	14 billion m^3
Total available water amount	112 billion m^3

In Turkey, the Ministry of Environment and Forestry (MoEF) has all the responsibilities of water resources management. It regulates the use of water, prevents its waste, conserves consumption, levies and collects tariffs, and gives water extraction permits. The table below reflects use and demand amounts of water for public supply.

Table 1.2. Water usage areas (DMI, 2011).

Area of Usage	Value
Irrigation	29.5 billion m^3
Drinking water	6.2 billion m^3
Industry	4.3 billion m^3
Total	40 billion m^3

In this respect, a methodology is to be developed in Turkey, to serve as a guide for all Turkish cities with similar issues and opportunities. A repeatable methodology will be developed during the project called “Turkish Cities of the Future” which is going to guide cities throughout the world (Taşlı, 2010a). In this study, a solution which encompasses use of different grades of water is going to be proposed for a specific area in Kayseri. A case study will be held in the context of Cities of the Future Project. In a City of the Future context, land planners and land development entities will make informed choices that use and reuse water most efficiently while protecting source and receiving waters (Moddemeyer, 2010).

Within the scope of the Cities of the Future Program, integrated water management plans will involve infrastructure and land use approaches for Kayseri. The main problems and the subjects related to the integrated water and wastewater management systems in Kayseri are presented in Table 1.3.

Table 1.3. Cities of the Future Kayseri, Problems and Subjects (Taşlı, 2010b).

Problems in Kayseri	Subjects
• Water Supply: Irresponsive to Drought • Uncertainty: Seasonal changes • Since the precipitate is in the form of rain instead of snow, more surface flow & less leakage is present. • Decrease in groundwater level • Storm water management	• Adaptation to climate change • Protection of Water Resources • Recharge of groundwater
• Flooding • Combined sewerage system • For whole water systems, combined sewerage systems are in a bad condition	• Water Resistant City Design • Storm water management
• Resource efficiency • Energy • Daily Timing Schedule affects management • Management of Biosolids • Primary Sludge Digestion, Mixing with treatment sludge, Dewatering, Open Disposal • Unsanitary Disposal of Solid Waste & Biosolids	• Energy Efficiency • Biosolids Management • Energy Supply • Intersections with Solid Waste Management
• Efficient Management • Non-revenue water • Regional Decrease Target: 10%, Total Target :%10 • Waste Trade • Effects on WWTPs • Separate Treatment of the most important discharges • Reaction to drought	• Management of ministrations • Non-revenue water • Industrial Management

2. THEORETICAL BACKGROUND

2.1. Integrated Management Strategy

2.1.1. Definition and General Implementations

Integrated Management Strategy (IMS) includes flood prevention; coastal pollution; urban planning and management; housing and infrastructure development; water conservation along with other social and economic matters, such as public health, recreational activities, property values, and tourism.

Integrated management system requirements (Figure 2.1) involve curb natural resource consumption, reduction of pollution; protection of the environment and its ecosystems, encouragement for sustainable economic growth, ensuring the equal allocation of public resources. This integrated strategy must be oriented towards long-term goals and should engage public participation to support social equity, economic welfare and environmental values of sustainable urban development.

Figure 2.1. Schema for integrated system requirements (USGS, 2011).

Effective water management combines economic concepts and methods with engineering and hydrologic expertise (Heinz et al., 2007).

The integrated water management approach, including urban planning, urban development and urban waste management, should be adjusted accordingly (Yu, 2008).

There are mainly 5 points where it is urgently necessary to attach water resources management to urban management (Braga et al., 2006):

- Inter-basin Water Transfers
- Flood Control
- Wastewater Collection and Treatment
- Water Supply
- Conservation and Rational Water Use

Conservation of the rural areas is fundamental for the water security of Kayseri since it has a direct bearing on groundwater recharge.

If an integrated strategy is put in place, large quantities of urban storm water could be properly collected and stored during periods of heavy rain. This could prevent urban flooding, mitigate rainwater pressure on the sewage system, and reduce wet weather sewage overflows.

2.1.2. Storm Water Management

The water that falls on the urban area as rain is regarded as a ‘problem’ so drainage systems have to be developed in order to drain the catchments on which the cities are built with the storm water drained to a waterway. Combined systems lose storm water as a resource since treatment is required. By means of separated storm water systems, the use of urban storm water could provide an effective solution to water-related problems. This means changing the way the resource is managed, not merely discharging it.

Arguably, urban storm water, recycling effluent and other resources will have to be included in the overall mix of water saving, supplementing, measures in the very near future. Home developers are required to use less water on indoor/outdoor plants, install rainwater tanks for toilet flushing and garden use, and fit water saving showerheads, taps and dual-flush toilets (Yu, 2008).

When storm water is left uncontrolled or is controlled inadequately, increases of the magnitude and duration of runoff cause downstream flooding, accelerate channel erosion, and impair aquatic habitat (Roesner et al., 2001).

Increases in the magnitude and duration of storm-water runoff that accompany uncontrolled development allow a stream to carry more sediment than it could prior to watershed development. When a watershed cannot supply the stream with the volume of sediment it has the capacity to carry, channel degradation may occur in the form of incision lateral migration, or a combination of both.

In-stream erosion potential due to development can be evaluated with continuous simulation and through identification of critical shear stress values. Regional storm-water controls designed to allow a match of the post development peak flow frequency exceedance to predevelopment values and to minimize erosion potential based upon local stream erosion characteristics (Pomeroy et al., 2008).

Typical basic storm water management systems are designed to accomplish the following seven foundational goals:

- ✓ Protect life and health
- ✓ Minimize property losses
- ✓ Enhance floodplain use
- ✓ Ensure a functional drainage system
- ✓ Protect and enhance the environment
- ✓ Encourage aesthetics

In this respect, a repeatable management strategy for a specific case in Kayseri is going to be proposed while ensuring a functional drainage system for storm water is introduced to the catchment area and encouraging aesthetics at the area.

2.1.3. Aquifer Management

Turkey has important groundwater renewable resources, average of 14 billion cubic meters. (Table 2.1). Groundwater has been increasingly used by farmers (especially by small holders) but also by cities and industries that consider it a strategic resource.

Table 2.1. Available water amounts for Turkey (DMI, 2011).

Annual available surface water amount	98	billion m^3
Annual available groundwater amount	14	billion m^3
Total available water amount	112	billion m^3

Generally, groundwater development has had positive economic and social impacts. However, in some cases, it has caused negative impacts such as depletion of hydraulic head, groundwater quality degradation, subsidence or land collapse, interference with other water developments, and adverse impact on aquatic ecosystems.

While the problems and causes of aquifer depletion and contamination are clear, immediate solutions are not. Water professionals agree, however, that active aquifer management must be undertaken in the wider context of watershed management. Most solutions involve some combination of increased recharge rate, reduced consumption rate, overall efficiency gains, and reduced or eliminated contaminant sources. (USAID, 2011).

2.1.3.1. Recharge of Aquifers Used for Non-potable Purposes. Where reclaimed water is recharged into non-potable aquifers and there is no possibility of the water migrating to potable aquifers, health concerns are mitigated, although the reclaimed water, upon extraction, is subjected to the appropriate water quality requirements for the subsequent use of the water.

2.1.3.2. Recharge of Aquifers Used for Potable Purposes. Groundwater recharge of potable aquifers is problematic, as many utilities distribute drinking water from potable water supply wells with little or no treatment. As a consequence, it is necessary for reclaimed water to meet all drinking water standards-and water quality limits for potentially toxic unregulated chemical contaminants and microbial pathogens-prior to extraction.

Injection directly into a confined aquifer provides little opportunity for additional water quality improvement in the subsurface resulting in the need to meet all water quality limits prior to injection via incorporation of advanced wastewater treatment processes.

Regulatory considerations relating to groundwater recharge of potable aquifers are complex and must be discussed in detail on behalf of potable water quality parameters (Metcalf et al., 2006).

2.2 Integrated Wastewater Management

Considering used urban water, currently still called wastewater, as a resource may require decentralization and source water separation into black water (water containing mostly fecal matter), yellow water (urine) and grey water (cleaner but still polluted used water without black water and urine).

Black water contains most of the organic matter while a small volume of urine flow (about 1 % of the total used water flow) contains most of nitrogen and about 50% of phosphorus (Hao et al., 2010).

In order to implement an integrated management for wastewater, wastewater characteristics of the wastewater source must be identified clearly. The variability in constituent mass loading affects the type and size of the process selected. The characteristics of wastewater from individual buildings are dependent on the activities occurring inside the building.

Household wastewater derives from a number of sources (Figure 2.2.). A common strategy for Cities of the Future Turkey is going to be developed in order to embed a pilot project of the mentioned concept. Related pilot project must involve creating buildings that can use 2 different grades of water. For Kayseri, it is an opportunity to develop such a strategy since KASKI has a positive look on integrated management strategies.

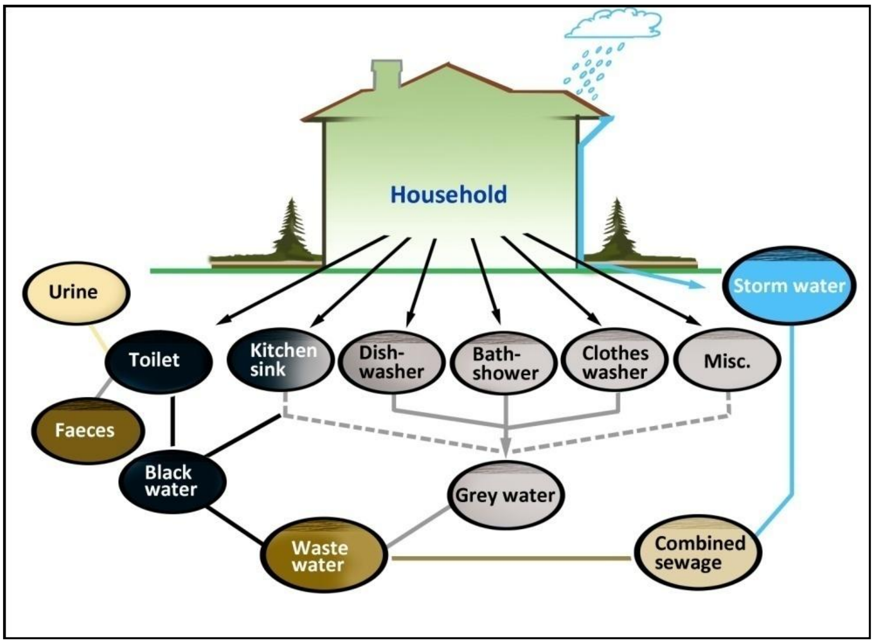


Figure 2.2. A range of possible sources of household waste water showing waste water from toilet, kitchen, bathroom, laundry and others (UNEPD, 2011).

In recent years, the traditional combined flow options for wastewater management have been criticized by several authors (Larsen and Gujer, 1996; Otterpohl et al., 1999; Wilderer and Schreff, 2000). These authors, among many others, argue that the traditional method of sanitation does not reuse the nutrients which wastewater contains, and that the invested capital is tied up in infrastructure system, pumping stations, wastewater treatment plants (WWTP), which is inflexible and designed for the maximum theoretical wastewater flow.

In order to overcome these weaknesses, alternative technologies based on the separation of the wastewater into its constituent parts, grey water and black water, where black water can further be separated into yellow water (urine) and brown water (faeces and flushing water), are proposed.

2.2.1. Grey Water Management

Grey water is wastewater generated from domestic activities such as laundry, dishwashing, and bathing, which can be recycled on-site for uses such as landscape irrigation and constructed wetlands. Grey water differs from water from the toilets which is designated sewage or black water to indicate it contains human waste.

Grey water composes 50–80% of residential wastewater generated from all of the house's sanitation equipment (except toilets). To reduce domestic water use, water conservation practices may be implemented, which also have the effect of reducing wastewater generations while proportionally increasing the concentration of wastewater constituents.

Grey water systems are usually expensive to retrofit into a building, and therefore should be included, if possible, during building planning and construction. In some areas facing water shortages, the use of grey water for irrigation and toilet flushing is recommended. Management of grey water systems may present challenges if there is insufficient planning (Metcalf et al., 2006).

In order to introduce a management strategy based on grey water reuse land planners and entities should succeed to gain public acceptance for the proposed concept.

2.2.2. Black Water Management

Black water is water contaminated by sewage. Wastewater from the toilet is termed ‘black water’. It has a high content of solids and contributes a significant amount of nutrients (nitrogen, N and phosphorus, P). Black water further can be separated into faecal materials and urine (UNEPD of Technology, Industry and Economics, 2011).

Proper treatment and handling of black water is critical due to the potentially dangerous pathogens in the water. Proper treatment is necessary to protect both public health and the environment, especially in remote and sensitive ecosystems.

For most onsite applications, black water is typically handled with a septic tank and drain field sometimes called a soil treatment unit. In some cases, secondary treatment is required for additional treatment of the wastewater between the septic tank and drain field. The septic tank separates the liquid from the solid portion of the waste while a drain field provides a discharge location for the liquid portion of the waste, commonly referred to as grey water.

Sufficient soil treatment in the drain field system renders the water clean enough to be released to the environment. Solids build up in the septic tank and are removed every few years (BEES, 2010).

A number of black water management options have been introduced. Four of these options are applicable to both the higher and lower altitude sites and include:

- Pit toilets
- Barrel Fly-Outs
- Incinerating Toilets
- Carry-out

In addition to these four options, a composting toilet option is introduced for each of the site types. Because of differing site conditions at the higher and lower residential quarters, a site-specific composting system can also be evaluated. The components of the system include:

- Higher residential systems: composting toilet, evaporation tank (for liquids) and barrel fly-outs
- Lower residential systems: composting toilet, onsite liquid infiltration field, and barrel fly-outs

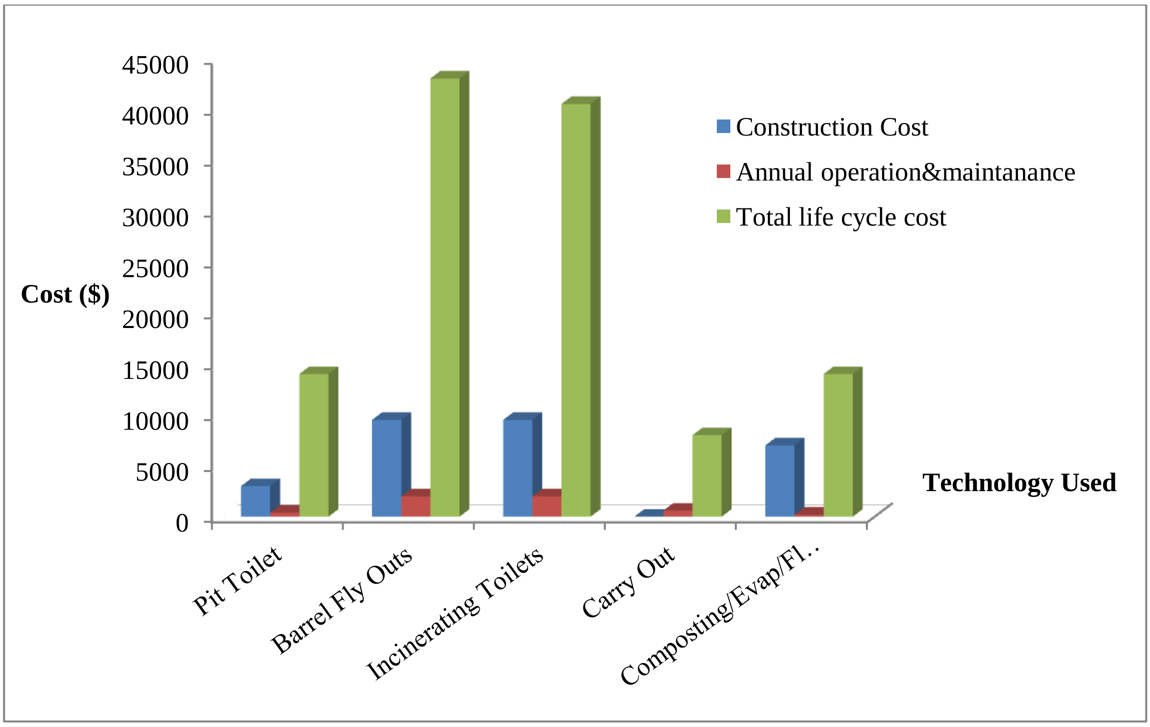


Figure 2.3. Cost comparison of black water management options (BEES, 2010).

Each of the five black water management options has unique situations critical to success. All black water management solutions can be successfully implemented if site conditions meet the requirements of the technology. Proper planning at each site is critical to determine the size and use of each facility to prevent against overloading and system abuse (BEES, 2010).

2.3. Current Management Practices in Kayseri

2.3.1. Aquifer Management in Kayseri

Kayseri is one of the leading cities in Turkey which meets its drinking water demand from groundwater resources. Kayseri which has a population of over 1.000.000, uses utility water the water from the aquifer flows with comparable quality of headwater. The water quality for domestic uses drains with hardness of 9-12 Fr^{o}. Within this respect, Kayseri may be the leading city in the world by using high quality of water. Kayseri owes this specialty to the volcanic originated Erciyes Mountain which is extended over a large area.

City’s drinking and utility water is being supplied from a headwater with the capacity of 500 L/s which is located at the piedmonts of Erciyes Mountain and from 50 drilling wells at a plain aquifer which Kayseri is constructed over (Figure 2.4).

On the other hand, city’s solid waste was openly dumped near the city center, within the boundary of drinking water catchments until 1997 for 15 years time. In addition, until 1980 for 15 years time, couple of hundred meters near the head water’s outlet, solid waste was spilled over. Between 1980-1997, for a 17 years time, similarly, over an area which is incorporated in drinking water catchment, near city center, Kumarlı, solid waste was dumped over. On that account, drilled wells for drinking water are in danger of being polluted because of the Beştepeler and Kumarlı waste storage areas (Değirmenci et al., 2006a).

Groundwater plays a strategic role in Kayseri region. In Kayseri, drinking water in households is supplied almost entirely from volcanic rock aquifer systems whose major recharge area extends over the Erciyes Mountain (Değirmenci et al., 2008). The annual precipitation is 350 mm for Kayseri. The amount of water that could reach into the aquifers from rainfall is 50 mm. Annually, the rest 300 mm precipitation diverges as runoff.

According to a study in Meriç transboundry river basin, groundwater levels of some major pollutants such as nitrate and coliform bacteria have shown an unacceptable and increasing trend over the years. Since groundwater is the main water supply resource for the city, an integrated water and wastewater project has been initiated to protect the drinking water resources (Kibaroğlu, 2008).

In accordance with an another study which was done in the Develi Closed Basin, Kayseri presumes that with the proper water management, 82×10^6 m^3/year spring water can feed Sultansazlığı and water shortage at this wetland can be prevented (Yıldız et al., 2008).

There are mainly two separated aquifer systems. The first one is supplied from melted glaciers, which used as drinking water supply.

The upper unconfined aquifer is under the effect of various polluting sources so that the possibility of the aquifer to become polluted is extremely high. If the piozometric head of first confined aquifer is higher than the shallow aquifer's piozometric head, since the related hydraulic gradient upwards, no leakage may occur from the unconfined aquifer to confined aquifer.

In Kayseri drinking water catchment area, the mentioned water table difference is about 10-20 cm. This difference assures confined aquifer's freshness. If the excess water is pumped from confined aquifer, the hydraulic head difference disappears, as a result, confined aquifer pollution risk arises (Değirmenci, 2010).

In this respect, groundwater recharge with rainwater arises as a solution in order to prevent pollution risk for the aquifers. Aykar (2009), analyzed Kayseri city drinking water basin and its hydro geochemical characteristics. According to the study conducted at 70 locations within the borders of Kayseri, field analyses showed that if the vadose zone thickness is relatively low and the aquifer is characterized as an unconfined aquifer, the pollution risk of the aquifer arises. As a result, if recharge of the aquifers is an option as a storm water management practice, the aquifer's thickness and type must be taken into consideration.

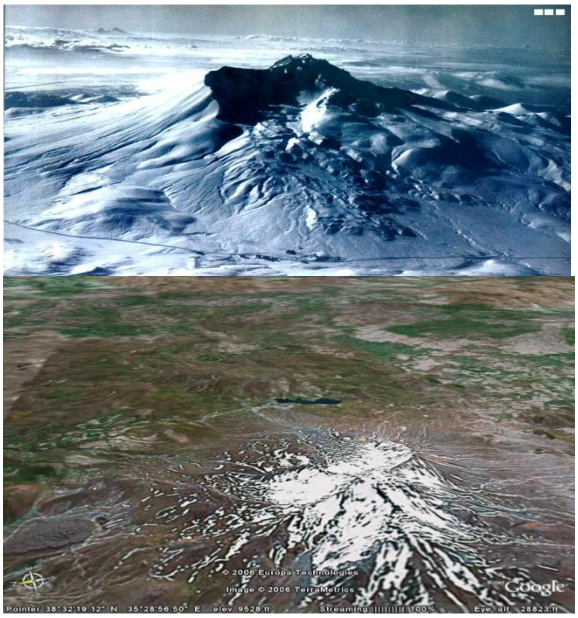


Figure 2.4. Snapshots from Erciyes Mountain Culmination and Kayseri Plain (Değirmenci et al., 2006b).

2.3.2. Storm Water Management in Kayseri

The effective application of the concept of sustainable water resources development in the coming century will depend on the implementation of a planning process that also encompasses storm water management practices.

Kayseri’s upcoming storm water plans should not be dissociated from other resources planning in the watershed. Furthermore, existing regional plans and urban master plans must be taken into consideration.

In Kayseri, after the revision of municipality law, Kayseri Municipality borders have changed. The situation in water distribution network and population is as follows (Nalbantoğlu, 2010).

Table 2.2. Situation before and after the amendment in municipality law of Kayseri (Nalbantoğlu, 2010).

	Before 01.01.2005	Current Situation
Catchment Area	395 km^2	2150 km^2
Catchment Population	694.026	911.984
Current Water Distribution System Length	1.968 km	2.740 km
Percentage of Channel Connection	98%	95%

The storm water system was separated throughout the main distribution system. But still, there are currently combined parts in the system network. The inadequate placement of distribution pipes cause problems during emergencies. In combined systems, during drought seasons, odor problem arises. On the other hand, during heavy rain flooding occur frequently.

Figure 2.5. Flood after heavy rain (Nalbantoğlu, 2010).

The problems including odor and flooding may persist unless an integrated management approach for storm water is put in place.

Figure 2.6. Obtaining image of the water distribution system while cleaning via image endorser robot (Nalbantoğlu, 2010).

From view point of an integrated management approach, combined systems lose storm water as a resource since it must be treated. Storm water is not a wastewater that has to be treated; it can be an environmental amenity as a resource. There are two solutions to gain storm water as a resource.

Action 1: Creating a separated storm water and sanitary system to use storm water as an aquifer recharge and storage.

•*Action 2:* Creating aesthetic drainage canals as well as retaining their drainage function.

These two solutions can be implemented as a long term solution to Kayseri’s storm water management. Creating urban concepts that reflect this philosophy and increase desirability of these districts, example projects of inland cities are necessary (Moddemeyer, 2010).

3. MATERIALS AND METHODS

3.1. Study Area and Main Properties

Working area encompasses Kumarlı district (Figure 3.1), Kayseri, near city center. Kayseri is located on the Central Kızılırmak plain forthcoming up on the south of Central Anatolia and Toros Mountains.

The watershed, bounded by latitudes 37^0 45’- 38^0 18’ N and longitudes 34^0 56’-34^0 58’. Kayseri, whose surface area is 16.917 km^2, is surrounded by Sivas from east and north east; Yozgat from north, Nevşehir from west, Niğde from south west, Adana and Kahramanmaraş from south.

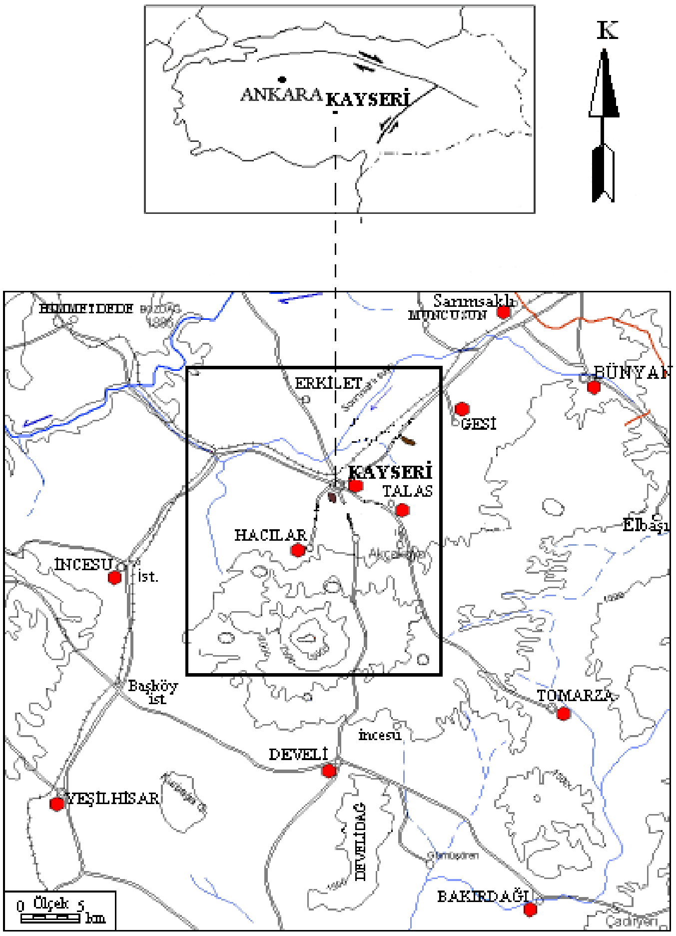


Figure 3.1. Location of Kayseri city center (Aykar, 2009).

Temperature: The climate of Kayseri is a steppe climate; where the temperature difference between day and night is high. Rainfall is low and there is more snow when compared to coastal regions of Turkey. On the other hand, city's overall climatic conditions can vary with the altitudes. Climate is much softer in the hollow places whereas the places near Erciyes Mountain face harsher conditions.

The hottest days recorded are in July and August where the temperature is recorded as 38^0C. The average temperature for these months is 23^0C. The cold months of the year are December, January and February. Average winter temperature ranges between –2 ^{0}C and –6 ºC. According to last 60 years data, the average temperature is 10.4^0C (DMI, 2011).

Evaporation and moisture: At the city center of Kayseri, according to 24 year record, the annual average evaporation rate is 101.39 mm where the maximum evaporation amount is recorded in July 1994, as 294.7 mm.

Precipitation: At the city center the annual average precipitation is 377 mm. March, April and May are the rainy months while the driest months are June, July and August. Especially the precipitation which starts in April until the end of May called "Kırk İkindi" is crucial. The number of snow covered days recorded as 40 days, for the city center.

Wind: Mistral is the predominant wind in Kayseri. In addition, southwest and south winds are also important winds for Kayseri city center. At the city center, especially during spring, southeaster speed exceeds 125 km/hr.

Green Areas: Similar to the plains of Kayseri, on the mountains and hills, moorlands are dominant. At the high altitudes, predominant vegetations are moorlands and forests. Regardless of the fact that between the mountains, in the plains, predominant vegetation was moorlands until the plains were turned into agricultural zones. Kayseri has 13,157 m^2 green area per person while the global standard is 15 m^2/person (Kayseri City-Environment Status Report, 2004).

Population: As of 2008, according to official census data based on the "Address based Population Registration System which was conducted by Turkish State Institute of Statistics, the total population of Kayseri is 1,184,386. 85% of population is living in the city center while 15% is living in rural areas of the city. Besides, population density is calculated to be 69 people/km^2.

Kayseri city has 16 towns. In Kayseri, after the revision of municipality law, Kayseri Municipality borders have changed. Previously, 2 metropolitan towns were constituted. The new law introduces Kayseri main municipality with 5 municipalities; Kocasinan, Melikgazi, Hacılar, İncesu and Talas (www.kayseri.gov.tr). On the table below city population distribution over towns is presented.

Table 3.1. Kayseri city population distribution over towns.

Towns	Total	Main District	Rural Areas
Kocasinan	365,153	350,698	14,455
Malikgazi	434,98	430,421	4,559
Akkışla	9,128	2,781	6,347
Bünyan	34,819	12,705	22,114
Develi	65,452	36,072	29,38
Felahiye	6,971	2,065	4,906
Hacılar	12,723	11,756	967
İncesu	21,433	16,69	4,743
Özvatan	5,367	3,686	1,681
Pınarbaşı	31,099	11,927	19,172
Sarıoğlan	18,844	3,452	15,392
Sarız	12,697	4,466	8,231
Talas	81,399	75,098	6,301
Tomarza	28,652	10,347	18,305
Yahyalı	38,198	19,909	18,289
Yeşilhisar	17,471	9,376	8,095
Total	1,181,386	1,001,449	182,937

3.2. Model Structure

Storm Water Management Model (SWMM) was developed by USA EPA (Environmental Protection Agency), which was first used for simulation of urban storm-runoff, planning, assessment and management for administrations and engineers (Rossman, 2009; Singh et al., 2005). Now it has been updated to be applicable in rural areas.

A three-step approach was employed for this study, primarily using program contained in Version 5.0 of the U.S. Environmental Protection Agency (US EPA) Storm Water Management Model (SWMM) suite of tools:

1. Continuous precipitation data was processed by SWMM as a rain gage for use in SWMM Runoff.
2. SWMM Runoff generated surface runoff based on local precipitation data continuous and event based, land use, and topography.
3. SWMM routed event-based flows through the drainage system and storm-water basin, creating stage–discharge relationships for use in the construction of a storage tank.

SWMM accounts for various hydrologic processes that produce runoff from urban areas. These include:

- time-varying rainfall
- 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

In addition to modeling the generation and transport of runoff flows, SWMM can also estimate the production of pollutant loads associated with this runoff. The following processes can be modeled for any number of user-defined water quality constituents (Rossman, 2009):

- dry-weather pollutant buildup over different land uses
- pollutant wash off from specific land uses during storm events
- direct contribution of rainfall deposition
- reduction in dry-weather buildup due to street cleaning
- reduction in wash off load due to BMPs
- entry of dry weather sanitary flows and user-specified external inflows at any point in the drainage system
- routing of water quality constituents through the drainage system
- reduction in constituent concentration through treatment in storage units or by natural
- processes in pipes and channels.

Since its inception, SWMM has been used in thousands of sewer and storm water studies throughout the world. Typical applications include:

- design and sizing of drainage system components for flood control
- sizing of detention facilities and their appurtenances for flood control and water quality protection
- flood plain mapping of natural channel systems
- designing control strategies for minimizing combined sewer overflows
- evaluating the impact of inflow and infiltration on sanitary sewer overflows
- generating non-point source pollutant loadings for waste load allocation studies
- evaluating the effectiveness of BMPs for reducing wet weather pollutant loadings

3.2.1. Visual Objects

Figure 3.2 depicts how a collection of SWMM's visual objects might be arranged together to represent a storm water drainage system. These objects can be displayed on a map in the SWMM workspace. The following sections describe each of these objects.

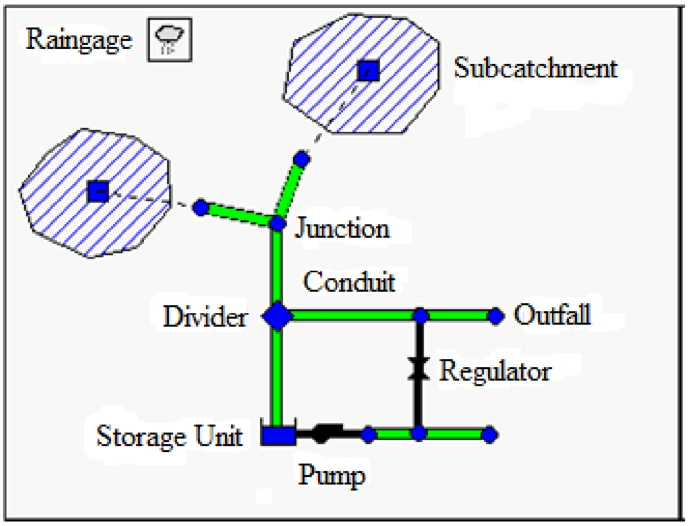


Figure 3.2. Example of physical objects used to model a drainage system (Rossman, 2009).

3.2.1.1. Rain Gages. Rain Gages supply precipitation data for one or more subcatchment areas in a study region. The rainfall data can be either a user-defined time series or come from an external file created by user. Several different popular rainfall file formats currently in use are supported, as well as a standard user defined format which is available for EPA SWMM 5.0.

The principal input properties of rain gages include: rainfall data type (i.e., intensity, volume, or cumulative volume) recording time interval (i.e., hourly, 15-minute, etc.) source of rainfall data (input time series or external file) name of rainfall data source.

DMI provides rainfall data as an intensity time series with time intervals of 24 hours which indicates the daily cumulative precipitation amount measured per 1 m^2 area. (mm/m^2.day)

3.2.1.2. Subcatchments. Subcatchments are hydrologic units of land whose topography and drainage system elements direct surface runoff to a single discharge point. The user is responsible for dividing a study area into an appropriate number of subcatchments, and for identifying the outlet point of each subcatchment. Discharge outlet points can be either nodes of the drainage system or other subcatchments.

In this study, the area is divided into 30 subcatchment areas according to the elevations and waterways when the project layout is taken into consideration. In the model simulation, 30 subcatchments were determined according to the water ways, slope of the catchment area and the altitudes. Subsequent to the determination of the subcatchment areas, the imperviousness percentages were specified according to the land use and cover categories (Table 3.2 and Table 3.3).

Imperviousness. Imperviousness has emerged as a key indicator for urban watershed management. It is an integrative indicator, and can be used to estimate or predict cumulative water resource impacts.

Subcatchments can be divided into pervious and impervious subareas. Surface runoff can infiltrate into the upper soil zone of the pervious subarea, but not through the impervious subarea. Several authors have proposed to use imperviousness as a unifying theme for urban watershed management (Schueler, 1994; Arnold et al., 1996). As presented in Figure 3.3, there are several ways for determination of imperviousness. Creating a land use/cover map should be done according to the following steps:

- ✓ Calculating the amount of each land use/cover within the study area.
- ✓ Multiplying each land use/cover category with an imperviousness factor, typical for each category.

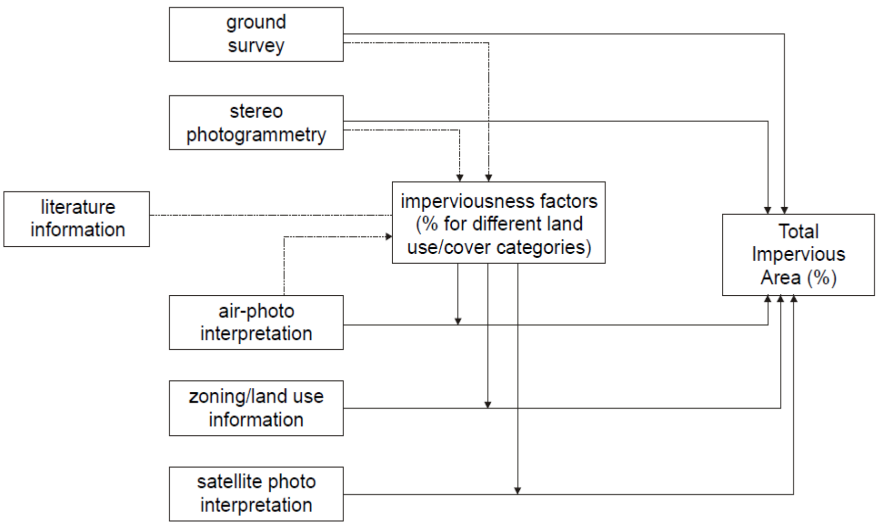


Figure 3.3. Alternative methods for determining imperviousness (CMN, 2011).

Table 3.2. Recommended land use and land cover categories (Storm Water Management Manual, 2011).

General Land Use Categories	General Land Cover Categoires
agriculture	built-up areas
single family residential- low density	forest
single family residential- medium density	shrubs
single family residential- high density	grass/lawns
townhouse residential	bare/exposed soil
multifamily residential	water
commercial	-
industrial	-
instutional	-
transportation (highways)	-
utilities & special infrastructure	-
recreation & parks	-
forestry	-
Open space (no apparent use)	-

While land cover provides in theory the most accurate information for calculating imperviousness, quite frequently land cover maps do not distinguish between various types of built-up areas, which are required for imperviousness calculations. Therefore, land use based on zoning information is most commonly used for imperviousness calculations (Storm Water Management Manual, 2011).

In the proposed study, a land use/cover map is created in accordance with the following steps. Firstly, the amount of each land use/cover area is calculated. Followingly, by means of the land use percentages, overall imperviousness of each subcatchment is determined.

Table 3.3. Recommended land use and imperviousness factors.

	Land Use or Surface Characteristics	% Imperviousness
Business	Commercial Areas	85
	Neighborhood Areas	70
Residential	Single-unit	n/a[1]
	Multi-unit	70
	Apartments	80
Townhouse		65
Instutional		80
Industrial	Light industrial	80
	Heavy industrial	90
Transport		90
Parks, cemeteries		5
Playgrounds		10
Schools		50
Undeveloped Areas	Historic flow analysis	2
	Greenbelts, agriculture	2
	Off-site flow analysis	45
Streets	Paved	100
	Gravel	40
Drives and walks		90
Roofs		90
Water		0
Recreation		3
Open space		3
Agriculture		3
Lawns (all soils)		0
Land Cover	Forest	1
	Clear cut	3
	Grass	3
	Shrub	3
	Bare	3

[1]No typical imperviousness factor is assigned to single family residential as this value varies greatly with housing density (Storm Water Management Manual, 2011).

Infiltration. Infiltration is the process by which water on the ground surface enters the soil (Figure 3.4). Infiltration rate in soil science is a measure of the rate at which soil is able to absorb rainfall or irrigation. Infiltration is another indicator for the determination of the amount of water which will diverge as runoff through the subcatchment area.

Infiltration is basically a component of the general mass balance hydrologic budget. There are several ways to estimate the volume and/or the rate of infiltration of water into a soil. There are three estimation methods available in EPA SWMM 5.0 model;

- Horton's method
- Green-Ampt method
- SCS method

The general hydrologic budget, with all the components, with respect to infiltration *F* is given in Equation 3.1. Given all the other variables and infiltration being the only unknown, simple algebra solves the infiltration question.

$$F = B_I + P - E - T - ET - S - R - I_A - B_O \tag{3.1}$$

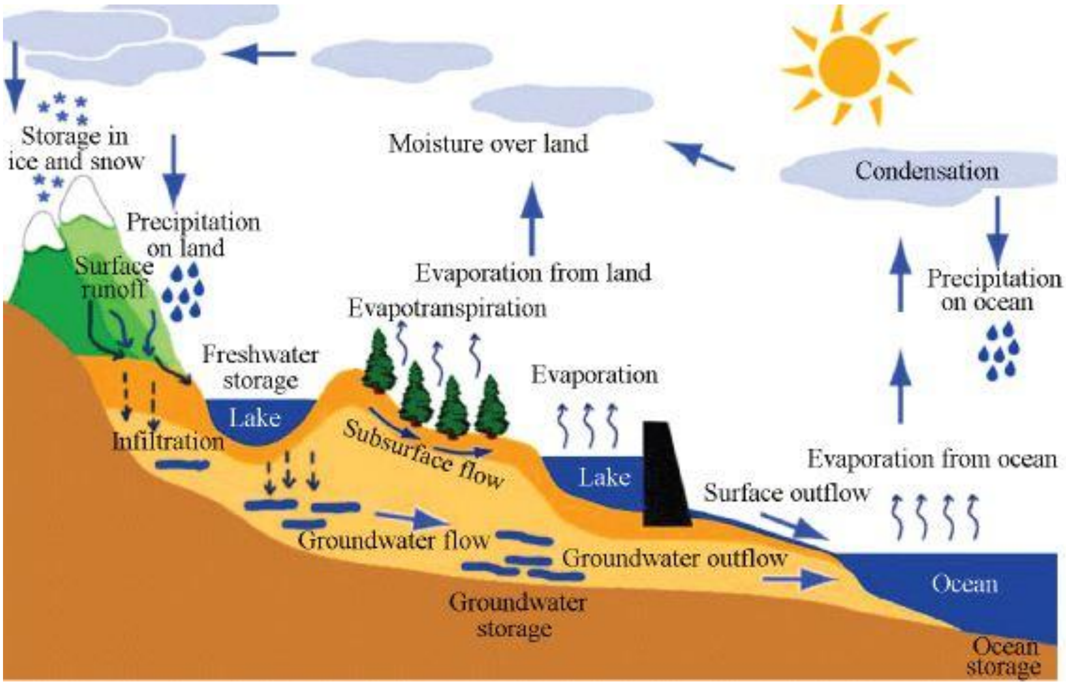


Figure 3.4. Hydrological Cycle (Technopure, 2011).

where;

F is infiltration, which can be measured as a volume or length (mm);

B_I is the boundary input, which is essentially the output watershed from adjacent, directly connected impervious areas;

B_O is the boundary output, which is also related to surface runoff, R, depending on where one chooses to define the exit point or points for the boundary output;

P is precipitation (mm);

E is evaporation (mm);

ET is evapotranspiration (mm);

S is the storage through either retention or detention areas (mm);

I_A is the initial abstraction, which is the short term surface storage such as puddles or even possibly detention ponds depending on size (mm);

R is surface runoff (mm).

For each fraction of the catchment (pervious and impervious) the rainfall loss is the difference between the rainfall depth and the depth of runoff. This is made up of various components as illustrated in Figure 3.5. Not all methods of modeling rainfall losses use all of these components.

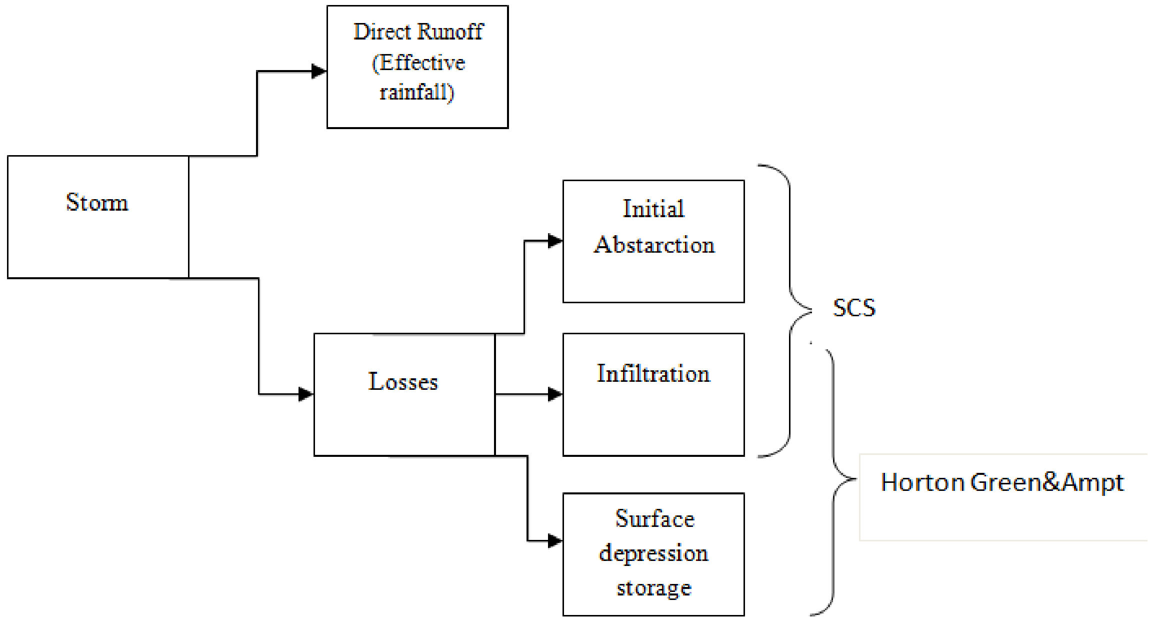


Figure 3.5. Different components for the use of rainfall abstraction models (Smith, 2004).

The infiltration capacity is assumed to decrease continuously throughout the storm as the storage potential in the soil is progressively reduced by the volume of infiltration. The reduction in infiltration capacity is a function of the infiltrated volume and not of the elapsed time from the start of rainfall.

<u>*Horton infiltration.*</u> It is an empirical formula indicates that infiltration starts at a constant rate, f_0, and is decreasing exponentially with time, t. After some time when the soil saturation level reaches a certain value, the rate of infiltration will level off to the rate f_c.

$$f_t = f_c + (f_0 - f_c)e^{-kt} \tag{3.2}$$

where;

f_t is the infiltration rate at time t (mm);
f_0 is the initial infiltration rate or maximum infiltration rate (mm);
f_c is the constant or equilibrium infiltration rate after the soil has been saturated or minimum infiltration rate (mm);
k is the decay constant specific to the soil (d^{-1}).

<u>*Green-Ampt infiltration.*</u> The Green-Amp method of infiltration estimation accounts for many variables that other methods, such as Darcy's law, do not. It is a function of the soil suction head, porosity, hydraulic conductivity and time (Wikipedia, 2011).

$$\int_0^{F(t)} \frac{1 - \psi \Delta\theta}{F + \psi \Delta\theta} d = \int_0^t K \, dt \tag{3.3}$$

where;

ψ is wetting front soil suction head (mm);
θ is water content (%, unitless);
K is Hydraulic conductivity (mm/s);
F is the total volume already infiltrated (mm)

Once integrated, the infiltration rate can be solved for either volume of infiltration or instantaneous infiltration rate:

$$f(t) = K\left[\frac{\Psi\Delta\theta}{F(t)} + 1\right] \tag{3.4}$$

SCS (US Soil Conservation Service) Model. The SCS curve number which is tabulated in the publication SCS (USDA TR-55, 1986), June 1986. By the use of Curve Number Table (Appendix C) for a listing of values by soil group, and the accompanying Soil Group Table (Appendix D) for the definitions of the various groups can be determined.

Adjustments will be needed when a subcatchment contains separate pervious and impervious fractions and a Curve Number is selected from a table where the two land uses are lumped together (Rossman, 2009).

As a result, each of the equations should provide a relatively accurate assessment of the infiltration characteristics of the soil. In the current model the infiltration rate is calculated through Horton's equation.

3.2.1.3. Junction Nodes. Junctions are drainage system nodes where links join together. Physically they can represent the confluence of natural surface channels, manholes in a sewer system, or pipe connection fittings.

External inflows can enter the system at junctions. Excess water at a junction can become partially pressurized while connecting conduits are surcharged and can either be lost from the system or be allowed to pond atop the junction and subsequently drain back into the junction. The principal input parameters for a junction are: invert elevation height to ground surface ponded surface area when flooded (optional) external inflow data.

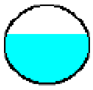
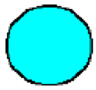
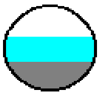
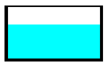
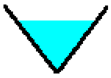
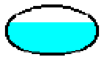
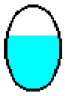
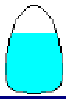
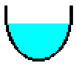
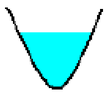
In this study, maximum invert elevation was set to be 0.5 m. Surface ponded areas were calculated by the model automatically when subcatchments are introduced to the system.

3.2.1.4. Conduits. Conduits are pipes or channels that move water from one node to another in the conveyance system. Their cross-sectional shapes can be selected from a variety of standard open and closed geometries as listed in Table 3.4.

Most open channels can be represented with a rectangular, trapezoidal, or user-defined irregular cross-section shape.

In this study triangular cross section conduits are used with a height of 0.5 m and a maximum width of 0.5 m.

Table 3.4. Examples for available cross section shapes for conduits (Rossman, 2009).

Name	Parameters	Shape	Name	Parameters	Shape
Circular	Full Height		Circular Force Main	Full Height, Roughness	
Filled Circular	Full Height, Filled Depth		Rectangular - Closed	Full Height, Width	
Rectangular – Open	Full Height, Width		Trapezoidal	Full Height, Base Width, Side Slopes	
Triangular	Full Height, Top Width		Horizontal Ellipse	Full Height, Max. Width	
Vertical Ellipse	Full Height, Max. Width		Arch	Full Height, Max. Width	
Parabolic	Full Height, Top Width		Power	Full Height, Top Width, Exponent	

The principal input parameters for conduits are: names of the inlet and outlet nodes offset height or elevation above the inlet and outlet node inverts conduit length Manning's roughness cross-sectional geometry entrance/exit losses (optional) presence of a flap gate to prevent reverse flow (optional).

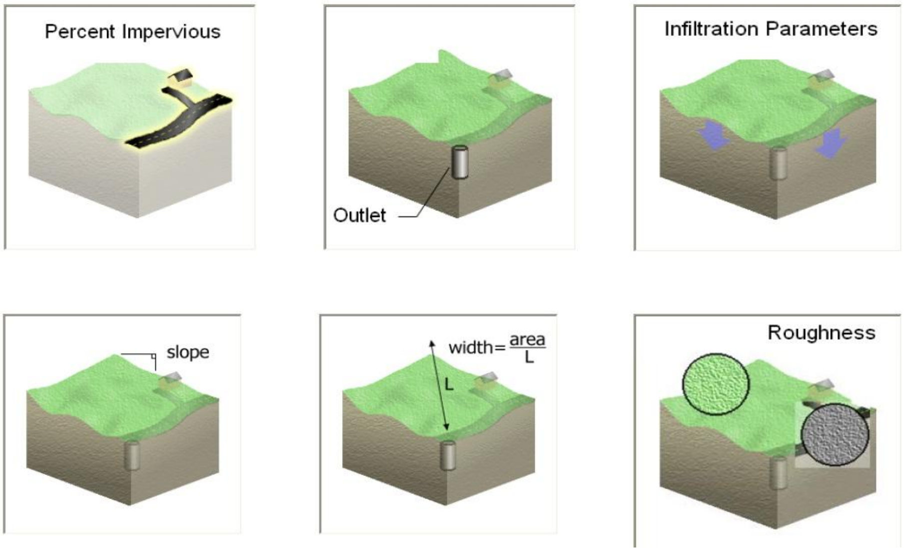


Figure 3.7. Representation of typical elements introduced in SWMM 5.0 (Rossman, 2009).

SWMM uses the Manning equation to express the relationship between flow rate (*Q*), cross-sectional area (*A*), hydraulic radius (*R*), and slope (*S*) in all conduits.

$$Q = 1.49\ A\ R^{2/3}\ S^{1/2} \tag{3.5}$$

The slope *S* is interpreted as either the conduit slope or the friction slope depending on the flow routing method used (Rossman, 2009). Slope of an open channel should have a minimum 1:12 (V:H) slope to ensure drainage (Knox County Tennessee Storm Water Management Manual, 2007). Unless otherwise approved due to unavoidable physical constraints, the minimum centerline grade for various channel types shall be as follows:

Table 3.5. Minimum permissible grades for different types of open channels (Green County, 1999).

Channel type	Minimum slope
Concrete channels	0.25%
Composite channels with concrete inverts	0.5%
Grass channels	1.0%
Grass channels Concrete trickle channels	0.5%
Riprap and gabion channels	1%

The selection of Manning's *n* is generally based on observation; however, considerable experience is essential in selecting appropriate *n* values. In Table 3.6, typical roughness coefficients for a specific channel fulfilled materials are presented.

Table 3.6. Typical Roughness Coefficients (Huang, 1995).

Channel Fulfilled Material	Manning Coefficient (n)	Hazen Williams Coefficient (C)	Darcy Weisbach Coefficient (F)
Asbestos cement	0.011	140	0.0015
Brass	0.011	135	0.0015
Clinker	0.015	100	0.6
Molten iron	0.012	130	0.26
Forms of steel	0.011	140	0.18
Rigorous forms	0.015	120	0.6
Copper	0.011	135	0.0015
Galvanized iron	0.016	120	0.15
Glass based	0.011	140	0.0015
Lead	0.011	135	0.0015
Plastic	0.009	150	0.0015
Coal toar rome	0.010	148	0.0048
Clinched	0.019	110	0.9
PVC	0.009-0.0011	130	0.019

Rigid concrete lines will be used during construction of the channel design. 0.011 is set as Manning's coefficient in model simulations.

3.2.1.5. Outlets. Outlets are flow control devices that are typically used to control outflows from storage units. They are used to model special head-discharge relationships that cannot be characterized by pumps, orifices, or weirs. Outlets are internally represented in SWMM as a link connecting two nodes. An outlet can also have a flap gate that restricts flow to only one direction. Outlets attached to storage units are active under all types of flow routing. If not attached to a storage unit, they can only be used in drainage networks analyzed with Dynamic Wave flow routing.

A user-defined rating curve determines an outlet's discharge flow as a function of either the freeboard depth above the outlet's opening or the head difference across it. Control Rules can be used to dynamically adjust this flow when certain conditions exist. The principal input parameters for an outlet include: names of its inlet and outlet nodes height or elevation above the inlet node invert function or table containing its head (or depth) - discharge relationship.

3.3. Proposed Storm Water Management in Kayseri

The aim of the study is based on developing ways of utilizing water as a central feature of Kayseri urban design within the frame of the IWA Cities of the Future Project.

In order to implement a full management strategy, it is best to perform proposed solutions for small catchment areas first. Kayseri “Harikalar Diyarı” Project was selected in order to implement the integrated management strategy and evaluate the distress over the catchment area.

There is a need for such study in Kayseri since storm water cannot be graded as wastewater, which must be treated and disposed of. Storm water is a water resource that has to be turned into an asset rather than being treated as a wastewater.

From the view point of Cities of Future, Kayseri’s opportunity is the use of rainwater harvesting and local storm water management. Along with water reclamation and reuse, supplementing the existing groundwater supply will also improve the drought resilience of water supply and relive the stress on wastewater system (Moddemeyer, 2010).

The volume of the water which becomes surface runoff is going to be calculated via EPA SWMM 5.0. Water that flows unstrained will turn into an asset for “Kayseri Harikalar Diyarı” Project.

Different uses may compromise after storing water via storage tanks under the surface level. Two outlets are going to be introduced to the system. The altitudes and slope of the area is taken into consideration while determining the number of outlets. Within the help of these two outlets, water is going to be stored in commercially available storage tanks with the capacity of 50.050 liters under surface level.

First storage tank is going to be located near the water-ski lake so that the stored storm water could be fed to the lake while a flushing system is going to be installed to each storage tanks.

The second outlet will be located at the north east of the study map where the slope of the area is taken into account. The mentioned outflow is located 200 m far away from the lake. The second storage tank will be used for different purposes such as irrigation needs of the area.

3.3.1. “Kayseri Harikalar Diyarı” Project Properties

In Kayseri, Kumarlı district a European style amusement park appealing to whole population in Kayseri and neighborhood areas (Figure 3.8) is going to be constructed. Park is going to be called as “Kayseri Harikalar Diyarı”. This park is going to be totally different from any other fun fairs that have been constructed before in Turkey. The park includes several different recreational areas such as amusement park, horseback riding areas, ice-skating and water-ski park.

Figure 3.8. Design snapshots of the Kayseri Harikalar Diyarı Project.

During the design of the park, international design criteria are going to be taken into consideration, thus secure construction, and land use of the area is crucial. Kayseri “Harikalar Diyarı” has a land area of 452,000 m^2. Land use must be planned accordingly in order to implement an integrated management strategy for water and wastewater.

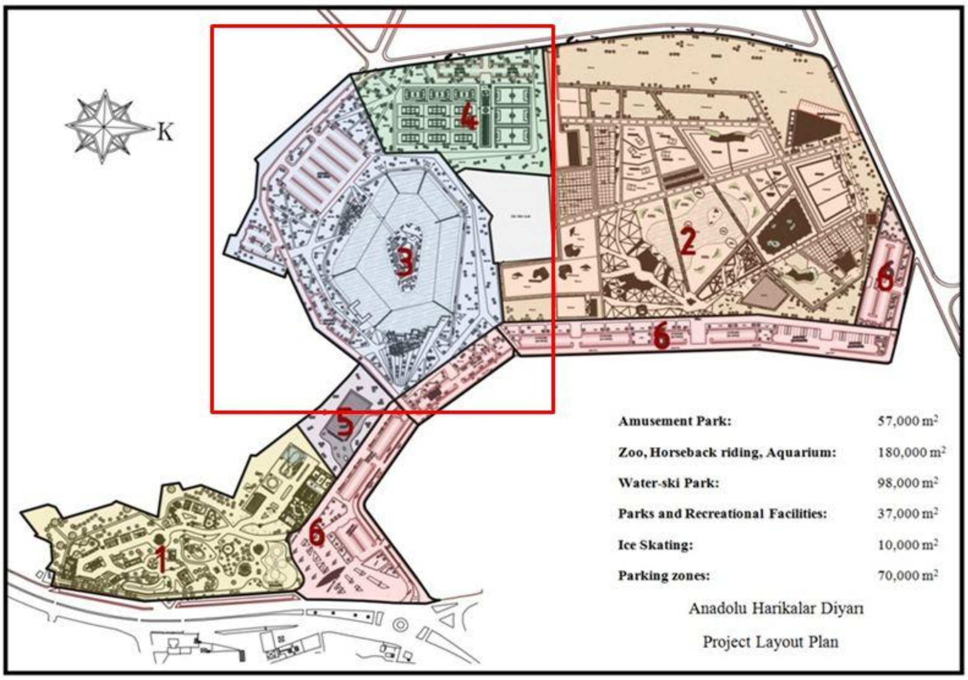


Figure 3.9. Anadolu Harikalar Diyarı Project Layout Plan.

In order to simulate water amounts for a 3-year period, water-ski park with a land area of 98,000 m^2 is selected (Figure 3.9) to determine the net runoff through the specific catchment area. Kayseri’s climatological statistics indicates that a management program is essential when the current water resources aspects are taken into consideration (Table 3.7 and Table 3.8).

Table 3.7. Climatological Statistics for long years, (DMI, 2011).

	Parameters	Av. Temperature (^0C)	Solar Radiation (hours)	Rainy Days	Av. precipitation (kg/m^2)	Highest Temperature (^0C)	Lowest Temperature (^0C)
Long Years (1975-2011) (Months)	January	-1.8	2.8	11.6	33.6	17	-28.1
	February	0	3.8	11.6	33.6	20.1	-28.4
	March	4.9	4.9	12.3	42.5	26.6	-28.1
	April	10.6	6	13.5	57.2	31.2	-11.6
	May	14.9	8.1	13.5	56.6	33.4	-5.5
	June	19.1	10.2	8.4	35.9	36	-0.4
	July	22.5	11.6	2.8	14.3	40.7	3.7
	August	22	11.3	2.4	8.5	40	2.1
	September	17.2	8.9	4.2	12.9	36	-2.5
	October	11.5	6.4	7.9	34.8	32.6	-0.83
	November	4.8	4.5	9.3	37.2	24.8	-16.2
	December	0.2	2.5	11.6	39	21	-25.5

Table 3.8. Extreme measurements for precipitation, wind and snow level (DMI, 2011).

Parameter	Month	Day	Year	Value
Max Precipitation	05	17	1999	51.8 kg/m^2
Most strong wind	01	19	1981	149.4 km/h
Highest snow level	02	19	2008	51.0 cm

Tensions (Table 3.8) over water during times of drought mount as the range of competing interests for this scarce resource expands. In addition to urban versus rural interests, water is also needed for environmental concerns, industry, and other uses. Kayseri’s main water concerning problem is to implement water to the center of life.

Currently 99% of the potable water consumption in Kayseri is based on ground water (Uyak et al., 2008). Based on the fact that the main source of drinking and utility water is ground water, the current resource must be managed properly and any pollution that might occur must be prevented accordingly. Conservation of the rural areas is fundamental for the water security of Kayseri since it has a direct bearing on groundwater recharge.

3.3.2. Open Channel Design

Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs. Manning’s equation or water surface profile information can be used to estimate average flow velocity in an open channel (USDA, 1986).

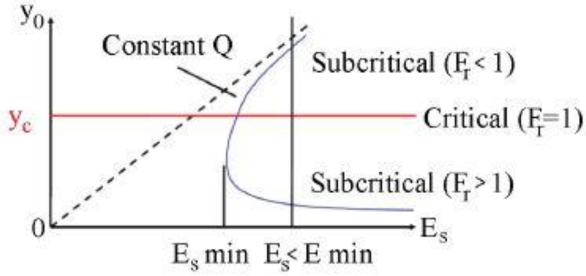


Figure 3.10. Specific energy schema for an open channel (White, 2006).

Specific Energy, E_s (m) (Equation 3.6) represents the height of the energy line above the channel bottom where Y_0 is the water surface elevation (m) and V is the velocity in an open channel (m/s). Critical water depth (Equation 3.7) is determined in order to prevent flow from ranging as super critical flow.

$$E_s=Y_0+V^2/2g \tag{3.6}$$

$$Y_c= (Q^2/gb^2)^{2/3}= (q^2/g)^{1/3} \tag{3.7}$$

In fluid mechanics, the Froude number (Equation 3.8) is used to determine the resistance of an object moving through water, and permits the comparison of objects of different sizes which is based on the speed/length ratio.

For any type of a channel, Froud's number is calculated by Equation 3.8. where b (m) is the bottom width of the channel, Q (m^3/s) is the flow rate and A (m^2) is the cross sectional area of an open channel.

$$F_r = (Q^2b/gA^3)^{1/2} \tag{3.8}$$

Critical depth, the depth of flow at which the specific energy is a minimum for a given flow rate and channel cross shape, and a unique relationship exists between depth and specific energy. Normal depth is the depth at which uniform flow occurs when the discharge rate is constant. Friction and gravity forces are in balance. Subcritical flow; lower energy, lower velocity flow, which occurs when the normal depth is greater than the critical depth.

Subcritical flow is controlled by downstream conditions. Supercritical flow is a high energy, high velocity flow, which occurs when the normal depth is less than the critical depth. Supercritical flow is controlled by upstream conditions.

Critical flow occurs at minimum energy level of $E_{s,min}$. The flow is classified as follows:

$y_0>y_c$, $V<V_c$: Subcritical (Fr<1)

$y_0=y_c$, $V=V_c$: Critical (Fr=1)

$y_0<y_c$, $V>V_c$: Supercritical (Fr>1)

The control of the water velocity flows through an open channel should be checked accordingly to flow classification limits of Froud's number (Nalluri et al., 2001).

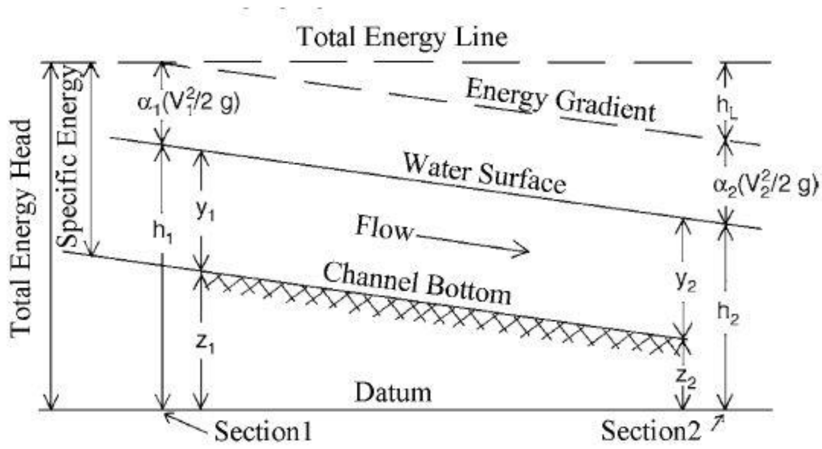


Figure 3.11. Schematic Representation of terms used in the energy equation (Mccarley, 1990).

The following equations are those most commonly used to analyze open channel flow.

Continuity Equation: The continuity equation is the statement of conservation of mass in fluid mechanics. For the special case of steady flow of an incompressible fluid, it assumes the simple form as indicated in Equation 3.9.

$$Q = A_1V_1 = A_2V_2 \tag{3.9}$$

where;

Q = discharge (m^3/s);

A = flow cross-sectional area (m^2);

V = mean cross-sectional velocity (m/s) (which is perpendicular to the cross section).

The continuity equation can be used with Manning's equation to obtain the steady uniform flow velocity via Equation 3.10.

$$V = Q/A = [(1.49/n)\, R^{2/3}\, S^{1/2}] \tag{3.10}$$

where;

R= the hydraulic radius of an open channel (m)

S= slope of an open channel (m/m)

Energy Equation: The energy equation expresses conservation of energy in open-channel flow expressed as energy per unit weight of fluid, which has dimensions of length and is therefore called energy head. The energy head is composed of potential energy head (elevation head), pressure head, and kinetic energy head (velocity head). These energy heads are scalar quantities that give the total energy head at any cross section when added. Written between an upstream open-channel cross section designated 1 and a downstream cross section designated 2, the energy equation is presented in Equation 3.11.

$$h_1 + a_1 (V_1^2 /2g) = h_2 + a_2 (V_2^2 /2g) + h_L \qquad (3.11)$$

where;

h_1 and h_2 are the upstream and downstream stages, respectively (m);

a = kinetic energy correction coefficient;

V = mean velocity (m/s); and

h_L = headloss (m)

Local cross sectional changes (minor loss) are effective factors on the water depth in an open channel as well as the boundary resistance (m). The stage h is the sum of the elevation head z at the channel bottom and the pressure head, or depth of flow y.

$$h = z + y \qquad (3.12)$$

Equation 3.12 states that the total energy head at an upstream cross section is equal to the energy head at a downstream section plus the intervening energy head loss. The energy equation can be applied only between two cross sections, at which the streamlines are nearly straight and parallel, so that vertical accelerations can be neglected.

Because rectangular and triangular sections are special cases of trapezoidal sections, it follows that the best hydraulic section for trapezoidal sections is the one of primary practical interest (CRC, 2001).

In this study both triangular and trapezoidal cross section design calculations are going to be evaluated. According to following design parameters, maximum water depth in the channel is going to be predicted and maximum water velocities will be obtained (Figure 3.12).

Given that the desired side slope, *M* to one, has been selected for a given channel, the minimum wetted perimeter (*P*) exists when:

$$P = 4y\ (1 + M^2)\ 0.5 - 2My \qquad (3.13)$$

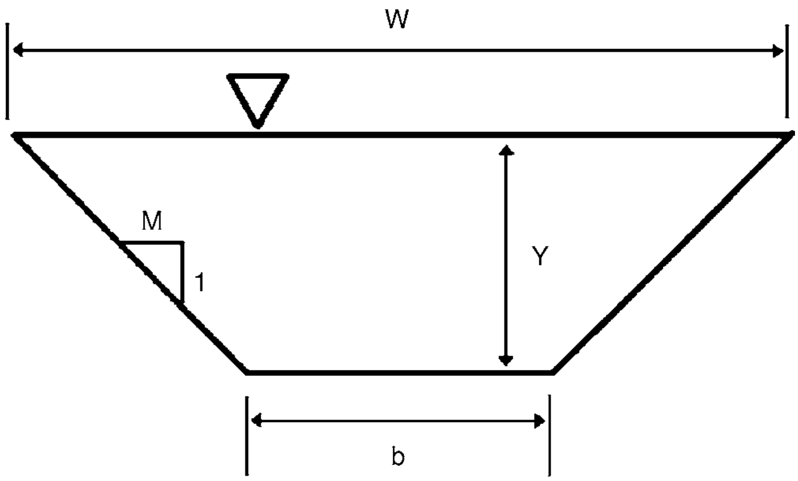


Figure 3.12. Cross-section of a trapezoidal channel - definition of variables (CRC, 2001).

From the geometry of the channel cross section and the Manning equation, design equations can be developed for determining the dimensions of the best hydraulic section for a trapezoidal channel. The depth of the best hydraulic section is defined by Equation 3.14.

$$y = C_M\ (Qn/(S^{1/2}))^{3/8} \qquad (3.14)$$

where;

$$C_M = \frac{\left[\left(k+2\left(M^2+1\right)^{(0.5)}\right)^{\frac{2}{3}}\right]^{\frac{3}{8}}}{\left[1.49(k+M)^{\frac{5}{3}}\right]^{\frac{3}{8}}} \qquad (3.15)$$

where;

M = slope of the desired side slop (m/m)

K and C_M = Empirical constants determined in accordance with M (unitless) (Table 3.8).

The associated bottom width is:

$$b = ky \tag{3.16}$$

The cross-sectional area of the resulting channel is:

$$A = by + My^2 \tag{3.17}$$

For triangular and trapezoidal cross section areas, for the selected slope of trapezoidal side slope ratio, the empirical values of C_M, k are constants for side slopes, are determined (Table 3.9).

Table 3.9. Values of C_M and k for determining bottom width and depth of best hydraulic section.

M	C_M	k
0/1	0.79	2
0.5/1	0.833	1.236
0.577/1	0.833	1.155
1.0/1	0.817	0.828
1.5/1	0.775	0.606
2.0/1	0.729	0.472
2.5/1	0.688	0.385
3.0/1	0.653	0.325
3.5/1	0.622	0.28
4.0/1	0.595	0.246
5.0/1	0.522	0.198
6.0/1	0.518	0.166
8.0/1	0.467	0.125
10.0/1	0.43	0.1
12.0/1	0.402	0.083

Table 3.10. Manning's n roughness coefficients for artificial channels (USDA, 1986).

		Depth Range		
Lining Category	Lining Type	0-0.15m	0.15-0.6 m	>0.6 m
Rigid	Concrete	0.015	0.012	0.013
	Grouted riprap	0.040	0.030	0.028
	Stone masonry	0.042	0.032	0.030
	Soil cement	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Unlined	Bare Soil	0.023	0.020	0.020
	Rack cut	0.045	0.035	0.025
Temporary	Woven paper net	0.016	0.015	0.015
	Jute net	0.028	0.022	0.019
	Fiberglass roving	0.028	0.022	0.019
	Straw with net	0.065	0.033	0.025
	Curled wood mat	0.066	0.035	0.028
	Synthetic mat	0.036	0.025	0.021
Gravel riprap	2.5 cm-D50	0.044	0.033	0.030
	5 cm-D50	0.066	0.041	0.034
Rock riprap	15 cm-D50	0.104	0.069	0.035
	30 cm-D50	-	0.078	0.040

3.3.3. Storage Tank Design

The conventional Rational Method for estimating peak discharges *Q* for channel design may provide inaccurate estimates.

In the formula,

$$Q = C.i.A \tag{3.18}$$

C is the runoff coefficient, *i* is the average rainfall intensity (mm) associated with the time of concentration, and *A* is the drainage area (m^2). Much of the debate in the literature is about how to select the average rainfall intensity, *i*. However, the analysis shown earlier indicates the large potential error in the estimate of *CA* if conventional design methods are used (Lee et al., 2003).

The runoff flowing through drainage channel is harvested by making tanks of suitable capacities at appropriate location in the site. Since storm flow is unsteady and intermittent, storage facilities are a necessary consideration for the system.

The flow is released by gravity. As presented in Figure 3.13, a storm-response system is going to be developed using EPA SWMM 5.0. The runoff that can be collected through outlets is going to be calculated in order to evaluate the water volume and storage tank capacity.

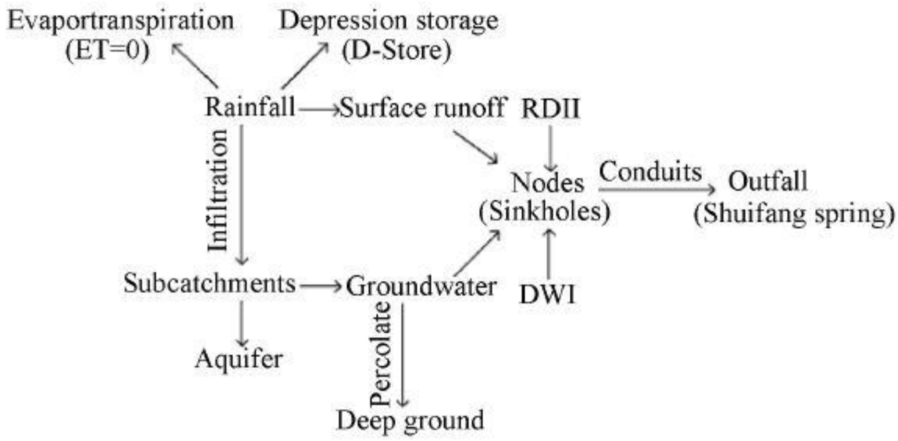


Figure 3.13. Storm responses of the system (Yuexia et al., 2007).

Within the context of cities of the future project, in order to maintain pre-development runoff conditions at a newly developed site such as Harikalar Diyarı, underground storm water retention/detention systems that captures and stores runoff is going to be designed (Figure 3.14). These systems can be installed quickly, and depending on the selected construction materials, maintenance work is minimum.

After the required storage volume is determined, with site examination, the configuration, and excavation requirements can be determined by the design engineer (EPA Storm Water Technology Fact Sheet, 2001).

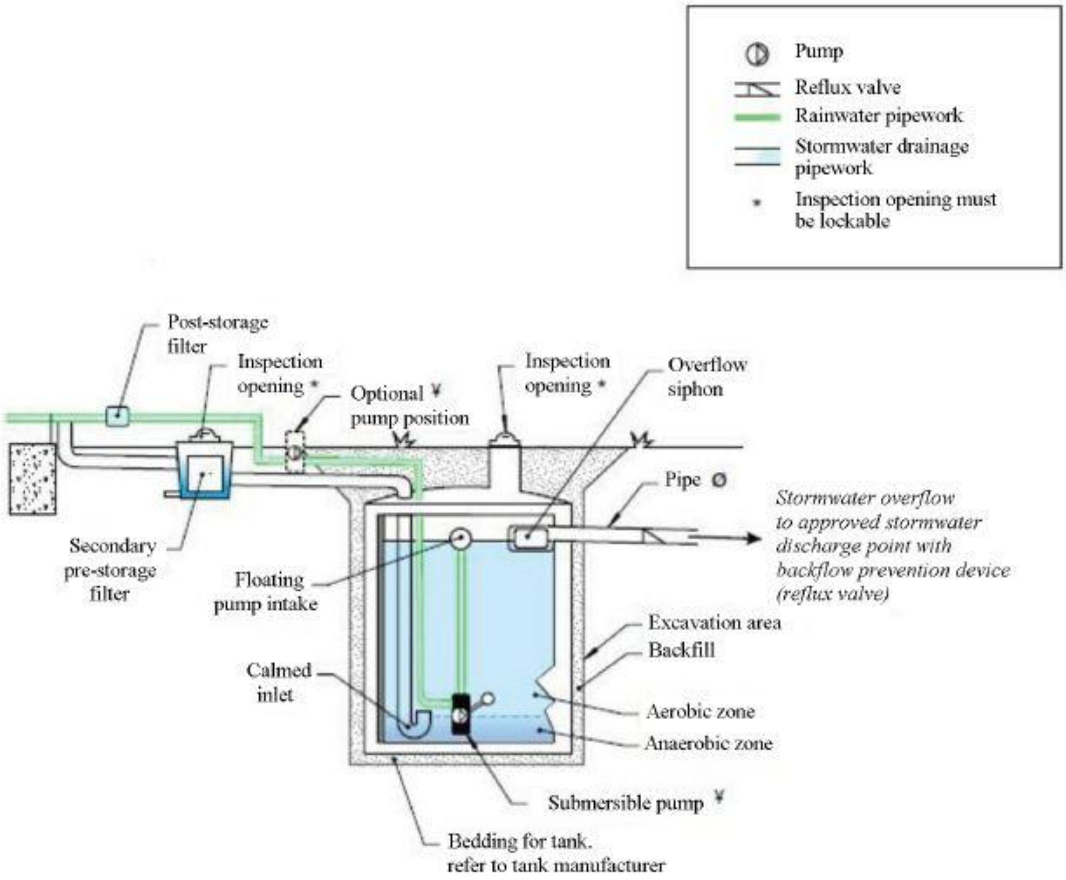


Figure 3.14. Schematic representation of the rain water storage tank, under the surface level (AUGNWC, 2008).

In this study, the water ways are going to be presented while two outlets are going to be introduced to the system so as two water tanks are going to be designed according to the outflow of the mentioned outlets.

Table 3.11. presents design considerations for construction materials of underground storage tanks. Plastic (HDPE) storage tanks are selected due to their convenient properties such as, easiness of handling, available sizes, minimum space requirements and minimum maintenance entailments.

Table 3.11. Comparison of design considerations for construction materials of underground retention/detention systems.

Shapes	Construction Material		
	Concrete	Plastic (HDPE)	Steel and Aluminum
Spatial Requirements	Rectangular vaults or circular pipes	Circular pipes	Circular pipes, semi circular pipe-arches, or other special shapes
Rigidty/Flexibility	Primarily continuous space with no angles	Can be fitted into irregular and angled spaces	Can be fitted into irregular and angles spaces
Fill Requirements	Very rigid, does not require fill to maintain rigidity; not flexible	Rigid, requires fill for stability; not flexible	Rigid, requires fill for stability; can withstand some shifting without breaking or buckling
Other Requirements	Requires minimum fill above structure	Requires minimum fill between and above pipes	Requires minimum fill between and above the pipes
Other Requirements	None	Requires minimum spacing between pipes. Water table must be below level of pipe	Requires minimum spacing between pipes
Available sizes	Multiple sizes that can be pre-cast or cast in place	Multiple pipe diameters are available; all are pre-manufactured	0.3 m-3.7 m diameters and pipe arches are available pre-assembled. Larger diameter pipe and pipe arches are available for assembly on-site.
Handling	Requires moving equipment	Can be moved by hand	Requires moving equipment

4. RESULTS AND DISCUSSION

4.1. Installation of Backdrop Map

The first step of setting up EPA SWMM 5.0 model is loading of the backdrop map. Layout map must be loaded aligned with the AutoCAD drawing of the layout of the project. The physical dimensions of the map are defined so that map coordinates can be properly scaled to the computer's video display. The coordinates of the study area map are set as they are in the AutoCAD version of the study area (Figure 4.1).

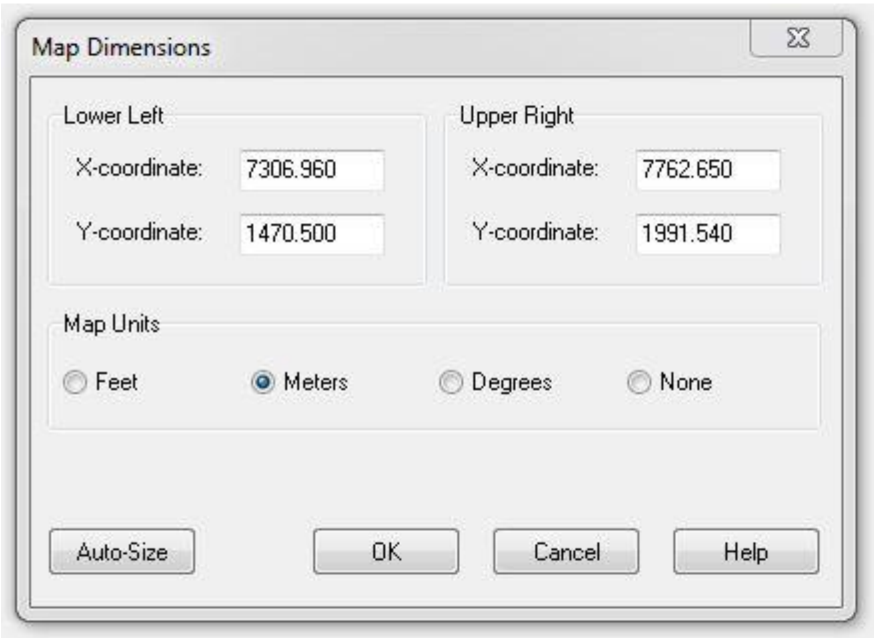


Figure 4.1. Map units and coordinates set up.

Loading of the back drop map image with the right coordinates is very important since whole measurements and calculations such as conduit lengths, areas, and node locations will be automatically set by EPA SWMM 5.0.

Imperviousness can be calculated using either land use or land cover maps, but the combination of land use and land cover provides the most meaningful information for a particular area. The layout of the Kumarlı, "Harikalar Diyarı" Project was obtained.

Subsequently, the contour map of the area was acquired and combined with the layout plan (Figure 4.2.). With the help of the combined map, the land uses and altitudes were specified clearly.

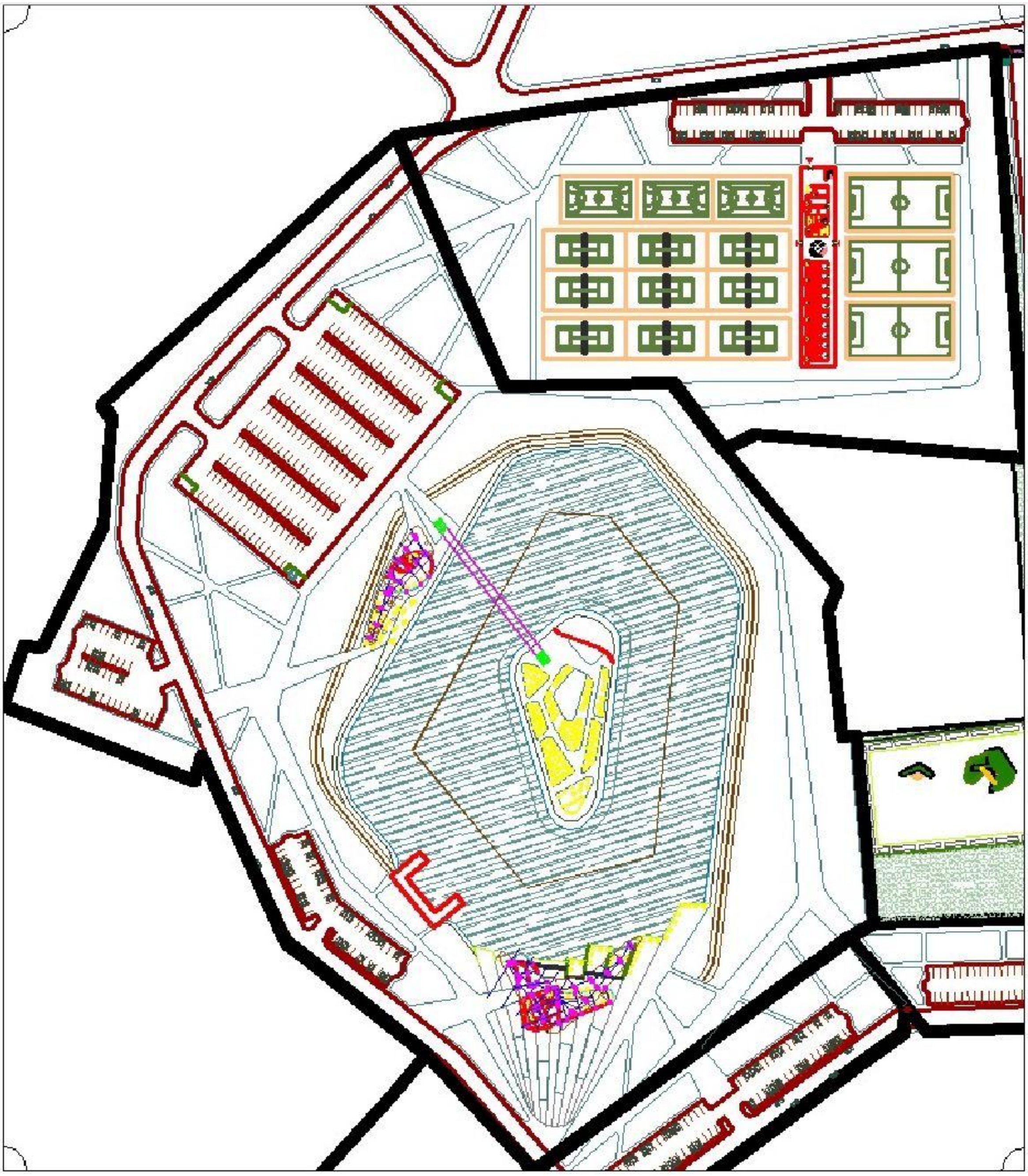


Figure 4.2. Back-drop image of the model.

4.2. Mitigation of Spatial Data into Hydrological Model

In this study, Kayseri "Harikalar Diyarı" Project Water-ski Park drainage system for separate storm water channel serving a 98.000 m^2 area will be modeled. The system layout is shown in Figure 4.3 consisting subcatchment areas *S1* through *S30*, storm sewer conduits *CA1-CA11; CB1-CB6; CC1-CC13*, and conduit junctions *JA1-J11*; *JB1-JB7; JC1-JC13.* The system discharges to two storage tanks at the point labeled as *OUTLET1 and OUTLET 2*.

Firstly, the visual objects on SWMM's study area map will be entered and the various properties of these objects will be set. Then model will simulate the water quantity and response to the events that will occur continuously for a 3-year rainfall record. (Appendix-A).

Average evaporation data and a 3-year temperature data (Appendix-B) set is introduced to the model as input parameters. Subcatchments are created having regarded to land use purposes. Conduit connections were designed in accordance with the layout plan and altitudes with a specific minimum slope for an open channel: 0.003.

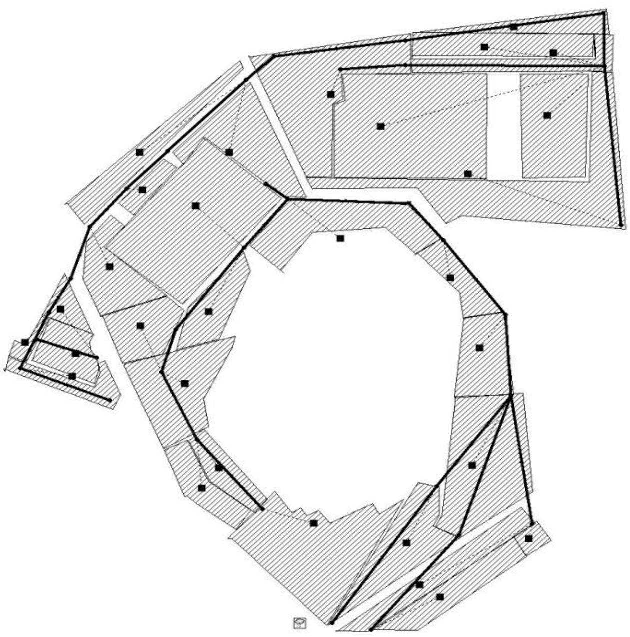


Figure 4.3. The system lay out consisting SWMM 5.0 elements.

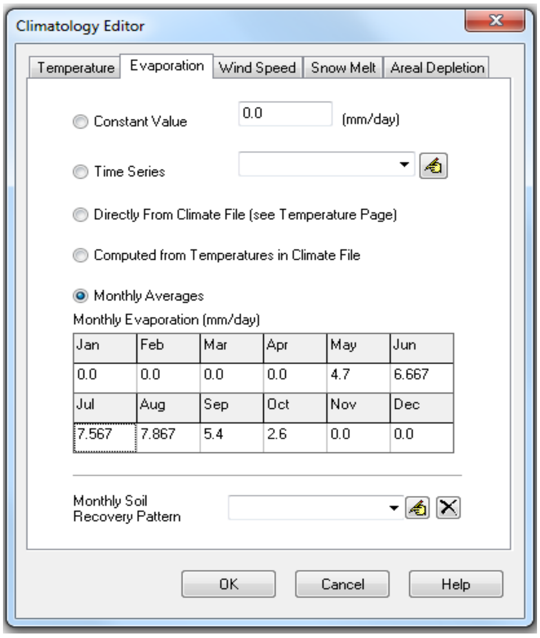


Figure 4.4. Introducing the evaporation data as an input parameter to the model.

Under climatology tab, the monthly average evaporations (Figure 4.4) for long years were introduced to the model in order to take evaporation losses.

In this design, water flows by gravity. The elevations of the study area decrease from the bottom of the study are to the top. The elevation difference between the highest and lowest points of the study area is 5.3 m. The conduits are installed on the paths instead of the grasslands. Not only the conduit properties but also junction properties were set in accordance with the decreasing elevation (Table 4.1 and Table 4.2).

Land use and surface characteristics are the main criteria determining the imperviousness of a specific catchment area. Subcatchment imperviousness values were calculated according to land use based on zoning information percentages in the created subcatchments (Table 4.3). Subcatchment areas were set by the model automatically while the widths perpendicular to the flow directions were measured via AutoCAD drawing of the study area. The automatically set areas are cross-checked via AutoCAD drawing in order to justify whether if the right area magnitudes are used in the module before running (Figure 4.5).

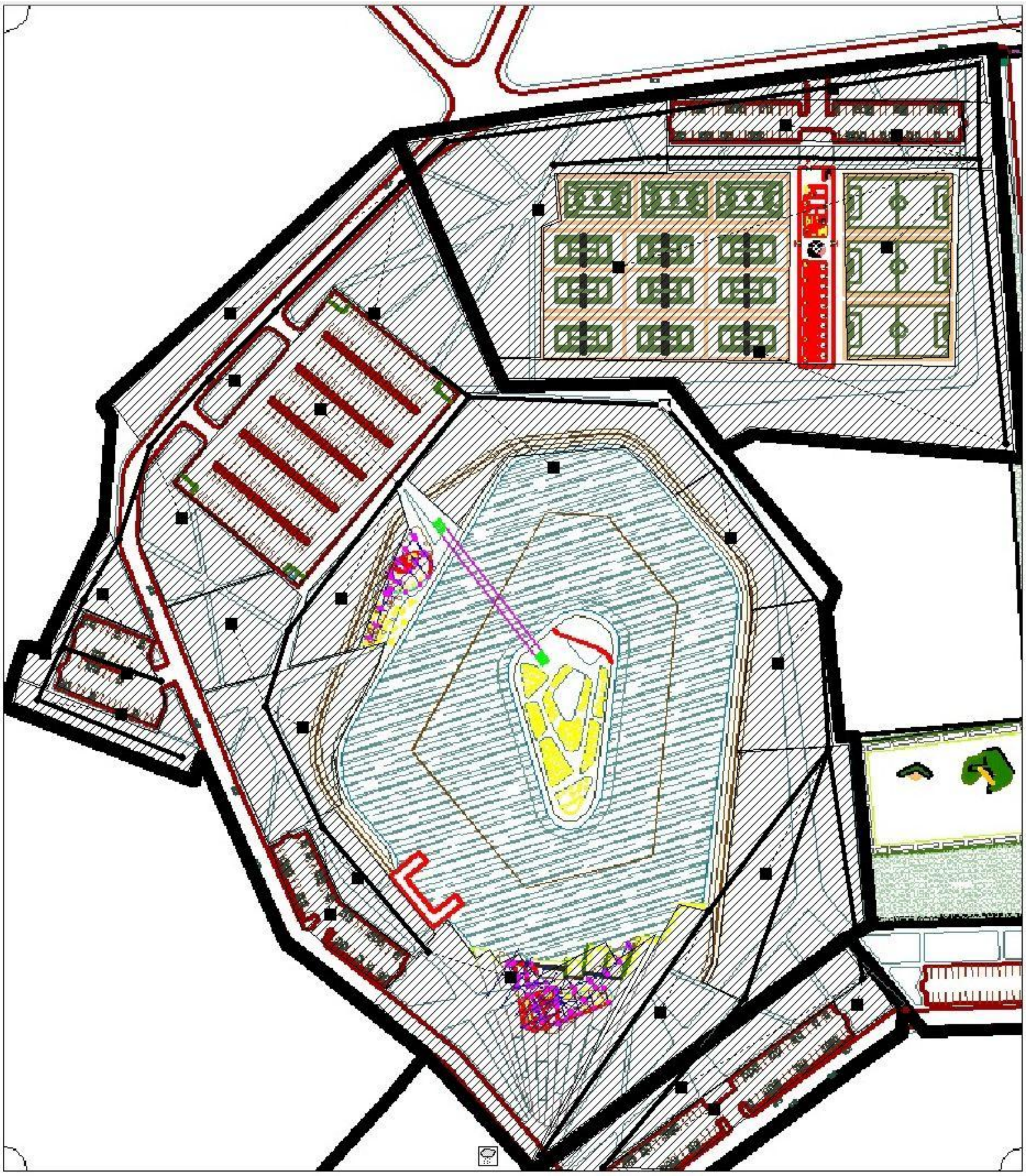


Figure 4.5. Developed layout after installation of the visual objects.

Table 4.1. Installed Conduit Properties.

Conduit Name	Inlet Node	Outlet Node	Length	Invert elevation (m)	Slope	Out (m)
A1-A5	A1	A5	98.37	1068.171	0.003	1067.876
A3-A4	A3	A4	131.13	1068.171	0.003	1067.775
A2-A6	A2	A6	102.65	1068.171	0.003	1067.863
A4-A6	A4	A6	89.53	1067.376	0.003	1067.107
A5-A6	A5	A6	116.60	1067.376	0.003	1067.026
A6-A7	A6	A7	64.69	1066.526	0.003	1066.332
A7-A8	A7	A8	74.51	1065.832	0.003	1065.609
A8-OUTFALL1	A8	OUTFALL1	38.16	1065.109	0.003	1064.994
A10-A11	A10	A11	126.98	1064.700	0.003	1064.319
A11-OUTFALL2	A11	OUTFALL2	41.84	1063.819	0.003	1063.694
B1-B2	B1	B2	72.44	1066.800	0.003	1066.583
B2-B3	B2	B3	59.75	1066.083	0.003	1065.903
B3-B4	B3	B4	35.10	1065.403	0.003	1065.298
B4-B5	B4	B5	81.58	1064.798	0.003	1064.553
B5-B6	B5	B6	49.83	1064.053	0.003	1063.913
B7-B6	B7	B6	20.09	1064.900	0.003	1064.840
B6-OUTFALL1	B6	OUTFALL1	87.27	1064.800	0.003	1064.538
C1-C3	C1	C3	68.88	1065.200	0.003	1064.993
C4-C2	C4	C2	45.56	1064.900	0.003	1064.763
C3-C5	C3	C5	49.15	1064.493	0.003	1064.346
C5-C6	C5	C6	31.43	1063.846	0.003	1063.752
C6-C7	C6	C7	42.94	1063.252	0.003	1063.123
C7-C8	C7	C8	40.29	1062.623	0.003	1062.502
C8-C9	C8	C9	42.15	1062.002	0.003	1061.875
C9-C10	C9	C10	80.01	1061.375	0.003	1061.135
C10-C11	C10	C11	27.33	1060.635	0.003	1060.553
C11-C12	C11	C12	94.69	1060.053	0.003	1059.769
C12-OUTFALL2	C12	OUTFALL2	144.29	1059.269	0.003	1058.837
C11*-C13	C11*	C13	46.52	1064.300	0.003	1064.160
C13-A11	C13	A11	142.96	1063.660	0.003	1063.232

Table 4.2. Installed Junction Properties.

Junction Name	Invert Elevation (m)	Max Depth (m)
A1	1069.60	0.50
A2	1068.80	0.50
A3	1069.90	0.50
A4	1068.60	0.50
A5	1068.70	0.50
A6	1067.50	0.50
A7	1066.30	0.50
A8	1065.60	0.50
A10	1065.20	0.50
A11	1064.50	0.50
B1	1067.30	0.50
B2	1066.10	0.50
B3	1065.40	0.50
B4	1065.35	0.50
B5	1065.30	0.50
B6	1065.25	0.50
B7	1065.40	0.50
C11*	1064.80	0.50
C1	1065.70	0.50
C2	1065.40	0.50
C3	1065.60	0.50
C4	1065.70	0.50
C5	1065.30	0.50
C6	1065.25	0.50
C7	1065.20	0.50
C8	1065.15	0.50
C9	1065.00	0.50
C10	1064.90	0.50
C11	1064.80	0.50
C12	1065.50	0.50
C13	1064.60	0.50

Table 4.3. Installed Subcatchment Properties.

Subcatchment	H 1	H 2	Length (m)	Width (m)	Slope (%)	Imperviousness (%)
1	1065.40	1065.30	112.00	72.00	0.0893	100
2	1064.90	1064.70	109.71	81.81	0.1823	100
3	1064.90	1064.30	82.50	48.00	0.7273	100
4	1069.70	1069.50	129.00	17.00	0.1550	100
5	1067.70	1066.70	72.29	17.00	1.3833	100
6	1065.70	1065.40	48.00	35.00	0.6250	100
7	1064.70	1064.30	129.00	17.00	0.3101	100
8	1064.70	1064.30	50.93	14.98	0.7853	15
9	1064.60	1064.30	133.00	14.64	0.2256	15
10	1064.90	1064.50	109.29	30.85	0.3660	20
11	1065.40	1065.20	201.22	37.26	0.0994	25
12	1065.90	1065.40	93.72	35.45	0.5335	15
13	1065.40	1065.30	50.00	15.59	0.2000	0
14	1065.50	1065.30	70.19	35.50	0.2849	10
15	1065.80	1065.40	91.77	40.96	0.4359	5
16	1065.40	1065.30	53.04	33.54	0.1885	15
17	1067.00	1065.70	76.46	28.04	1.7003	17
18	1067.00	1066.30	34.51	26.37	2.0283	17
19	1068.70	1067.90	84.27	71.92	0.9493	25
20	1069.10	1068.80	160.30	19.51	0.1872	7
21	1069.10	1068.60	127.30	17.04	0.3928	10
22	1068.80	1067.70	92.43	43.79	1.1901	30
23	1067.30	1066.10	71.78	23.37	1.6718	15
24	1066.10	1065.80	67.93	49.50	0.4417	23
25	1065.40	1064.90	178.29	26.55	0.2804	50
26	1065.40	1065.30	30.89	29.87	0.3237	0
27	1065.60	1065.30	35.00	17.65	0.8571	0
28	1065.70	1065.50	88.13	15.32	0.2269	0
29	1065.70	1065.40	58.34	61.89	0.5142	10
30	1069.50	1069.20	17.00	21.96	1.7647	7

4.3. Preliminary Results

EPA SWMM 5.0 model attempts to describe the total runoff amount of the system that can be stored for different purposes of water. Stored water can be used for different areas such as recharge of the water-ski lake and irrigation of the parks. Rainwater analyses were conducted in accordance with *Standard Methods for the Examination of Water and Wastewater* (APHA, 1998) at KASKI. With respect to the EPA's storm water quality standards, rainwater samples taken from Kayseri-city center were analyzed in KASKI laboratories. Results are compared with WHO Guidelines for water reuse for different purposes (WHO/CEHA, 2006) and EU Directive for bathing water (EU, 2006/7/EC).

Table 4.4. Kayseri's rain water analyses results versus WHO guidelines for water reuse for different purposes and EU directive for bathing water.

	WHO guidelines for water reuse for different purposes (WHO/CEHA, 2006)			EU Directive for Bathing Water (2006/7/EC)		
				Excellent Quality	Good Quality	KASKI Analyze Results
Parameter	Irrigation of ornamentals, fruit trees and fodder crops	Irrigation of vegetables likely to be eaten uncooked	Toilet flushing			
Sensory test	no abnormal change in color	no abnormal change in color	no abnormal change in color	no abnormal change in color	no abnormal change in color	no abnormal change in color
Total suspended solids (mg/L)	≤ 140	≤ 20	≤ 10	n/a	n/a	22
Oxygen saturation%				80-120	n/a	80.40
BOD_5 (mg/L)	≤ 240	≤ 20	≤ 10	n/a	n/a	8
pH	neutral	neutral	neutral	neutral	neutral	6.5
Total coliforms (cfu/100mL)	≤ 100	≤ 100	≤ 10	n/a	n/a	<100
Fecal coliforms (CFU/100mL)	≤ 1000	≤ 200	≤ 10	250	500	<200

As a result, the storm water quality objectives meet requirements for use of both irrigation purposes and bathing water purposes. The harvested rainwater has the tremendous potential not only for meeting the requirements for irrigation but also recharging of an artificial lake. By constructing storage tanks, and preventing seepage losses through the whole system, it is possible to meet water demands of the study area.

Kayseri, Kumarlı region receives 406 mm/m^2 rainfall of which, 35 mm/m^2 water is lost annually through evaporation. Although the region receives a moderate amount of rainfall, it lacks appropriate rainwater management technology, coupled with lack of suitably designed soil and water conservation measures which lead to severe water shortages, particularly in the summer season. The summer months of June to September are the major part of the water deficit period, while the months from March to May are considered as the water surplus period. The mean values of water surplus (157 mm) and water deficit (18 mm) also indicate that net water surplus in the region exceeds water deficits.

As a result of the simulation by means of EPA SWMM 5.0, the maximum velocities and depth through conduits are presented in Table 4.5.

Two outlets were introduced for the entire catchment by EPA SWMM 5.0, using simulations of 3-year precipitation data. In this study, the designed collectors have adequate hydraulic capacity to pass forward predicted dry weather and storm flows for events of a 2.4-year return period or less (Table 4.7).

Collection of harvested runoff water in micro/macro water harvesting tanks/structures of small to medium size covering reasonably large catchment area has proved successful, provided seepage as well as evapotranspiration losses are checked (Hadda et. al., 2009).

Table 4.5. Maximum velocities and maximum depths through conduits.

Conduit Name	Inlet Node	Outlet Node	Length (m)	Max velocity (m/sn)	Max depth (m)
A1-A5	A1	A5	98.37	0.61	0.10
A3-A4	A3	A4	131.13	0.00	0.00
A2-A6	A2	A6	102.65	0.72	0.09
A4-A6	A4	A6	89.53	0.75	0.09
A5-A6	A5	A6	116.60	0.52	0.01
A6-A7	A6	A7	64.69	0.93	0.16
A7-A8	A7	A8	74.51	0.75	0.19
A8-OUTFALL1	A8	OUTFALL1	38.16	0.79	0.19
A10-A11	A10	A11	126.98	0.52	0.15
A11-OUTFALL2	A11	OUTFALL2	41.84	0.71	0.30
B1-B2	B1	B2	72.44	1.06	0.12
B2-B3	B2	B3	59.75	0.72	0.15
B3-B4	B3	B4	35.10	0.36	0.27
B4-B5	B4	B5	81.58	0.27	0.31
B5-B6	B5	B6	49.83	0.37	0.36
B7-B6	B7	B6	20.09	0.00	0.00
B6-OUTFALL1	B6	OUTFALL1	87.27	0.31	0.42
C1-C3	C1	C3	68.88	0.00	0.00
C4-C2	C4	C2	45.56	0.00	0.00
C3-C5	C3	C5	49.15	0.32	0.07
C5-C6	C5	C6	31.43	0.20	0.10
C6-C7	C6	C7	42.94	0.18	0.11
C7-C8	C7	C8	40.29	0.34	0.15
C8-C9	C8	C9	42.15	0.36	0.13
C9-C10	C9	C10	80.01	0.25	0.16
C10-C11	C10	C11	27.33	0.46	0.19
C11-C12	C11	C12	94.69	0.44	0.19
C12-OUTFALL2	C12	OUTFALL2	144.29	0.34	0.22
C11*-C13	C11*	C13	46.52	0.45	0.15
C13-A11	C13	A11	142.96	0.28	0.21

The geometry of conduits designed in the model for each subcatchment drained through “Harikalar Diyarı”, modeled as a triangular cross section by the use of combined layout plan and contour map. Channel bed slopes are set to be 0.003, and the Manning roughness coefficient is selected as 0.012 (Table 3.6 and Table 3.7). The total length of the modeled channel is 2147 m.

4.3.1. Outflow Volume Determinations

Monthly peak flow rate of the system is presented in Table 4.6. with the return periods of the monthly occurred rainfall events. In order to determine the net runoff potential of the catchment area, total runoff resulting from the subcatchment areas and introduced 2 outfalls, OUTFALL1 and OUTFALL2 (Table 4.8 and Table 4.9) are listed.

Table 4.6. Monthly peak flows of the system.

Start Date	Monthly Duration (hours)	Monthly Peak (CMS)	Exceedance Frequency (%)	Return Period (months)
4/1/2010	312	0.021	4.35	29
3/1/2010	336	0.021	8.7	14.5
3/1/2009	504	0.019	13.04	9.67
2/1/2010	336	0.017	17.39	7.25
10/4/2010	168	0.016	21.74	5.8
4/4/2009	336	0.014	26.09	4.83
7/7/2009	72	0.011	30.43	4.14
12/1/2009	384	0.011	34.78	3.63
11/2/2009	240	0.011	39.13	3.22
1/4/2010	528	0.01	43.48	2.9
2/1/2009	312	0.01	47.83	2.64
1/1/2011	360	0.009	52.17	2.42
2/1/2011	360	0.009	56.52	2.23
9/9/2010	24	0.005	60.87	2.07
1/2/2009	336	0.004	65.22	1.93
3/1/2011	360	0.003	69.57	1.81
12/12/2010	168	0.003	73.91	1.71
6/15/2009	24	0.003	78.26	1.61
5/1/2009	96	0.002	82.61	1.53
5/20/2010	48	0.002	86.96	1.45
9/23/2009	24	0	91.3	1.38
11/26/2010	48	0	95.65	1.32

Table 4.7. Total runoff amounts and imperviousness percentages of subcatchments.

Subcatchment	Imperviousness (%)	Total Runoff , (10^6 L)
1	100	7
2	100	7.8
3	100	3.44
4	100	1.84
5	100	1.33
6	100	1.33
7	100	2.1
22	30	2.05
11	25	1.96
19	25	1.7
10	20	1.43
25	50	1.13
24	23	1.05
16	15	0.75
12	15	0.7
18	17	0.55
21	10	0.53
9	15	0.36
29	10	0.35
17	17	0.34
20	7	0.34
8	15	0.31
23	15	0.29
15	5	0.26
28	0	0.12
14	10	0.4
26	0	0.08
30	7	0.07
13	0	0.06
27	0	0.03

According to the total runoff results for each subcatchment area, the areas that have higher imperviousness which indicates water infiltration is lower and water diverges as runoff, the cumulative water runoff level is higher. Higher water elevations are expected in the constructed conduits bordering in the related subcatchment areas.

Table 4.8 and 4.9 shows the peak flow frequency exceedance for each of the two outfalls.

Table 4.8. Monthly inflow levels for OUTFALL1.

Start Date	Monthly Duration (hours)	Monthly Peak (CMS)	Exceedance Frequency (%)	Return Period (months)
4/1/2010	312	0.01	4.35	29
3/1/2010	336	0.01	8.7	14.5
3/1/2009	504	0.009	13.04	9.67
2/1/2010	336	0.009	17.39	7.25
10/4/2010	168	0.008	21.74	5.8
4/4/2009	336	0.007	26.09	4.83
7/7/2009	72	0.005	30.43	4.14
12/1/2009	384	0.005	34.78	3.63
11/2/2009	240	0.005	39.13	3.22
1/4/2010	528	0.005	43.48	2.9
2/1/2009	312	0.005	47.83	2.64
1/1/2011	360	0.004	52.17	2.42
2/1/2011	360	0.004	56.52	2.23
9/9/2010	24	0.002	60.87	2.07
1/2/2009	336	0.002	65.22	1.93
3/1/2011	360	0.001	69.57	1.81
12/12/2010	168	0.001	73.91	1.71
6/15/2009	24	0.001	78.26	1.61
5/1/2009	96	0.001	82.61	1.53
5/20/2010	48	0.001	86.96	1.45
9/23/2009	24	0	91.3	1.38
11/26/2010	48	0	95.65	1.32

Exceedence frequencies and returning periods of the rainfall events results in similar trends of monthly peak flows for OUTFALL 1 and OUTFALL 2 (Figure 4.6). Runoff exceedances and flow frequency curves were produced for the catchment.

The flow frequency curves were then analyzed to determine what physical changes in the watershed might occurred. As a result, it was determined that the net runoff is diversed homogenously throughout the catchment area. No apparent physical change was observed in the water levels at the outfalls. No net flooding is recorded in the final status report of the model simulation and all links are stable.

Table 4.9. Monthly inflow levels for OUTFALL 2.

Start Date	Monthly Duration (hours)	Monthly Peak (CMS)	Exceedance Frequency (%)	Return Period (months)
4/1/2010	312	0.01	4.55	29
3/1/2010	240	0.01	9.09	14.5
3/3/2009	456	0.009	13.64	9.67
2/1/2010	312	0.008	18.18	7.25
10/4/2010	168	0.008	22.73	5.8
4/4/2009	288	0.007	27.27	4.83
7/7/2009	72	0.006	31.82	4.14
12/6/2009	312	0.006	36.36	3.63
11/3/2009	192	0.005	40.91	3.22
1/4/2010	528	0.005	45.45	2.9
2/1/2009	312	0.005	50	2.64
1/1/2011	336	0.004	54.55	2.42
2/1/2011	336	0.004	59.09	2.23
9/9/2010	24	0.002	63.64	2.07
1/2/2009	288	0.002	68.18	1.93
3/1/2011	336	0.002	72.73	1.81
12/12/2010	168	0.001	77.27	1.71
6/15/2009	24	0.001	81.82	1.61
5/1/2009	96	0.001	86.36	1.53
5/20/2010	48	0.001	90.91	1.45
9/23/2009	24	0	95.45	1.38

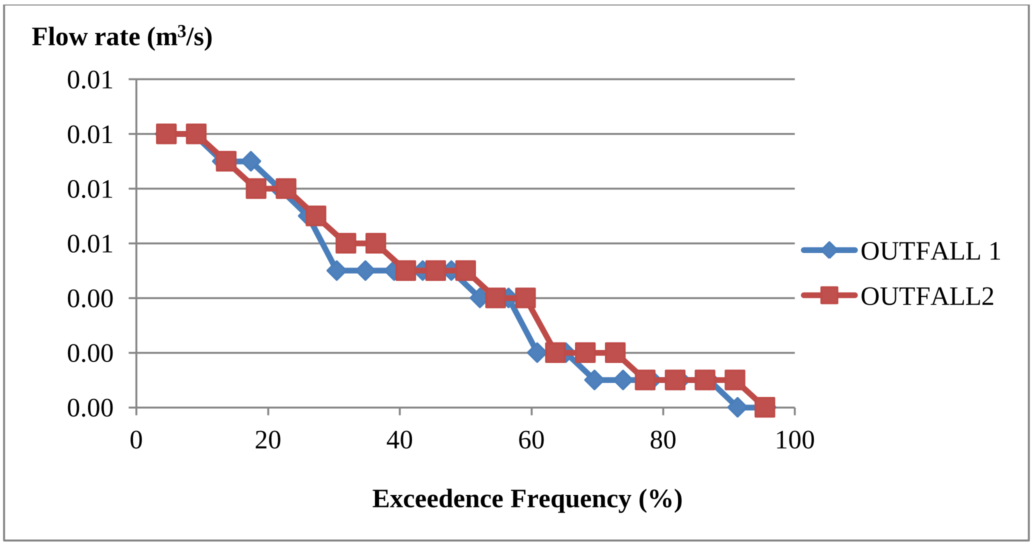


Figure 4.6. Exceedence frequencies for OUTFALL 1 and OUTFALL 2.

Total runoff amounts for OUTFALL 1 and OUTFALL 2 are presented in table 4.10. As a result, in a 3 year period of time maximum total runoff water volume that can be stored in a storage tank at OUTFALL 2 is 19,856 m^3 whereas the maximum storable runoff amount at OUTFALL 1 is 18,444 m^3.

Table 4.10. Net water volumes for outfalls and system.

Outfall Node	Flow Frequency Percent (%)	Average Flow (CMS)	Maximum Flow (CMS)	Total Volume 10^6 lt
Outfall 2	25.94	0.001	0.01	19.8
Outfall 1	25.65	0.001	0.01	18.4
System	25.75	0.002	0.021	38.2

4.3.2. Proposed Channel Design for the Catchment Area

Storm water open channels are generally designed to operate in a free-surface flow condition, with filling percentages of individual pipes being less than 70-80% (Ferreri et. al., 2010).

According to the model results, the maximum outflow rate was found to be 0.01 m^3/s. Slope of the channel was set to be 0.3%, for water flowing without any external forces through the study area. The latitudes of the area are convenient for water flowing down this slope. As expected, the role of geometry in modeling fluid throughout the channel is highly important.

In order to choose the best hydraulic cross section for the system, both trapezoidal and triangular cross sections are evaluated. Within the help of equation 3.14, critical water level is calculated for both cross section types. For trapezoidal channel side slope is chosen to be 0.5/1 and the bottom width as 0.2 m, while for triangular channel the total width and height is chosen to be 0.5 m (Table 4.11 and Table 4.12).

Table 4.11. Design Results of the Channel Design.

	b (m)	y (m)	M	A (m^2)
Trapezoidal	0.2	0.36	0.5	0.100
Triangular	0	0.50	0.4	0.100

Table 4.12. Critical water depth values of the channel according to the selection of best hydraulic section.

Trapezoidal									
k	C_M	A (m^2)	P (m)	M	$y_{critical\ (m)}$	R	$V_{manning\ (m/s)}$	F_R	R_e
1.236	0.833	0.1	2.55	0.5/1	0.08	0.04	0.79	0.04	4.41
Triangular									
k	C_M	A (m^2)	P (m)	M	$y_{critical\ (m)}$	R	$V_{manning\ (m/s)}$	F_R	R_e
2	0.79	0.1	1	0/1	0.08	0.1	1.47	0.05	11.24

For trapezoidal and triangular cross-section channels, water levels are found to be 0.36 m and 0.50 m respectively. In conclusion, the maximum depth of the channel is set to be 0.5 m. In order water to flow through the system with a maximum water level of 0.5, a triangular cross-section area for the conduits is set as the shape of the conduits Figure 4.7.

With the same cross sectional area amount, the wetted perimeter of a triangular channel is smaller, thus the cost of excavation and lining will be lower.

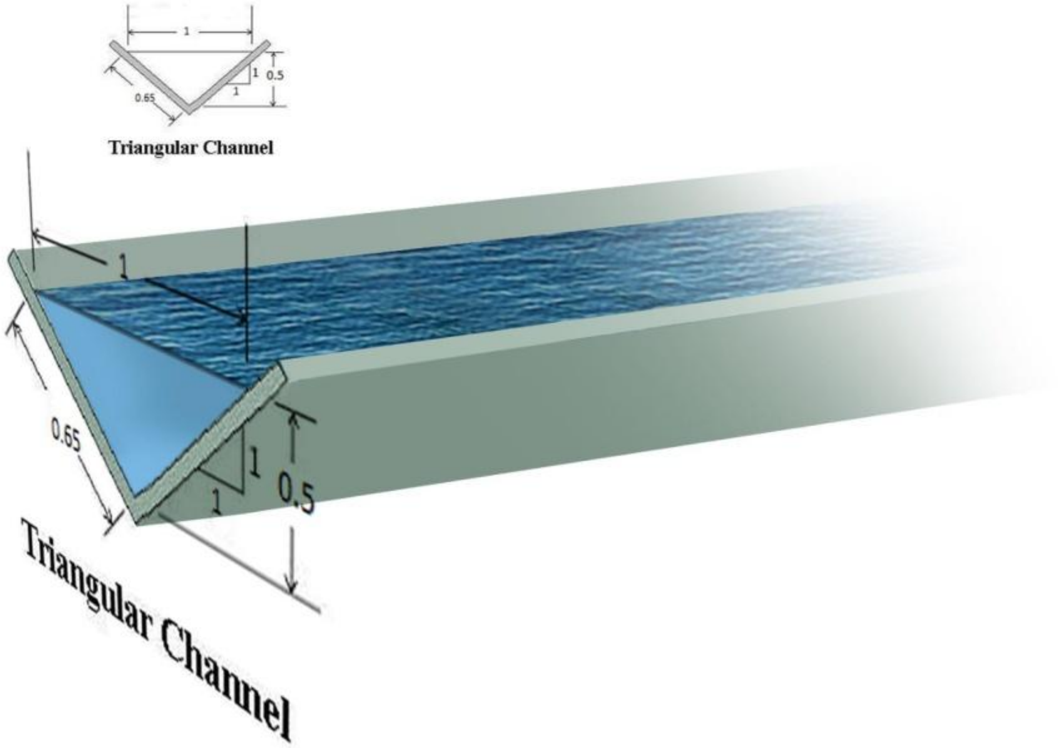


Figure 4.7. Cross section of the storm water channel.

The force of moving water on objects in its path increases with the square of velocity. Table 4.13 lists approximate flow depths that a child (20 kg) would be able to withstand while standing in a concrete bottom channel or gutter flowing at the selected velocities. If the public has access to the flow route, these combinations of gutter velocity and flow depth should not be exceeded in open-channel design.

Table 4.13. Permissible water levels and maximum velocities in an open channel (Alberta, 1999).

Water velocity (m/s)	Permissible depth (m)
0.5	0.8
1.0	0.32
2.0	0.21
3.0	0.09
Note: Based on a 20-kg child and concrete lined channels.	

In Table 4.14, for each conduit, the maximum water velocities are presented with the time occurrence rates. Also the maximum full depths of the conduits are listed. None of the conduits are surcharged by means of design considerations of channel width and height (Figure 4.6). Besides, in open channel hydraulics critical flow state plays an outstanding role since discharge and flow depth are related uniquely for a given channel geometry. So the selection of a cross sectional area type for an open channel becomes very important. For each type of cross-sectional area the water flow may vary as subcritical (Fr<1), critical (Fr=1) and supercritical (Fr>1). In order to have a steady flow and avoid surcharging on the conduits, the best practical geometry of a channel cross section area must be selected. Maximum velocity does not exceed the limits and the water flow is classified as subcritical flow (Fr=0.05) through the whole channel.

Table 4.14. Conduit properties.

Link	Type	Time of Max Occurance Days	Max Velocity (m/s)	Max/Full Flow	Max/Full Depth
A1-A5	Conduit	483	0.55	0.00	0.11
A3-A4	Conduit	483	0	0.00	0.00
A2-A6	Conduit	483	0.69	0.00	0.09
A4-A6	Conduit	483	0.68	0.00	0.10
A5-A6	Conduit	483	0.45	0.00	0.11
A6-A7	Conduit	483	0.81	0.01	0.17
A7-A8	Conduit	483	0.65	0.01	0.21
A8-OUTFALL1	Conduit	483	0.69	0.01	0.21
A10-A11	Conduit	483	0.6	0.01	0.16
A11-OUTFALL2	Conduit	483	0.62	0.05	0.32
B1-B2	Conduit	483	1.02	0.00	0.13
B2-B3	Conduit	483	0.63	0.01	0.16
B3-B4	Conduit	483	0.32	0.04	0.28
B4-B5	Conduit	483	0.24	0.05	0.33
B5-B6	Conduit	483	0.33	0.08	0.39
B7-B6	Conduit	483	0	0.00	0.00
B6-OUTFALL1	Conduit	483	0.27	0.12	0.45
C1-C3	Conduit	483	0	0.00	0.00
C4-C2	Conduit	483	0	0.00	0.00
C3-C5	Conduit	483	0.28	0.00	0.08
C5-C6	Conduit	483	0.18	0.00	0.11
C6-C7	Conduit	483	0.16	0.00	0.12
C7-C8	Conduit	483	0.31	0.01	0.16
C8-C9	Conduit	483	0.32	0.01	0.14
C9-C10	Conduit	483	0.22	0.01	0.17
C10-C11	Conduit	483	0.4	0.01	0.20
C11-C12	Conduit	483	0.38	0.01	0.21
C12-OUTFALL2	Conduit	483	0.29	0.02	0.24
C11*-C13	Conduit	483	0.52	0.01	0.16
C13-A11	Conduit	483	0.26	0.02	0.22

After the selection of the most appropriate design parameters of conduits, selected conduit properties such as the shape and invert elevations are entered to the model. Before launching to run the model, all data sets must be entered into the model in order to attain the accurate results for the simulation.

Figure 4.8. Before the implementation of the proposed channel and storage tanks.

In Figure 4.8, the study area is illustrated. After the implementation of the proposed channel, the Water-ski park of Kayseri, Harikalar Diyarı will attract more attention of people and the primary objective of the Cities of the Future Project will be succeeded. Adaptation of water to the center of the life was the main purpose of the study. The installation of the channels will come into prominence and will promote people’s awareness of the water scarcity threat.

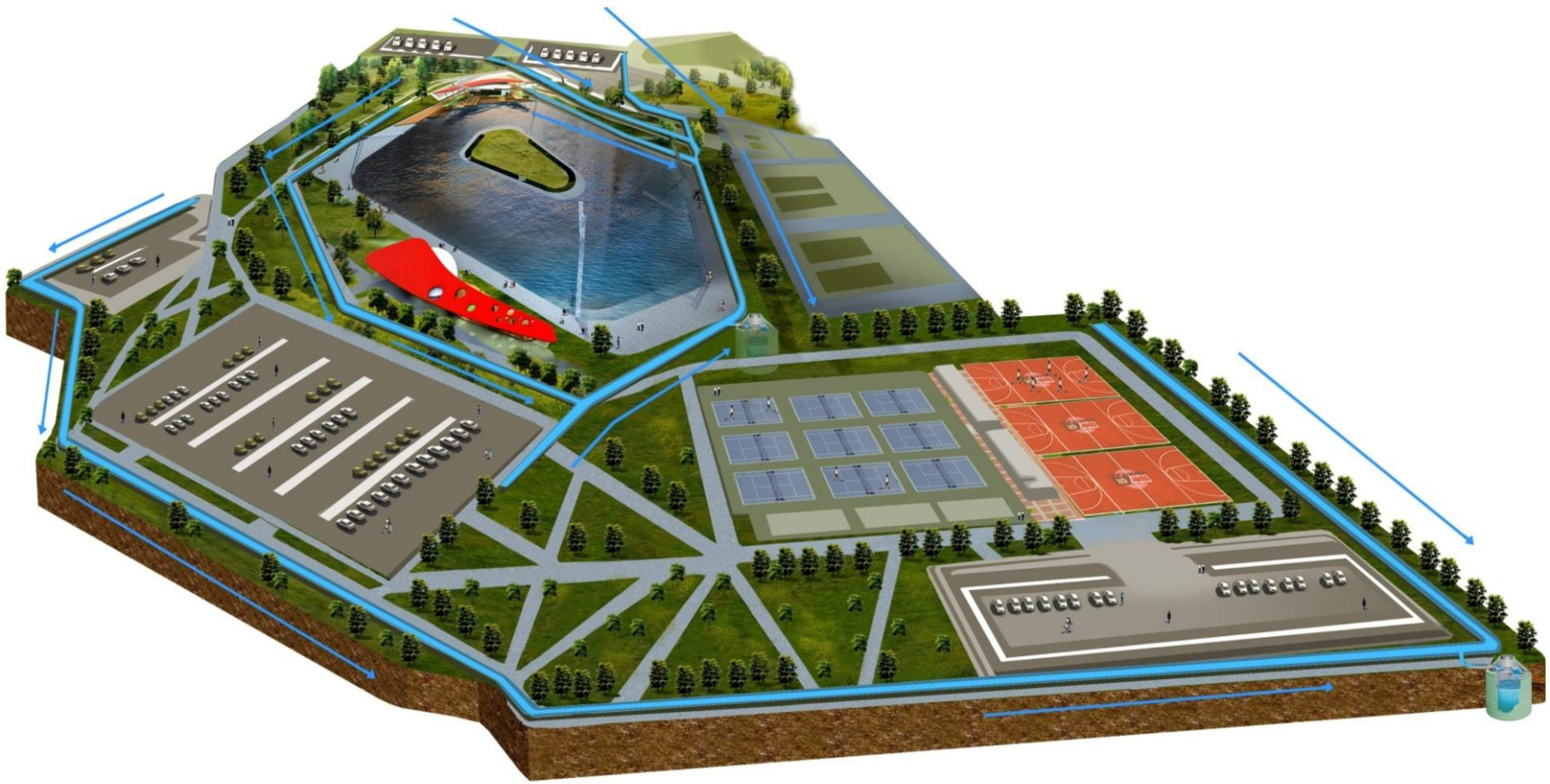


Figure 4.9. After the implementation of the proposed channel to water-ski park of Kayseri Harikalar Diyarı.

4.3.3. Proposed Storm Water Storage Tank Design for the Catchment Area

Several different types of storm water Best Management Practices (BMPs), including retention/detention ponds, storm water wetlands and underground storage structures can provide storm water control. These systems capture flow and release it slowly over time.

On-site underground retention/detention systems are designed to provide predetermined amount of storage volume within a specified area. System designs can range from simple storage pipes or chambers to complex systems consisting of multiple pipes or chambers with accompanying joints, crossovers, multiple inlets and access points. At a minimum each system must have an inlet an outlet and a structure to access the chamber. (Dizon, 2000).

The primary advantage of the on-site underground storm water retention/detention system is that it captures and stores runoff, thus helping meet the requirement to maintain pre-developed run-off conditions at newly developed sites (Storm Water Technology Fact Sheet). Because these systems are underground, local residents are less likely to have access to them, making them safer than ponds or other aboveground storm water BMP's.

Two underground storage tanks are proposed in the Kayseri Harikalar Diyarı Water-Ski Park. One is going to be located at north-west side of the lake while the latter is going to be located at very right corner of the layout (Figure 4.9). For storage tank-1, runoff is then released into the water-ski lake at a controlled rate, where further removal of particulate matter occurs via the post storage filters. Besides, for the second storage tank, water can be released for different purposes in accordance with needs and necessities of the area.

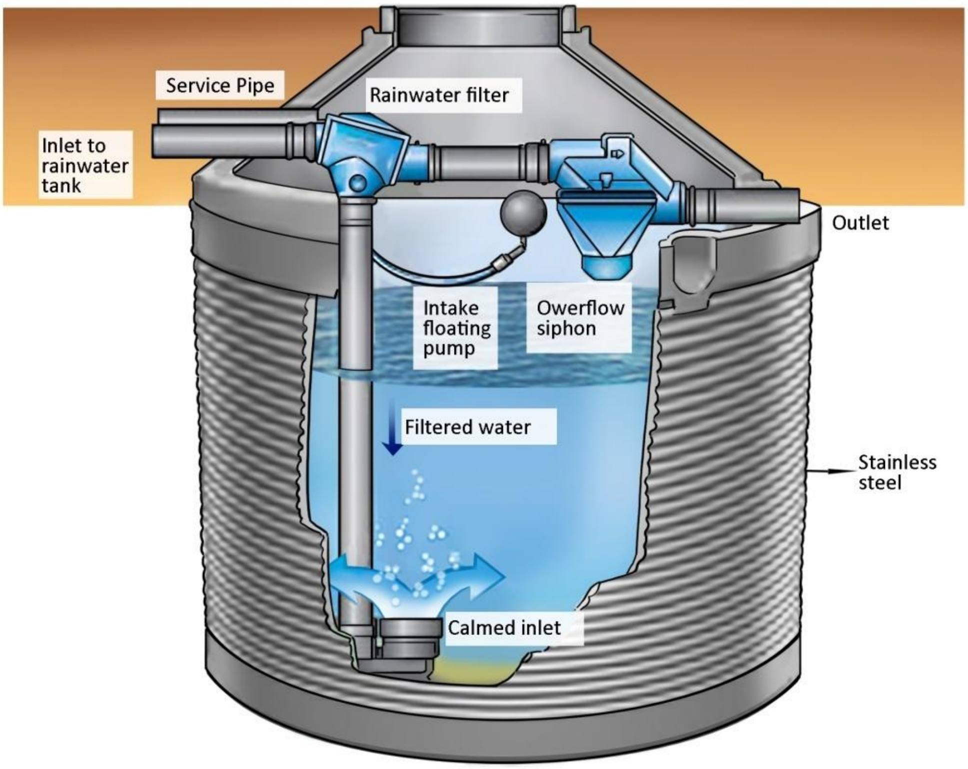


Figure 4.10. Illustration of proposed underground rain water storage tank.

In Figure 4.10, the scheme of the proposed storage tank is presented. There will be two pre-storage filters in order to remove particulate matter from storm water. Also, an overflow siphon is recommended in order to prevent any overflows. The maximum water volumes and design considerations are listed in Table 4.15. The tank will be located underground, in average, flushing will be performed 2 times once a week (Table 4.15).

Table 4.15. Total runoff volumes and particular flushing times for each tank.

	Outfall 1-Storage Tank 1		Outfall 2-Storage Tank 2	
	Volume (m^3)	Times flushing	Volume (m^3)	Times flushing
3 years	19595	392	18444	369
1 month	544	11	512	10
1 week	136	3	128	3
3 days	58	1	55	1

Commercially, the maximum available horizontal cylindrical stainless steel tanks have a volume of 50.050 liters have a diameter of 4.2 m whereas the height of each tank is 3.42 m. In addition, tanks are corrosion resistant for 25 years.

After the required storage volume has been determined, site can be examined in order to determine what configuration will maximize storage while minimizing the size of excavated area.

The runoff can be stored below ground where suspended solids are allowed to settle out before the water is released back into the environment.

Once underground storm water retention/detention systems are installed, they require very little maintenance. Because of the weight of the soil above them the inlet and outlet pipes have to be chosen as corrugated steel pipes. A study in Washington D.C. metropolitan area conducted by the National Corrugated Steel Pipe Association (NCSPA, 1999) found that all of the system was performing well with the construction of corrugated steel pipes. None of the pipes were inspected while some of which had been in place for up to 25 years.

Once appropriate construction material is determined for a specific application, design must determine the amount of storage volume required by the system. The construction of such a system with mentioned properties and design criteria, costs 5 700 $ for each storage tank installation (PJT Environmental Plumbing Solutions, 2011).

5. CONCLUSIONS AND RECOMMENDATIONS

Within the context of Cities of the Future project, Kayseri is one of the pioneer cities in Turkey where integrated management of water and wastewater projects are going to be developed. In this study, within the scope of Cities of the Future, Kayseri, a pilot project to demonstrate a resilient strategy is introduced.

Project includes rainwater collection and storage design aligned with the U.S. Environmental Protection Agency Storm Water Management Model (SWMM 5.0) results.

The fundamental goal is to determine the maximum flows with the exceedence of occurrence, assuming that the design rainfall has the same probability as the relevant maximum flows in Kayseri Harikalar Diyarı water-ski park. In Kumarlı, Harikalar Diyarı Park, rainwater harvesting/local storm water management, along with water reclamation and reuse will improve drought resilience of water supply and relief stress on wastewater system.

The recorded peak water levels and their exceedence periods showed a similar trend. According to the simulation results, placement and design of the conduits, subcatchment areas and junctions are consistent. No flooding and surcharge on conduits were recorded throughout the system according to the simulated results.

An open concrete, triangular channel with a bed slope of 0.003 and total length of 2147 m is proposed for water collection of the study area. Water will drain in the channel without requiring any external forces for it to flow.

Two storage tanks with a capacity of 50 m^3 are proposed for capturing of rainwater in order to help meeting the requirements to maintain pre-developed run-off conditions at the newly developed site.

The obtained results demonstrated that after 3 years, net 38039 m^3 rain water can be stored in two proposed underground storage tanks. 42% of the water-ski lake with a volume of 90,000m^3, could be recharged with the stored rain water.

As a result, the net storm water amount is determined and reuse strategy assessment was conducted. Rainwater collected from Kayseri, Kumarlı district meets WHO guidelines for water reuse for different purposes (WHO/CEHA, 2006) and EU Directive for Bathing Water (2006/7/EC). The harvested and stored water is appropriate for usage in different purposes such as recharge of the water-ski lake and irrigation of the subjected area.

In order to store water in water harvesting tank for longer periods during off season, in addition to the runoff from the study area, roof-top water harvesting through tanks from the existing buildings located in the area may be linked up wherever possible to the proposed system.

It should be noted that there must be an available control scenario which is required for matching the predevelopment peak flows for various return periods. Based on these design criteria, the expectation would be that under the scenario, the peak runoff rates for the design storms would match the developed scenario.

Areas have to develop a policy of no net increase in runoff for a design storm water event for newly developed areas. Local requirements should dictate how much of a given storm must be captured and treated, and the required storage volume can be calculated using this value.

APPENDIX A

RAINFALL DATA

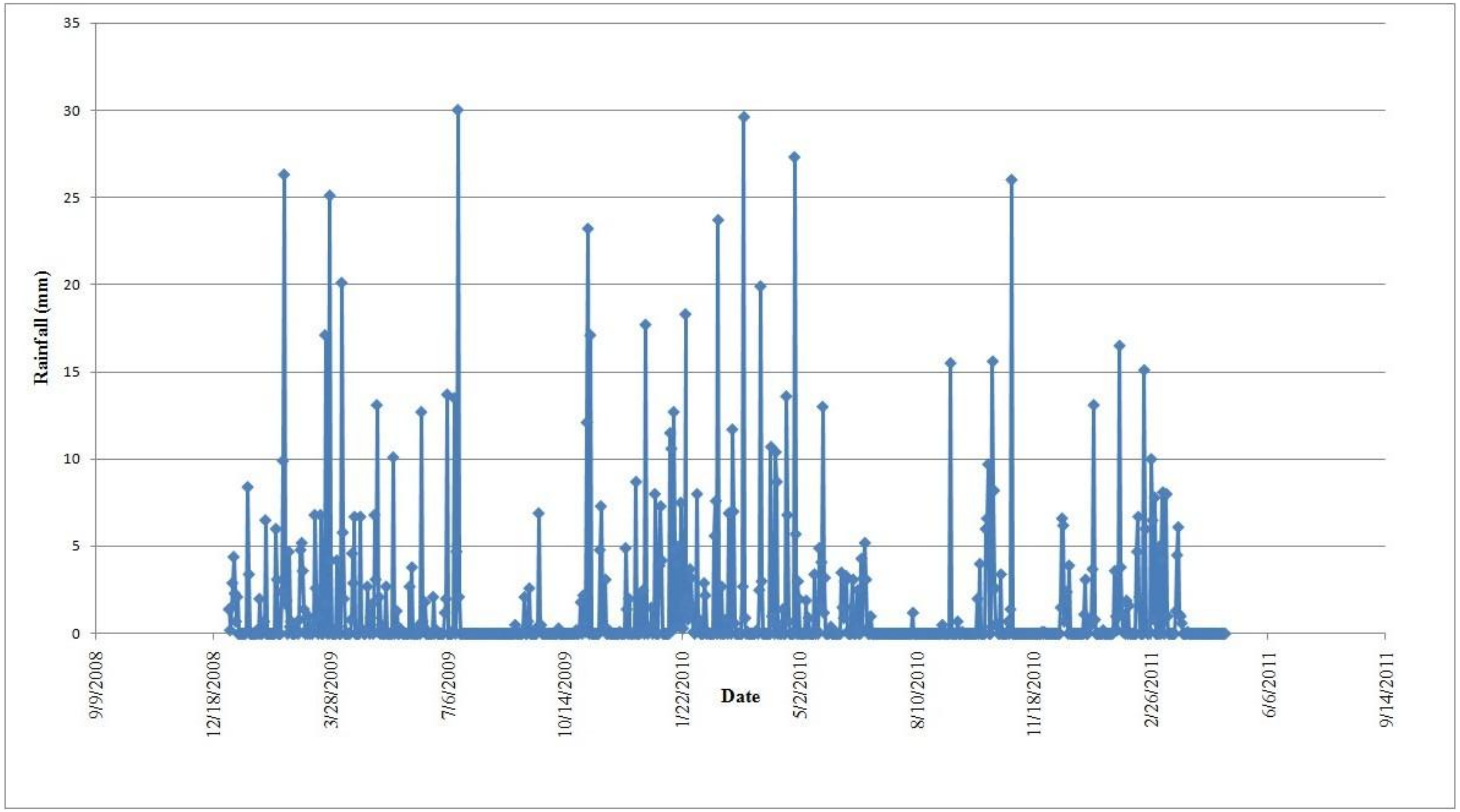


Figure A.1. Rainfall Data for a time period of 3 years.

APPENDIX B

TEMPERATURE DATA

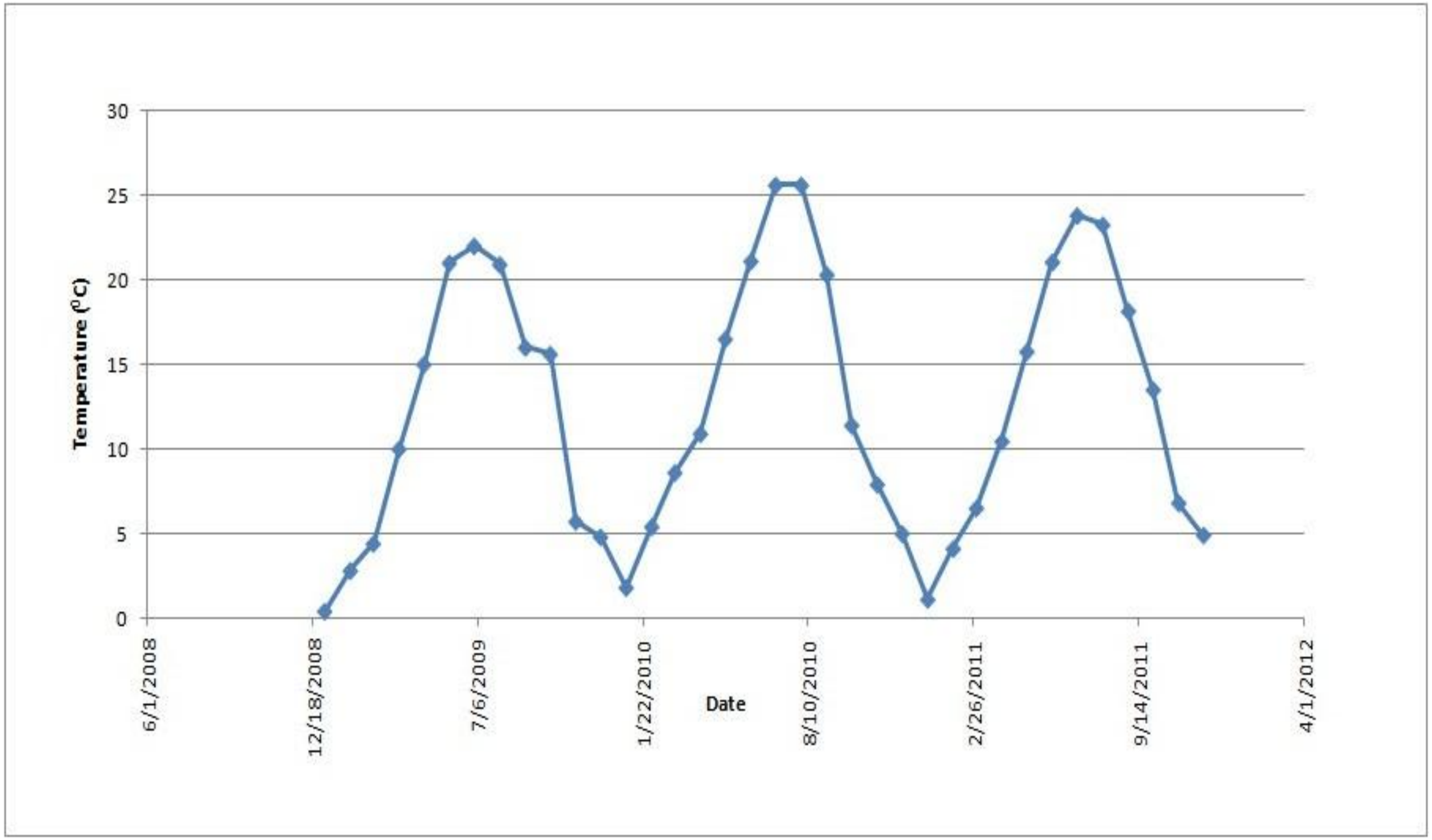


Figure B.1. Temperature Data for a time period of-3 years.

APPENDIX C

SCS RUNOFF CURVE NUMBERS

Table C.1. SCS Runoff Curve Numbers (USDA TR-55,1986).

	Hydrological Soil Group			
Land Use Description	A	B	C	D
Cultivated land				
Without conservation treatment	72	81	88	91
With conservation treatment	62	71	78	81
Pasture or range land				
Poor condition	68	79	86	89
Good condition	39	61	74	80
Meadow				
Good condition	30	58	71	78
Wood or forest land				
Thin stand, poor cover, no mulch	45	66	77	83
Good cover[1]	25	55	70	77
Open spaces, lawns, parks, golf courses, cemeteries, etc.				
Good condition: grass cover on 75% or more of the area	39	61	74	80
Fair condition: grass cover on 50-75% of the area	49	69	79	84
Commercial and business areas (85% impervious)	89	92	94	95
Industrial districts (72% impervious)	81	88	91	93
Residental[2]				
Average lot size (% impervious[3])				
1/8 ac or less (65)	77	85	90	92
1/4 ac (38)	61	75	83	87
1/3 ac (30)	57	72	81	86
1/2 ac (25)	54	70	80	85
1 ac (20)	51	68	79	84
Paved parking lots, roofs, driveways, etc.[4]	98	98	98	98
Streets and roads				
Paved with curbs and storm sewers[4]	98	98	98	98
Gravel	76	85	89	91
Dirt	72	82	87	89

1. Good cover is protected from grazing and litter and brush cover soil.
2. Curve numbers are computed assuming that the runoff from the house and driveway is directed toward the street with a minimum of roof water directed to lawns where additional infiltration could occur.
3. The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.
4. In some warmer climates of the country a curve number of 95 may be used.

APPENDIX D

SOIL GROUP DEFINITIONS

Table D.1. Soil groups and their definitions (USDA TR-55,1986).

D	C	B	A	Group
High runoff potential. Water movement through the soil is restricted or very restricted. Group D soils typically have greater than 40 percent clay, less than 50 percent sand and have clayey textures.	Moderately high runoff potential. Water transmission through the soil is somewhat restricted. Group C soils typically have between 20 percent and 40 percent clay and less than 50 sand and have loam, silt loam, sandy clay loam, clay and silty clay loam textures.	Moderately low runoff potential. Water transmission through the soil is unimpeded. Group B soils typically have between 10 percent and 20 percent clay and 50 percent to 90 percent sand and have loamy sand or sand loam textures.	Low runoff potential. Water is transmitted freely through the soil. Group A soils typically have less than 10 percent clay and more than 90 percent sand or gravel and have gravel and sand textures.	Meaning
<0.06	0.06-0.57	0.57-1.42	> 1.42	Saturated Conductivity (in/h)

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