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Evaluation the Effects of Rainfall Intensity on Storm Network and the Infrastructure Projects [Al-Hur District, Kerbala as a Case Study]

A thesis
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بسم الله الرحمن الرحيم

قَالُوا سُبْحَانَكَ لَا عِلْمَ لَنَا إِلَّا مَا عَلَّمْتَنَا ۖ إِنَّكَ أَنْتَ الْعَلِيمُ الْحَكِيمُ ﴿٣٢﴾ (البقره-32)

صدق الله العظيم

To my

Father, Mother and

My husband

With love and

respect

I dedicate this work

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First, thanks to Allah his majesty for enabling me to complete this work. This project would not have been possible without the assistance of many individuals. I am grateful to those people, who volunteered their time and advice, specially, my supervisors Dr. *Basim Khlail Nile* and Dr. *Waqed H. Hassan* for their guidance, advice, invaluable remarks and fruitful discussions throughout the preparation of my thesis.

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Abstract

The variation of rainfall intensity higher than the design standards due to the global warming considers as one of the big problems that are facing the urban area. Urbanization in cities led to the spread of impervious area quickly and the increase in the amount of the runoff quantity due to the decrease in the infiltration rate. The misuse of the storm drainage system by the inhabitants decreases the drainage capacity and led to urban flooding. Modeling of the urban flooding considers as an important method that help in discovering and managing the flooding problems.

In this research Storm Water Management Model [SWMM] has been used to simulate the flooding of the storm drainage network in Al-Eskari quarter, Kerbala, Iraq. The hydrological data (i.e. rainfall intensity, temperature and wind speed) was collected for a period from 2008 to 2016 by Agricultural Meteorology Iraqi Network (AMIN) . The hydraulic data was provided for the storm drainage system of Al-Eskari quarter regarding to the manholes, pipes and pumps. The calibration data was used for two rainfall events in 18/1/2016 with rainfall intensity 2 mm/hr and 22/1/2016 with rainfall intensity of 4mm/hr.

The major aim of this study is to develop a model to evaluate the effect of the climate variation and the misuse of the storm drainage system through adding an illegal sewage quantity to the storm network by the inhabitants. Also, this study aims to specify the damage cost of the infrastructure that caused by flooding. Finally, this study tries to provide a technical support for the decision makers by developing and accessing a solution to mitigate the flooding in the study area.

The results indicate that the storm drainage network is suitable for the design intensity of 2-years return periods [9.6 mm/hr intensity for 1-hour] without considering the effect of an illegal sewage quantity. The climate variation plays the major role in the flooding of the study area with a rainfall intensity reach in an

event to more than three times the design intensity with 33.54 mm/hr for 1-hour. Under 33.5 mm/hr rainfall intensity and without considering the sewage quantity, the equivalent water depth in the outlet reached to 5.55 m measured from the invert of the outlet, 47% from the manholes of the system flooding with discharge greater than 0.04 m^3/sec and the flooding duration was 43 hours. The existent of sewage increases the flooding problem due to the clogging that happened in the system, where the system started to flood under rainfall intensity lower than the design intensity. The equivalent water depth with the maximum rainfall intensity in addition to the sewage amount reached to 5.62 m, 48% of the manholes in the system flooding with flow reach to 0.04 m^3/sec and the flooding duration was 72 hours. The sewage quantity doubled the flooding duration. The flooding event with long duration led to damage in the infrastructure, especially the roads and houses. The total cost of flooding damage reached to 354800 $ and it's a repeated cost if no solutions are provided to mitigate the flooding. In this study a solution was suggested by adding a transmission pipe line with cost of 1154 $ for each meter of the pipe. This performance of this solution under the maximum rainfall intensity and with consider the sewage amount was as follow: The equivalent water depth with the maximum rainfall intensity and with consider the sewage amount reached to 5 m, the volume of water flooding reached to 129 m^3, 33% of the manholes flooding with discharge reached to 0.04 m^3/sec and the flooding duration was 20 hours. The results of adding a pipe line is so promising and decreasing the flooding in the study area. The simulation results can offer resilient technical support for the urban drainage system controlling and planning.

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Symbol	Definition	Dimension
A	Sub catchment area	L^2
A1,A2,A3,B1and B2	Coefficients appear in the equation to compute the groundwater flow	-
d	The water depth on the sub catchment [depth of storage in the reservoir]	L
$\mathbf{D_{p,pout}}$	The distance between the outlet point of the sub catchment and another arbitrary point	-
f	Infiltration capacity	L/T
F	Mass infiltration	L
g	The acceleration due to gravity	L/T^2
G	The water consumption per capita	L^3/T
$\mathbf{H_{gw}}$	The height of saturated zone above bottom of aquifer	L
$\mathbf{H_{sw}}$	The height of surface water at receiving node above aquifer bottom	L
H*	The threshold groundwater height	L
$\mathbf{I_e}$	Rainfall excess which is the rainfall intensity minus the evaporation and infiltration rate	L/T
K	Saturated hydraulic conductivity	L/T
$\mathbf{L_{max}}$	The maximum runoff length in the sub catchment	-
p	The population number	thousand
q	The flow rate in the system	L^3/T

Q	The runoff flow from the sub catchment	L^3/T

Symbol	Definition	dimension
Q_{avg}	The average sewage discharge for each manhole	L^3/T
Qgw	The groundwater flow	$L^3/T/L^2$
n	The manning's coefficient	-
N	Available porosity	-
n,n+1	The subscripts representing boundary conditions at the end of time step n [or start of time step of time n+1] and the end of time step n+1	-
R	The hydraulic radius of flow over the sub catchment	L
S	The slope of the sub catchment	L/L
$S_\circ$	The bed slope of the pipe	L/L
S_f	The friction slope of the pipe	L/L
t	Time	T
t_{n+1} , t_n	The time step size	T
V	The volume of water on the sub catchment	L^3
W	The width of the sub catchment	L
x	The distance along the channel	L
y	The depth of flow	
Ψ	Suction head	

List of Abbreviation

Abbreviation	Description
AMIN	Agricultural Meteorology Iraqi Network
GIS	Geographic Information System
HSPF	Hydrological Simulation Program Fortran
IMAEM	Iraqi meteorological agency and earthquake monitoring
IPM	Iraqi planning ministry
KSD	Kerbala's Sewage Director
PVC	Polyvinyl Chloride
SWMM	Storm Water Management Model

CERTIFICATE OF THE EXAMINING COMMITTEE

We certify that we have read the thesis entitled " **Evaluation the effects of rainfall nsity on storm network and the infrastructure projects [Al-Hur district,Kerbala as ıse study]**" and as an examining committee, we examined the student **"Batul Abdulla ıaeel AL-Mas'sod"** in its content and in what is connected with it, and that in our ıion it is adequate as a thesis for degree of Master of Science in Civil Engineering rastructure Engineering).

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Chapter one

Introduction

1.1 General

The flooding of storm system in urban areas has negatively effect on infrastructure. The most important parameter that effects on the flooding rate is climate change and variation, which increases the runoff quantity by increasing the rainfall intensity. The development in the cities has also increased the urbanization and causing an increase in impervious area, which in turn decreases the infiltration rate that causes an increase in the runoff quantity, peak flow and the decrease concentration time (Saghafian et al., 2008).

The effect of the fluctuations of the hydrological processes and all the other condition on the flooding event can be simulated through mathematical model of the storm system. The urban modeling has an important role in planning, designing, developing and controlling the urban drainage system (Rossman., 2010).

Models have the ability to simulation different circumstances effect on the hydraulic performance of the storm system and provide a lot of options to describe this effect such as estimating the quantity of water flooding and depths of water for each point in the drainage system.

The runoff-rainfall process modeling requires numerous information and parameters to simulate the connection between the runoff and drainage system. This kind of information is not always available or can be easily estimated.

Kerbala, Iraq [Where the study area in] is a rapidly urbanization city because it is characterized by religious nature. The political conditions in the country has also caused the immigration of thousands of citizens from other cities to Kerbala. The urbanization in the study area [Al-Eskari quarter, Kerbala] converts the most of the lands from pervious into impervious area, which causes an increase in the urban flooding. As a result, the control of the urban flooding quantity becomes an important issue in reducing the cost of the infrastructure damage.

1.2 Statement of Study Area Problems

The case study is in Al-Eskari quarter, which is located in the north part of Kerbala, Iraq. The study area suffers from flooding of the storm water drainage system during rainy season due to climate variation, urbanization and the exist of the illegal sewage quantity from the residents. The climate variation led to the increase in the rainfall intensity and quantity to a value higher than the design intensity of the storm drainage network in the study area. This increase exceeded the drainage capacity of the system and resulted in flooding. The exist of the illegal sewage quantity that drainage through the storm drainage network has worsen the situation. The sewage decreases the drainage capacity of the system and causes a sedimentation on the pipes' walls. As results the existence of the sewage during the rainfall season makes the system flooding at lower, rainfall intensity than its design intensity.(Kerbala's Sewage Director (KSD), 2015)

1.3 Objectives

The main purposes of this study are to provide a technical support regarding the performance of the storm drainage network under different circumstances in the study area and as follow:-

- Develop a model to simulate the performance of the storm system in the study area using **Storm Water Management Model (SWMM)** for two cases [climate variation only, current state with the existence of sewage quantity] and analysis the storm system after adding a solution to mitigate the flooding effect.

- 2.Estimate the damage cost in the infrastructure caused by the flooding of storm system in the study area.

1.4 Methodology of the Current Study

The steps of work in this study could be summarized as follows:

1- Collecting data, which involves hydrologic and hydraulic, topography, land use data, digital image of the study area, soil properties, flow quantity observed data , etc.

2- Verifying the model using the observed data in order to find the best simulation results.

3- Carrying out a views on the:-

A- study area during a rainy event to specify the flooding extended in the reality

B- damage that happened to the infrastructures due to the flooding of the storm system.

4-To achieve the above, the following soft wares have been used: Storm Water Management Model [SWMM] v.5.1 to simulate the storm drainage system and ArcGIS 10.3, to view the digital image of the study area, the hydraulic data map of the storm system and the topography map.

1.5 Assumptions and limitation

1. The flow routing in the proposed model is a dynamic flow routing for unsteady non uniform ,and manning roughness formula is use for the measure the flow.
2. Green-Ampet method was used for the infiltration calculation in this model .

Chapter two

Basic Concepts and Literature Review

2.1 Introduction

This chapter displays the main factors that influenced the flooding in urban areas. These factors were the climate change, urbanization and the misuse of the drainage system. Besides that, this chapter will display previous studies regarding the modeling of the urban flooding by SWMM. The cost of the infrastructure damage due to the flooding events will also be discussed.

2.2 The Relation Between Urbanization and Flooding

The impact of land use changes rises the hazard of urban flooding, which increase the volume and peak discharge of the water flooding, and reduce time to peak discharge (Figure 2.1).(Butler and Davies., 2011).

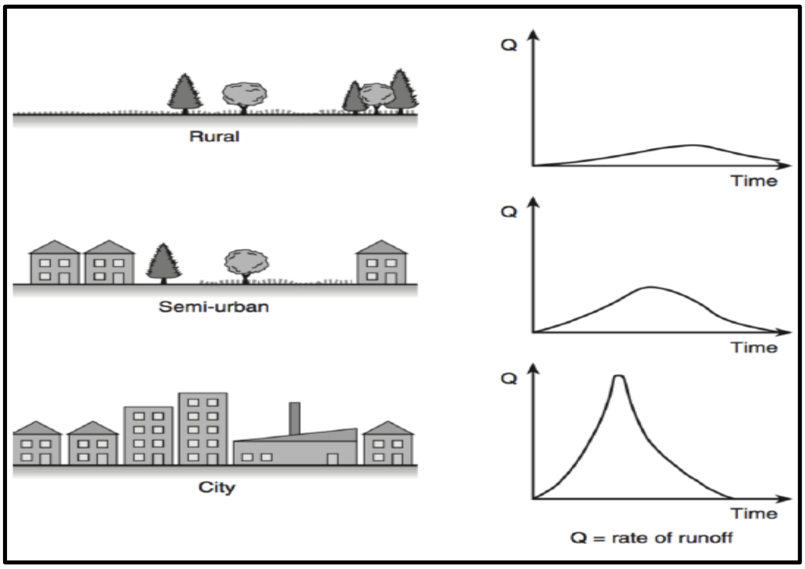

Fig. 2.1: The effect of land use change on flooding phenomena (Butler and Davies., 2011)

(Sheng and Wilson., 2009) stated that in Los Angeles and in the urban area, there was loss of 90% of the precipitation as a runoff due to the decrease in the infiltration rate as a result of the change of the land use cover. While 25% of the precipitation in the non-urban forested areas retain as a runoff and the residual quantity of water lost due to the infiltration and evaporation.

When rainfall falls on non-urban area, some of the runoff losses through evaporation, or absorbs by plants; some infiltrates through ground and converts to groundwater storage; and some runs off the ground (Figure 2.2-a) shows that. The urbanization includes covering the natural ground to impervious surface. The impervious surface rises the amount of surface runoff and decreases the infiltration rate, and therefore increases the total volume of water flooding during or quickly after the rainfall (Figure 2.2-b) (Butler and Davies ,2011).

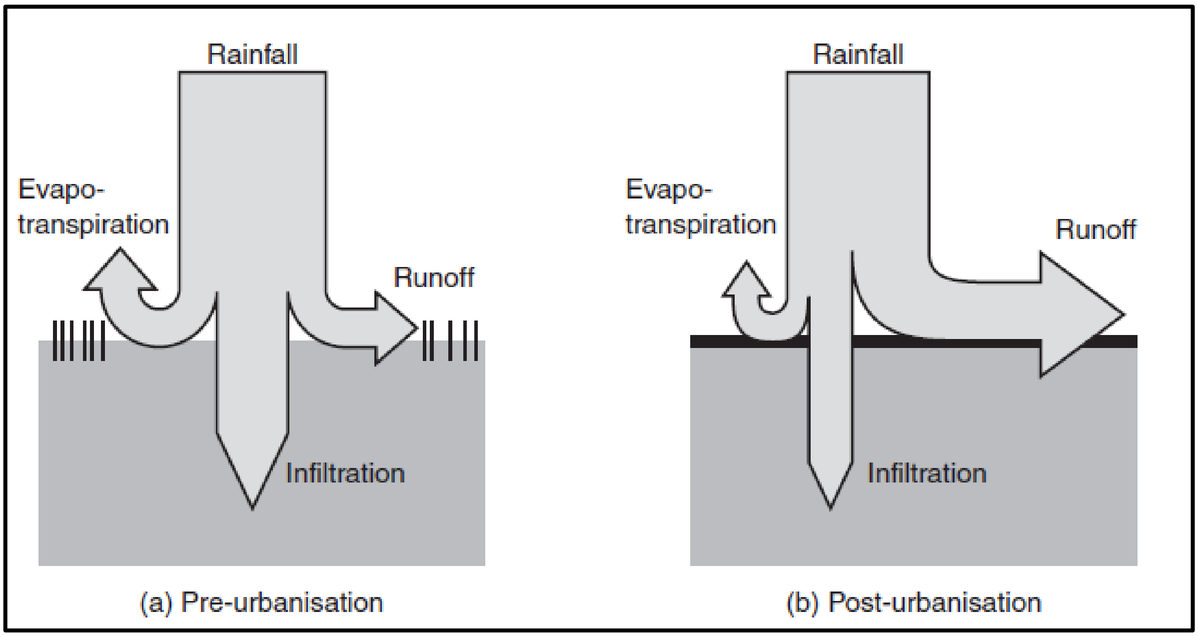

Fig. 2.2: The effect of urbanization on the hydrology cycle (Butler and Davies,2011)

2.3 The Relation Between Climate Change and Flooding

It is very essential to recognize the different between climate change and climate variability, where the former denotes to a long-term alteration in the climate, whereas the latter is the natural variation in the climate from one period to the next. Climate variability looks to have a very obvious impact on various hydrological processes (Kundzewicz and Robson ., 2004).

The growth of the urbanization and the likely increases in extreme rainfall due to climate change was the most meaningful impact in the evaluation of hydraulic capacity of urban drainage systems. Also, strong evidences that the possibilities and hazards of sewer surcharge and flooding are increase due to the global warming (IPCC., 2007).

In regions where the mean precipitation reductions, precipitation intensity is predictable to rise, but there would be longer periods between precipitation happenings (Meehl et al., 2007).

In general, the urban area is projected to different effects due to climate changes. In this regard, low-intensity precipitation events will not damage the urban drainage system directly, but it is possible that this it may multiply the effect of following precipitations if pervious areas can become saturated and may affect on the levels of groundwater. On the other hand, extreme precipitation and very high-intensity events are likely to cause increased surface floods, basement floods, combined sewer overflow and influx to treatment instruments. Even if the entire precipitation volume reduced, the increased peak intensity will cause quick runoff, and the satisfactory infiltration ability might not be obtainable(Olsson et al., 2009)

2.4 Modeling of Urban Flooding

Chiew et al., (1995) evaluated the effect of climate change projected in 2030 on the runoff in 28 catchments in Australian by using a hydrologic daily rainfall-runoff model. They found that the runoff quantity increased by 25% in north-east Australia, 4-20% in south-east and up to 10% in Tasmanian.

(Hsu et al., 2000) simulated the overflow of storm network and pumping stations shortage in Muchia, Taiwan by using a combined model of SWMM and (2D) diffusive overland-flow model. The results showed that total runoff surpassed the capacity of pumping stations and the runoff cannot be drainage instantaneously with peak intensity of 78 mm/hr, which results in the overflow from sewer outlets and manholes.

For the most current dominanted models , explained that SWMM have the ability to simulates water quality and quantity of sewer overflow and urban water runoff for continuous or single precipitation event efficiency.Also SWMM has the ability to direct the inflow through open channel and a closed conduit system by means of an explicit numerical solution of the shallow water wave equations, flows are directed through the pipe system consuming a modified kinematic wave. In SWMM, the generation of the runoff depends on land use, topography, rainfall intensities and antecedent moisture conditions, (Zoppou ., 2001).

The watershed modelling defined as mathematical descriptions of components of the hydrologic cycle. This description developed by different science approach by linking the field data with the basic physical concepts. The question of "what modeling skill is better?" has the difficulty in answering because poverty of the unanimity on which model has advancement on other models, (Singh and Woolhiser ., 2002).

(Sands et al., 2002) carried out two simulations accompanied with SWMM model in New York. The first was to model the rainfall distribution that was happened on August 26, 1999 with rainfall intensity of 104 mm for six-hour. This simulation permitted of establishment a database that tracks each manhole and specifies the flow rate that contributing to each manhole over a 24-hour model run. The second simulation involved the estimation of numerous design storms and specified the drainage capability of the storm sewer network to discharge them. The results of running the design storms through the storm sewers network showed that the system was possibly designed to deal with 10-year return periods with 24-hour event, because it starts to flood severely at greater design storms. When sewage quantity of 10% of the conduit capacity was added into the model, the system showed signs of only being able to handle the 7 to10-years return periods.

(Suarez et al., 2005) collected a historical rainfall data from 1970-2000 and future predict data from 2020-2050, which they been used in (Hydrological Simulation Program Fortran) HSPF hydrologic model to simulate the urban flooding in central Puget Sound, Washington. The hydrologic modeling indicated that an increase would be observe in the future rainfall intensity. From this results that the urban discharge system designed using historical rainfall records may be subject to a future rainfall patterns differs from existing design criteria so that there was uncertainty in the efficiency of urban discharge system that result from using assumption of constant condition in climate especially the distribution of precipitation.

(Semadeni et al., 2008) specified the impact of the climate change and urbanization on drainage system in Helsingborg, Sweden. This city has a problem in flooding of combined sewer and pump station during heavy rainfall and oblige to discharge the waste water without treated to the point of discharge in waters body during the flooding time leading to pollution. Sewer flows resulting from different urbanization storylines were simulated for two 10-year periods corresponding to present (1994–2003) and future climates (nominally 2081–2090) resulted that the low rainfall intensity expected to decrease by 50% and the high rainfall intensity expected to increase by 500%.

(Cambez et al., 2008) used SWMM to reveal the percentage of overflow discharge that decrease due to the existence of storage tank with capacity of 70 m^3/s for continuous rainfall event of 5-year . The simulation results indicated that the overflow discharge decreased from 14% to 20% comparing with the case of tank absence.

In a study conducted by (Park et al., 2008), which investigated the impact of the division level and spatial resolution of the sub catchment on the surface runoff and pollution level in SWMM. The division of the study area in korea done with GIS overlaying technique. The division taken into account the land use type, the surface slope and the flow direction in the storm drainage system. The results reported that the surface runoff did not have that much impact by the division level of the sub catchment ,where the total, cumulative and the peak of the runoff impact slightly,but the pollution loads impacted .

Li et al., (2010) used SWMM to determine the effect of proposal solutions on drainage efficiency of network drainage system in China. They found that in case of increase the pipe diameter, the overflow in the study area decreased by 81.62%. on the other hand, the change the location of manholes decreased the overflow in the study area by 44.78 %. These results revealed the ability of SWMM in the planning and designing of the urban drainage network saving time, cost and effort.

Meierdiercks et al., (2010) simulated the hydrological response that affected by the increase of growth rate , urbanaization and the age of the drainage network structure in three subbasin in Baltimore metropolitan region, United States of America , using SWMM . In Baltimore, the oldest subbasin contains nearly less than 1% stormwater controls management, while the two newest subbasins have stormwater controls reach to 48.8% and 24.6%, respectively . The authors of the study made a comparison between two subbasin : the first have a percentage of impervious and stormwater controlling ponds higher than the second with a drainage structure density less than the second . They found the hydrological response was similar in both subbasin for the same storm event . Eliminating the stormwater controlling ponds from two subbasins in the simulation with remaining the other parameters would increase peak discharge during the same event by 50%

and 48% respectively. As a result, providing a surface storage or improving the efficiency of the drainage network is of high significant and have better effect in decreasing peak discharge than reducing the urbanaization.

Beling et al., (2011) calibrated SWMM results to simulate the drainage system in four sub catchment in Brazil to deacreas the differnce between the simulation and observed results. The calibration done with different number of rainy event for each sub catment depending on the provided data where range from 4 to12 events. The results of calibration indicated that the coefficient od determination (R^2) ranged from 93% to 99% , the average erorr percentage in runoff peak between observed and simulation results ranged from 0.63% to 8.3% and the average erorr in the runoff volume ranged from 4.72% to 22.3% .

Junhua et al., (2012) simulated urban drainage network in northern China using SWMM for one year return periods and they stated that the high ratio of impervious area and the small diameter of downstream conduits were the main reason for increasing the rate of flooding in the study area .

Wenting et al., (2013) analyzed the storm water network capacity during different return periods (0.25, 0.5, 1, 2, 5, 10) years in Nanjing, China using storm water management model (SWMM). The city has a high percentage of population and urbanization. The main aim of this analysis is to provide base for planning and re-establishment the network to avoid the flooding problems. The results of analysis showed that the capacity of the network under small return period less than 5-year return period was satisfying, but with the increase of return period to 10-year return periods, the storm water network had 57.4% of the conduit pressurized under long time. Moreover, the water began to escape from the system causing flooding the area.

Zhao et al., (2013) utilized SWMM model to simulate the urban flooding in rainstorm drainage system in Kunming Dongfeng East Road catchment, China. The model was constructed to evaluate the storm network under urbanization scenario in actual case of urbanization by increasing the impervious area to 40% and 84% with different return periods of 1,5,10,20 years with rainfall duration of 120 min. The result of the simulation for the total runoff and peak flow in the outlet under different return periods and urbanization scenario are shown in (Figure 2.3).

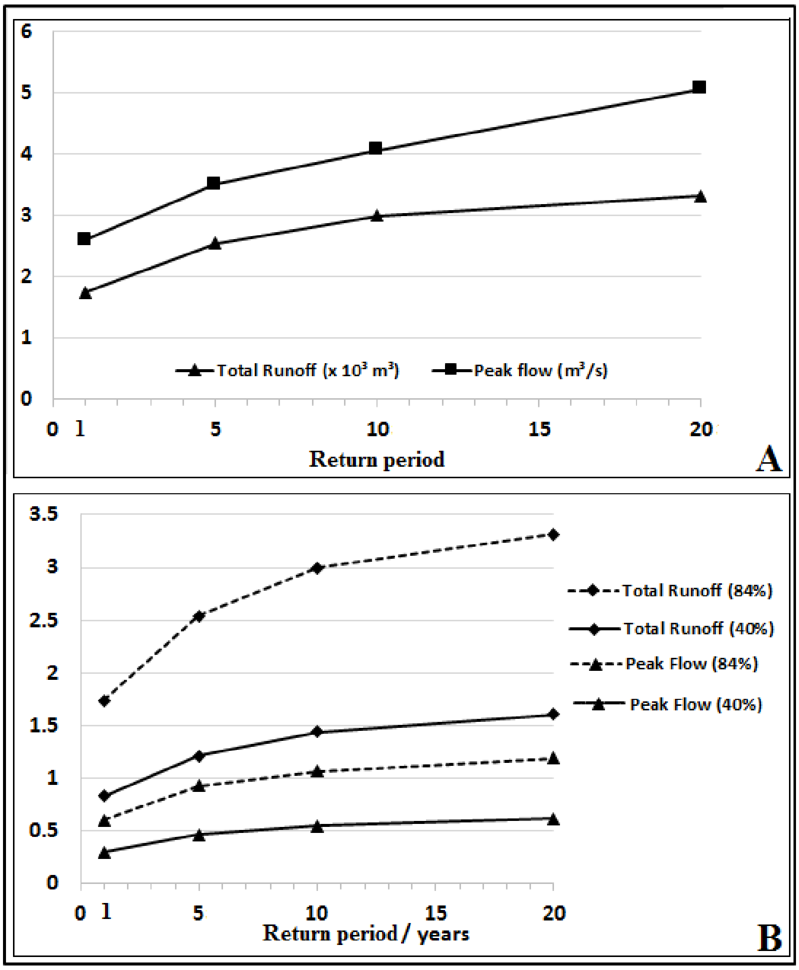

Fig. 2.3 :The simulation results of total runoff and peak flow in outlet : A-under different return periods , B- under different urbanization degree (Zhao et al., 2013)

Jung et al., (2015) made a comparison by using a combination of SWMM or other urban hydrology model with synthetic hydrograph methods like (Soil Conservation Service) SCS–SWMM or Clark–(illiuoise urban drinage area simulator) ILLUDAS or use traditional synthetic hydrograph methods such as SCS–SCS or Clark–Clark to estimate the urbanization influence on the hydraulic preformance for the same strom drainage network and same location in Korea. The result of the comparison stated that the using of SWMM model give appropriate and predictable results for the influence of the urbanization approximate to reality .The results of SWMM can be used for design purpose comparing with the traditional synthetic hydrograph methods. SWMM gave a good result even for simulation of an uncalibrated natural watershed.

Lei et al., (2015) carried out a flooding simulation in Dongguan City, China using SWMM to simulate the storm drainage system for 1,2,5,10, 25 return periods. The result explained that the capacity of the drainage network was enough only for 1-year recurrence period precipitation and it started to overflow in higher recurrence period due to the fast urbanization in China. The results also indicated that SWMM was good in forecasting the flooding quantity, but without the surface flow information.

Satyaji and Venkata (2015) revealed that increasing of the urbanization and the encroachments on the storm drainage system by the resident were the main reasons for growing of the flooding problem in Patna and Chennai, India. The drainage system modeled using SWMM to understand the behavior of the storm drainage system under the flooding problem. SWMM produced that the network will be flooded in 2 year return period and attributed that to the poor of rehabilitation and the lack of concern from people for the health of the system as they throw the rubbish and polythene bags to the drainage stream.

2.5 The Effect of Flooding on Infrastructure

AASHTO (1993) classified the types of failure that infect the flexible pavemets that caused or accelected by the moisture and runoff.The distress types include depressions, rutting, potholes, and fatigue cracking.

(Merz et al., 2004) conducted flood damage assessments that are largely emphasis on direct financial losses by consuming damage functions which relate property losses to damage-causing factors. The flood loss of a construction is affected by many factors and usually only flooding depth and construction use are taken into account as damage-causing factors. In this research a data set of nearly 4000 losses records was considered. Each record denote the direct monetary losses to a flooded construction. The data set of nine flood happenings in Germany during the duration from 1978 to 1994 covered. The depth-damage functions, which relate the water depth to the total damage were not useful in assessment the variability of the damage data since damage is estimated by numerous factors in addition to the water depth. Due to that, it has to be projected that flood damage analyses are accompanying with vast doubts. The assessments were compared to inform flood losses of an intense flood in 1993.

(Genovese., 2006) made a flood risk assessment to estimate approximately the cost of the infrastructure and roads damage due to the flooding based on formula provided by (Kok., 2001):

$$[DAMAGE = p * A * H * V]$$

where: [p= % of urban covered surface in land use, A= area (m^2) of the land use, H= water depth damage factor, V= average price for m^2 for a house]. The assessment based on flood depth, land use and maps of the flooded areas that providing valuable information. The results provided an average estimate and should not be considered as a detailed cost assessment of the damage since they are strongly depending on the quality of the damage functions and the availability of the detailed datasets.

One of the most influential factor that contributes to pavement damages is the climate condition such as torrential rains and flooding, in which the runoff water absorbed by pavement deteriorates the structural pavement base. The pavement will fail due to increasing humidity content of the subgrade. Therefore, drainage is essential in the streets' system to save the low water table, (Kordi et al., 2010).

2.6 Summary

The previous studies indicated that SWMM model was effective and able to simulate the urban flooding and produce results that are close to realty. Also, it can provide for the decision makers a technical support in planning and management improvement and hydraulic rehabilitation plans to control the urban flooding in order to reduce the effort and economic losses and help them in taking sound decisions toward development implementation. The studies outlined a critical role for SWMM model in the design, planning, evaluation and analysis the storm, sanitary and combined drainage system. The effect of urbanization and climate change on the urban flooding can be simulated and analyzed by SWMM. In this study, the constructed of the rainfall-runoff simulation model in Al-Eskari quarter, Al-Hur district, Kerbala, Iraq was based on SWMM model. It aims to evaluate and analyse the performance and drainage capacity of the storm drainage system under climate change and with a percent of illegal sewage that carry by the network and evaluation the network performance under proposal improvements to provide technical support to the decision makers. Also assessment the damage cost that happened in the infrastructure due to the flooding. The results will address the flooding problems and helpful to improve the drainage capacity of the storm drainage network.

(Sands et al., 2002), Li et al., (2010), Beling et al., (2011) and (Lei et al., 2015) was the main sources that considered in the research due to the converge in the main idea of the research especially in the part of using [SWMM] to simulate the flooding events but in except the data supply for the study area.

For the estimation of the damage to the infrastructure, the work of (Merz et al., 2004) considered but due to the deficit in the data in the study area only the estimate of the damage was depended on a survey of the current situation, and flooding damage happened for roads and house's walls.

Chapter three

Modeling and Case Study

3.1 Introduction

In this chapter, the setup and performance of the storm drainage network simulation model of the study area were discussed. All the computation of hydrological and hydraulic process in Storm Water Management Model (SWMM) will be outlined. Also, all the input data that needed for SWMM to create the most realistic simulation to the study area will be included. A plenteous data is needed to build any rainfall-runoff model to get an accurate representation that describe the intricate relation between the characteristic of the sub catchment, storm drainage, rainfall and runoff.

The data needed can be divided into two types: the first describes data regarding to the hydrology processes (rainfall, temperature, wind speed, evaporation and etc.) running by SWMM. The second describes the parameters of study area such as (sub catchment's width, area, impervious percent, infiltration and groundwater data) and the drainage system such as (pipe's diameter, length, slope and manhole's depth). Since it was difficult to provide all the data required from the specialized authorities, this chapter will be stated the basic data and the information references that needed to satisfy the requirement of the model building to simulate a runoff-rainfall process.

This chapter begin with a description of the study area. Also, it will contain a short introduction about SWMM model and its computational processes of hydraulic flow and surface runoff. The methodology for computation the parameters that need to build the model will be also explained. These parameters include the sub catchment slope, width, impervious, storm drainage system, evaporation, infiltration, sewage amount. (Figure 3.1) illustrated the methodology of the study work.

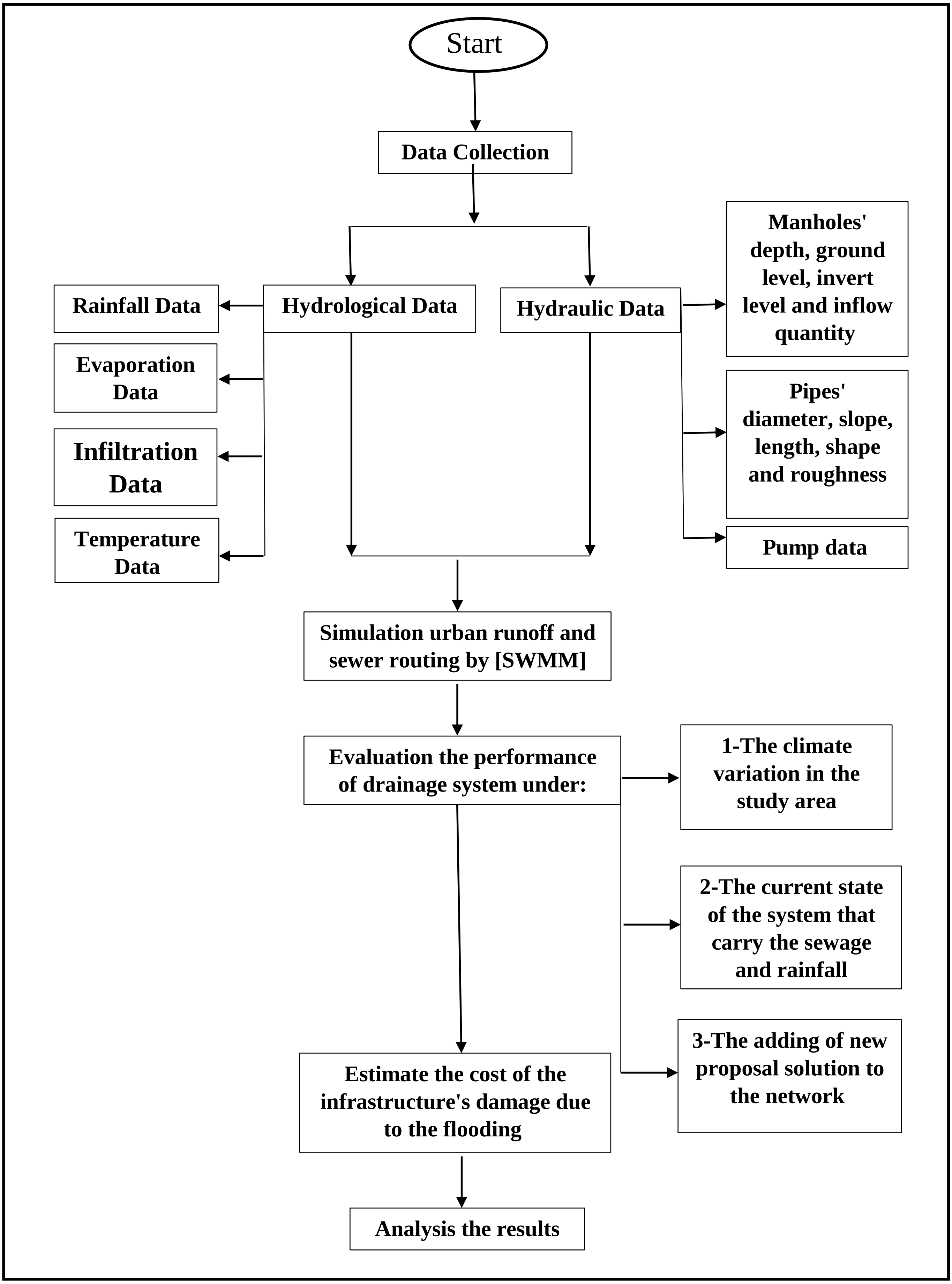

Fig. 3.1: Methodology of the work

3.2 The Study Area

3.2.1 The Location of the Study Area

Geographically, the study area (Al-Eskari quarter) is located in the north of Kerbala province center, Iraq between latitudes (32˚ 39′ 18″ N - 32˚ 38′ 42″ N), and longitudes (43˚ 09′ 06″ E - 43˚ 57′ 54″ E) as shown in (Figure 3.2). It is approximately flat surface of low slope and sandy clay soil (Kerbala's Sewage Director (KSD), 2015). The study area is located in a region with elevation ranges from 24m to 28m above the sea level. The total area are around 1.1 Km^2, 0.372 Km^2 from the study area are pervious (34% of the total area) and 0.728 Km^2 are impervious (about 66% of the total area) included 0.062 km^2 of paved roads. The impervious area can be classified as roofs, roads, and sidewalks, while the pervious can be classified as gardens and unpaved roads. The impervious percent of the Al-Eskari quarter was determined directly from plan maps and digital aerial image by computing the percent of the roofs, roads, and sidewalks to the entire area at each sub catchment. Depending on the land use type and slope of the study area, the total area of Al-Eskari quarter is divided into (104) sub catchments. All of the sub catchments were modelled and directly linked to the storm drainage system. The climate in the study area has a desert weather with extremely hot, long and dry summer from May to October and short cold winter with rain from November to April. The average temperature in winter reaches to 8°C and to 48°C during summer. The mean of the total annual rainfall in Kerbala is less than 92 mm (Agricultural Meteorology Iraqi Network (AMIN) ,2015). The area of AL-Eskari quarter is serviced by only storm drainage network where constructed in 2008 .The system carry a quantity of illegal sewage connections. The study area is suffering from the flooding during the rainy seasons.

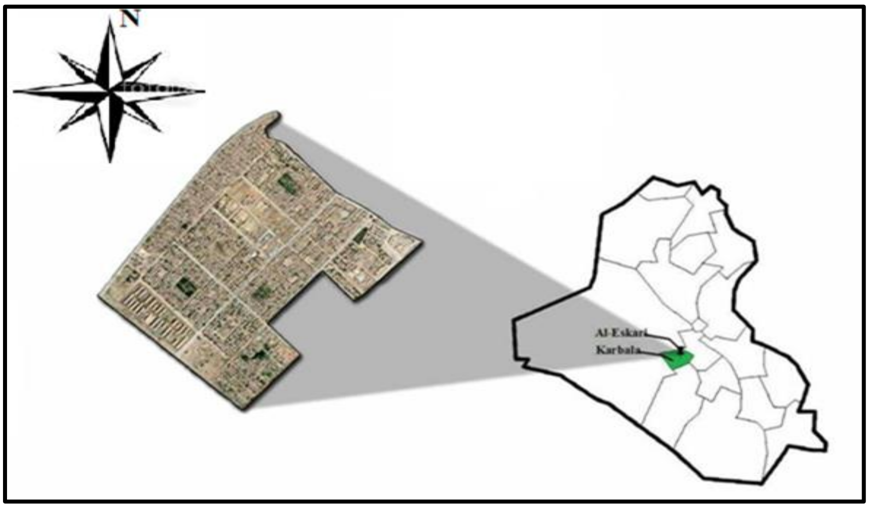

Fig. 3.2: Geographical location of the study area relative to Iraqi map

3.2.2 Land Use

The study area is a resident quarter including 13.5 % gardens and services buildings, 6% paved roads and 80.5 % houses. (Figure 3.3) shows the land use type for the study area in details.

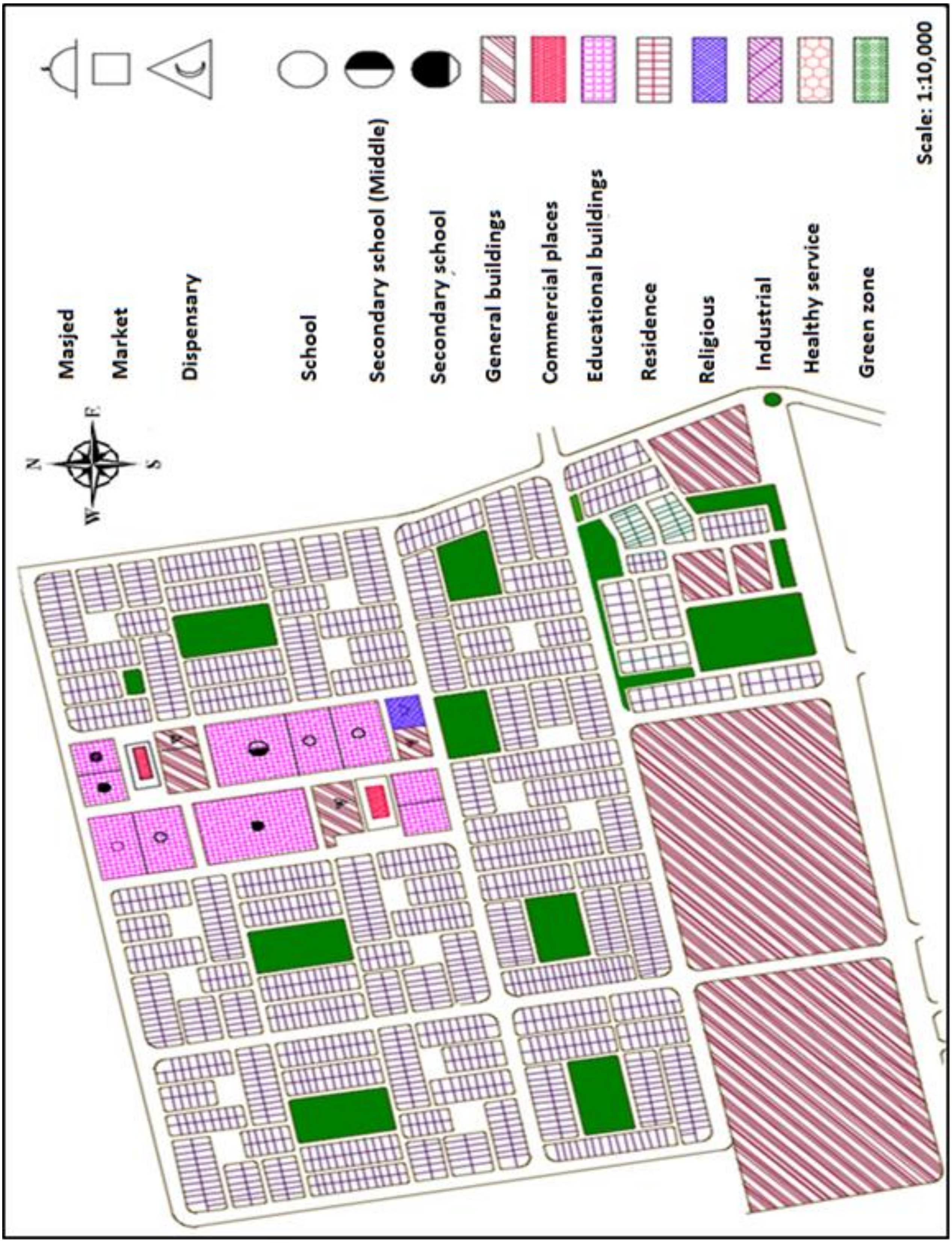

Fig. 3.3: Land use map of the study area (Directorate of Urban Planning.,2016).

3.2.3 The Flooding in the Study Area

The study area (Al-Esqari quarter) has a storm drainage network only that covered all the area. This storm drainage system is suffering from the flooding during the rainy days that happens for many reason. The main reason was the illegal quantity of sewage discharge through the storm drainage system and its decrease of the drainage capacity of the system that causing clogging in the pipes and causing flooding. The illegal sewage quantity causes flooding to system under rainfall intensity lower than the design intensity. The climate variation in the recent years increased the rainfall intensity of higher than the design intensity in which in one of the rainy event, it reached to 3.5 times the design intensity in 11/5/2015. The second main reason that contributes to the increase in the intense of the flooding was the urbanization, where most of the ground in the study area is covered by an impervious area that led to increase the time to peak and peak discharge and decrease the infiltration rate which in turn led to increase the volume of runoff. (Figure 3.4) shows an example of the flooding event that happened in the study area at 28/3/2016 with peak rainfall intensity reach to 24.5 mm/h for 5-hrs.

Fig. 3.4: Flooding event (28/3/2016)

3.3 SWMM Model

Storm Water Management Model (SWMM) was first built-up in 1971 by the U.S. Environmental Protection Agency, it is one dimensional, dynamic rainfall-runoff simulation model handle single or continuous event simulation of runoff quantity and quality in urban regions (Rossman et al.,2006). The runoff module in SWMM deal with the precipitation flow over a collection of sub catchments to generate the runoff and then the routing module transport this runoff through a series of pipes, manholes, pumps station, storage etc. The data necessary for modelling contain geology data (soil type), land use, hydraulic data [drainage system dimension and the Spatial distribution], climate data (essentially precipitation and temperature) , topography and hydrologic properties .SWMM widely desirable model in the world due to its ability in planning, analysis and design of storm water, sanitary and combined networks (Rossman et al., 2006). The version 5.1 of SWMM running under Microsoft windows system and giving an integrated environment for data editing and easily can download free by the internet.

3.4 The Computational Processes in SWMM

SWMM is dynamic rainfall–runoff simulation model. It used principles of conservation of mass and momentum and water balance wherever suitable (Rossman .,2010). This section will be describe all the computational practices of the hydrology and hydraulic process that effect in the study area.

3.4.1 Surface Runoff Routing

The runoff theoretical view of SWMM is shown in (Figure 3.5).

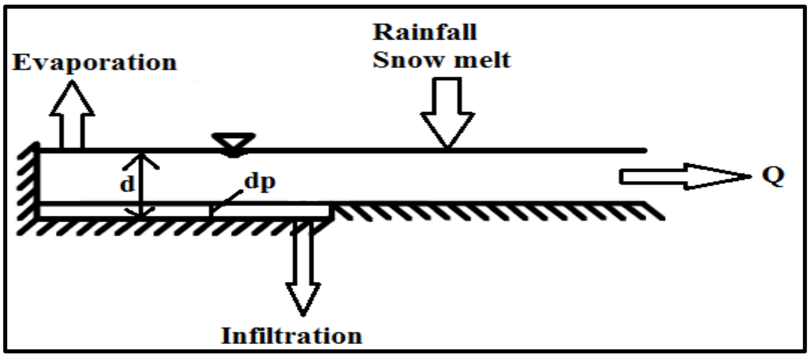

Fig. 3.5: SWMM runoff theoretical view, (Rossman., 2010)

Each sub catchment surface is handled as a nonlinear in which the storage of the reservoir is considered to be a function of inflow and outflow as shown in water balance equation (Eq 3.1). The Inflow comes from precipitation and any nominated upstream sub catchment flow. The outflow has several sources including evaporation, infiltration, and surface runoff Rossman (2010).

$$Q_{storage} = Q_{input} - Q_{output} \qquad \text{.... [3.1]}$$

where:

$Q_{storage}$ = The maximum surface storage provided by ponding, surface wetting, and interception (m^3/sec).

Q_{input} = The inflow that including precipitation and any nominated upstream sub catchment flow (m^3/sec).

Q_{output} = The outflow that including evaporation, infiltration, and surface runoff (m^3/sec).

The water balance is either positive or negative storage. The positive balance happens in the wet seasons where the precipitation higher than the evaporation, which generates a water excess convert and fill the sub catchment stores causing flooding and surface runoff. On the other hand, the negative balance happens in drier seasons where the evaporation exceeds precipitation and the plants absorb the water store in the sub catchment causing the decrease in water.

The capacity of the nonlinear reservoir is the maximum depression storage, which is the maximum surface storage provided by wetting, ponding, and detention of the sub catchment surface. The surface runoff occurs only when the depth of water in the reservoir exceeds the maximum depression storage (dp).

The concept of account the surface runoff for each sub catchment depends on the continuity of mass (Eq 3.2) (Pitt et al. ,1999).

$$\frac{dV}{dt} = \frac{d\,[A*d]}{dt} = A * I_e - Q \qquad \text{.... [3.2]}$$

where:

[$\frac{dV}{dt} = \frac{d\,[A*d]}{dt}$] = The change of volume stored over the sub catchment over time.

V=A*d =The volume of water on the sub catchment (m^3).

A= The sub catchment area (m^2).

d = The water depth on the sub catchment (depth of storage in the reservoir) (m).

I_e = rainfall excess, which is the difference between the rainfall intensity, the evaporation and the infiltration rate (m/sec).

A* I_e = The precipitation excess from the sub catchment.

Q = The runoff flow rate from the sub catchment (m^3/sec) and the account depends on Manning (Eq 3.3).

$$Q = \frac{A_{cs} * R^{\frac{2}{3}} * S^{\frac{1}{2}}}{n} \qquad \text{.... [3.3]}$$

where:

A_{cs} = The cross sectional area of flow over the sub catchment (m^2) and it is equal to the **[W * [d-d_p]]**.

R = The hydraulic radius of flow over the sub catchment (m).

S = The slope of the sub catchment.

W = The width of the sub catchment (m).

n = The manning roughness coefficient for overlanded .

The hydraulic radius (Eq 3.4) is defined as the ratio of cross section area to the wetted perimeter.

$$\mathbf{R = \frac{A}{p} = \{W \cdot [d - d_p]\}/W = d - dp} \quad \text{....[3.4]}$$

So the Manning equation for calculating surface runoff become as (Eq 3.5).

$$\mathbf{Q = \frac{W * (d - d_p)^{\frac{5}{3}} * S^{0.5}}{n}} \quad \text{.... [3.5]}$$

By substituting (Eq 3.5) in (Eq 3.2): -

$$\mathbf{\frac{dd}{dt} = \frac{d_{i+1} - d_i}{t_{i+1} - t_i} = I_e - \frac{W * (d - d_p)^{5/3} * S^{0.5}}{A * n}} \quad \text{.... [3.6]}$$

where:

i, i+1 = The subscripts representing boundary conditions at the end of time step **i** (or start of time step i+1) and the end of time step **i+1** (e.g., d_{i+1} is the depth at the end of time step i+1).

$\mathbf{t_{i+1} - t_i = \Delta t}$ = The time step size (sec).

Q = The average runoff flow rate through time step n+1 (m^3/sec).

$\mathbf{I_e}$ = The average rainfall intensity through time step n+1 (m/sec).

d = The average depth of flow during time step n+1 and represented as $\mathbf{d = (d_i + d_{i+1})/2}$ (m).

Finally, there is only one unknown at any time in (Eq 3.6), which is the d_{n+1}. (The value of d_i is known from the end of the past time step). The Newton-Raphson method for numerically solving a nonlinear equation is used to solve for

$\mathbf{d_{i+1}}$. The calculated value of $\mathbf{d_{i+1}}$ is then used in (Eq 3.5) to calculate the surface runoff at the end of the time step.

3.4.2 System Flow Routing

The process of determining the time and magnitude of the flow at any point of the drainage system based on known or assumed hydrographs at one or more point in upstream is called as the flow routing. There are three levels of sophistication used in SWWM for flow routing to solve the conservation of mass and momentum equations for conduits of open channel and this equation is a comprehensive one-dimensional Saint-Venant (Rossman.,2010). The SWMM allows the modeler to select the level of sophistication to solve the equations. The three level of flow routing in SWMM are steady flow routing, kinematic flow routing and the dynamic flow routing. In this study, the dynamic flow routing has been used because it has the ability to account for pressurized flow, channel storage, flow reversal, backwater and entrance/exit losses as the dynamic flow routing considers the most theoretically precise consequences. In dynamic routing, the full flow in closed pipe represents as pressurized flow and the flooding happens when the water depth skips the maximum available depth at the node.

For the Saint-Venant, the flow can be represented by the two partial differential equations Pitt et al. (1999). Firstly, the momentum (Eq 3.7):

$$\frac{1}{A}\frac{\partial q}{\partial t}+\frac{1}{A}\frac{\partial}{\partial x}\left[\frac{q^2}{A}\right]+g\frac{\partial y}{\partial x}-g\,[S_0\text{-}S_f]=0 \qquad \text{....} [3.7]$$

Secondly, the continuity (Eq 3.8):

$$\frac{\partial A}{\partial t}+\frac{\partial q}{\partial x}=0 \qquad \text{....} [3.8]$$

where:

q = The flow rate in the system (m^3/sec).

g = The acceleration due to gravity (m/s^2).

y = The depth of flow (m).

S$_\circ$ = The bed slope (m/m).

S$_f$ = The friction slope (m/m).

A = The cross-sectional area (m^2).

x = The distance along the channel (m).

and **t** = The time (sec).

The terms in the momentum equation can be described as follows:

$\frac{1}{A}\frac{\partial q}{\partial t}$ = The change in momentum due to the change in velocity over time.

$\frac{1}{A}\frac{\partial}{\partial x}[\frac{q^2}{A}]$ = The change in momentum due to the change in velocity along the channel.

$g\frac{\partial y}{\partial x}$ = The change in the water depth along the channel.

g [S$_0$-S$_f$] = gravity force term, proportional to the bed slope and friction force term, proportional to the friction slope.

The terms in the continuity equations can be represented as follows:

$\frac{\partial A}{\partial t}$ = The rate of change of area with time

$\frac{\partial q}{\partial x}$ = The rate of change of channel flow width distance

The two partial differential equations are solved numerically as done for runoff surface routing. The dynamic flow routing uses the Manning equation to determine flow rate (Q). The Hazen-Williams or Darcy-Weisbach equation is used for circular force main shapes under pressurized flow (Rossman .,2010).

3.4.3 Infiltration Model

The infiltration in SWMM can be modeled in three different formulas Horton, Green-Ampt and SCS curve number method. In this research, the Green-Ampt model has been used. There is no unified vision for preference one way for others, and the Green-Ampt model is more physically-based according to Gironás et al. (2009).

The Green -Ampt method is a simplify, empirical model to represent the infiltration process. It develops from the application of Darcy's law and the law of conservation of mass. Green-Ampt supposes that a sharp wetting front exists in the soil column separating the soil where the above wetting front is fully saturated and the soil below is at the initial moisture content (Rawls et al.,1983). This method is a function of the soil's hydraulic conductivity, soil suction head, porosity and initial moisture deficit of the soil. The general (Eqs.3.9 and 3.10) of the Green-Ampt is given below (Rawls et al., 1983):

$$f = K * \{ [\frac{\Psi * N}{F}] + 1 \} \quad \text{....[3.9]}$$

$$F = K * t + \Psi * N * \ln \{1 + [\frac{F}{\Psi * n}]\} \quad \text{....[3.10]}$$

where:

f = Infiltration capacity (mm/h).

K = Saturated hydraulic conductivity (mm/h).

Ψ = Suction head (mm).

N = Available porosity.it can be calculated by subtract wilting point from field capacity.

F = Mass infiltration [mm].

Rawls et al., (1983) considered these equations under the assumption that the depth of ponding on the soil surface is negligible and analyzed approximately 5000 soils samples through the United States and published values for the Green-Ampt parameters as shown in Table 3.1:

Table 3.1: Parameters of Green-Ampet for different soil type, (Rawls et al., 1983)

Soil texture class	**K** *in/hr*	**Ψ** *in*	**Φ**	**FC** *in*	**WP wilting point**
Sand	*4.74*	*1.93*	*0.437*	*0.062*	*0.024*
Loamy Sand	*1.18*	*2.4*	*0.437*	*0.105*	*0.047*
Sandy Loam	*0.43*	*4.33*	*0.453*	*0.19*	*0.085*
Loam	*0.13*	*3.5*	*0.463*	*0.232*	*0.116*
Silt Loam	*0.26*	*6.69*	*0.501*	*0.284*	*0.135*
Sandy Clay Loam	*0.06*	*8.66*	*0.398*	*0.244*	*0.136*
Clay Loam	*0.04*	*8.27*	*0.464*	*0.31*	*0.187*
Silty Clay Loam	*0.04*	*10.63*	*0.471*	*0.342*	*0.21*
Sandy Clay	*0.02*	*9.45*	*0.43*	*0.321*	*0.221*
Silty Clay	*0.02*	*11.42*	*0.479*	*0.371*	*0.251*
Clay	*0.01*	*12.6*	*0.475*	*0.378*	*0.265*

3.4.4 Groundwater and Aquifer

In SWMM, the aquifer is used to model the vertical movement of water infiltrating from the sub catchments that located overhead them (Rossman., 2010). Aquifers are denoted by two zones: un-saturated and saturated zones and their performances are described by parameters such as hydraulic conductivity, soil porosity, initial moisture content, the water table elevation, the unsaturated zone, bottom elevation, and evapotranspiration depth. (Eq 3.11) is used to calculate the groundwater flow: -

$$Q_{gw} = A_1 [H_{gw} - H^*] B_1 - A_2 [H_{sw} - H^*] B_2 + A_3 H_{gw} H_{sw} \quad \text{....} [3.11]$$

Where:

$Q_{\mathbf{gw}}$ = The groundwater flow (m^3/sec per hectare).

$H_{\mathbf{gw}}$ = The height of saturated zone above the bottom of the aquifer (m).

$H_{\mathbf{sw}}$ = The height of surface water at receiving node above aquifer bottom (m).

H* = The threshold groundwater height (m).

The coefficients (A_1, A_2, B_1, B_2, and A_3) appear in the equation compute the groundwater flow according to the (Rossman., 2010) as A_1=A_2=0.5, B_1=B_2=1 and A_3=0.

3.5 Sub catchment, Pipe and Junction

The urban sub catchment is the area discharged by a network of connecting pipes or by streams and the surface runoff generated in this area drainage into single outlet (Reddy., 2005). The sub catchment is divided into pervious and impervious surfaces, which is in turn divided into two areas: one has a depression storage and another that has not. The impervious area losses the rainfall only in the depression storage and the remain convert to a runoff in the street or flow in the pipes, but in the pervious area, the runoff can infiltrate through the upper soil zone (Rossman., 2010).

Spatial differences of the study area into sub catchments characteristic such as land use, drainage pathway and topography make the division of the drainage area to rectangular sub catchment is a general practice in the hydrological models but the spatial resolution to division the drainage basin has not a considerable influence on the surface runoff results (Park et al., 2008). The runoff block in SWMM generate the runoff from a collection of the sub catchment received the rainfall, then the routing block in SWMM transport this runoff to the system of pipes and junction where SWMM during this processes allows to determine the runoff flow for each sub catchment and flow quantity for each pipe in the system. The conduit is a circular stream transport water between two point and the junction is a point used to link a conduit together and when there is a change in the conduit characteristic. Outfalls are terminal junctions of the drainage system employed to describe the final downstream boundaries. After defined the component of the study area, the SWMM sketch could be drawn as shown in (Figure 3.6).

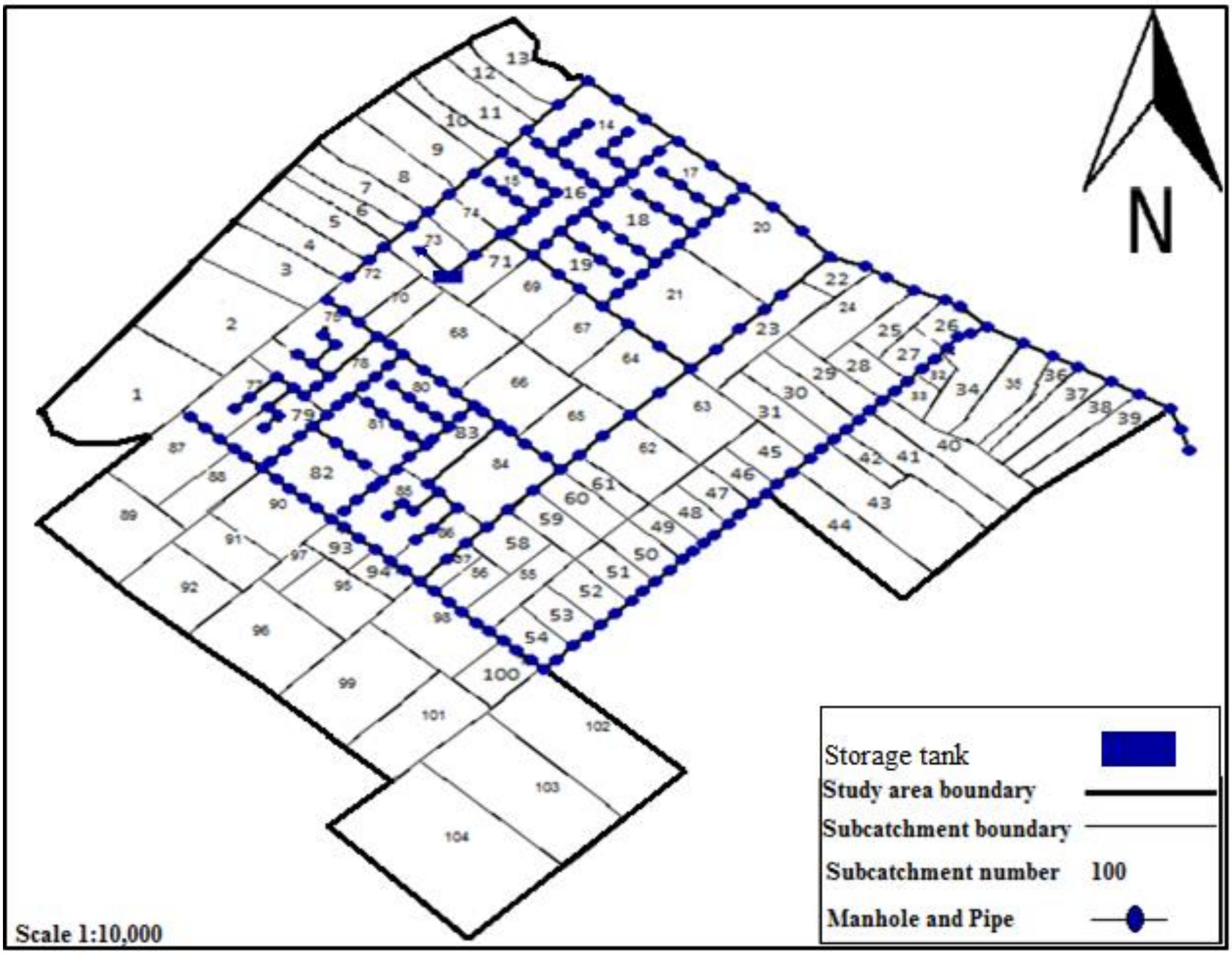

Fig. 3.6: Sketch representing the study area in SWMM

Each component in SWMM (sub catchment, conduit and junction) has several parameters that must be provided to get an accurate simulation and to solve the equations described in the previous parts.

3.5.1 SWMM Sub Catchment Parameters

3.5.1.1 Area and Width

The aerial image has been provided from (KSD., (2015)) (Kerbala's sewage directors), which displays the spatial distribution and assists in estimating the area of sub catchment easily by the tool of measurement in the GIS. The area of the sub catchment in Al-Eskari quarter ranges from 0.078 to 4.45 hectare. There was no real physical meaning for the width of the sub catchment (Cantone and Schmidt., 2011) . The width can be calculated as the surface area divided by the runoff length in which the latter represents the length of the longest surface flow route (Shen and Zhang., 2014) . The width can be calculating by the Eqs. (3.12) (3.13):

$$W=\frac{A}{L_{max}} \quad \text{....[3.12]}$$

Where:

A = The area of the sub catchment. (m^2)

L_{max} = The maximum runoff length in the sub catchment, which can be computed Eq. (3.13):

$$L_{max} = \max [D_{p,\,pout}] \quad \text{....[3.13]}$$

where:

$D_{p,pout}$ = The distance between the outlet point of the sub catchment and another arbitrary point. This arbitrary point represents the farthest point to the outlet and should be one of the vertexes of the sub catchment, (Figure 3.7) describes the process.

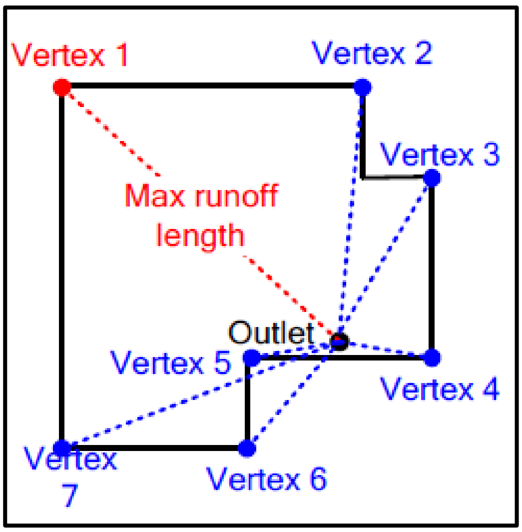

Fig. 3.7: The calculation of the maximum runoff length, (Shen and Zhang., 2014)

3.5.1.2 Slope

It is the tendency of the overland flow surface. The value for each sub catchment can be found from subtracting the ground elevation of the manholes in downstream from the upstream at each sub catchment divided by the distance between them.

3.5.1.3 Impervious Percent

The sub catchments area can be divided into two part: pervious and impervious surfaces. The impervious area in turn is divided into two areas in which one has a depression storage and the other has not. The impervious area losses the rainfall only in the depression storage and the remain converts to a runoff in the street or flow in the pipes. However in the previous area, the runoff can infiltrate through the upper soil zone (Rossman.,2010). The imperviousness percentage has indirect effects on downstream receiving waters and a direct effect on local surface water (Chabaeva et al., 2009). The percent of the impervious and pervious area can be estimated directly from the aerial image and land use map supplemented from urban planning and (KSD.,2015) by dividing the area of impervious area by the total area of the sub catchment. The impervious percent range from 10% to 98% in the study area.

3.5.1.4 Manning Roughness

It is one of the most effective parameter in the hydrologic modelling. The values for each sub catchment depend on the land use type. Values of Manning roughness for different overland type is tabulated in Table 3.2.

Table 3.2: The Manning roughness coefficient for overland, (McCuen et al., 1996)

n	Surface type
0.011	*Smooth asphalt*
0.012	*Smooth concrete*
0.013	*Ordinary concrete lining*
0.014	*Good wood*
0.014	*Brick with cement mortar*
0.015	*Vitrified clay*
0.05	*Fallow [no residue]*
Cultivated soils	
0.06	*Residue cover ≤ 20%*
0.17	*Residue cover > 20%*
0.13	*Range [natural]*
Grass	
0.15	*Short, prairie*
0.24	*Dense*
0.41	*Bermuda grass*
Woods	
0.4	*Light underbrush*
0.8	*Dense underbrush*

3.5.1.5 Depth of Depression Storage

The depth of depression storage for previous and impervious area is an important parameter and the values were taken based on Table 3.3.

Table 3.3: The depth of depression storage for various land use, (Rossman., 2010)

Depth of depression storage	
Impervious surfaces	*0.05 - 0.1 in*
Lawns	*0.1- 0.2 in*
Pasture	*0.2 in*
Forest litter	*0.3 in*

3.5.2 SWMM Drainage Network Parameters

Data regarding the pipes, manhole and pump station was provided by (KSD., 2015) and is shown in (Figure 3.8).

3.5.2.1 Pipes Properties

The spatial distribution and the upstream and downstream altitude must be supplied for each pipe in the model to get the slope and specify the flow direction of the fluid. The drainage system of the study area consists from circular pipes with diameter range from 315 mm to 600 mm and the length of pipes range from 11m to 114m. The material of the pipes is polyvinyl chloride (PVC), and the Manning roughness coefficient values for the pipes are given in the Table 3.4 in which the Manning value for the PVC pipe (plastic pipe) is 0.009.

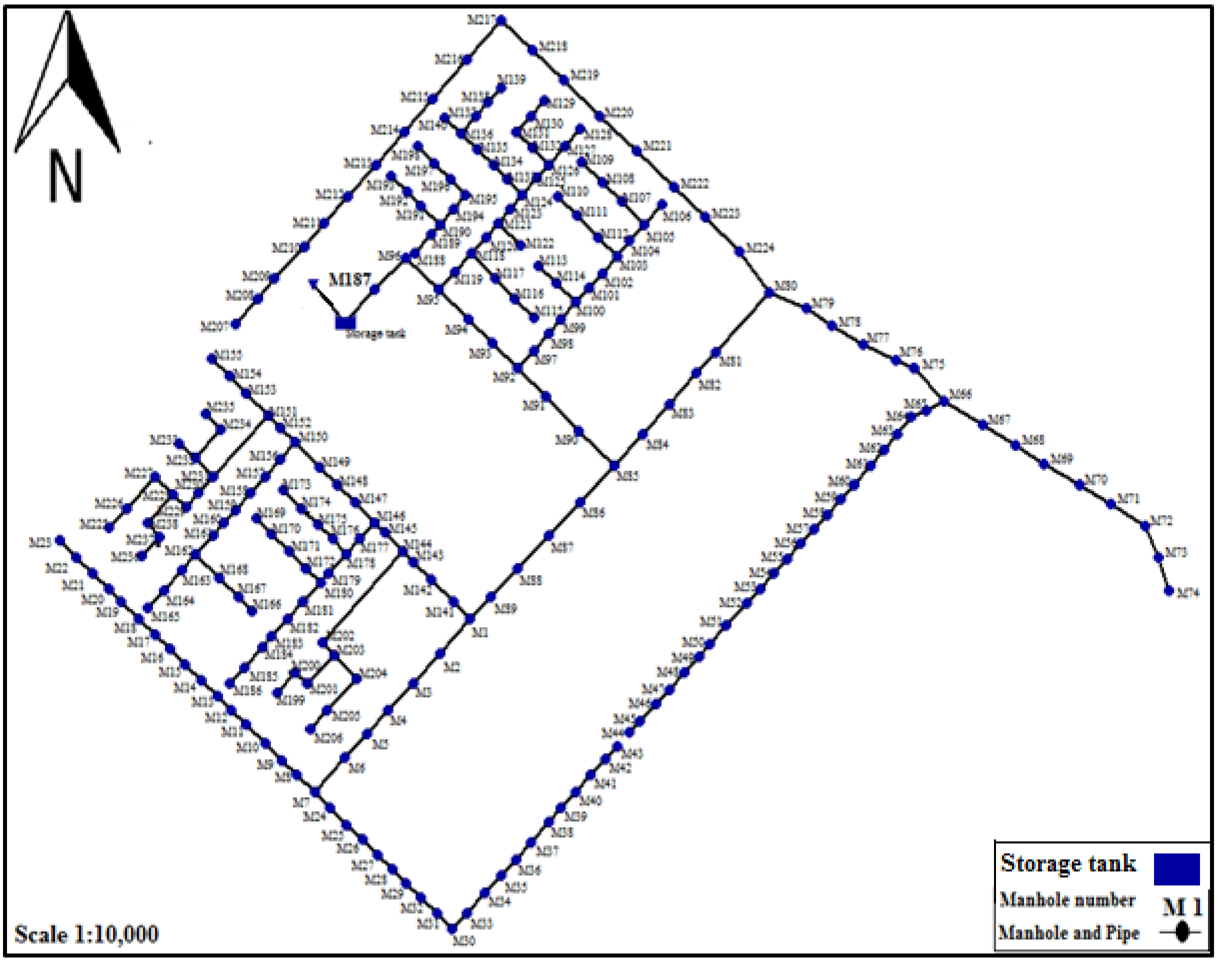

Fig. 3.8: The storm drainage network of Al-Eskari quarter.

Table 3.4: Values of Manning roughness coefficient for pipe (steel ., 1979).

n_p	The material of pipe
0.009	*Plastic pipe*
0.009	*Well-planed timber evenly laid*
0.01	*Neat cement. Very smooth pipe*
0.012	*Unplanned timber. Cast-iron pipe of ordinary roughness*
0.013	*Well-laid brick work. Good concrete. Riveted steel pipe. Well-laid vitrified clay pipe*
0.015	*Vitrified tile and concrete pipe poorly jointed and unevenly settled. Average brick work*
0.017	*Rough brick. Tuberculate iron pipe*
0.02	*smooth earth or firm gravel*
0.03	*Ditches and rivers in good order , some stones and weeds*
0.04	*Ditches and rivers with rough bottoms and much vegetation*

3.5.2.2 Junction Properties

Three parameters are needed to define the junction: the maximum depth, invert level and the inflow that enters to each junction due to the discharge of the sewage to the storm drainage network in AL-Eskari quarter. The first two parameters are provided from Kerbala sewage director. According to report from the central statistical organization of planning ministry the water consumption per capita in the central statistical organization of Iraqi planning ministry [IPM], was 422 liters/day per capita. The estimation of the average sewage for each manhole according to (steel., 1979) can be determined as follows:

$$Q_{avg}=0.8*P*G \quad \text{....[3.14]}$$

where: -

Q_{avg} is the average sewage discharge for each manhole (liter/day).

P is the population number (thousand).

G is the water consumption per capita (liter/day).

The percent of the sewage per capita from the water consumption ranges from 70 to 130 % (steel., 1979), and in Iraq, its approximately equal to 80% (KSD.,2015). The SWMM requires a time pattern for the (hourly, daily, monthly) variation in sewage, (Choi., 2016) has been supplied a time pattern for the dry weather flow as shown in Table 3.5. This variation is suitable for the resident area.

Table 3.5: Sewage variation pattern (Choi., 2016)

Daily pattern	
Sun	*1.02*
Mon	*1*
Tue	*0.99*
Wed	*0.95*
Thu	*0.94*
Fri	*1.01*
Sat	*1.1*

Monthly pattern	
Jan	*1*
Feb	*1*
Mar	*1*
Apr	*1*
May	*1*
Jun	*1*
Jul	*1*
Aug	*1*
Sep	*1*
Oct	*1*
Nov	*1*
Dec	*1*

Hourly pattern			
AM 12:00:00	*0.9*	*PM 12:00:00*	*1.17*
AM 01:00:00	*0.82*	*PM 01:00:00*	*1.17*
AM 02:00:00	*0.7*	*PM 02:00:00*	*1.13*
AM 03:00:00	*0.64*	*PM 03:00:00*	*1.1*
AM 04:00:00	*0.6*	*PM 04:00:00*	*1.07*
AM 05:00:00	*0.63*	*PM 05:00:00*	*1.08*
AM 06:00:00	*0.75*	*PM 06:00:00*	*1.1*
AM 07:00:00	*0.91*	*PM 07:00:00*	*1.12*
AM 08:00:00	*1.06*	*PM 08:00:00*	*1.12*
AM 09:00:00	*1.18*	*PM 09:00:00*	*1.15*
AM 10:00:00	*1.23*	*PM 10:00:00*	*1.11*
AM 11:00:00	*1.22*	*PM 11:00:00*	*1.01*

3.5.2.3 Pump Station

The pump station in the Al-Eskari quarter receives the water amount from the study area only. The station contains three submersible pumps that have one inlet pipe with a diameter of 600 mm and two pressurized outlets with a diameter of 315 mm drain in the river. The pump station has storage tanks with a dimension of (12*10*5) m^3.

3.6 Climate Data

The rainfall intensity, temperature and the wind speed are the most important hydrological data that should be provided in SWMM.

3.6.1 Rainfall Data

Finding the rainfall data was not simple, but the network meteorology of the Agricultural Meteorology Iraqi Network (AMIN) (2015) provided online an hourly data of the rainfall intensity for different Iraqi cities. For Kerbala, there are three meteorology stations in which the closet one for the study area was Al-Razaza station, which is located to the north of Kerbala city center in (43º 97′ E) and (32 ° 55′ N) (it lies about 20 km for the study area). The records covered the period of time from March of 2008 to March of 2016. Since there is not too high spatial-temporal variability of rainfall in Kerbala, this data has been used to model the hydrological process in the study area. All this data has been inserted in one rain gage for all the sub catchment. (Figure 3.9) gave the rainfall data from 2008 to 2016 that used in the modeling.

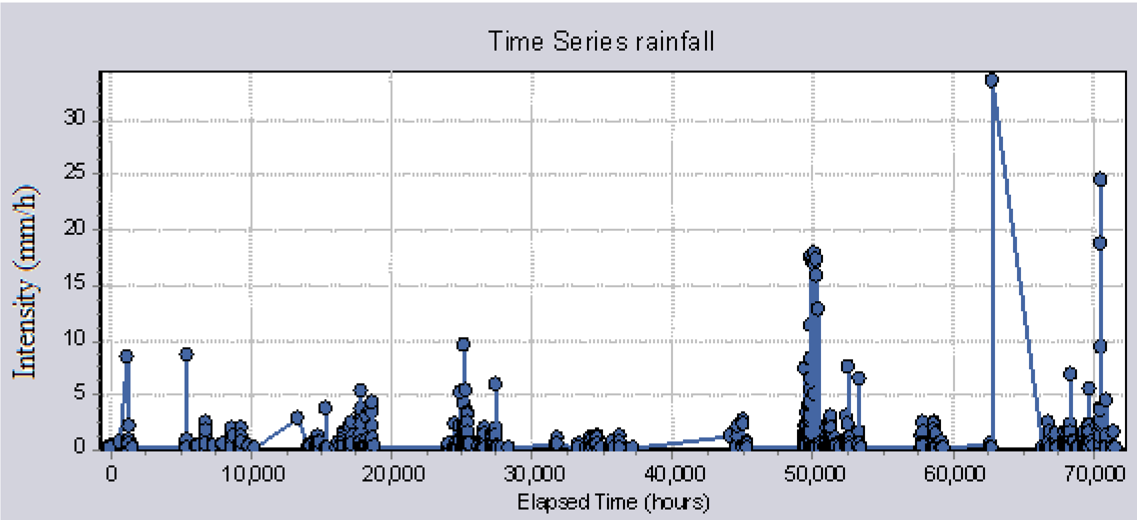

Fig. 3.9: The rainfall data from [2008 – 2016]

3.6.2 Evaporation

The evaporation in the natural environment is the phenomenon convert of water on the land surfaces covered by plants or from free water surfaces (lake, reviver, etc.) to the atmosphere. It is one of the main elements of the hydrological cycle. This cycle consists of continuous transmission of water from atmosphere to the ground's surfaces by precipitation, where it discharges to rivers, seepage, or directly as surface runoff. The cycle is closed as the water evaporates back into the atmosphere, the evaporation from land surfaces, together with rainfall , manages the amount of runoff that is existing for the sub catchment (Brutsaert., 1982). There is a different style in defining the evaporation in SWMM. In this study area, the monthly average style has been used to feed an average monthly rate. The monthly rate is from 2008-2014 that been supplied from the Iraqi meteorological agency and earthquake monitoring (IMAEM.,2016) as given in Table 3.6.

Table 3.6: monthly rates (mm/day) of evaporation in Kerbala from 2008-2014 (IMAEM.,2016)

Dec.	Nov.	Oct.	Sep.	Aug.	Jul.	Jun.	May	Apr.	Mar.	Fab.	Jan.
2.1	3	6.3	9.3	13	14	13	9.7	7.3	5.5	3.1	2

3.6.3 Wind Speed

The data of the wind speed supplied from the Iraqi meteorological agency and earthquake monitoring (IMAEM.,2016) as shown in Table (3.7). For the study area, the monthly average style has been employed.

Table (3.7): monthly rates (Km/h) of wind speed in Kerbala from 2008-2016 (IMAEM.,2016)

Dec.	Nov.	Oct.	Sep.	Aug.	Jul.	Jun.	May	Apr.	Mar.	Fab.	Jan.
6.92	6.68	7.2	8.9	12	16	15	12	12	11.2	9.6	7.9

3.7 The Input Data

All the data that input in the SWMM model for sub catchment and pipes was summarized in the 3.8 and 3.9 below:-

Table (3.8): the input data for the sub catchment

No.	Symbol	Description	value	Source
1.	n	Manning roughness coefficient for impervious area (Brick with cement mortar)	0.014	Table 3.2
2.	n	Manning roughness coefficient for impervious area (Smooth asphalt)	0.011	Table 3.2
3.	n	Manning roughness coefficient for impervious area (vitrified clay)	0.015	Table 3.2
4.	d_p	Depth of depression storage for impervious area	1.27 mm	Table 3.3
5.	d_p	Depth of depression storage for impervious area	2.54 mm	Table 3.3
6.	Ψ	Suction head (Sandy clay)	240 mm	Table 3.1
7.	K	Conductivity	0.508 mm/hr	Table 3.1

Table (3.9): The input data for the pipes

No.	Symbol	Description	Value	Source
1.	n_p	Manning roughness coefficient for pipes	0.009	Table 3.4

Chapter four
Results and Discussion

4.1 Introduction

SWMM modeling results have been introduced in this chapter. These results evaluate and analyses the flooding events of the storm drainage network in Al-Eskari quarter, Karbala, Iraq. To fulfill the aim of the study, calibration of the SWMM model has been conducted. After that, the results have been evaluated for three different cases:1- the results of flooding events under the climate variation only. 2-the results of flooding events after adding the sewage amount to the storm drainage network and 3- the results under a suggestion solution to reduce the flooding events. Moreover, an estimation for the cost of damage of the infrastructure has been determined due to the flooding events.

4.2 Model Calibration

A manual trial and error method has been used to calibrate the simulation model of storm drainage network of the study area. There was a lack in data that are needed to calibrate the model. Only data of two rainy days was provided. The first event was in 18/1/2016 with rainfall intensity of 2 mm/h for 60 min, and the second event was in 22/1/2016 with rainfall intensity of 4 mm/h for 60 min. The data was the total discharge at the outlet of the storm system for the two days. This total discharge represents the total sewage and rainfall quantities of the study area at the two days. This total discharge re-distribution for the whole system depending on the number of population contribution for each manhole to provide a data base for the calibration process. In the process of calibration, the most affected parameter on the results was the impervious percent, the width and Manning’s roughness coefficient of the sub catchments that has also been noted by (Beling et al., 2011). These parameters were changed in each trial until the simulated data draw down the observed data.

In order to check the validity of all the input parameters in the model and estimate the model parameters, cross validation is carried out on the data. The cross validation, which results from two rainfall intensity events are shown in Table 4.1. The cross validation results showed that the mean error (ME) for two events are very close to zero (0.0068) and (0.0032) for events in [18/1/2016] and [22/1/2016] respectively. Moreover, the mean square error (MSE) is very low as compared to the variance of the observed data for both events. The coefficient of determination [R^2] for the first rainfall event was 0.95 and for second rainfall event was 0.94. The above cross validation results showed that the chosen models and their parameters are adequate.

Table 4.1: Fitted parameters (Cross-Validation) of model.

No.	Parameters	Event	
		18/1/2016	22/1/2016
1.	ME	0.0068	0.0032
2.	MSE	0.0609	0.0417
3.	R^2	0.952	0.940

4.3 Effect of Climate Variation

The climate variation is a continuous and dynamic process. This variation projected to get rapidly in the future. In the middle east, especially in Iraq (Kerbala –Al Eskari quarter), the variance in the rainfall intensity from 2008 to 2016 is illustrated in Figure 4.1. In the study area the storm drainage network was design to carry a rainfall intensity for 2yr return period of 9.6 mm/h for 60 min (Karbala's Sewage Director [KSD], 2015).

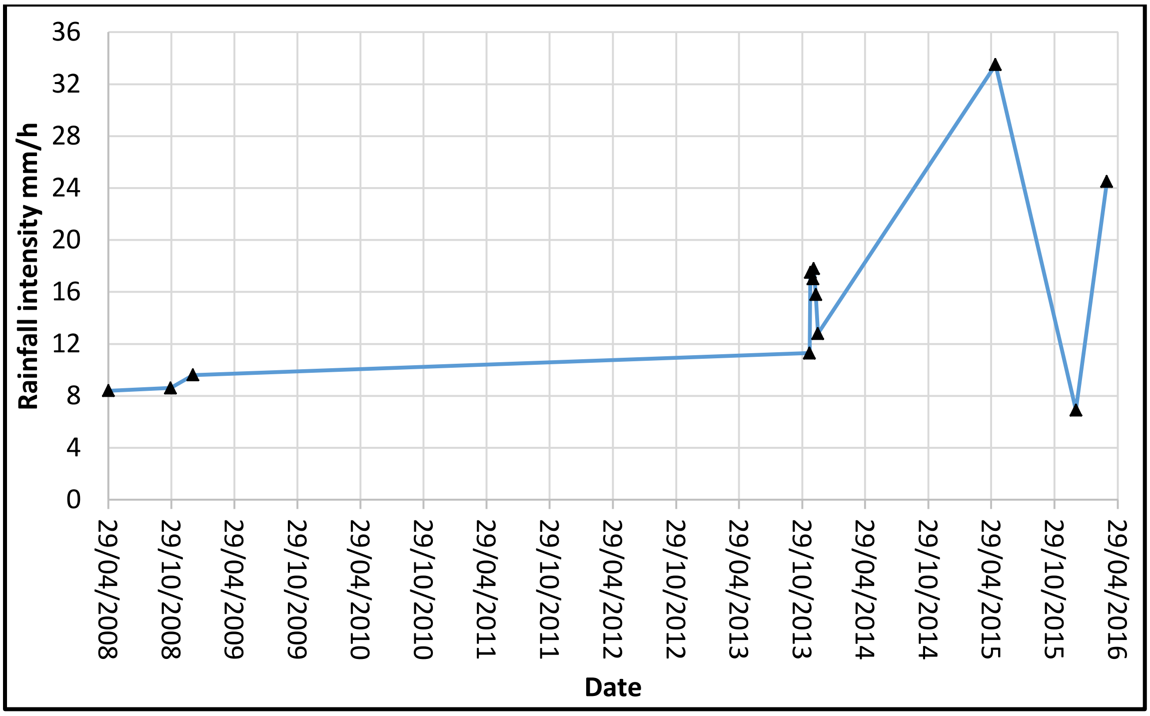

Fig. 4.1:The peak rainfall intensity during 2008 to 2016 for the study area

It can be observed from Figure 4.1 that the climate gets variety after year 2012. In year 2013 and specifically in November, only 3 rainy events have been occurred with intensity higher than the design intensity (9.6 mm/h). The first event was happened in 19/11/2013 with a rainfall intensity of 11.3 mm/h. The second event was occurred in 21/11/2013 with a rainfall intensity of 17.5 mm/h. The third event was happened in 29/11/2013 with a rainfall intensity of 17 mm/h. In December, 2013, there was three events. The first event was in 1/12/2013 with a rainfall intensity of 17.8 mm/h, the second event was in 7/12/2013 with a rainfall intensity of 15.8 mm/h and the third event was in 13/12/2013 with a rainfall intensity of 12.8 mm/h. It can be observed from these events that it happened in convergent periods. The maximum rainfall event was in 5/11/2015 with a rainfall intensity of 33.5 mm/h. The characteristic of this event was its happening in a summer season and this an evidence for the climate variation.

4.3.1 Results of Climate Variation by SWMM

The climate variation in the study area has been simulated for all the rainy events from 2008 to 2016. The study analyzed the effect of the highest rainy events using SWMM and compared it with the design intensity (9.6 mm/h for 1-hour) depending on the volume of water flooding and the equivalent water depth in the outlet manhole [M187]. The flooding discharge amount of the manholes divided into five stages according to SWMM and as follows:

- Stage1 (no flooding) ranges from (0 to 0.001 m^3/sec),
- Stage2 (very light flooding) ranges from (greater than 0.001 to 0.01 m^3/sec),
- Stage3 (medium flooding) ranges from (greater than 0.01 to 0.02 m^3/sec),
- Stage4 (high flooding) ranges from (greater than 0.02 to 0.04 m^3/sec),
- Stage 5 (very high flooding) for (greater than 0.04 m^3/sec).

The extend of the area flooding (the total area of open space and roads) has been classified approximately depending on the manholes that linked to it.

4.3.1.1 Results of Climate Variation by SWMM under Rainfall Intensity of 9.6 mm/hr

The event with the design intensity of (9.6 mm/h) for 1-hour was in (30/12/2008) and it has been analyzed. Figure 4.2 illustrates the performance of the system under the design intensity.

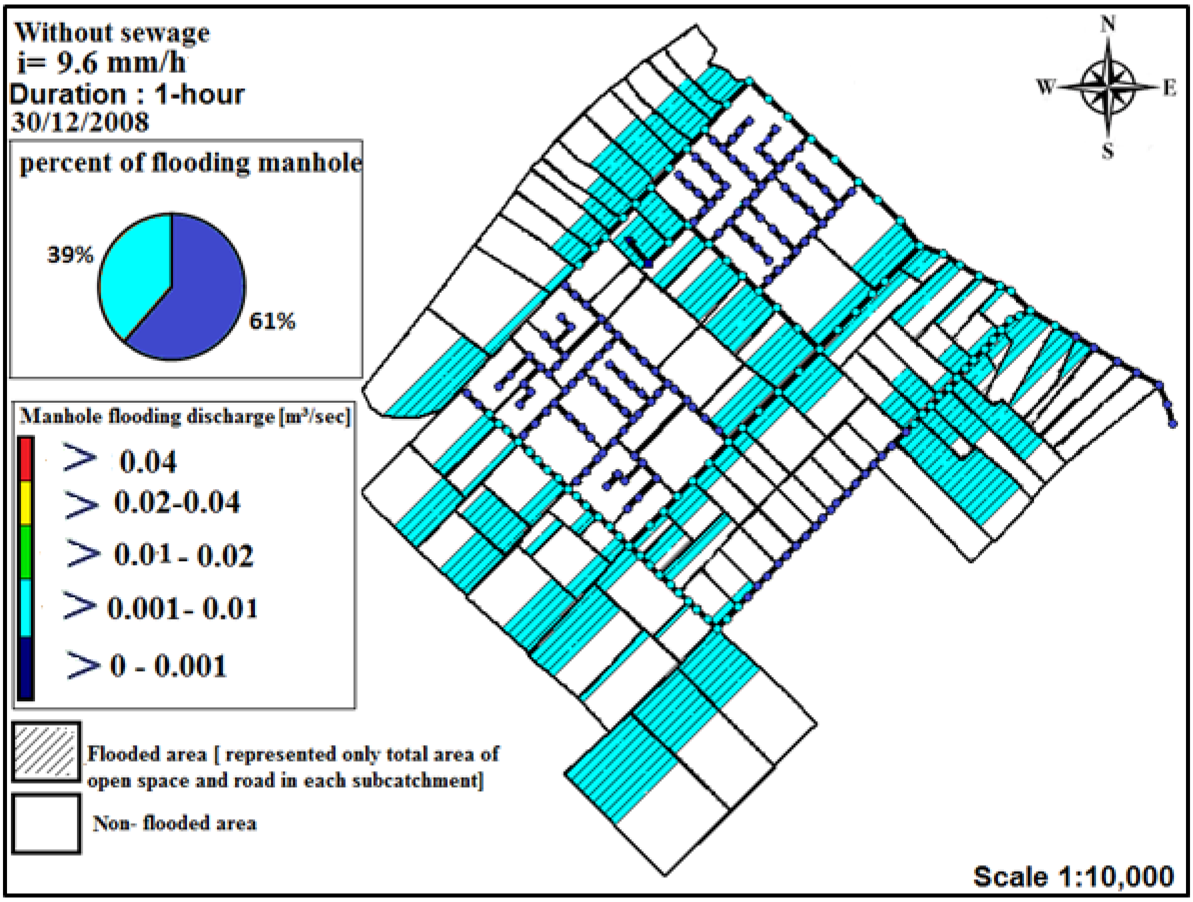

Fig. 4.2:The flooding manholes under design intensity of (9.6 mm/h) at peak time

The percent of flooding manholes under design rainfall intensity of (9.6 mm/h), where 61% of the manholes was within stage1 (from 0 to 0.001 m^3/sec) (no flooding) and 39% of the manhole was within stage2 (greater than 0.001 to 0.01 m^3/sec) (very light flooding). The total flooding duration was half-hour.

The equivalent water depth was measured from the invert level of the manhole. The depth of the manhole (M187) (from invert to the ground level) was 3.47 m. Figure (4.3) illustrates the equivalent water depth in the outlet manhole (M187) during the flooding.

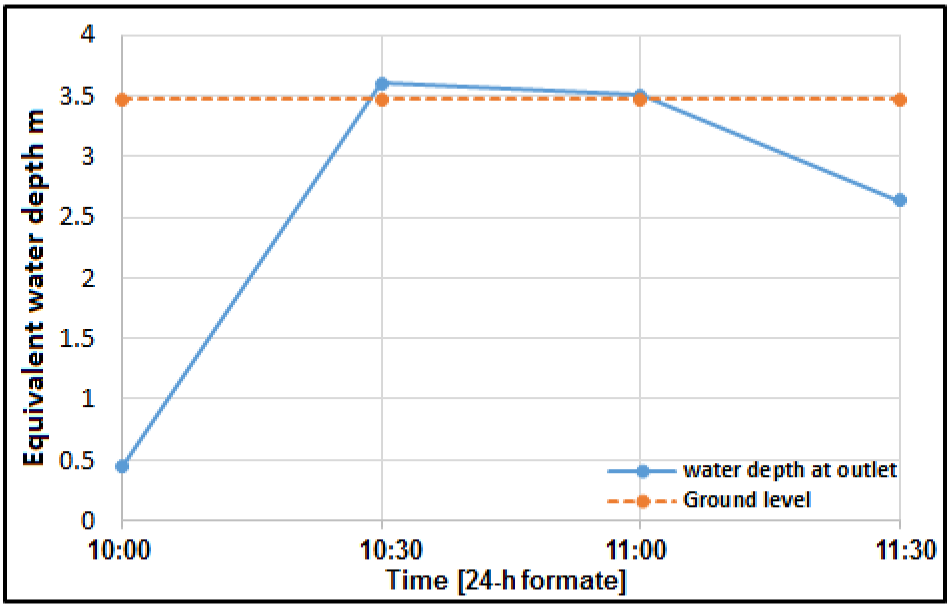

Fig. 4.3:The equivalent water depth in the outlet manhole (M187) with rainfall intensity 9.6 mm/h

Also, Figure (4.4) stated that the volume of water flooding in the outlet manhole (M187).

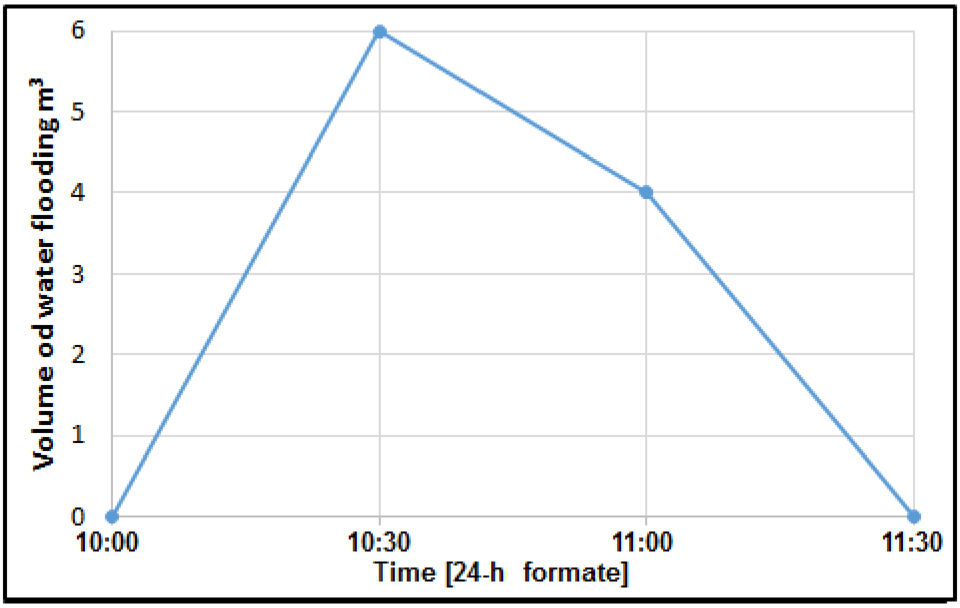

Fig. 4.4:The flooding volume in the outlet manhole (M187) with rainfall intensity 9.6 mm/h

The flooded area around the outlet manhole can be estimated by divided the maximum volume of water flooding on the maximum depth that can be reached when the water flooded on the surface (the sidewalk depth with 0.25 m). Under rainfall intensity of 9.6 mm/h, the flooded area around the manhole was 24 m^2. The performance of the storm network with flooding duration of half-hour, maximum water level of 3.6 m, flooding volumes of 6 m^3 and flooded area of 24 m^2 around the outlet. It can be revealed say that the system has a good performance under the design intensity of 9.6 mm/h and without considering the sewage amount. This is because the flooding event was continuous for only half-hour and the system during that period discharge all the water flooding and that can be acceptable.

4.3.1.2 Results of Climate Variation by SWMM under a Continuous Storm with Peak Rainfall Intensity of 11.3 mm/hr

The precipitation storm happened in 19/11/2013 with peak rainfall intensity 11.3 mm/h continuous for 18-hours illustrated in figure 4.5.

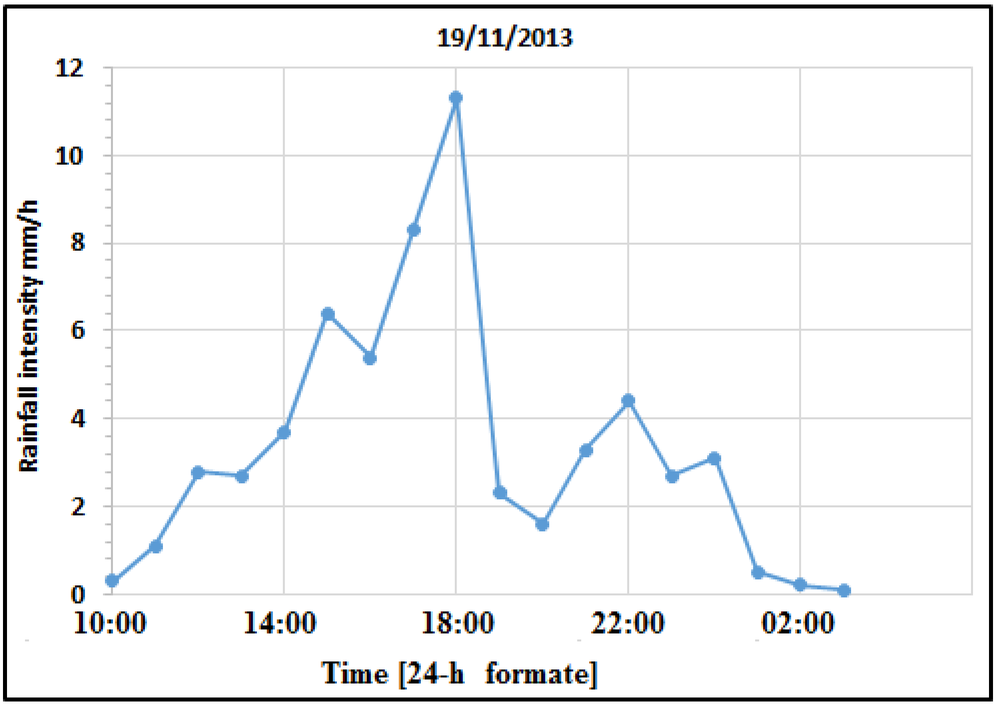

Fig. 4.5:The storm intensity at 19/11/2013

It can be observed that the storm happened in 19/11/2013 was with high rainfall intensity and with long time. This storm has a large effect on the system performance as illustrated in Figure 4.6, which observe the performance of the storm drainage system under this storm.

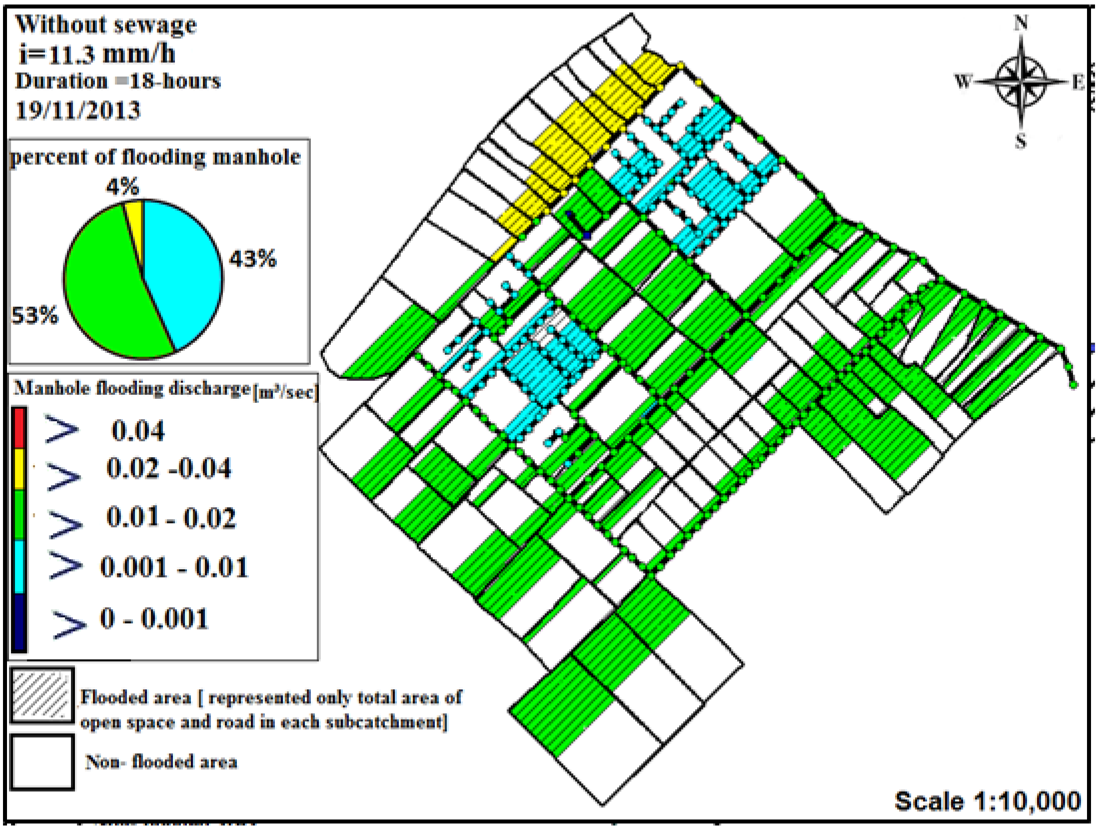

Fig. 4.6:The flooding manholes under rainfall intensity 11.3 mm/h at peak time

Figure 4.6 states the flooding manhole percent under peak rainfall intensity of 11.3 mm/h, where 4% of the manholes flooded within stage4 (greater than 0.02 to 0.04 m^3/sec) (high flooding rate), 53% of the manholes had a flooding discharge within stage3 (greater than 0.01 to 0.02 m^3/sec) (medium flooding rate), and 43% of the manholes flooded within stage2 (greater than 0.001 to 0.01 m^3/sec), (very light flooding rate). According to model analysis results, the total duration of the manhole flooding was about 45 hour, which is due to the long time of the storm and the high rainfall intensity. Figure 4.7 illustrates the equivalent water depth at the outlet (M 187) during the flooding event 19/11/2013.

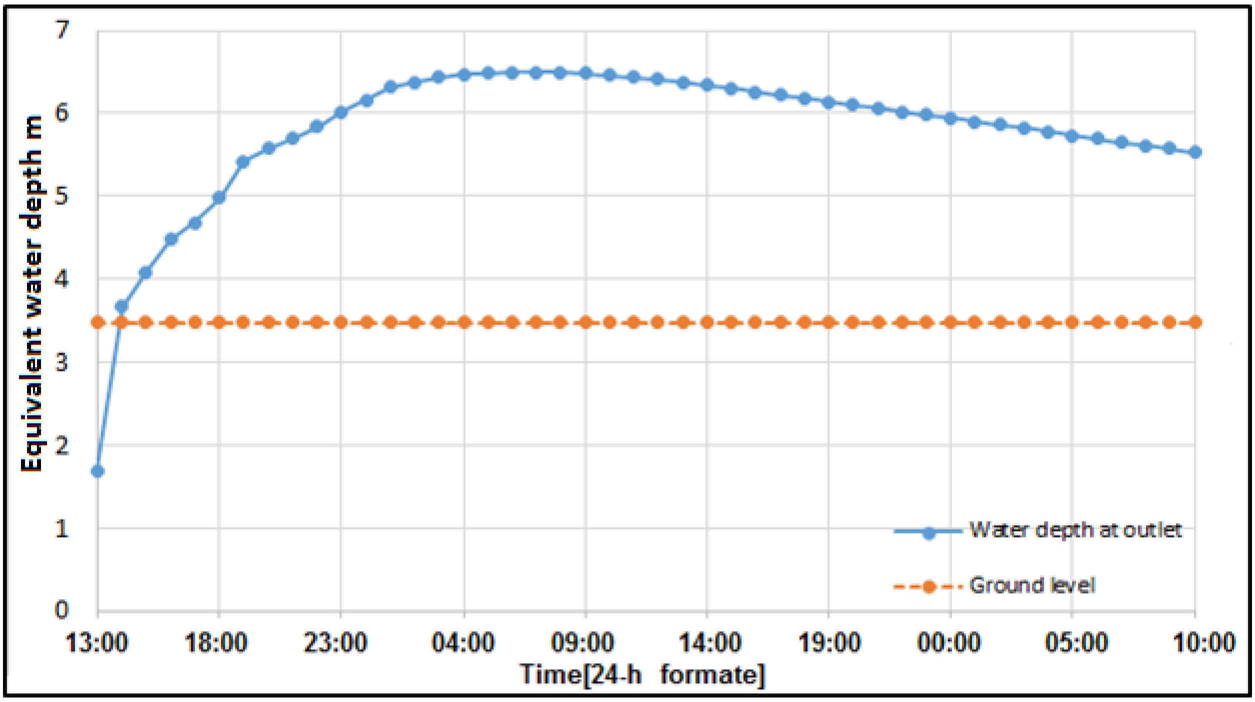

Fig. 4.7The equivalent water depth in the outlet manhole M187 with rainfall intensity 11.3 mm/h

The maximum equivalent water depth reaches to 6.5 m above invert of manhole (M187). In 19/11/2013 the equivalent water depth increased by 1.8 times comparing with the equivalent water depth under design rainfall intensity (3.6 m). Figure 4.8 showed the volume of flooding water under the rainy event of 19/11/2013 at outlet M187.

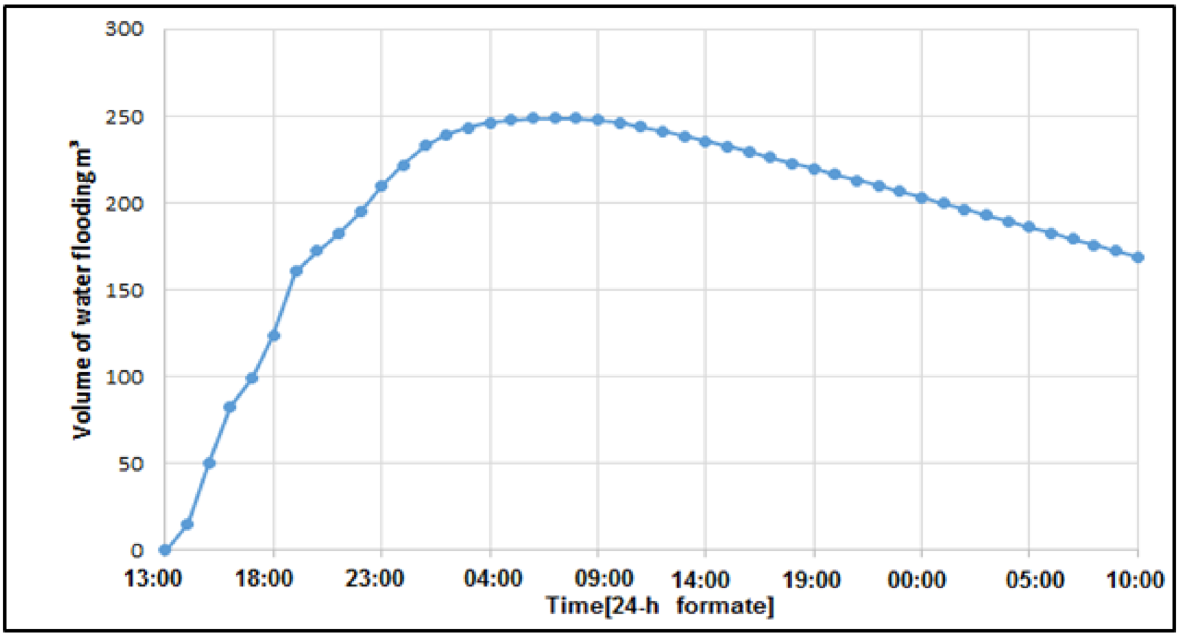

Fig. 4.8: Flooding volume in the outlet manhole M187 with rainfall intensity 11.3 mm/h

The maximum volume of water flooding was 248.82 m^3 under rainfall intensity of 11.3 mm/hr. By comparison with the maximum volume of water flooding under design rainfall intensity (6 m^3), it increased by 41.5 times because the rainfall intensity was higher than the design intensity where the peak intensity reach to 11.3 mm/h and with long duration reach to 16-hour . The flooding area under continues storm of 18-h with peak rainfall intensity of 11.3 mm/h was 995 m^2.

4.3.1.3 Results of Climate Variation by SWMM under Rainfall Intensity of 17.5 mm/hr

Prior the system discharges all the flooding volumes in 19/11/2013 another rainy event at (10:00:00) in 21/11/2013 occurred with rainfall intensity that reached to 17.5 mm/h for one hour. Figure 4.9 illustrates the flooding manholes under rainfall intensity of 17.5 mm/h.

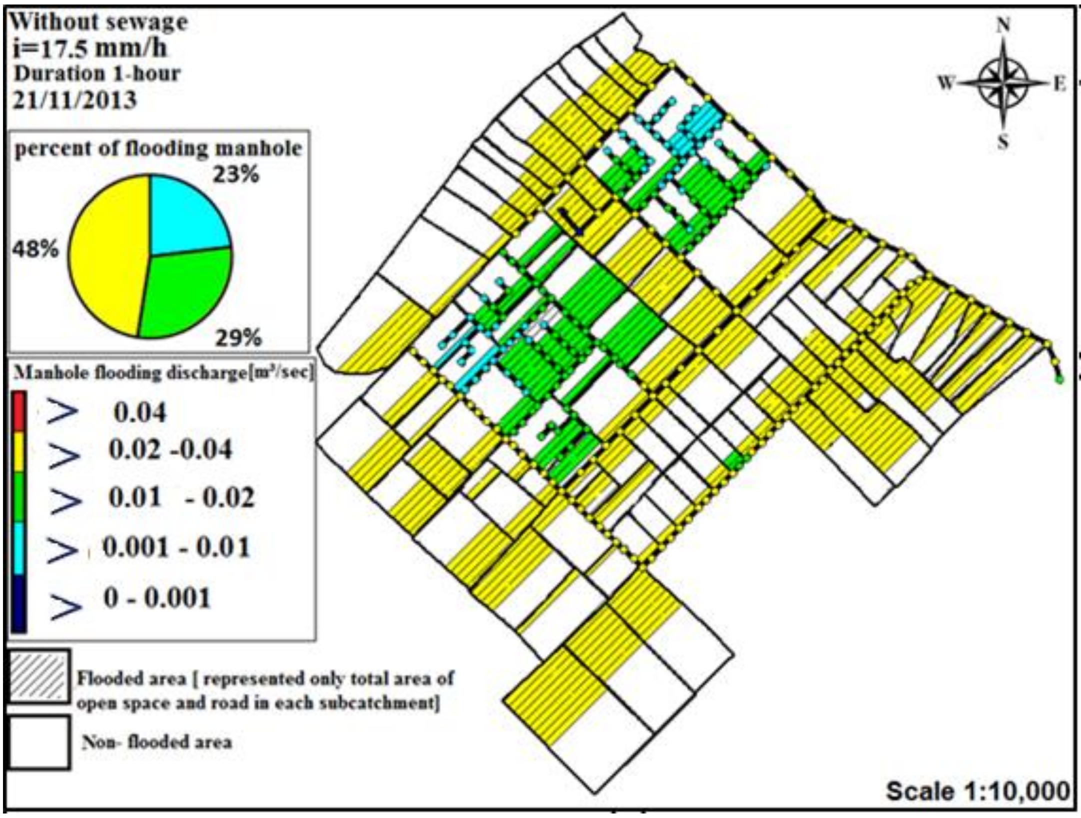

Fig. 4.9:The flooding manholes under rainfall intensity 17.5mm/h at peak time

The percent of flooding manholes under the rainfall intensity of 17.5 mm/h, where 48% flooded with stage4 (greater than 0.02 to 0.04 m^3/sec) (high flooding rate), which led to submerge the neighboring subcatchment as approximate in the figure, in which most of the 48% of the manhole were located at the main transporter pipe. Also, about 29% of the manholes flooded with stage3 (greater than 0.01 to 0.02 m^3/sec) (medium flooding rate) and 23% of the manholes flooded with stage2 (greater than 0.001 to 0.01 m^3/sec) (very light flooding rate], in which all the 23% of the manhole were located at the upstream. The total flooding duration was 50 hours under rainfall intensity of 17.5 mm/h after (10:00:00) 21/11/2013. Figure 4.10 shows the equivalent water depth in the outlet manhole (M187) under the intensity of (17.5 mm/h).

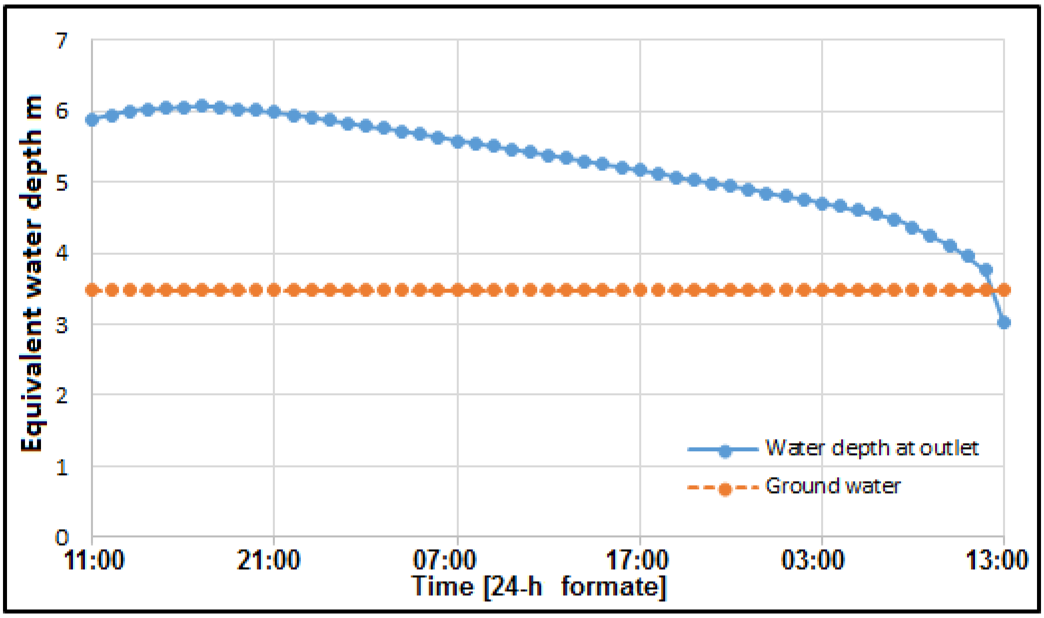

Fig. 4.10:The equivalent water depth in the outlet manhole (M187) with rainfall intensity 17.5 mm/h

The equivalent water depth in the outlet manhole (M187) under rainfall intensity of 17.5 mm/h, where the maximum equivalent water depth reached to 6.03 m above the invert level and it increased by 1.7 times comparing with the design intensity (3.6 m). From this comparing, one can imagine the percent of the flooding and the damage to the infrastructure. Also, the load that the system is exposed to as a result to climate Variation. Figure 4.11 shows the volume of water flooding under rainfall intensity of 17.5 mm/h.

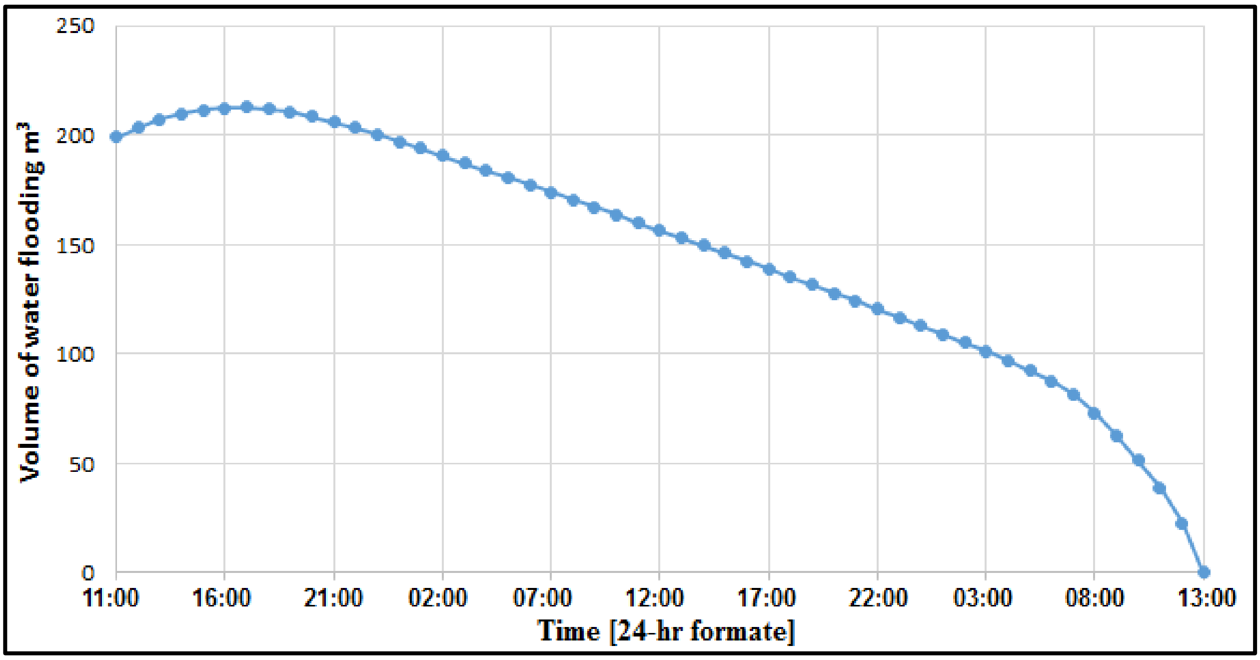

Fig. 4.11: Flooding volume in the outlet (M187) with rainfall intensity 17.5 mm/h

Under rainfall intensity of 17.5 mm/h, the maximum volume of water flooding was 212.76 m^3 comparing with 6 m^3 under the design intensity (increased by 35.5 times). The flooded area around the outlet was 851 m^2 under 17.5 mm/h.

4.3.1.4 Results of Climate Variation by SWMM under Rainfall Intensity of 33.5 mm/hr

Figure 4.12 shows the behavior of the storm network under the maximum rainfall intensity happened in 11/5/2015 with rainfall intensity of 33.54 mm/h for one hour.

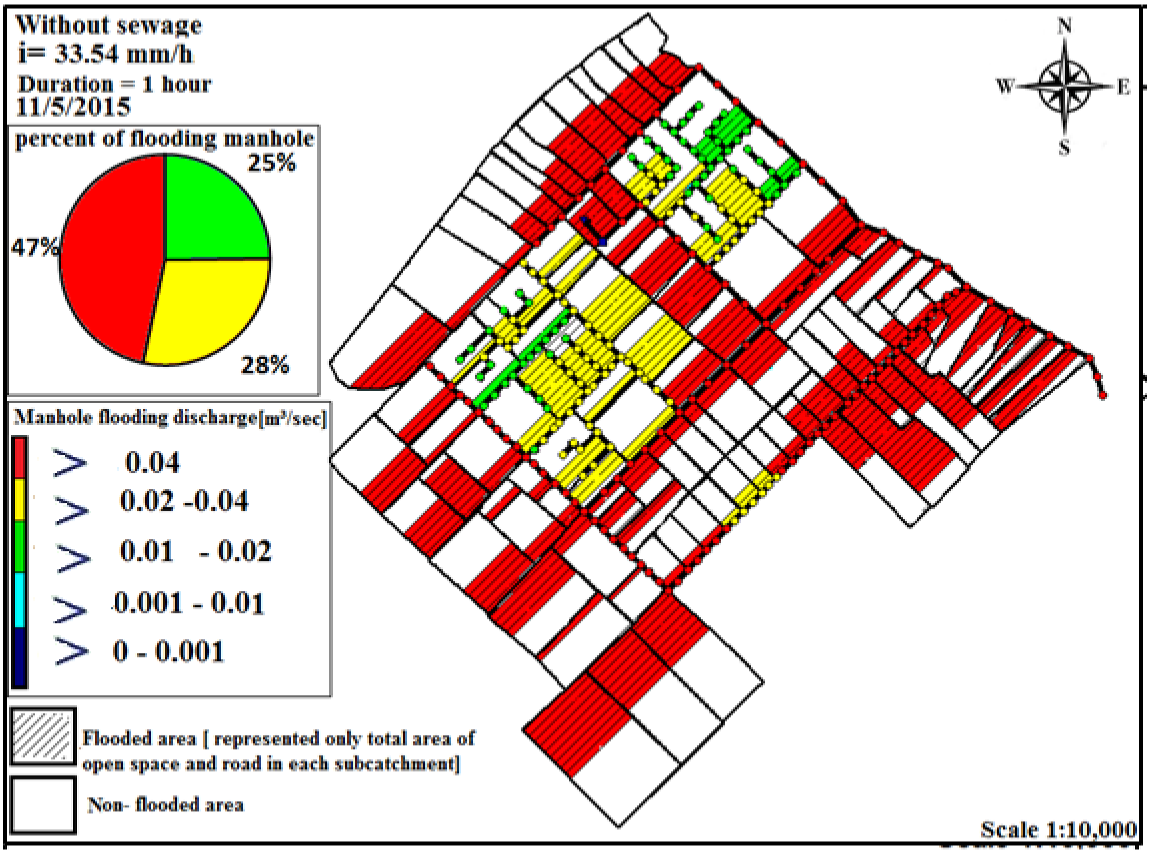

Fig. 4.12:The flooding manholes under design intensity [33.54mm/h] at peak time

Under the maximum rainfall intensity there is 47% of the flooding manholes was within stage5 (greater than 0.04 m^3/sec) (very high flooding rate),28% was within stage4 (greater than 0.02 to 0.04 m^3/sec) and 25% was within stage3 (greater than 0.01 to 0.02 m^3/sec) (medium flooding rate). The total duration of the flooding was 43 hours.

Figure 4.13 displays the equivalent water depth above the outlet manhole (M187) during the rainfall intensity of 33.54 mm/h.

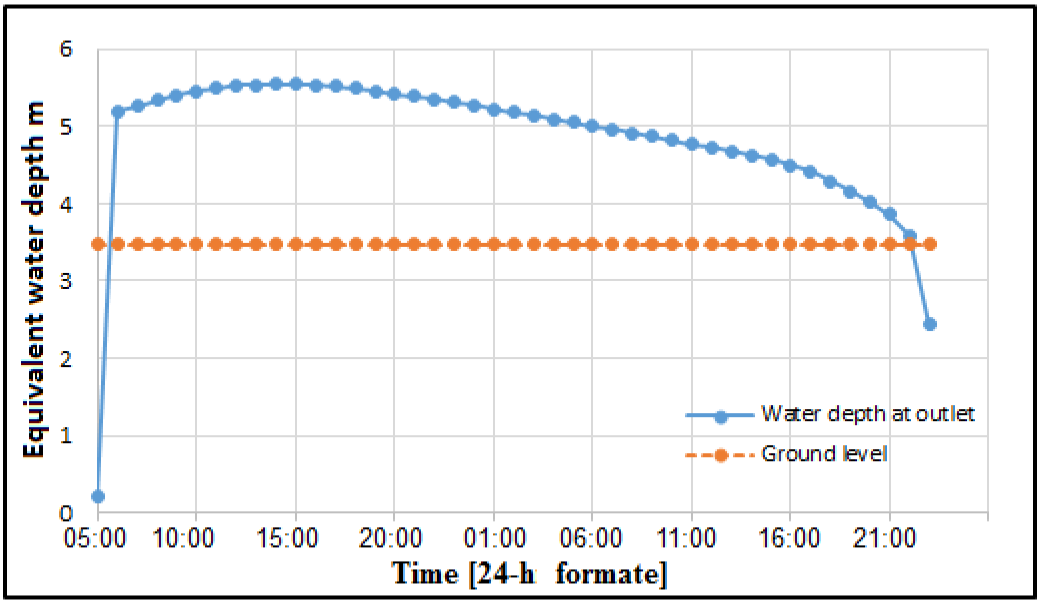

Fig. 4.13:The equivalent water depth in the outlet manhole (M187) with intensity 33.54mm/h

The maximum equivalent water depth under the rainfall intensity (33.54 m/hr) was 5.55 m that equals to 1.5 times comparing to the water level at the design intensity. Figure 4.14 displays the volume of water flooding in intensity of 33.54 mm/h.

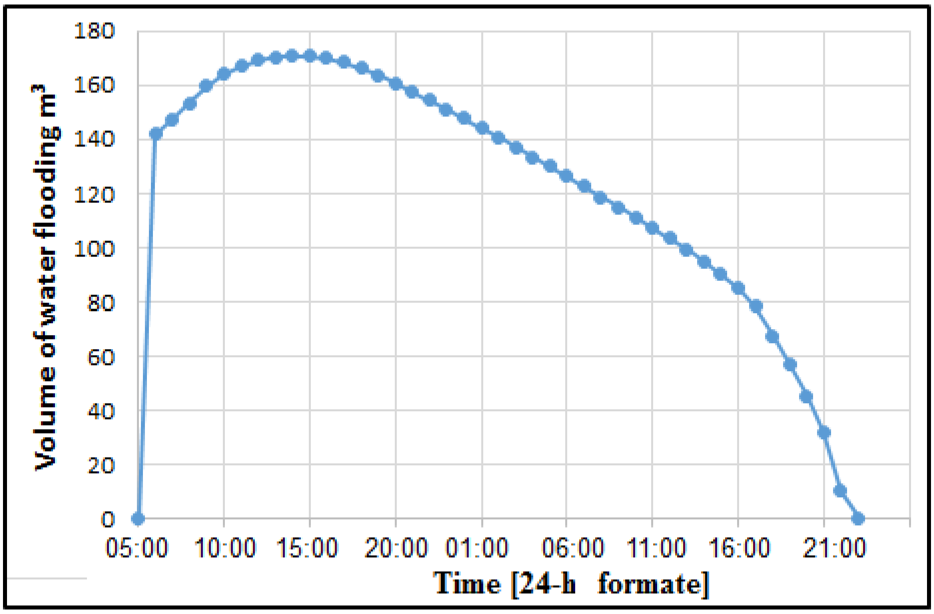

Fig. 4.14:The flooding volume in the outlet manhole (M187) with intensity 33.54mm/h

maximum volume of water flooding was 170.79 m^3. It is equivalent to 28.5 times the maximum volume of water flooding under design intensity of 6 m^3. The total flooding area was 683.2 m^2.

Other results of flooding events under different rainfall intensity without cosidering the sewage quantity was insert in Appendix A from figure A-1 to A-20.

4.4 Effect of Sewage Quantity

Beside the climate variation effect on the discharge capacity of the storm drainage system, the residents in the study area decrease the capacity of the storm system by discharge the sewage amount through it. The main reason of discharging the sewage through the storm system was the lack of the study area to sanitary system.

Figure 4.15 illustrates the observed quantity of sewage in the outlet during two months (January and February) of 2015 (KSD., 2015). These two months had not got any rainfall events.

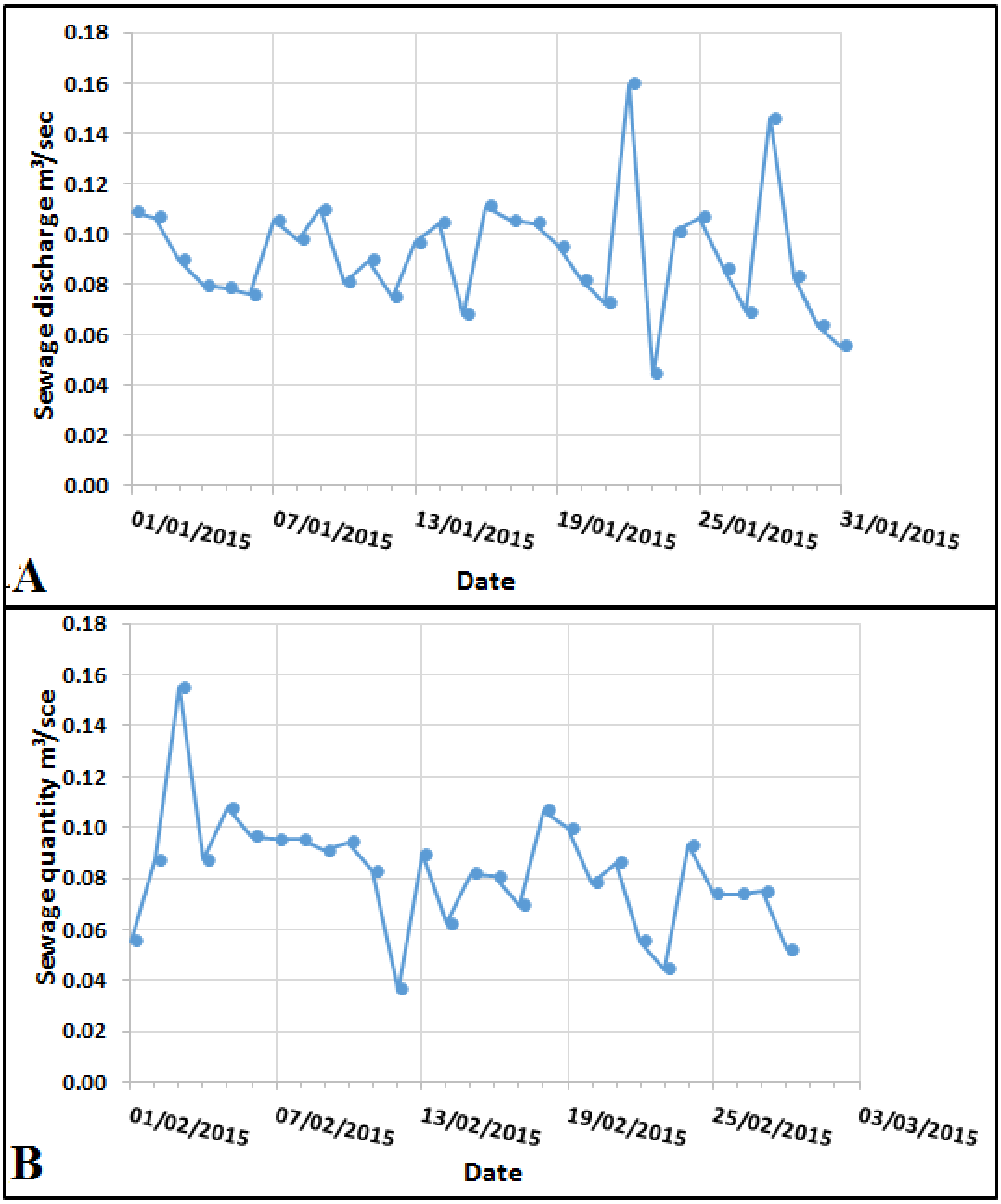

Fig. 4.15: Sewage quantity in outlet of two months (A-January ,B-February) 2015

a variation was notice in the sewage quantity during days and also between the two months. The monthly average of sewage discharge for January was 0.091 m^3/sec and for February was 0.082 m^3/sec . The results of SWMM when the sewage quantity exist will analyses and evaluate the effect of sewage by comparing the performance of the system with and without the sewage exist.

4.4.1 Results of the Sewage Effect by SWMM under Rainfall Intensity of 8.6 mm/hr

Figure 4.16 shows the performance of the system with rainfall intensity lower than the design intensity.

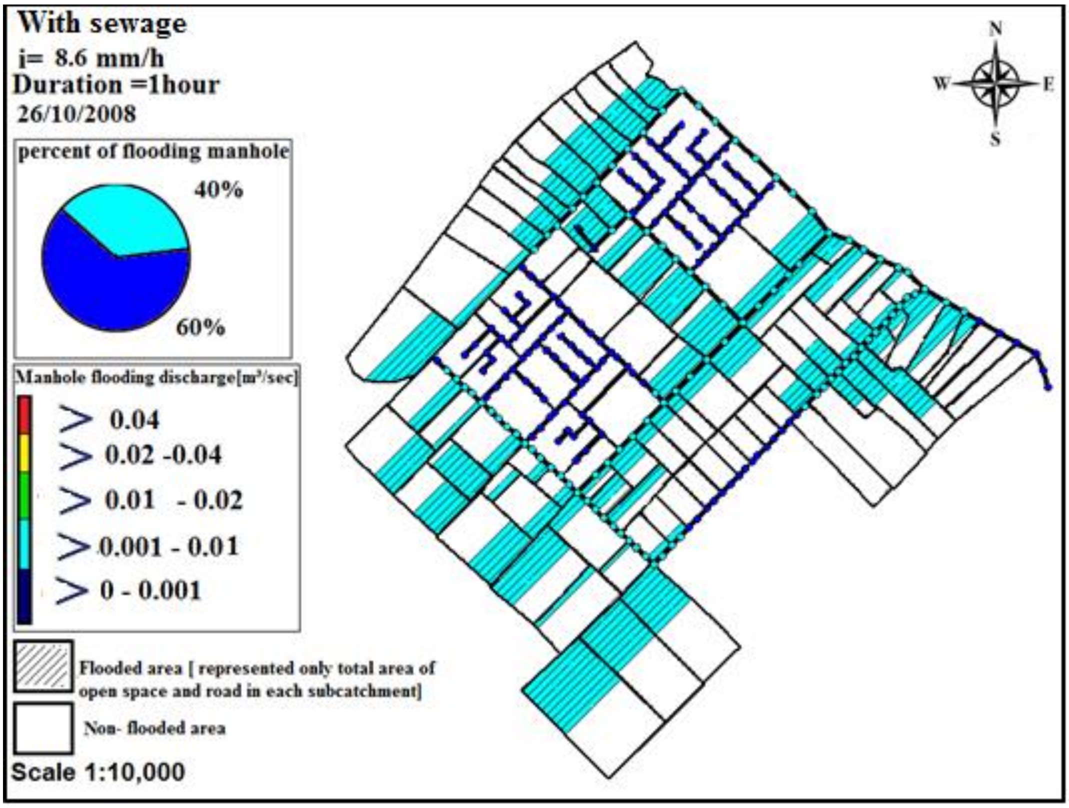

Fig. 4.16: Performance of storm drainage with sewage under 8.6 mm/h

From Figure 4.16, it is obvious that the effect of the sewage exists once on the system under rainfall intensity lower than the design intensity. The percent of flooding manholes was within stage1 was 60% and the flooding manholes within stage2 were 40%. The total duration of the flooding was 6-hours.

Figure 4.17 states that the equivalent water depth at the outlet in the two case (with and without sewage) under rainfall intensity of 8.6 mm/h for 1-hour.

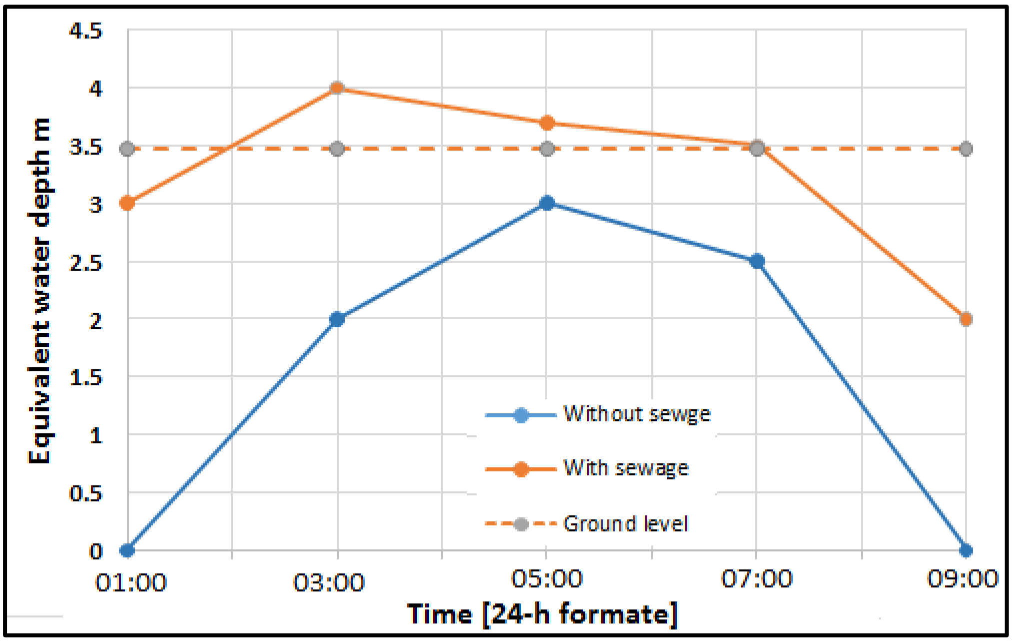

Fig. 4.17: Equivalent water depth at (M187)(with and without sewage) with intensity 8.6mm/h

Figure 4.17 shows that there was a difference in the peak equivalent water depth where as in the case of no sewage, the peak equivalent water depth was 3 m. On the other hand, in case of sewage existences the peak equivalent water depth was 4 m.

Figure 4.18 illustrates the volume of flooding water in the outlet manhole in the two cases (with and without sewage).

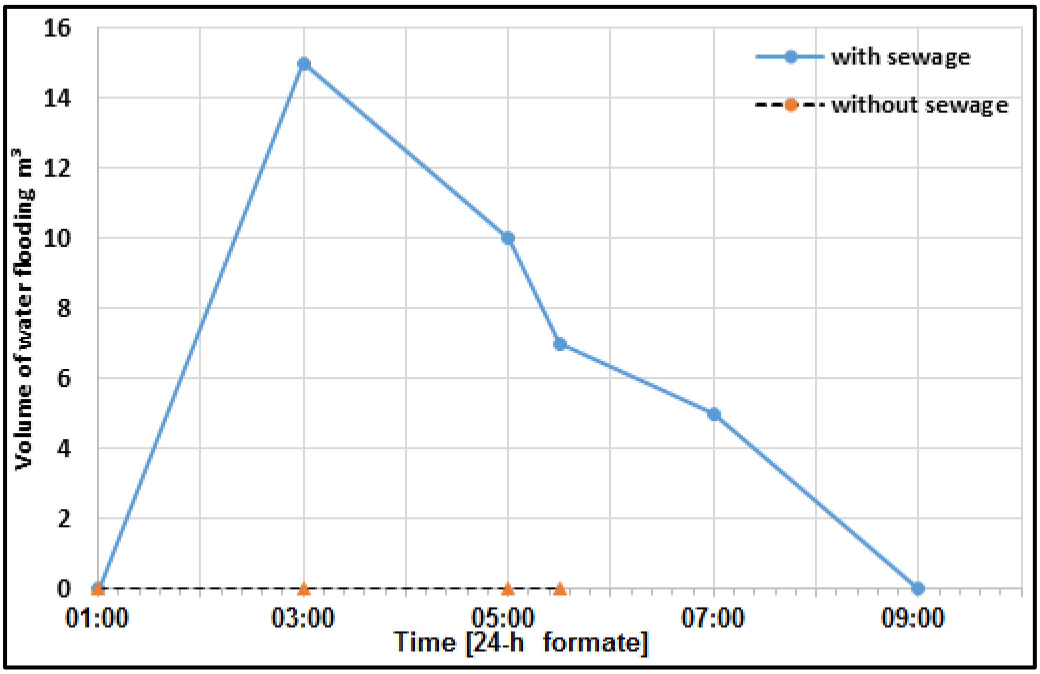

Fig. 4.18:Flooding volume in (M187) (with and without sewage) with intensity 8.6mm/h

the peak volume of water flooding with sewage existence was 15 m^3 due to the pipes clogging and the second case was without sewage with 0 m^3 because the rainfall intensity was lower than the design intensity. The flooding area around the outlet manhole during the peak period was 60 m^2.

4.4.2 Results of Climate Variation by SWMM under Design Rainfall Intensity 9.6 mm/hr

Figure 4.19 illustrates the behavior of the storm drainage system under the design rainfall intensity of 9.6 mm/h for 1-hour.

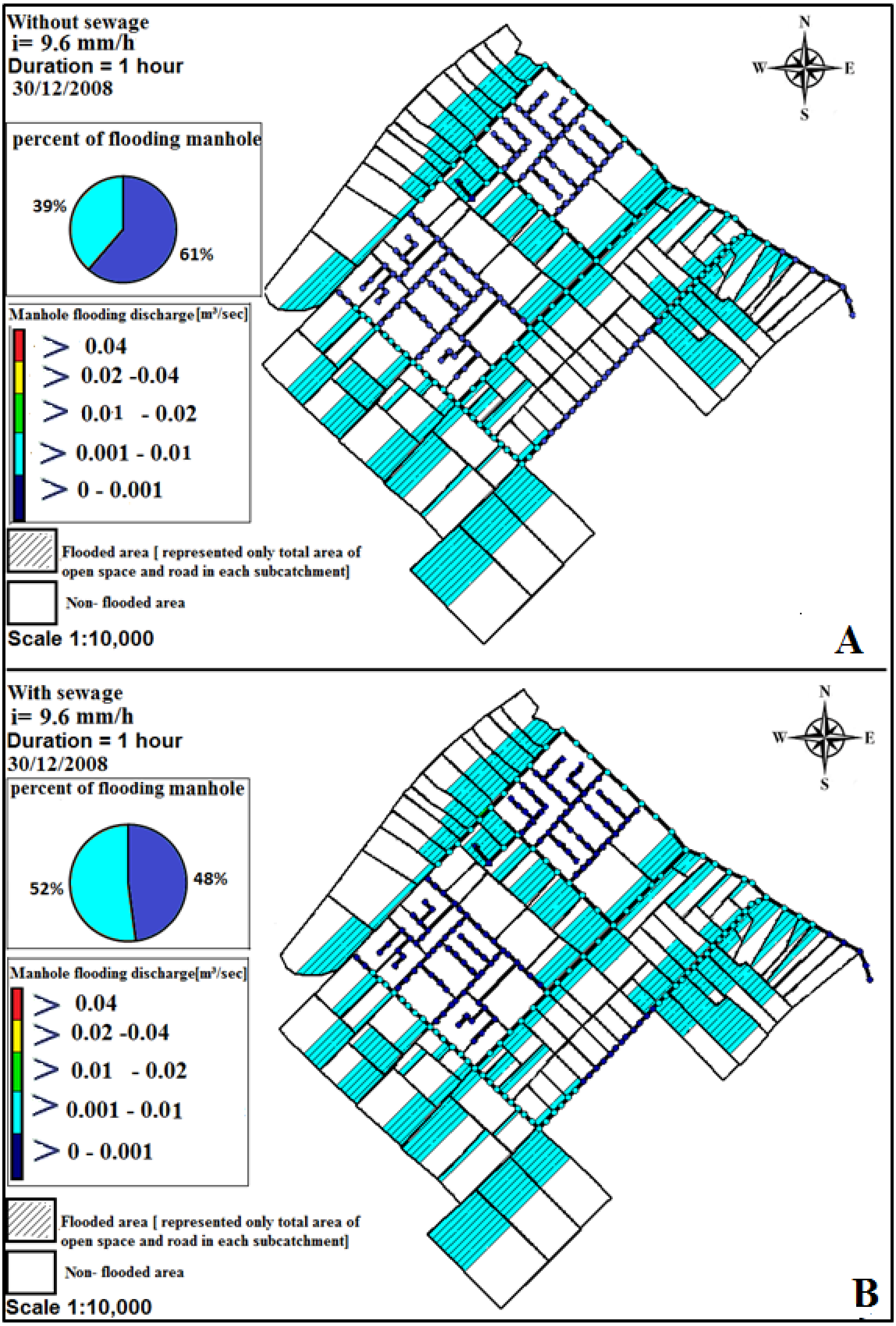

Fig. 4.19: Performance of storm drainage(A-without,B-with) sewage under 9.6 mm/h

The performance of the storm system under the design intensity of 9.6 mm/h for 1-hour.It is clear from Figure 4.19 (A) 61% of the flooding manholes was within stage1 (from 0 to 0.001 m^3/sec)(no flooding) and 39% was within stage2 (greater than 0.001 to 0.01 m^3/sec)(very light flooding rate).While in Figure 4.19 (B), the percent of flooding discharge within stage1 was 48% (the flooding manholes increase by 21 % comparing with the case of without sewage). Also 52% of the flooding manholes flood was within stage2. Figure 4.20 shows the equivalent water depth for the two cases (with and without sewage) at the outlet manhole (M187). The total duration of flooding in case of no sewage was half-hour and in case of sewage existence was 7-hours.

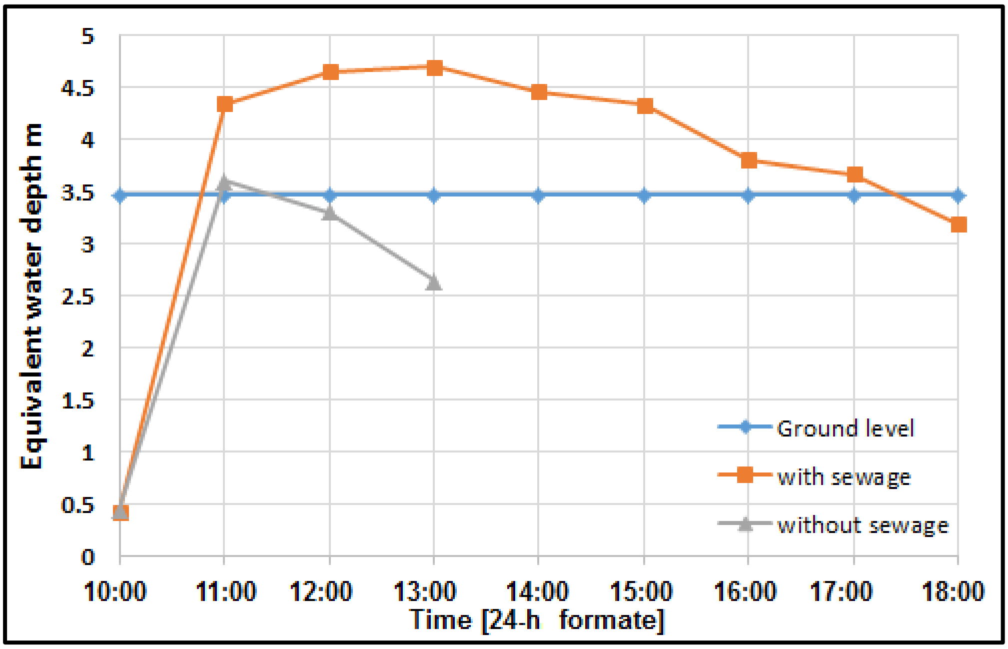

Fig. 4.20: Equivalent water depth at (M187)(with and without sewage) with intensity 9.6mm/h

The difference in the equivalent water depth at the outlet manhole between the two cases under rainfall intensity of 9.6 mm/h. In case of no sewage, the maximum equivalent water depth reached 3.6 m and in case of sewage existence, the peak equivalent water depth was 4.7 m. The percent of difference of equivalent water depth under design intensity and in two case was 23 %.

Figure 4.21 shows the volume of water flooding under rainfall intensity of 9.6 mm/h.

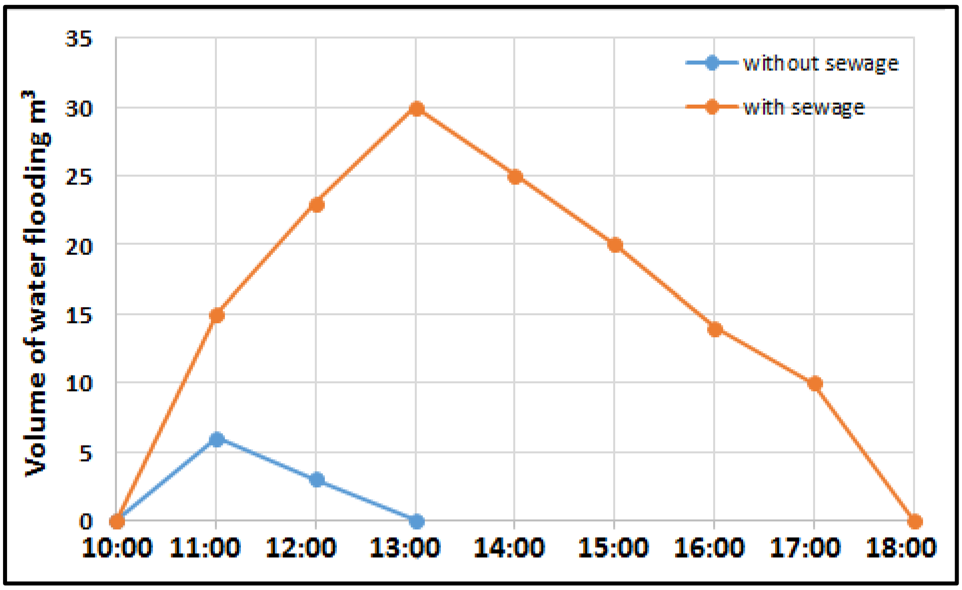

Fig. 4.21:Flooding volume in (M187) (with and without sewage) with intensity 9.6mm/h

It is clear that the maximum volume of water flooding reached to 6 m^3 in case of no sewage and reached to 30 m^3 in case of sewage existence. The flooding area was 120 m^2.

4.4.3 Results of Climate Variation by SWMM under Design Rainfall Intensity 17.8 mm/hr

Figure 4.22 displays the performance of the storm system under rainfall intensity of 17.8 mm/h for 1-hours.

Fig. 4.22 Performance of storm drainage(A-without sewage,B-with sewage) under 17.8 mm/h

The difference in the percent of flooding manholes in case of no sewage and with sewage was low, which is due to the quantity runoff was the demonstrated and the effect of sewage as a quantity was small and have a greater effect in the suffocation of pipes. In Figure 4.22(A) 44% of the flooding manholes was within stage4 (greater than 0.02 to 0.04 m^3/sec) comparing with 48% in Figure 4.22B) ,30% was within stage3 (greater than 0.01 to 0.02 m^3/sec) comparing with 28% in Figure 4.22 (B) and 26% of the flooding manholes was within stage2 (greater than 0.001 to 0.01 m^3/sec) (very light flooding) comparing with 24% in case of existence the sewage quantity. The total duration of flooding under rainfall intensity of 17.8 mm/h for 1-hour without sewage was 21 hours and in case of sewage existence was 50 hours. The high difference in the flooding duration belongs to the suffocation.

Figure 4.23 illustrates the equivalent water depth at the outlet manhole in the two case (with and without sewage).

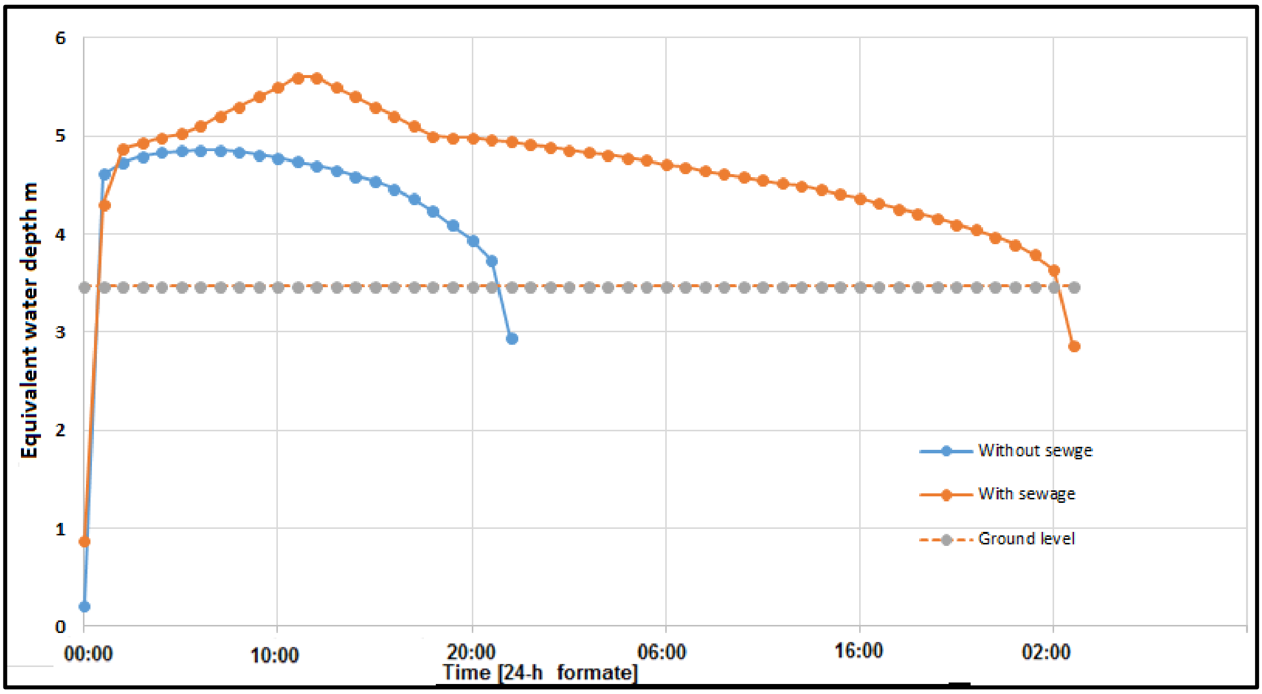

Fig. 4.23: Equivalent water depth in (M187)(with and without sewage) under 17.8 mm/h

The maximum equivalent water depth under rainfall intensity of 17.8 mm/h for 1-hour in case of no sewage was 4.86 m and in case of sewage existence was

5.6 m. Figure 4.24 shows the volume of water flooding under the rainfall intensity of 17.8 mm/h in two cases (with and without sewage).

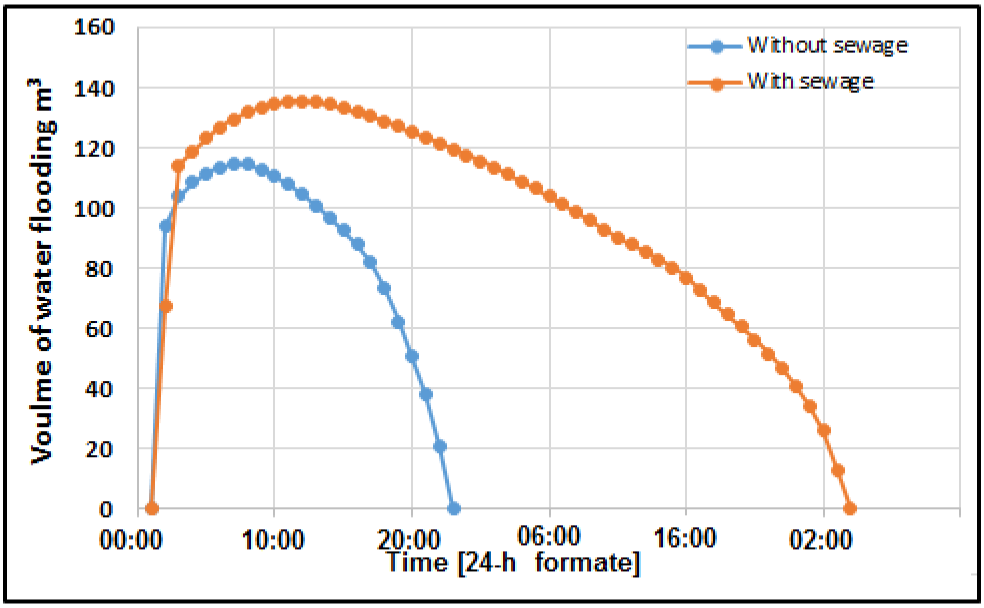

Fig. 4.24:Flooding volume in (M187) (with and without sewage) under 17.8 mm/h

The maximum volume of water flooding reach to 135.55 m^3 in case of sewage exist and reached to 114.53 m^3 in case of no sewage. The volume of flooding increased to 15.5% when the sewage exists under rainfall intensity of 17.8 mm/h (the storm network under the effect of sewage and climate variation). The flooding area around the outlet was 542 m^2.

4.4.4 Results of Climate Variation by SWMM under Design Rainfall Intensity 33.5 mm/hr

Figure 4.25 illustrates the performance of the storm system under the maximum rainfall intensity occurs in the study area (33.54 mm/h) for 1-hour.

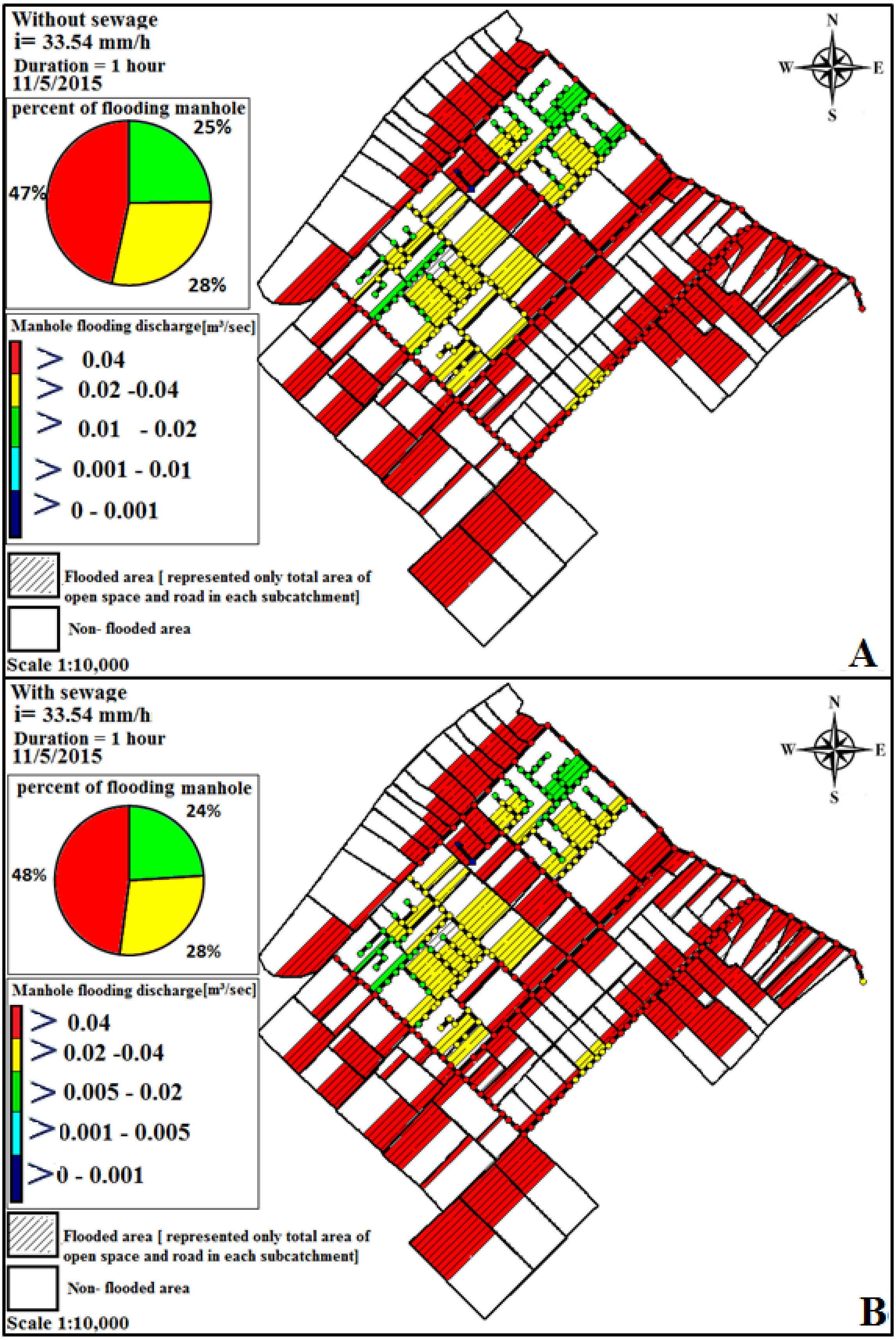

Fig. 4.25: Performance of storm drainage(A-without sewage,B-with sewage) under 33.54 mm/h

The difference in the percent of flooding manholes was very small, whereas in case of no sewage, 47% of manholes flooding was within stage 5 (very high flooding rate) and in case of sewage existence, the percent was 48% (increase only 1%) .Also, 28% of manholes flooding was within stage 4(high flooding rate) in both cases. Moreover, 25% and 24% of manholes flooding within stage 3 (medium flooding rate) from case of no sewage and with sewage respectively. This percent give for the first view that the sewage had not affected on the performance of the storm system under the high rainfall intensity. The difference in the duration of the flooding shown the high effect of exsit of sewage quantity .The total flooding duration was 43.5 in case of no sewage and 72 hours in case of sewage.

Figure 4.26 displays the equivalent water depth at the outlet under the rainfall intensity of 33.54 mm/h .

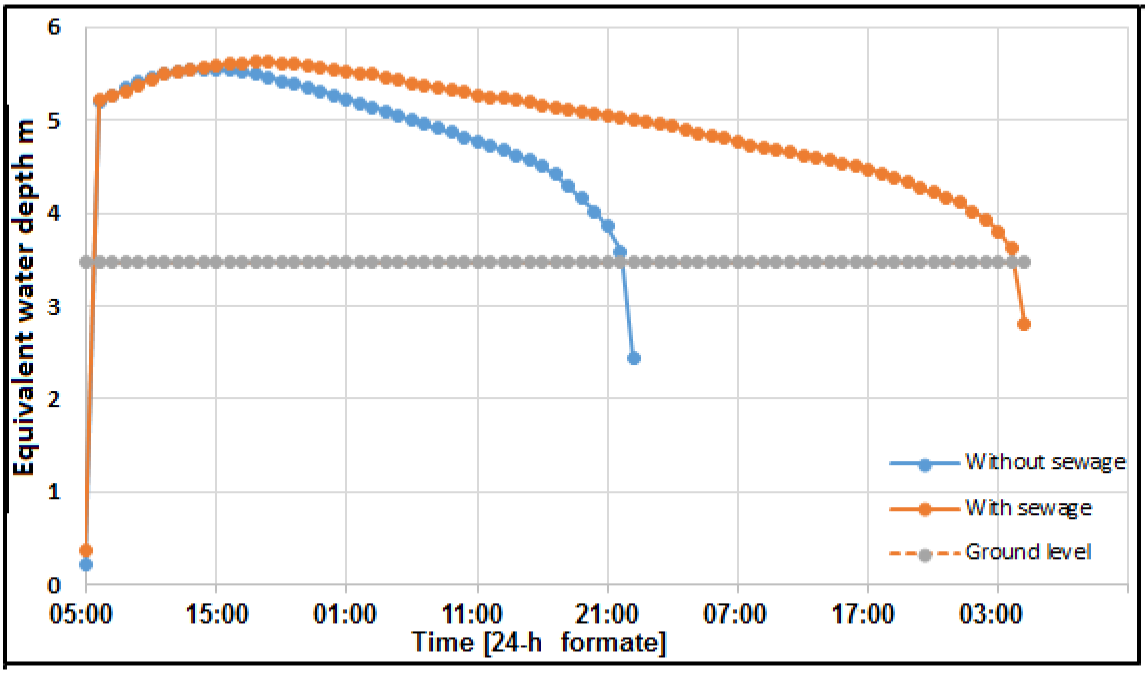

Fig. 4.26 Equivalent water depth in the (M187)(with and without sewage) under 33.54 mm/h

The maximum equivalent water depth under rainfall intensity of 33.54 mm/h in case of sewage existence was 5.62 m and in case of no sewage, it reached to 5.55 m .

Figure 4.27 illustrates the maximum volume of water flooding under rainfall intensity of 33.54 mm/h.

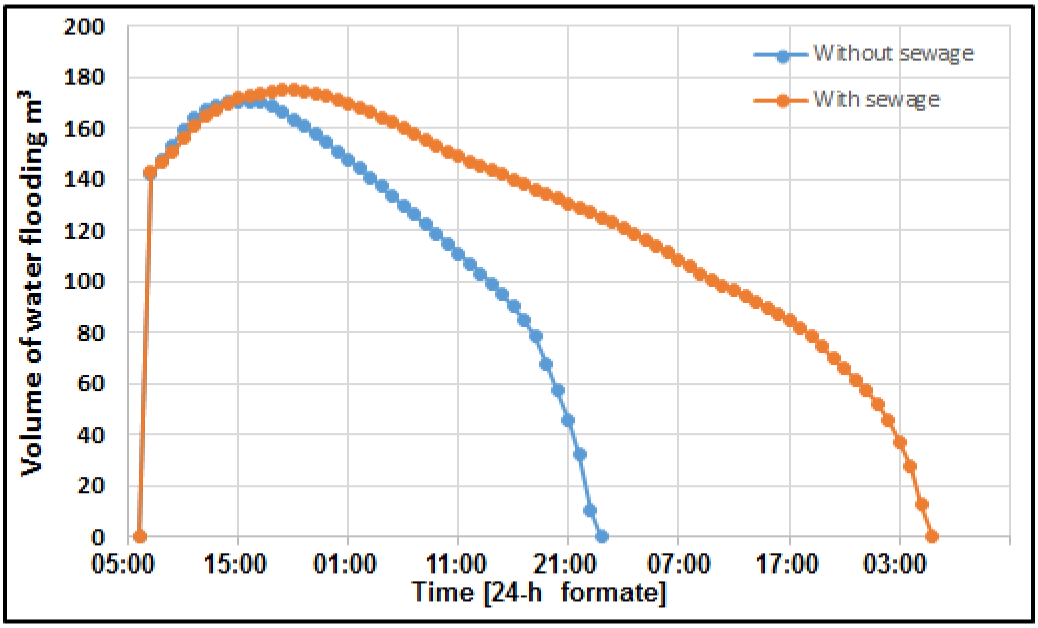

Fig. 4.27:Flooding volume in (M187) (with and without sewage) under 33.54 mm/h

The maximum volume of flooding in the outlet manhole was 175.12 m^3 in case of sewage existence and 170.79 m^3 in case of no sewage.The difference between the two case was 3% under the rainfall intensity of 33.54 mm/h.The flooding area under rainfall intensity of 33.54 mm/h was 700 m^2.As a result, the existence of sewage accompanied with the climate variation increases the duration required to the storm system to discharge all the flooding water to the double.

Other results of flooding events under different rainfall intensity with cosidering the sewage quantity was insert in Appendix A from figure A-21 to A-23.

4.5 Evaluation a Proposal Solution

The flooding events that occured in the study area (Al-Eskari quarter) have a direct and indirect losses. The direct losses related to the damage in infrastructure and roads, while indirect losses related to the restriction of the traffic movement, delayed business and pollution. For all this reasons, it is important to provide solution to mitigate the effect of flooding damage. In this section, one of proposed solution was the evalution by using SWMM. This solution is carried out by adding another pipeline transmission. The evalution of this solution using SWMM will provide a technical support before it is implemented on the ground. Figure 4.28 states the proposal pipeline.

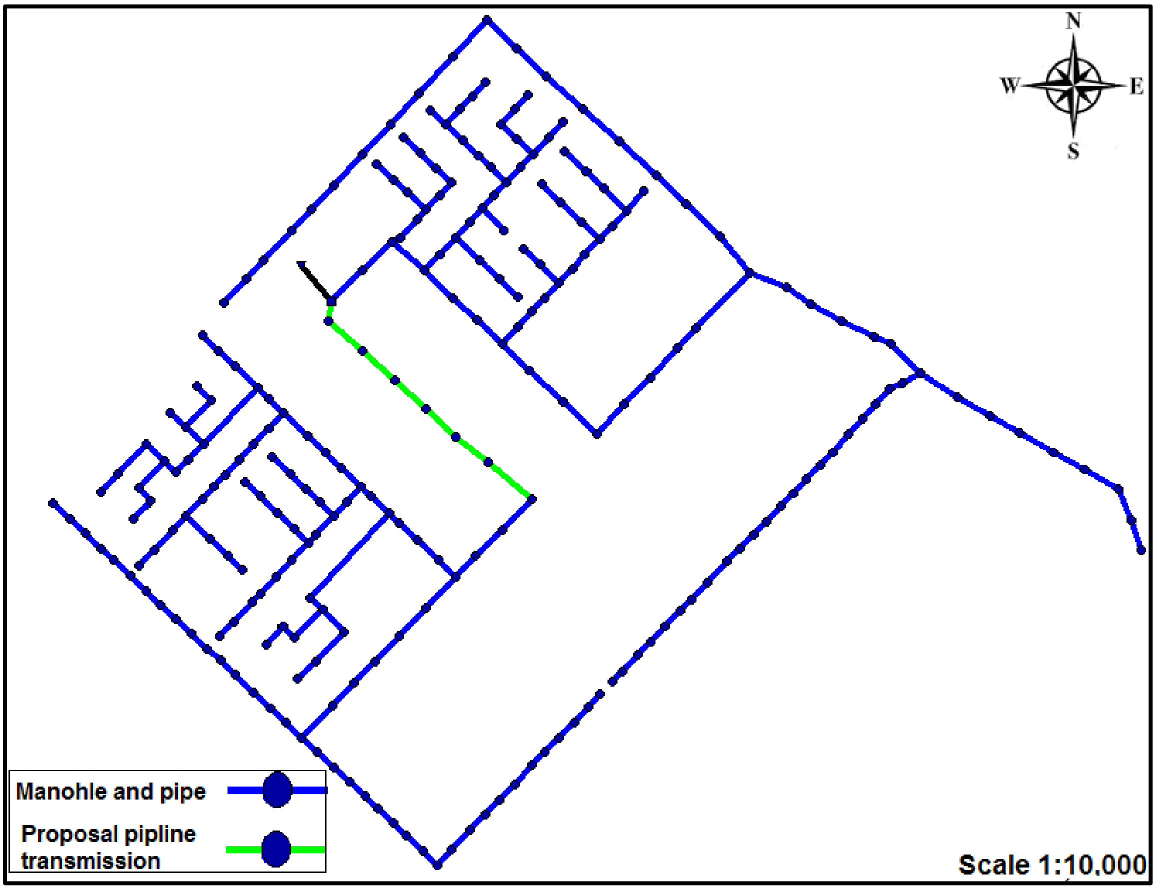

Fig. 4.28:The proposal pipeline transmission

The new pipeline transmission have a length of 344m and a diameter of 900 mm.The new pipeline is extended in unpaved area. The effect of this new pipeline will be shown under the rainfall intensity happened in the study area and with considering the sewage quantity.

4.5.1 Results of Adding a Mitigation Solution by SWMM under Design Rainfall Intensity 9.6 mm/hr

Figure 4.29 shows the performance of the strom network under rainfall intensity 9.6mm/h in two cases (A-current case , B-under new pipe transmission).

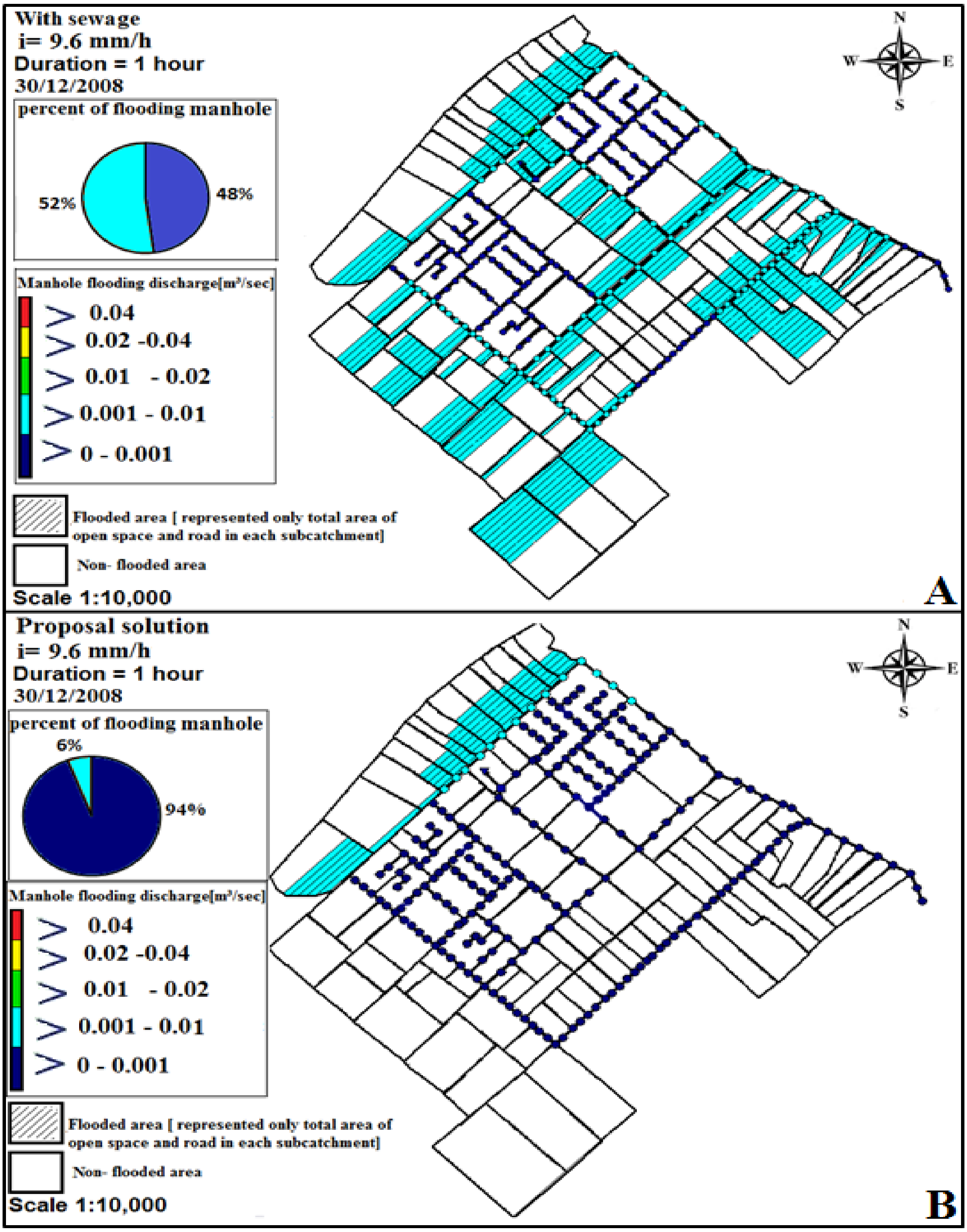

Fig. 4.29: Performance of storm drainage (A-with sewage,B-proposal solution) under 9.6 mm/h

After adding the new pipeline, the percent of flooding manholes in stage2 degreases from 52% to only 6% and the no flooding manholes increases from 48% to 94%. The total flooding duration decreased due to the new pipeline from 7-hours to 3-hours.

Figure 4.30 illustrates the maximum equivalent water depth that reached in some selected manholes under rainfall intensity of 9.6 mm/h in two cases (A- current case , B- after add the new pipeline transmission).

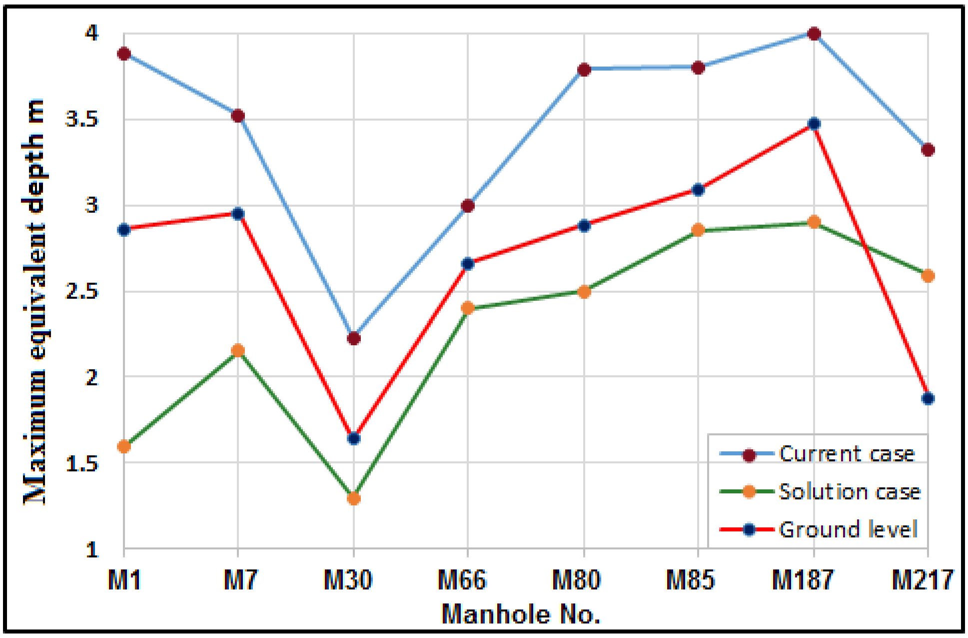

Fig. 4.30: Maximum equivalent water depth in selected manholes (current case+proposal solution) under 9.6 mm/h

The difference in the maximum equivalent water depth in some selected manholes,in which in manhole (M217), the maximum equivalent water depth decreased from 3.32 m to 2.6 m (decrease by around 22%) and so on for other manholes . This difference was due to the adding of the new pipeline proposal.

Figure 4.31 displays the maximum volume of water flooding.

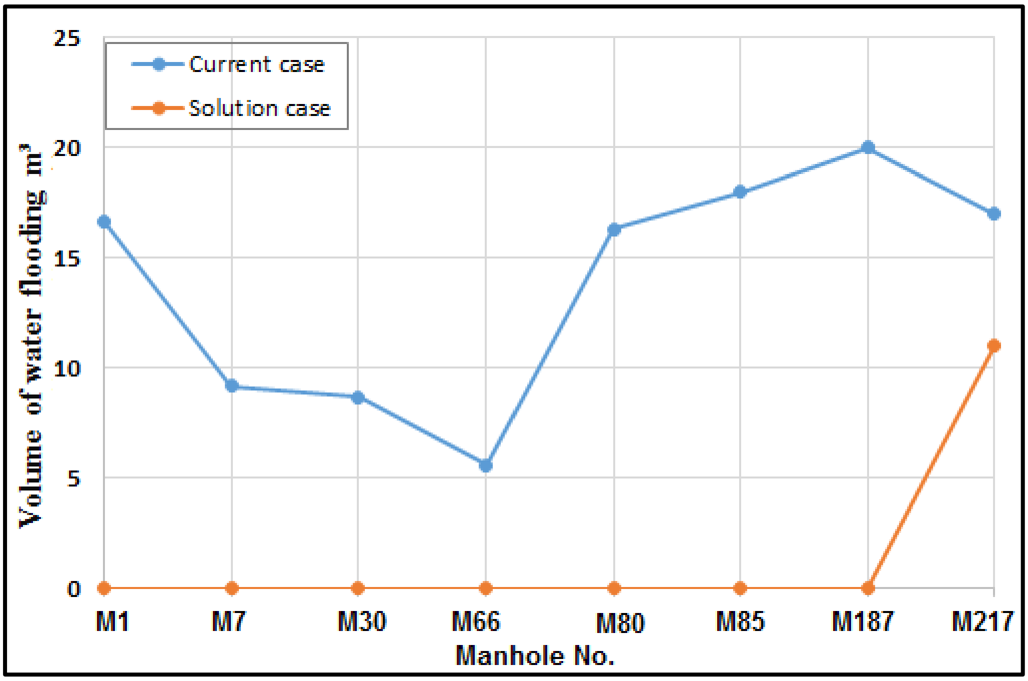

Fig. 4.31:Volume of water flooding in a selected manholes (current case+proposal solution) under 9.6 mm/h

In case of the proposal solution, there is no flooding water in the manholes. In manhole (M217), the volume decreased from 17 m^3 to 11 m^3.

4.5.2 Results of Adding a Mitigation Solution by SWMM under Design Rainfall Intensity 17.8 mm/hr

Figure 4.32 illustrates the performance of the storm system under rainfall intensity of 17.8 mm/h for 1-hour under the current situation and in case of adding the new pipeline.

Fig. 4.32: Performance of strom drainage (A-with sewage,B-proposal solution) under 17.8 mm/h

It is very clear the effect of the new pipeline in mitigation the effect of flooding.When comparing Figure 4.32 (A) and 4.32 (B) it can be found that stage4 decreased from 48% to only 6% and most of the monholes became flooding with stage 3 and 2 with 41% and 53% respectively. The total duration of flooding under rainfall intensity of 17.8 mm/h for 1-hour in case of sewage existence was 50 hours and in case of new proposal pipeline was 10 hours.

Figure 4.33 shows the difference in maximum equivalent water depth in some of manholes in the two cases (current and new pipeline).

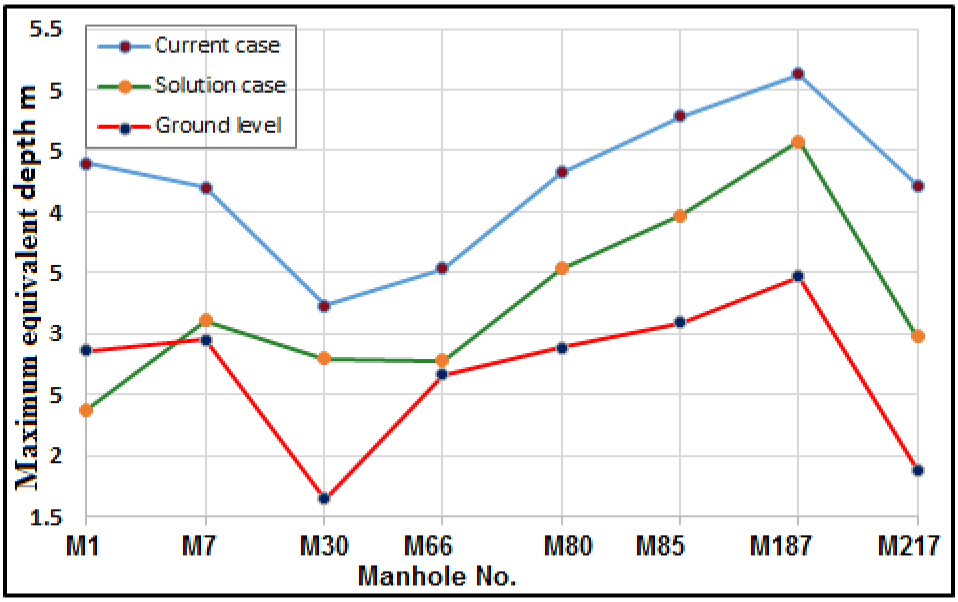

Fig. 4.33: Maximum equivalent water depth in some selected manholes (current case+proposal solution) under 17.8 mm/h

The high decrease in the peak maximum equivalent water depth at the selected manholes due to the new pipeline transmission solution. In manhole (M1), the equivalent water depth reduced from 4.4 m to 2.4 m, as well as in manhole (M7), in which the equivalent water depth decreased from 4.2m to 3.1m and so on for all other manholes.

Figure 4.34 shows the difference in volume of water flooding in some of manholes in the two cases (current and new pipeline).

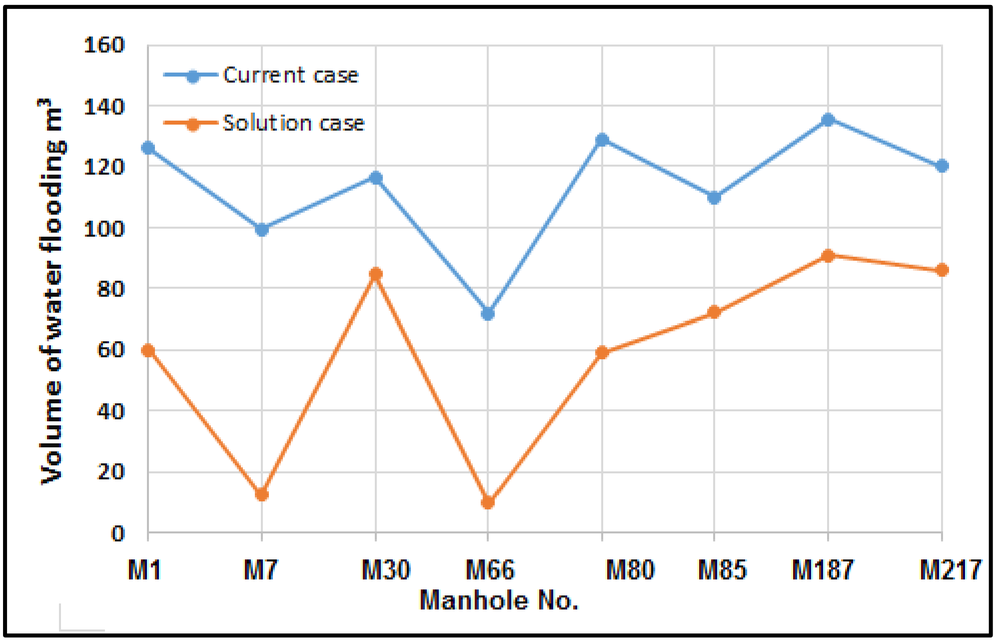

Fig. 4.34:Volume of water flooding in some selected manholes (current case+proposal solution) under 17.8 mm/h

the maximum volume of water flooding under rainfall intensity of 17.8 mm/h in the two cases (current case and new pipeline). The Figure shows a difference after try the new pipeline where in manhole (M1), the volume of water flooding decreased from 126.14 to 60 m^3 and as so on for all other manholes.

4.5.3 Results of Adding a Mitigation Solution by SWMM under Design Rainfall Intensity 33.5 mm/hr

Figure 4.35 illustrates the performance of the storm drinage network under the maximum rainfall intensity of 33.54 mm/h in two cases (current and new pipeline cases).

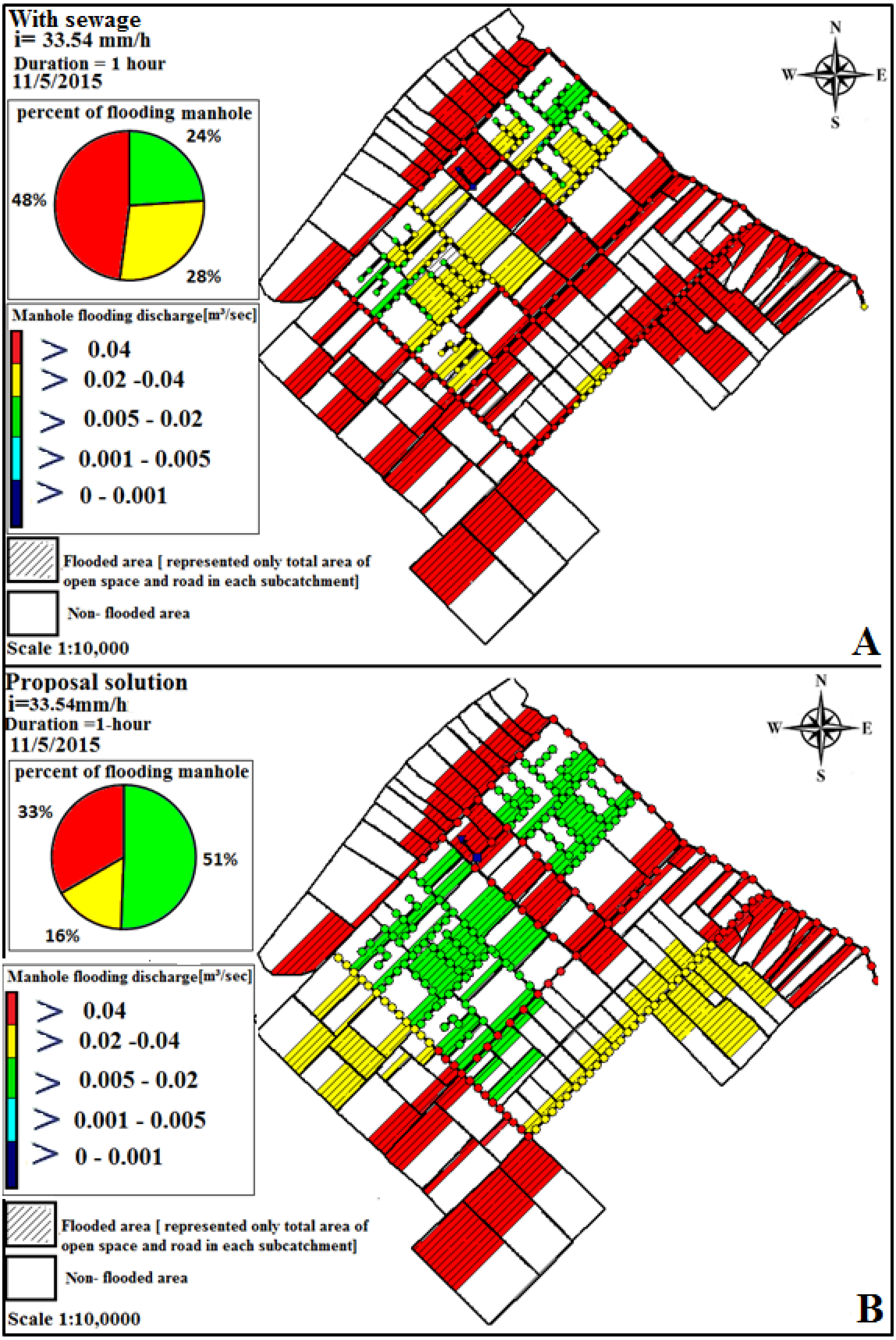

Fig. 4.35:Performance of storm drainage (A-with sewage,B-proposal solution) under 33.54 mm/h

The new pipeline, the flooding within stage5 decreases from 48 to 33% , stage4 decreased from 28 to 16% and 51% of the manholes in case of new pipeline flooding within stage3 (medium flooding rate). The total flooding duration was 72 hours in case of sewage existence and 20 hours in case of new proposal pipeline.

Figure 4.36 states the maximum equivalent water depth under rainfall intensity of 33.54 mm/h for some selected manholes in the two cases (current and new pipeline case).

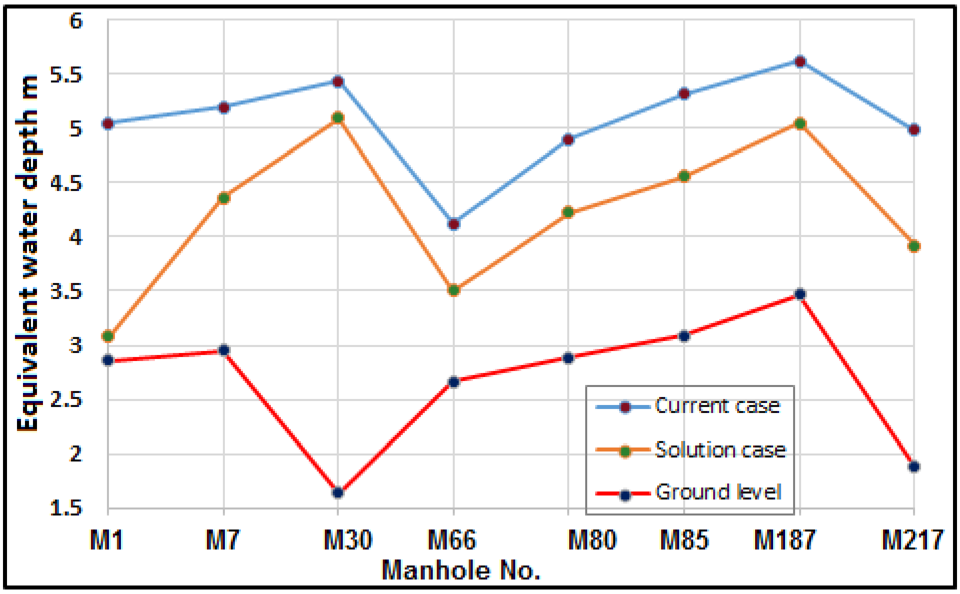

Fig. 4.36: Maximum equivalent water depth in some selected manholes (current case+proposal solution) under 33.54 mm/h

The high decrease in the peak equivalent water depth at the selected manholes due to the new pipeline transmission solution. In manhole (M1), the equivalent water depth reduced from 5 to 3.1 m ,as well as in manhole (M7), the equivalent water depth decrease from 5.2 to 4.4 m and so on for all other manholes.

Figure 4.37 shows the difference in volume of water flooding in some of manholes in the two cases [current and new pipeline cases].

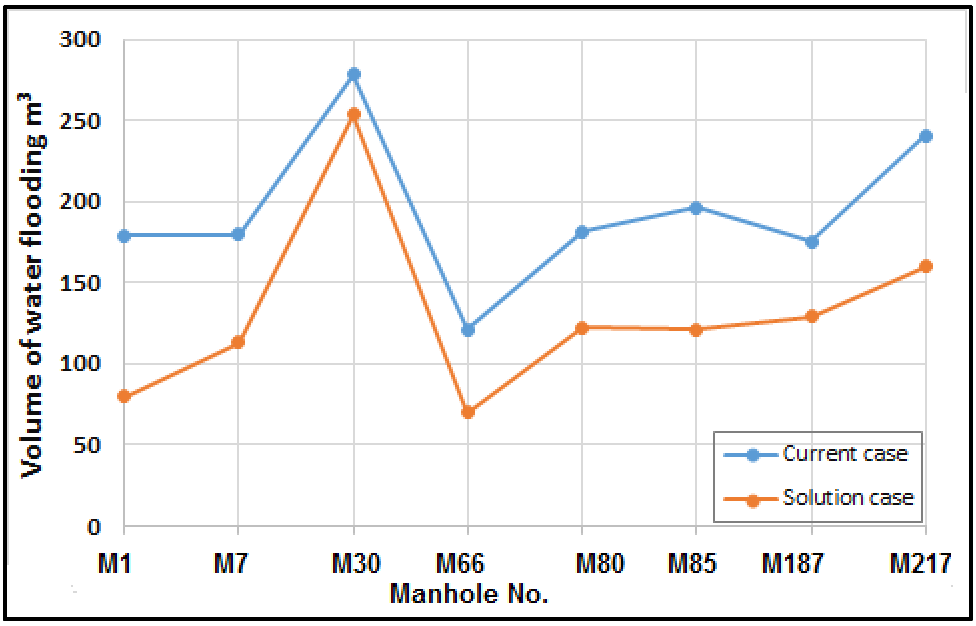

Fig. 4.37:Volume of water flooding in some selected manholes (current case+proposal solution) under 33.54 mm/h

The Figure shows a difference after trying the new pipeline where in manhole (M1), the volume of water flooding decreased from 179.24 to 80 m^3 and as so on for all other manholes as shown in Figure.

Other results of flooding events under different rainfall intensity with cosidering the sewage quantity was insert in Appendix A from figure A-23 to A-26.

Tables (4.2,4.3 and 4.4) summarized the results of all the three cases, which has been contained in this chapter.

Table 4.2: The summarization of the results in case of the climate variation effect only

Under the climate change only	Depth of outlet [M187]	Peak rainfall intensity mm/hr	Duration of the storm hour	Percent of flooding manholes					Max. water volume flooding m³ at outlet manhole [M187]	Max. depth of equivalent water flooding at the outlet manhole [M187] m	Total flooding duration Hour	Flooded area m²
				Stage 1 %	Stage 2 %	Stage 3 %	Stage 4 %	Stage 5 %				
	3.47 m	9.6	1	61	39	0	0	0	6	3.6	0.5	24
		11.3	18	0	43	53	4	0	248.82	6.5	45	995
		17.5	1	0	23	29	48	0	212.76	6	50	851
		17	2	0	29	27	44	0	123.54	4.97	28	494
		17.8	1	0	26	30	44	0	114.53	4.86	23	458
		15.8	1	0	39	29	32	0	104.5	4.74	20	418
		12.8	1	1	47	45	7	0	97.82	4.66	19	391
		33.54	1	0	0	25	28	47	170.79	5.55	43	683.16
		6.9	2	0	93	7	0	0	99	4.68	19	396
		24.5	9	0	0	43	29	28	364	7.9	110	1456

Tableخطأ! لا يوجد نص من النمط المعين في المستند..3: The summarization of the results in case of the sewage exist effect on the system

The sewage effect on the system	Peak rainfall intensity mm/h	Duration of the storm Hour	Percent of flooding manholes					Max. water volume flooding m³ at outlet manhole [M187]	Max. depth of water flooding at the outlet manhole [M187] measured from the invert of [M187] m	Total flooding duration Hour	Flooding area m²
			Stage 1 %	Stage 2 %	Stage 3 %	Stage 4 %	Stage 5 %				
	8.6	1	60	40	0	0	0	15	4	6	60
	9.6	1	48	52	0	0	0	30	4.7	7	120
	12.8	1	1	45	41	13	0	105	5.3	23	420
	17.8	1	0	24	28	48	0	135.55	5.6	50	542
	33.5	1	0	0	24	28	48	175.12	5.62	72	700.48

Table[خطأ! لا يوجد نص من النمط المعين في المستند.4]: The summarization of the results in case of the proposal

	Peak rainfall intensity mm/h	Duration of the storm	Percent of flooding manholes					Manhole no.	Max. water volume flooding m³ without solution	Max. water volume flooding m³ with solution	Max. depth of water flooding at the outlet without solution m	Max. depth of water flooding with solution m	Total flooding duration without solution at the outlet Hour	Total flooding duration with solution at the outlet Hour	Flooding area without solution	Flooding area with solution
			Stage 1 %	Stage 2 %	Stage 3 %	Stage 4 %	Stage 5 %									
The effect of the proposal solution	9.6	1	94	6	0	0	0	M1	16.616	0	3.88	1.594	7	3	66.464	0
								M7	9.166	0	3.52	2.152			36.664	0
								M30	8.662	0	2.23	1.3			34.648	0
								M66	5.576	0	3	2.4			22.304	0
								M80	16.3	0	3.79	2.5			65.2	0
								M85	18	0	3.8	2.85			72	0
								M187	20	0	4	2.9			80	0
								M217	17	11.03	3.32	2.591			68	44.12
	12.8	1	77	9	14	0	0	M1	71.84	0	3.74	1.994	12.5	5	287.36	0
								M7	39.49	0	3.44	2.648			157.96	0
								M30	60.5	37.3	2.47	2.148			242	149.2
								M66	14.3	0	2.83	2.378			57.2	0
								M80	66.84	23.37	3.63	3.138			267.36	93.48
								M85	93.07	43.29	4.15	3.618			372.28	173.16
								M187	105	67.53	5.3	4.295			420	270.12
								M217	90.08	49.61	3.04	2.513			360.32	198.44
	17.8	1	0	53	41	6	0	M1	126.14	60	4.4	2.371	50	10	504.56	240
								M7	99.48	12.38	4.2	3.105			397.92	49.52
								M30	116.42	84.57	3.23	2.792			465.68	338.28
								M66	71.73	9.71	3.53	2.777			286.92	38.84
								M80	128.97	58.92	4.32	3.531			515.88	235.68
								M85	110	72.04	4.78	3.968			440	288.16
								M187	135.55	90.75	5.13	4.578			542.2	363
								M217	120	85.81	4.22	2.975			480	343.24
	33.5	1	0	0	51	16	33	M1	179.24	80	5.05	3.08	70	20	716.96	320
								M7	179.61	113.29	5.2	4.364			718.44	453.16
								M30	278.41	253.46	5.44	5.093			1113.64	1013.84
								M66	121.04	70.03	4.12	3.501			484.16	280.12
								M80	181.25	122.02	4.9	4.228			725	488.08
								M85	196.56	120.87	5.32	4.563			786.24	483.48
								M187	175.12	129.2	5.62	5.048			700.48	516.8
								M217	240.86	159.96	4.98	3.922			963.44	639.84

4.6 The Damage Cost the Infrastructure due to the Flooding

The intensive flooding events and with long duration had affected negatively on the infrastructure, especially roads. AASHTO (1993) specified the failure that happened in the flexible pavements which may cause or accelerate by moisture in the pavements structure. This failure could be stripping, rutting, depressions, fatigue and potholes.

A field survey has been done to specify the damage in roads that may happen as a results of flooding in streets that are exposed to longest flooding duration. Figure 4.38 illustrates the location of the streets in the study area that are exposed to the heaviest flooding as a results to the system clogging and climate variation.

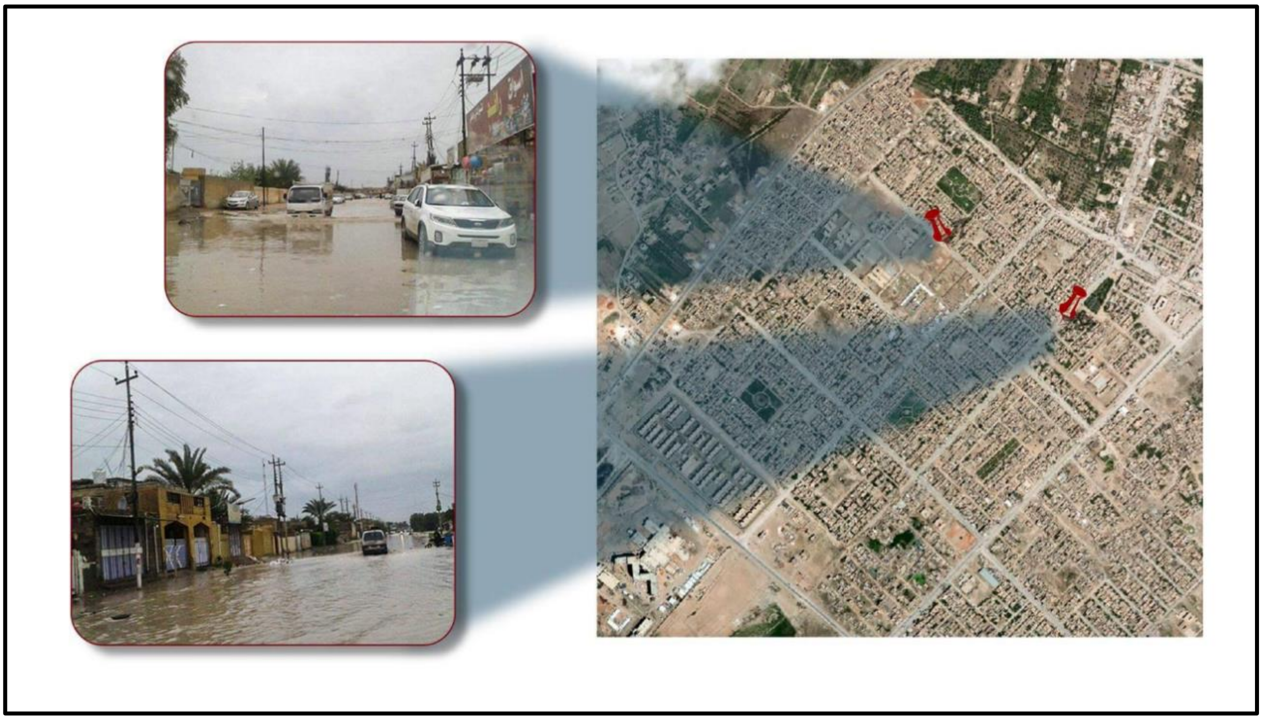

Fig. 4.38:The streets that has the most flooding events in duration

From the survey,it has been found that the most type of failure that happened to the streets and flooding of the storm drinage system was the stripping failure. Figure 4.39 shows the failure in the roads.

Fig. 4.39:The stripping failure in the streets that has the most flooding events in duration

From the field survey and according to (Shahin.,1994), the stripping failure is the intensive type which (Shahin.,1994) described this type of failure as follow "aggregate or binder has been worn away considerably. The surface texture is very rough and severely pitted. The pitted areas are less than 10 mm in diameter and less than 13 mm in deep. The larger pitted areas are counted as potholes".

4.6.1 Calculation of the Damage Cost

According to Shahin , this type of failure was treated by overlay, where an asphalt layer of 5cm put along all the roads and the cost of each square meter was 18000 ID, which equivalent to 14 $.

To calculate the damage cost used the (Kok., 2001) formula:-

[DAMAGE = p * A * H * V]

p= % of urban covered surface in land use=5.6%

A= area (m^2) of the land use=110 ha

H= water depth =0.25

V= average price for m^2 for a house=18000 ID

Damage = 0.056%*1100000*0.25*18000= 277200000 ID

The area of the road that have a strriping damage due to the flooding was 0.0062 km^2. This damage cost 277200000 ID for maintenous of the total area of the road that have flooding for very long duration.

4.7 Summary

Storm Water Management Model [SWMM] were used to simulate the storm drainage network in Al-Eskari quarter, Karbala, Iraq. The aim of this simulation was to detect the reasons that led to flooding of the storm system, specify the amount of the damage that happened to the infrastructure in the study area and evaluate a solution to mitigate the flooding in the study area. The SWMM simulation results indicates that the main reasons of the flooding in the study area were the climate variation and the illegal adding of the sewage quantity to the storm drainage system.

The values of the infrastructure damage were concentrated in the roads. These damages cost totally around 277200000 ID to be maintained only. Providing a solution to mitigate the flooding of the storm drainage network was an essential purpose. The adding of a transmission pipeline with 900mm diameter give good results and mitigate the flooding to considerable level. This pipeline shows a good result also under the maximum rainfall intensity and with the existence of the sewage amount.

The SWMM model that developed in this study can also be utilized to estimate the fluctuations in sewer parameters due to the growth of residents, variations in water use due to life-style alterations, and variations in precipitation caused by the climate variation.

Chapter Five
Conclusions and Future Recommendations

5.1 Conclusion

The conclusions have been made based on the results reported previously. They can be summarized as follow: -

1. The study area have a sandy clay soil and the depth of the ground water level was 0.9 m ,the total number the population in the study area was 19584 and the water consumption for each capita was 422 l/day.
2. SWMM model was effective in the analysis of the flooding problem of the storm drainage network of the study area. The calibration of the SWMM model for two rainfall event with rainfall intensity of 2 mm/h and 4 mm/h was good, the cross validation results showed that the mean error (ME) for the two events are very close to zero (0.0068) and (0.0032) respectively. The coefficient of determination [R^2] for the first rainfall event was 0.95 and for second one was 0.94.
3. The analysis of the storm drainage network by SWMM model under the design rainfall intensity of 9.6 mm/h and without considering the illegal sewage quantity indicated that the system was effective in drainage the rainfall quantity when the flooding duration continuous for half hour.
4. The simulation for the storm drainage system faced a rainfall intensity with a peak rainfall intensity of 11.3 mm/h continuous for 18 hours happened in 19/11/2013 and without considering the illegal sewage quantity indicated that 53% of the system's manholes flooding with discharge from 0.01 to 0.02 m^3/sec. The flooding duration under this rainfall intensity was 45 hours.
5. The SWMM model analysis for the storm drainage system faced a rainfall intensity with a peak rainfall intensity 17 mm/h for 1 hours happened in 29/11/2013 and without considering the illegal sewage quantity indicated that 44% of the system's manholes flooding with discharge of from 0.02 to 0.04 m^3/sec. The flooding duration under this rainfall intensity was 28 hours.

6. The analysis for an event with rainfall intensity with a peak rainfall intensity 33.54 mm/h for 1 hours happened in 11/5/2015 and this intensity was the maximum intensity that happened in the study area during the period from 2008 to 2016 and without considering the illegal sewage quantity indicated that 47% of the system's manholes flooding with discharge greater than 0.04 m^3/sec. The flooding duration under this rainfall intensity of 33.54 mm/h was 43 hours.
7. The illegal sewage quantity that drainage through the storm system increases the flooding problem in the study area. The system become flooding under a rainfall intensity lower than the design intensity. The SWMM model analysis for the rainfall intensity of 1 hour and peak rainfall intensity 8.6 mm/h and with considering the illegal sewage quantity indicated that 40% of the system's manholes flooding with discharge from 0.001 to 0.01 m^3/sec. The flooding duration under this rainfall intensity 8.6 mm/h was 6 hours.
8. About 52% of the system's manholes flooding with discharge from 0.001 to 0.01 m^3/sec under the design rainfall intensity of 9.6 mm/h due to the exist the illegal sewage quantity. The flooding duration under the design intensity increased from half hour to 7 hours due to the illegal sewage quantity.
9. The considering of the illegal sewage quantity with higher rainfall intensity as 33.54 mm/h for 1 hour indicated that the sewage did not effect on the quantity on the flooding discharge in the two case (with and without sewage) where around 48% of the system's manholes was flooded with discharge (greater than 0.04) m^3/sec. On the other hand, the flooding duration under rainfall intensity of 33.54 mm/h and due to the sewage quantity existence increased from 43 hours in case of no sewage to 72 hours in case of the illegal sewage.
10. After studying the flooding under all the condition and during the rainfall intensity from 2008 to 2016, several solutions have been suggested to mitigate the flooding intensity and duration in the study area. From these solutions, the best one has been chosen which represents the balanced between the cost of construction and the

maximum percentage reduction of flood quantity and duration. This solution is by providing a new pipeline with 344 m in length and 900 mm in diameter.

11. The SWMM model analysis for the design rainfall intensity of 1 hours and peak rainfall intensity 9.6 mm/h and with considering the illegal sewage quantity and the new solution indicated that the system's manholes flooding with discharge from 0.001 to 0.01 m^3/sec decreases from 52% in current case (the illegal sewage existence) to 6% under the new solution. The flooding duration also decreased from 7 to 3 hours.
12. The evaluation for the maximum rainfall intensity of 1 hours and peak rainfall intensity 33.54 mm/h and with considering the illegal sewage quantity and the new solution by SWMM indicated that the system's manholes flooding with discharge (greater than 0.04) m^3/sec decreased from 48% in current case (the illegal sewage existence) to 33% under the new solution. The flooding duration also degrease from 72 to 20 hours.
13. The flooding was an assistant factor in the damage of the infrastructure especially roads and houses' walls. The field survey indicated that the roads exposed to intensive striping as a direct or indirect effect from flooding events. The cost of repairing the damage was 277200000 ID .

5.2 Future Recommendations

1. Detect the environmental damage that may happened from the flooding of the storm network as it has been investigated in this study.
2. Detect new mitigate solution to reduce the effect of the flooding such as impervious pavement, increase the percent of gardens and use an artificial storage tanks. These solutions are of low cost, effective and considering a new approach in flooding mitigation in Iraq.
3. Try a new case study in the region.
4. Use other numerical models to analyze events.

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Appendix (A)

Results of the Flooding under Different Rainfall Intensity and Cases

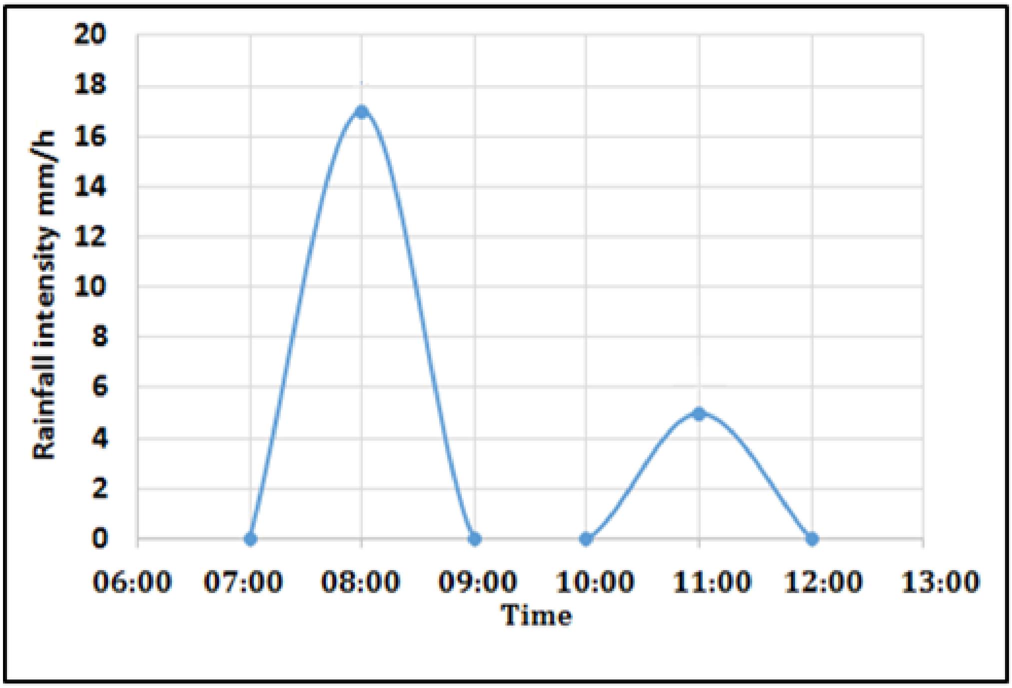

Fig. A.1:The storm intensity at 29/11/2013

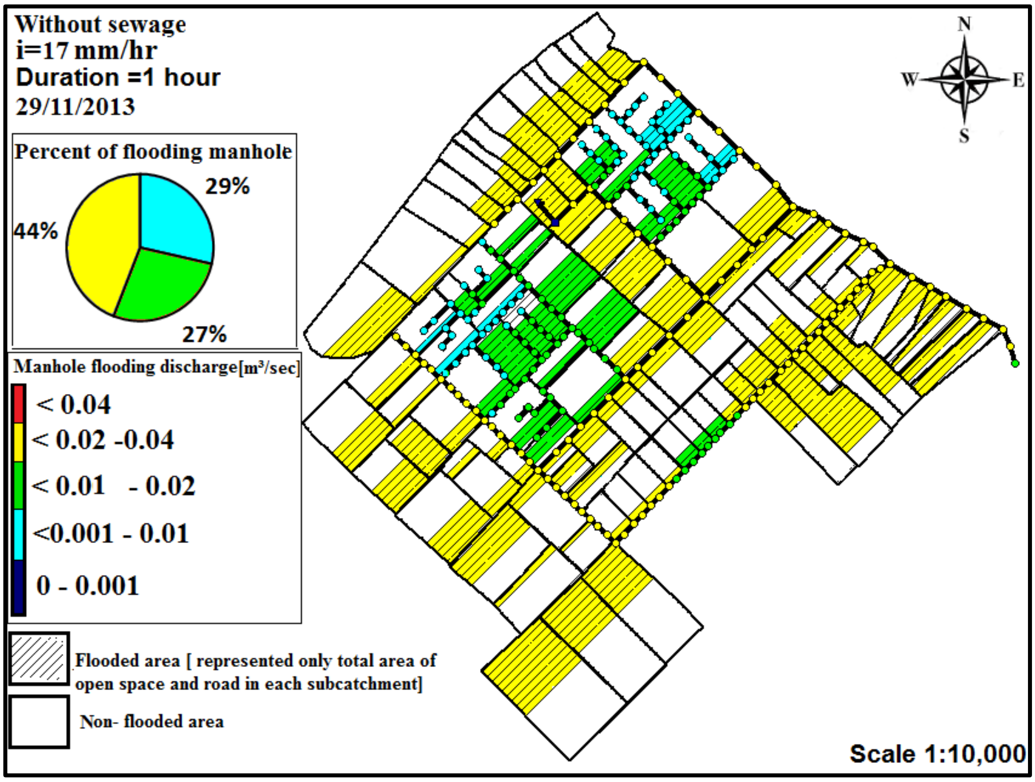

Fig. A.2:The flooding manholes under a peak intensity of 17mm/hr at peak time

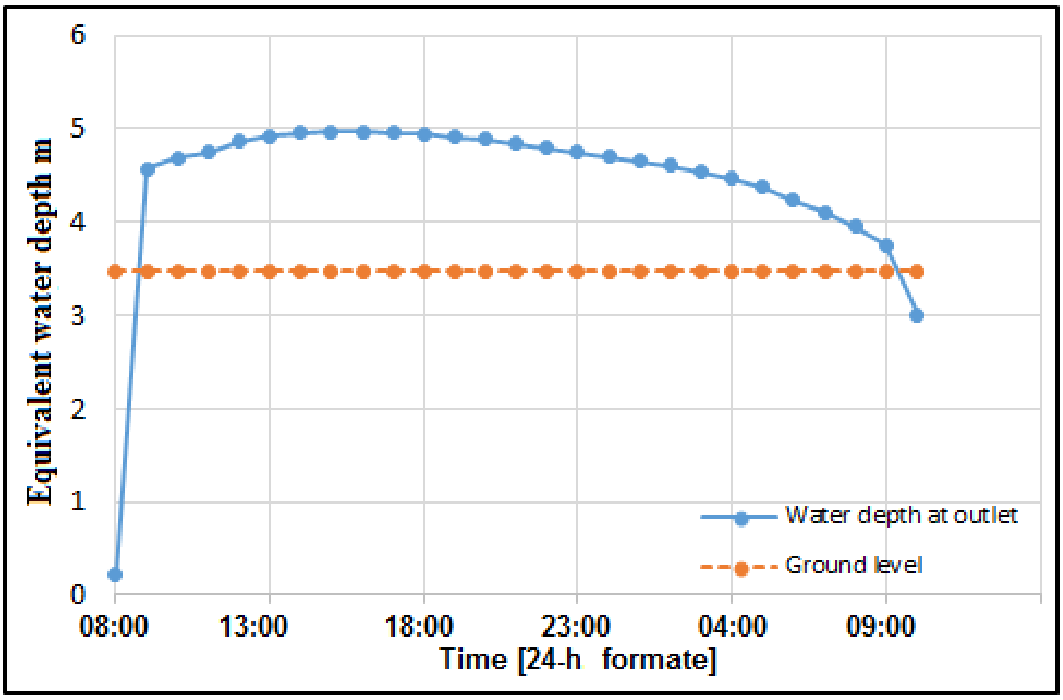

Fig. A.3:The water depth in outlet manhole (M187) with rainfall intensity 17 mm/hr

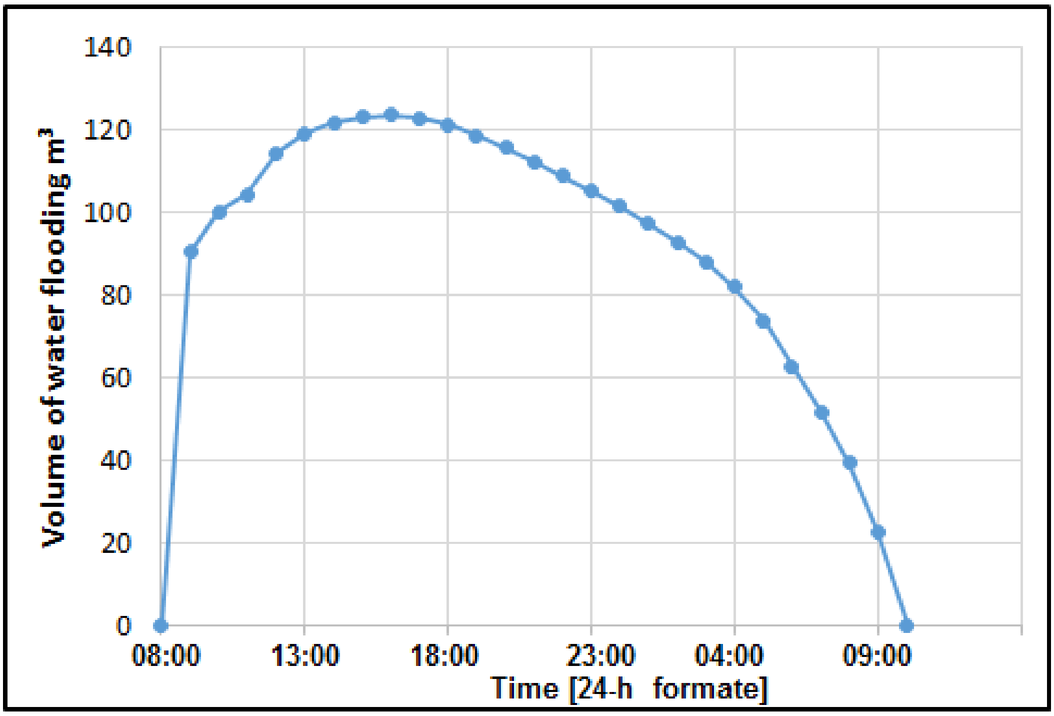

Fig. A.4:The flooding volume in the outlet manhole M187 with storm with peak intensity 17 mm/hr

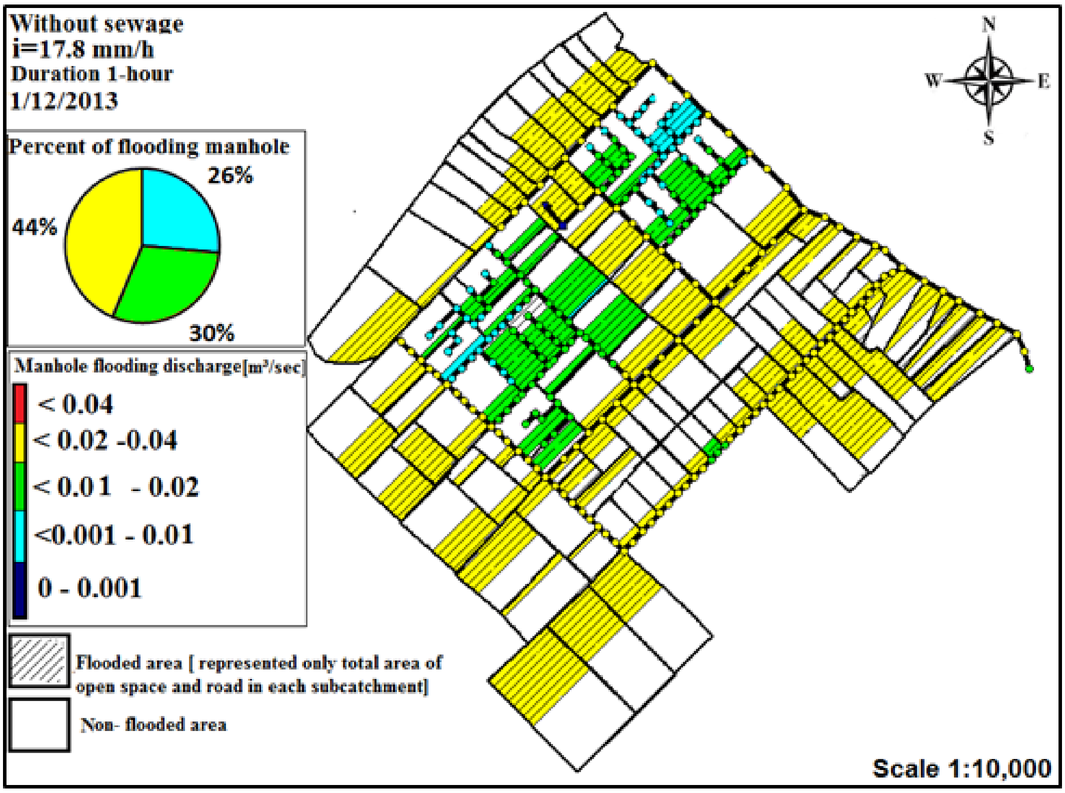

Fig. A.5:The flooding manholes under rainfall intensity 17.8mm/hr

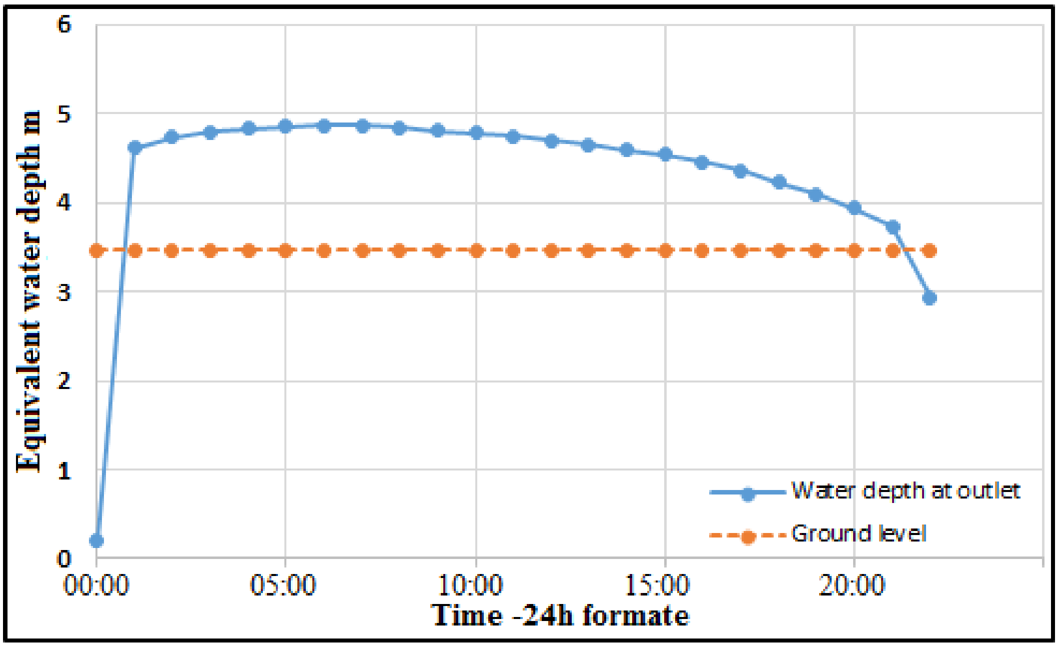

Fig. A.6:The water depth in outlet manhole (M187) with rainfall intensity 17.8mm/hr

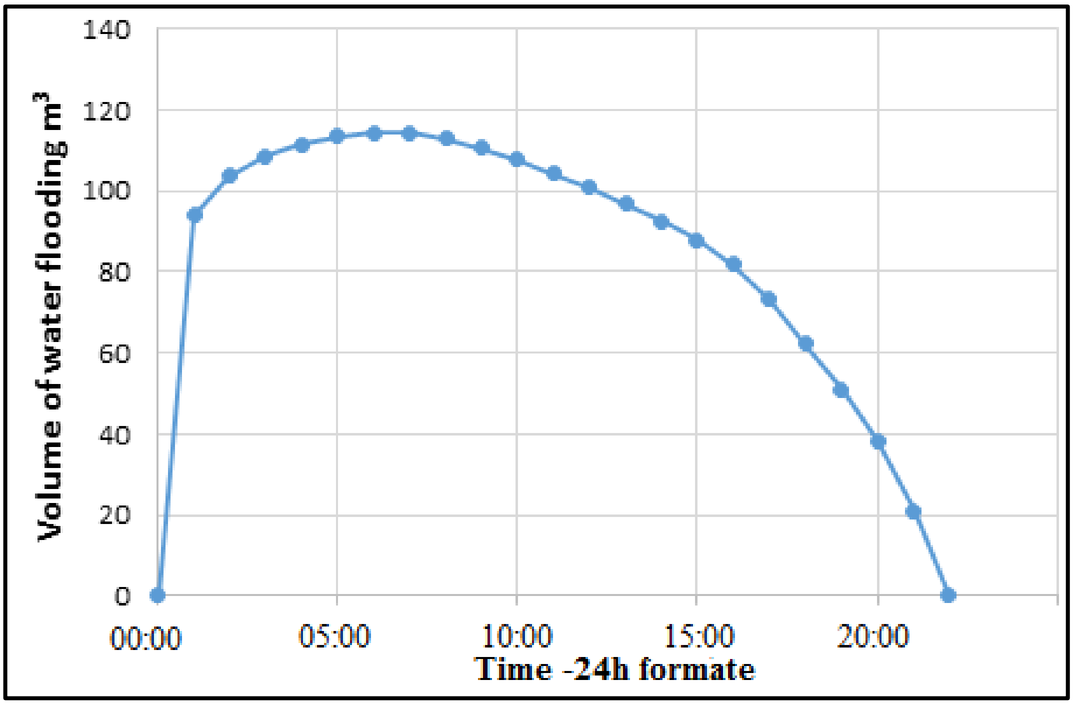

Fig. A.7: Flooding volume in outlet (M187) with rainfall intensity 17.8 mm/hr

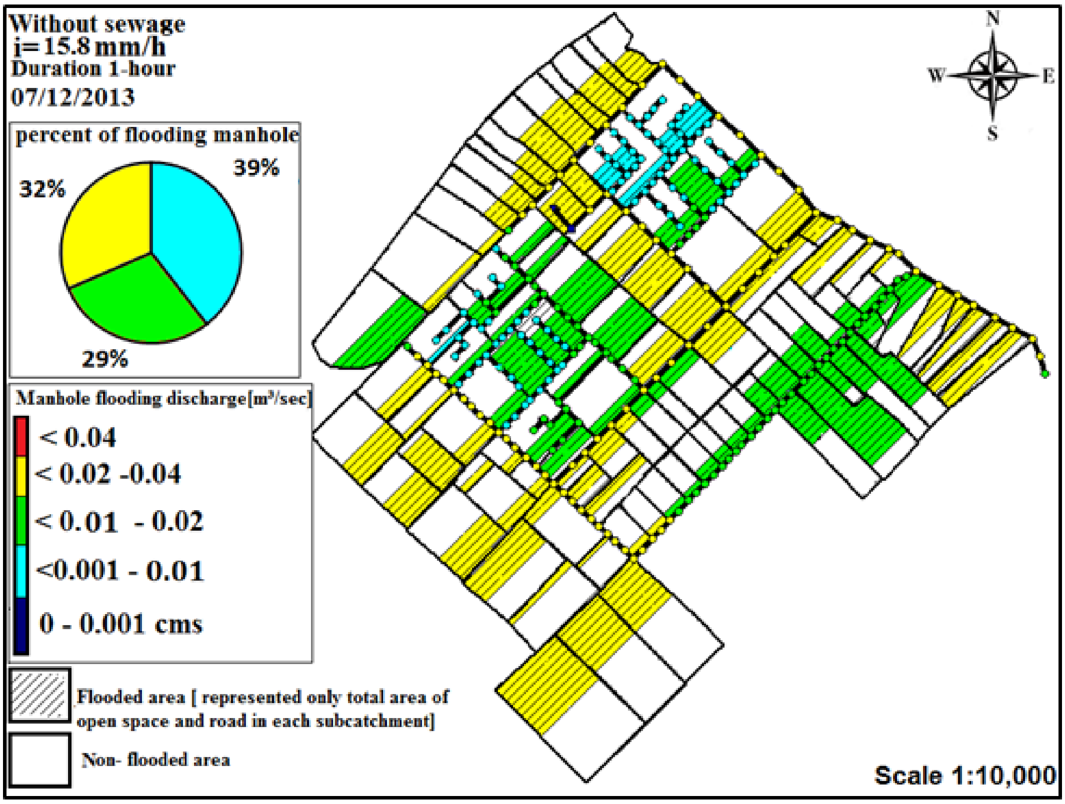

Fig. A.8:The flooding manholes under design intensity 15.8mm/hr at peak time

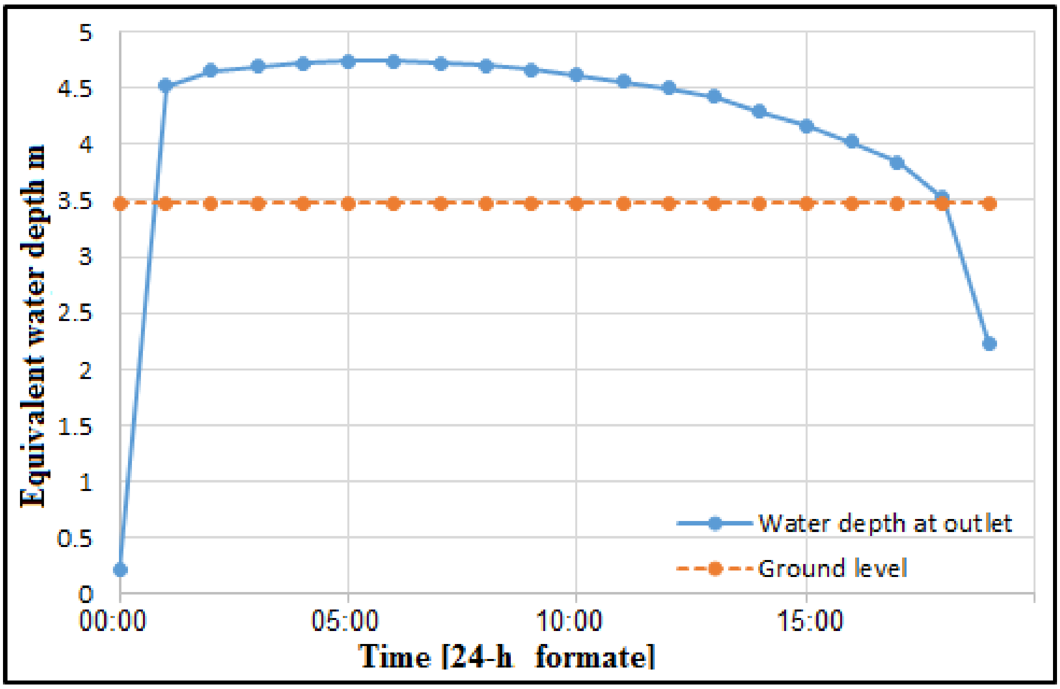

Fig. A.9: Water depth in outlet manhole (M187) with rainfall intensity 15.8mm/hr

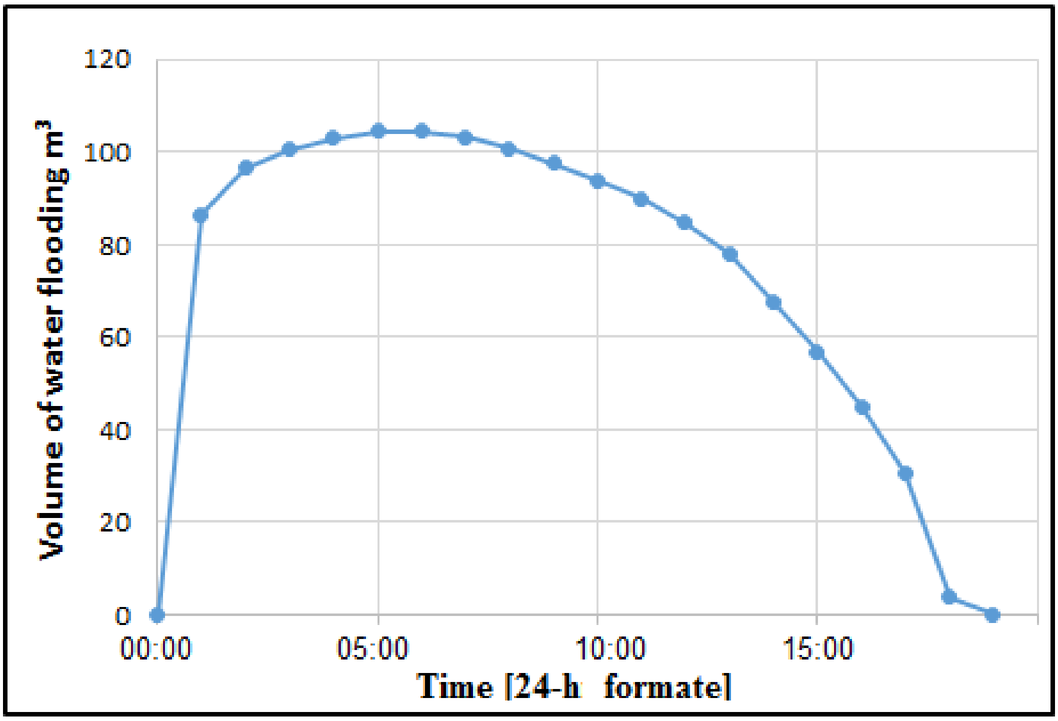

Fig. A.10: Flooding volume in the outlet (M187) with rainfall intensity 15.8 mm/hr

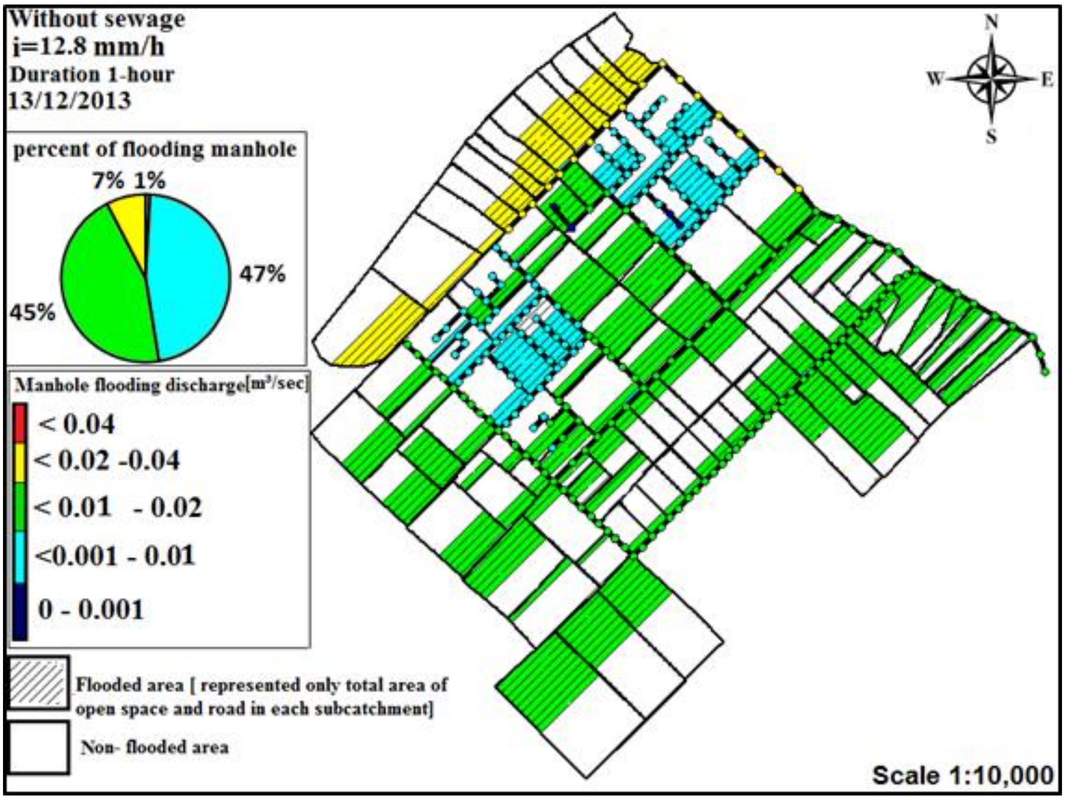

Fig. A.11:The flooding manholes under rainfall intensity 12.8mm/hr at peak time

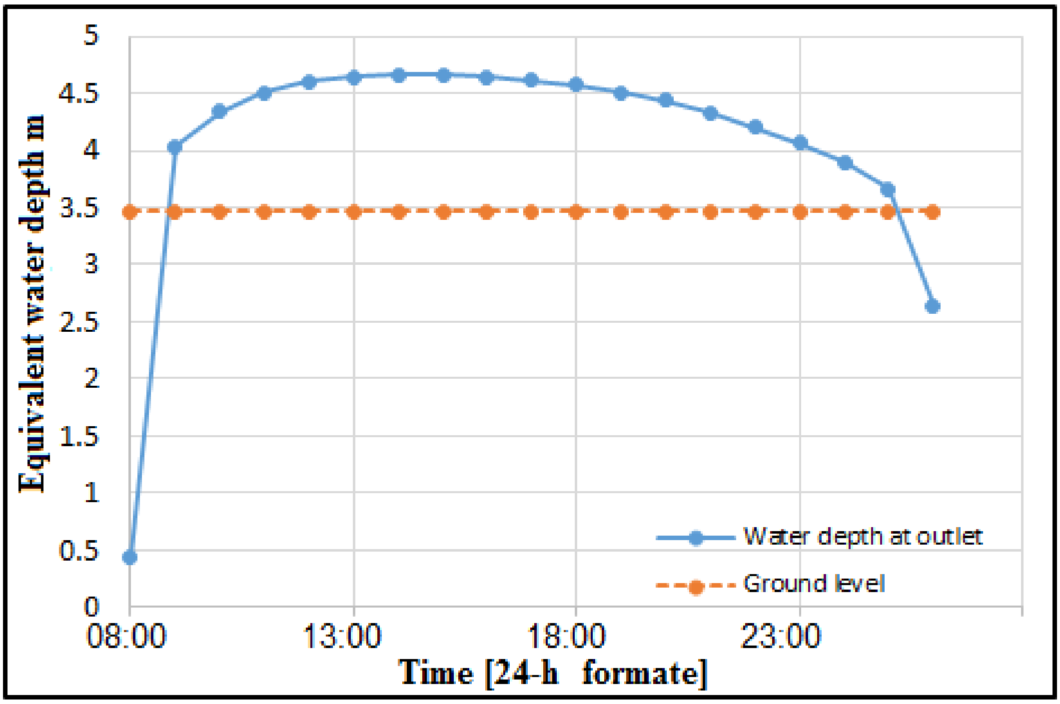

Fig. A.12:The water depth in the outlet manhole M187 with intensity 12.8mm/hr

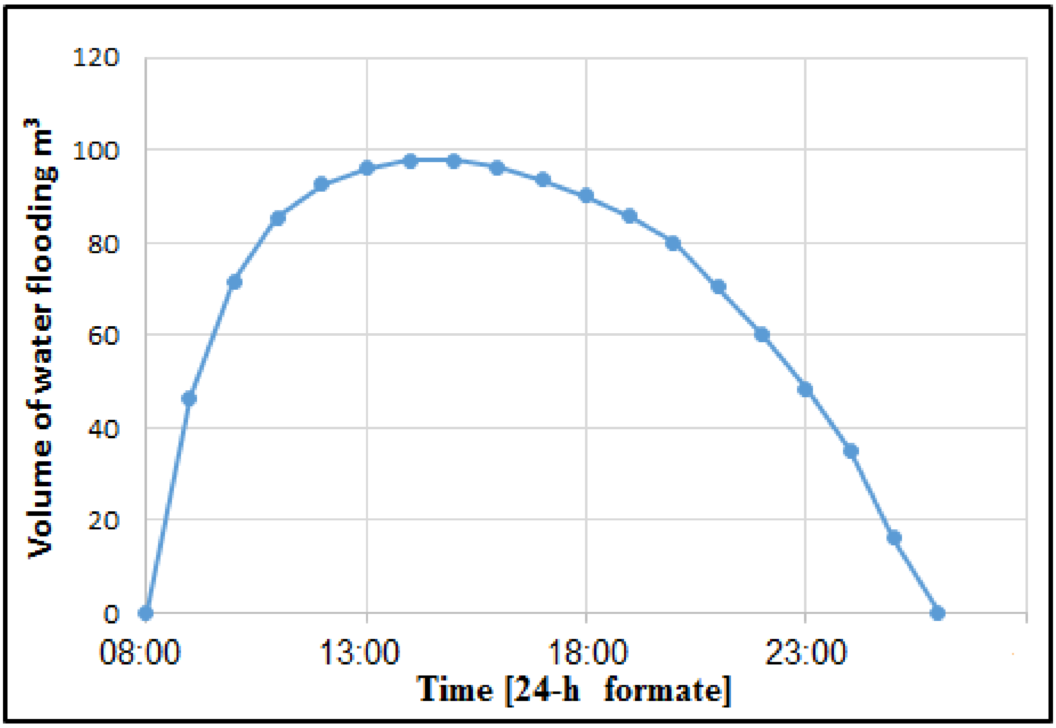

Fig. A.13:The flooding volume in the outlet manhole (M187) with intensity 12.8mm/hr

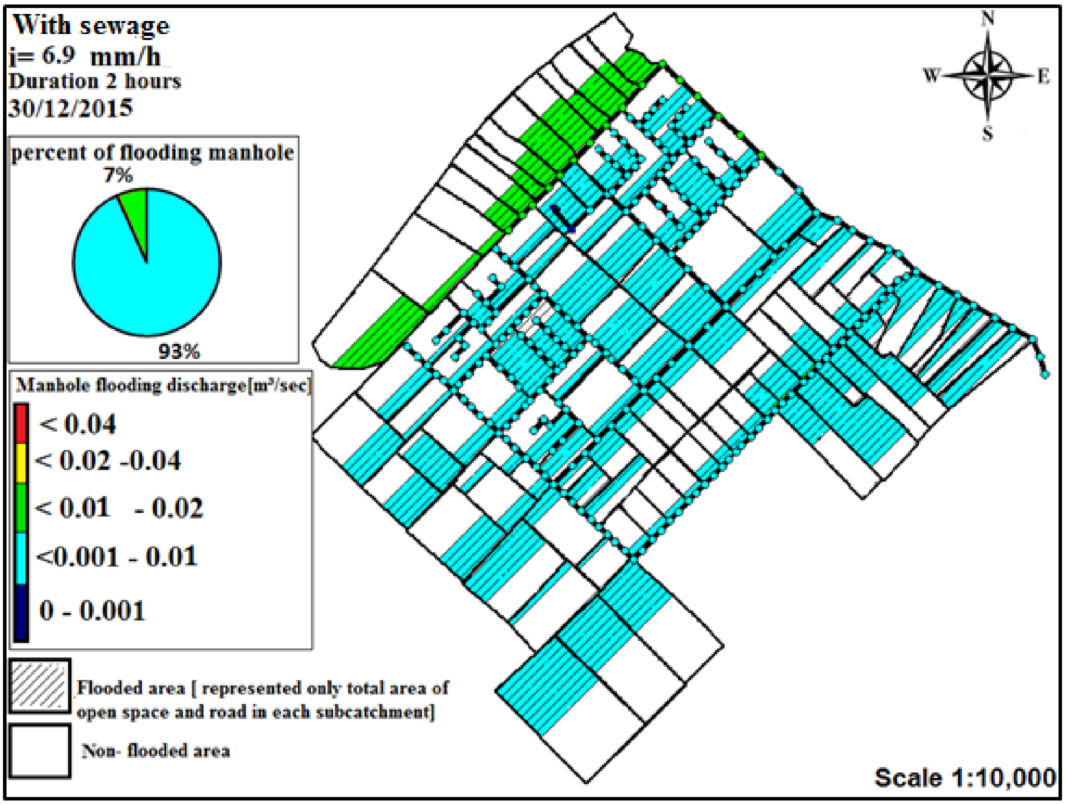

Fig. A.14:The flooding manholes under rainfall intensity 6.9 mm/hr at peak time

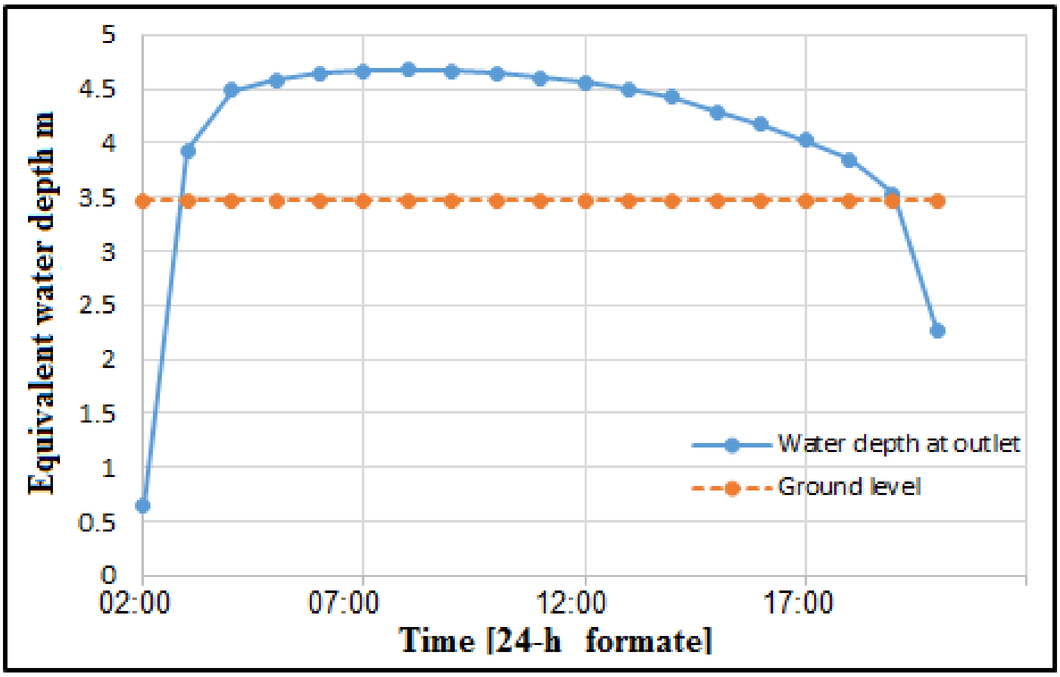

Fig. A.15:The water depth in the outlet manhole (M187) with intensity 6.9mm/hr

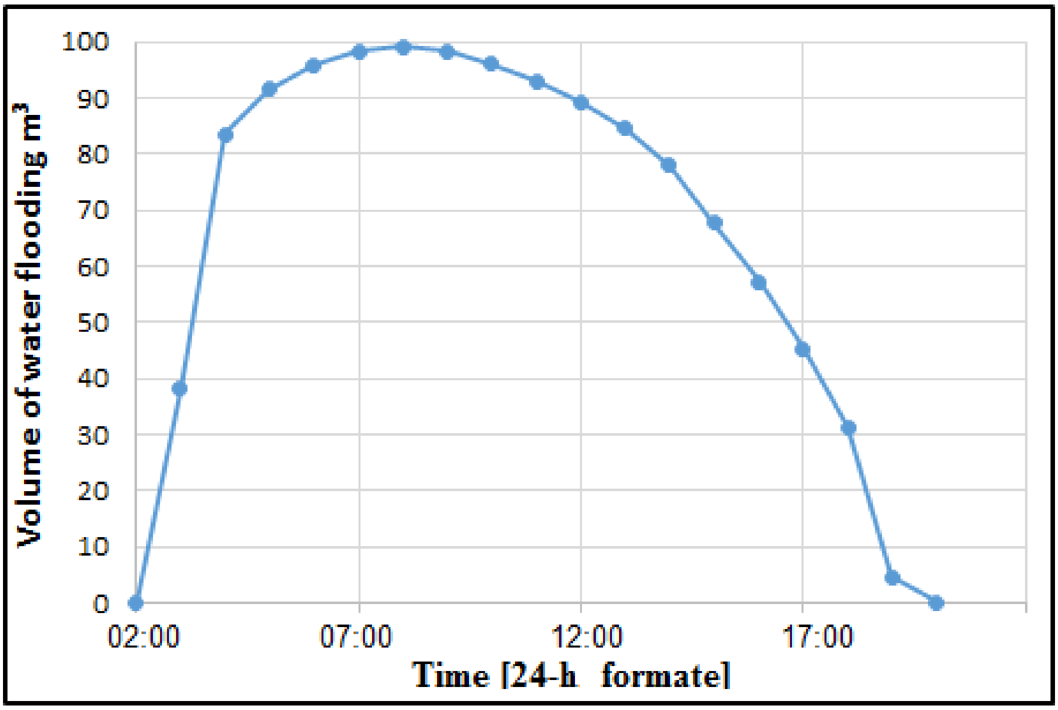

Fig. A.16:The flooding volume in the outlet manhole (M187) with intensity 6.9mm/hr

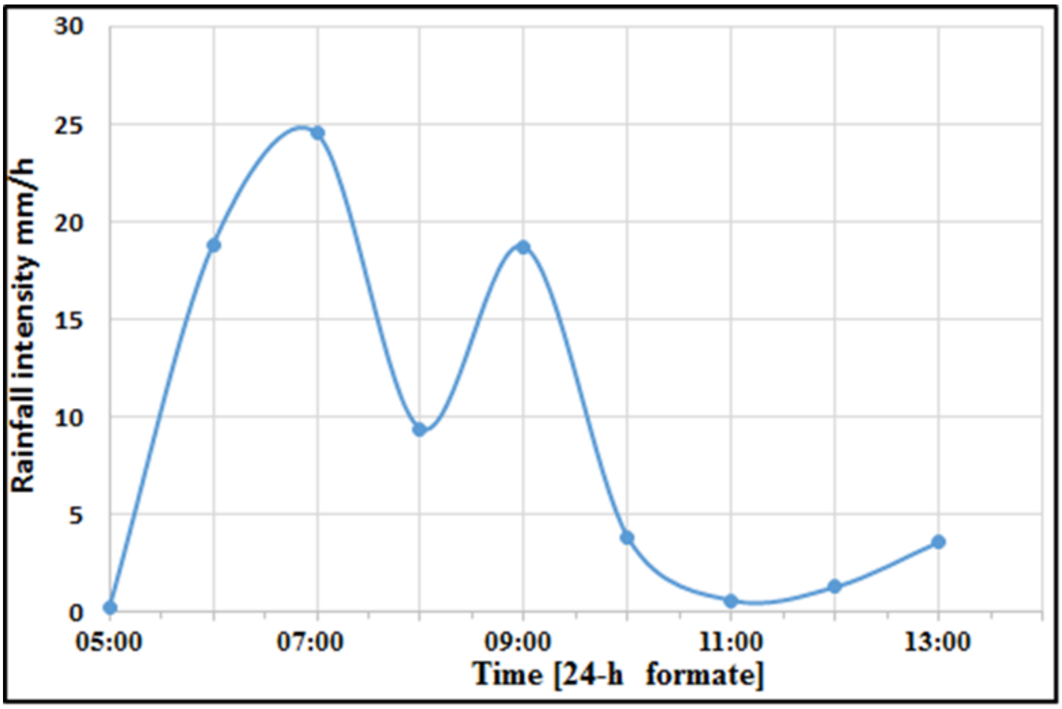

Fig. A.17:The storm intensity at 28/3/2016

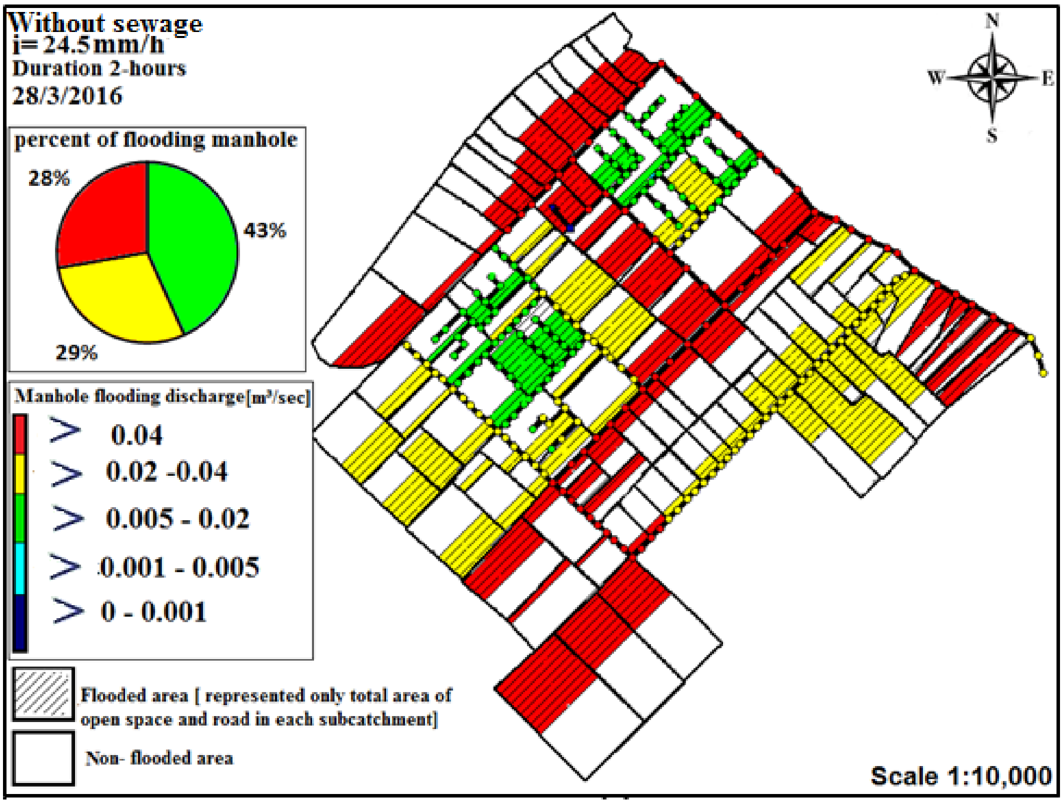

Fig. A.18: Flooding manholes under rainfall intensity 24.5 mm/hr at peak time

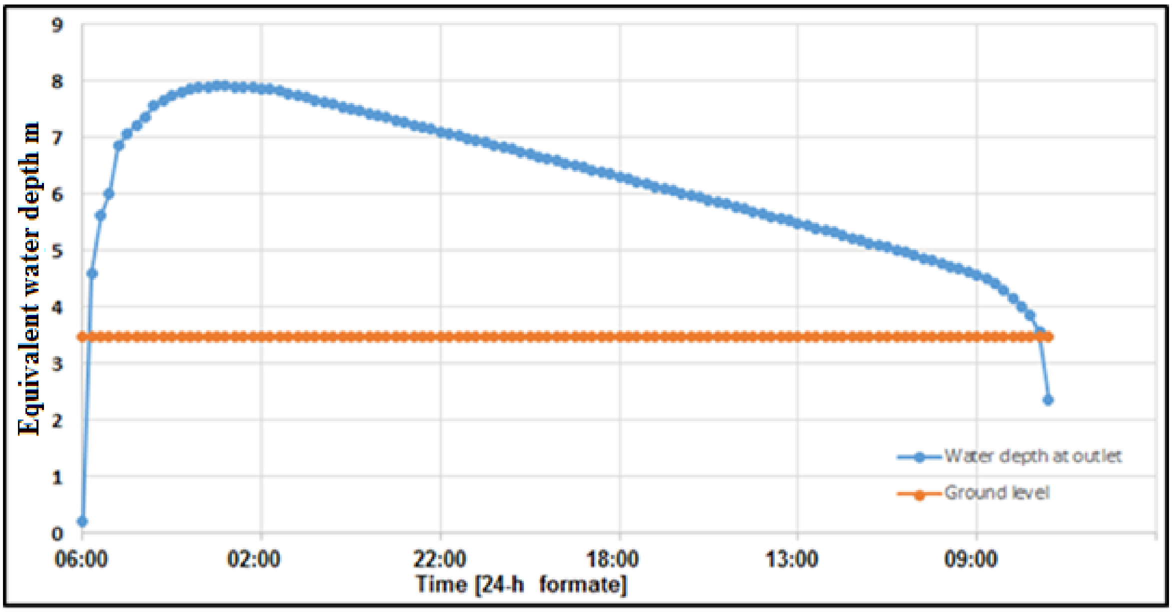

Fig. A.19:The water depth in the outlet manhole (M187) with intensity 24.5 mm/hr

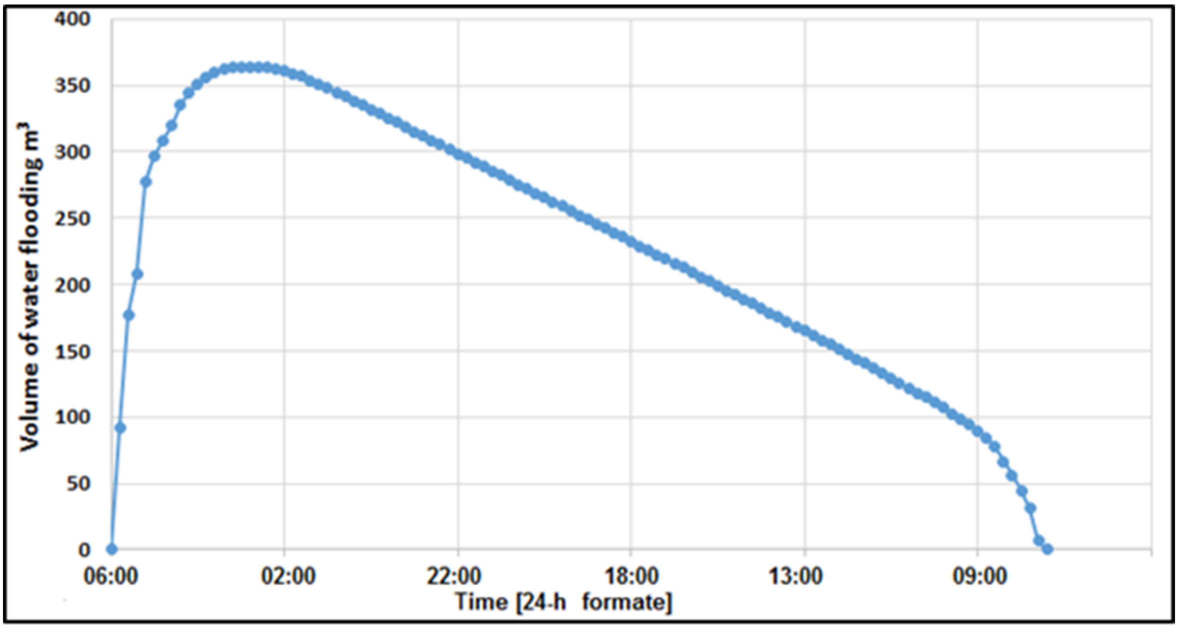

Fig. A.20: Flooding volume in the outlet manhole (M187) with intensity 24.5 mm/hr

Fig. A.21: Performance of storm drainage (A-without sewage,B-with sewage) under rainfall intensity 12.8mm/hr

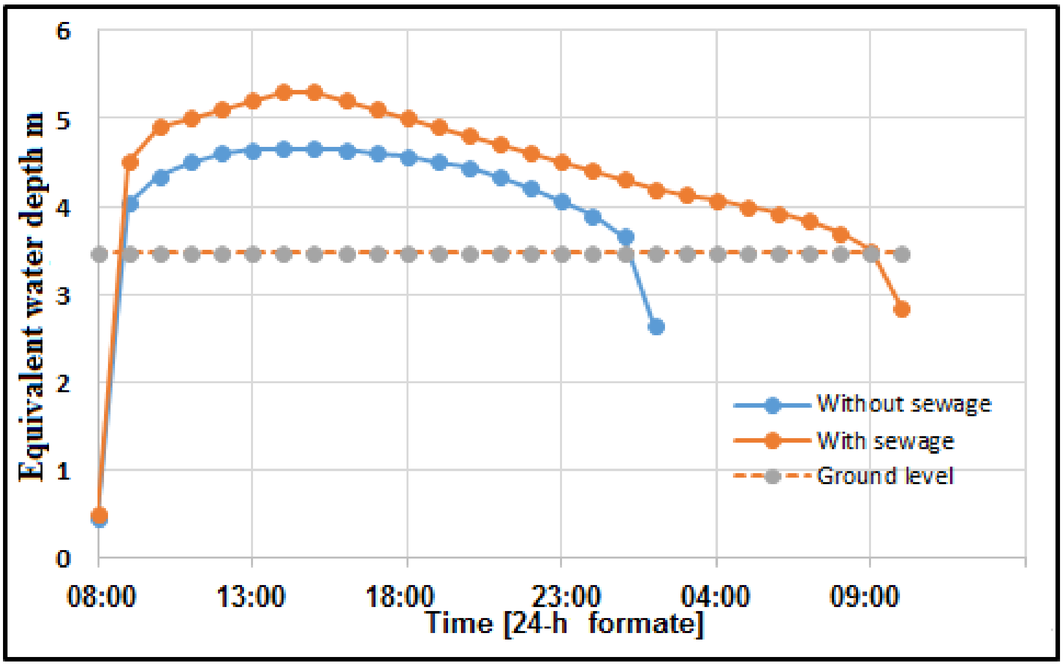

Fig. A.22: Water depth in manhole (M187)(with and without sewage) under 12.8 mm/hr

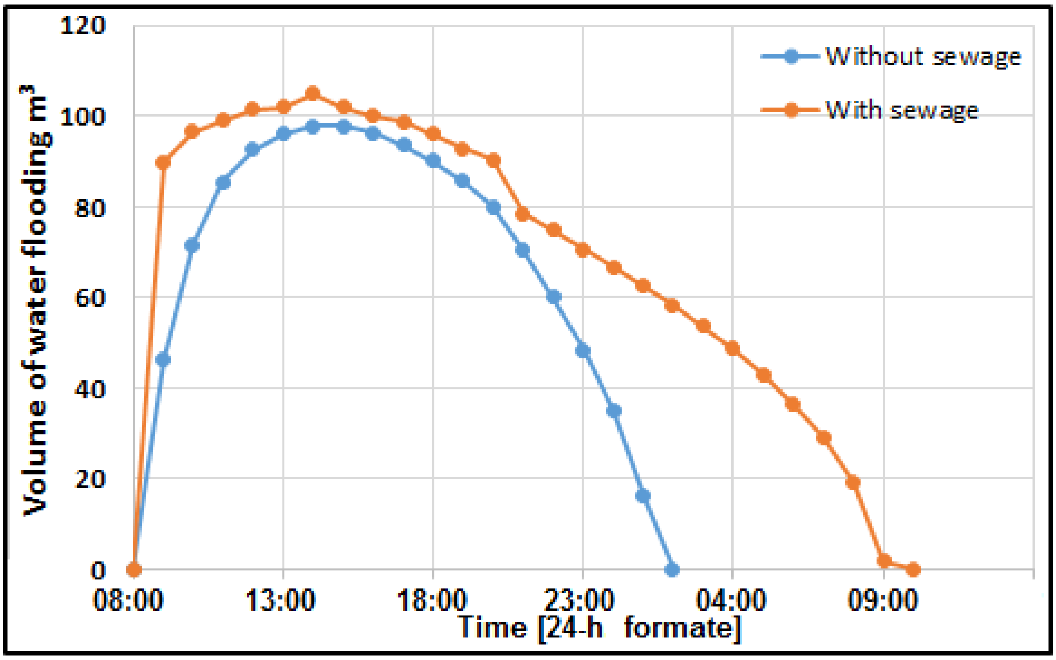

Fig. A.23:Flooding volume in (M187) (with and without sewage) under 12.8 mm/hr

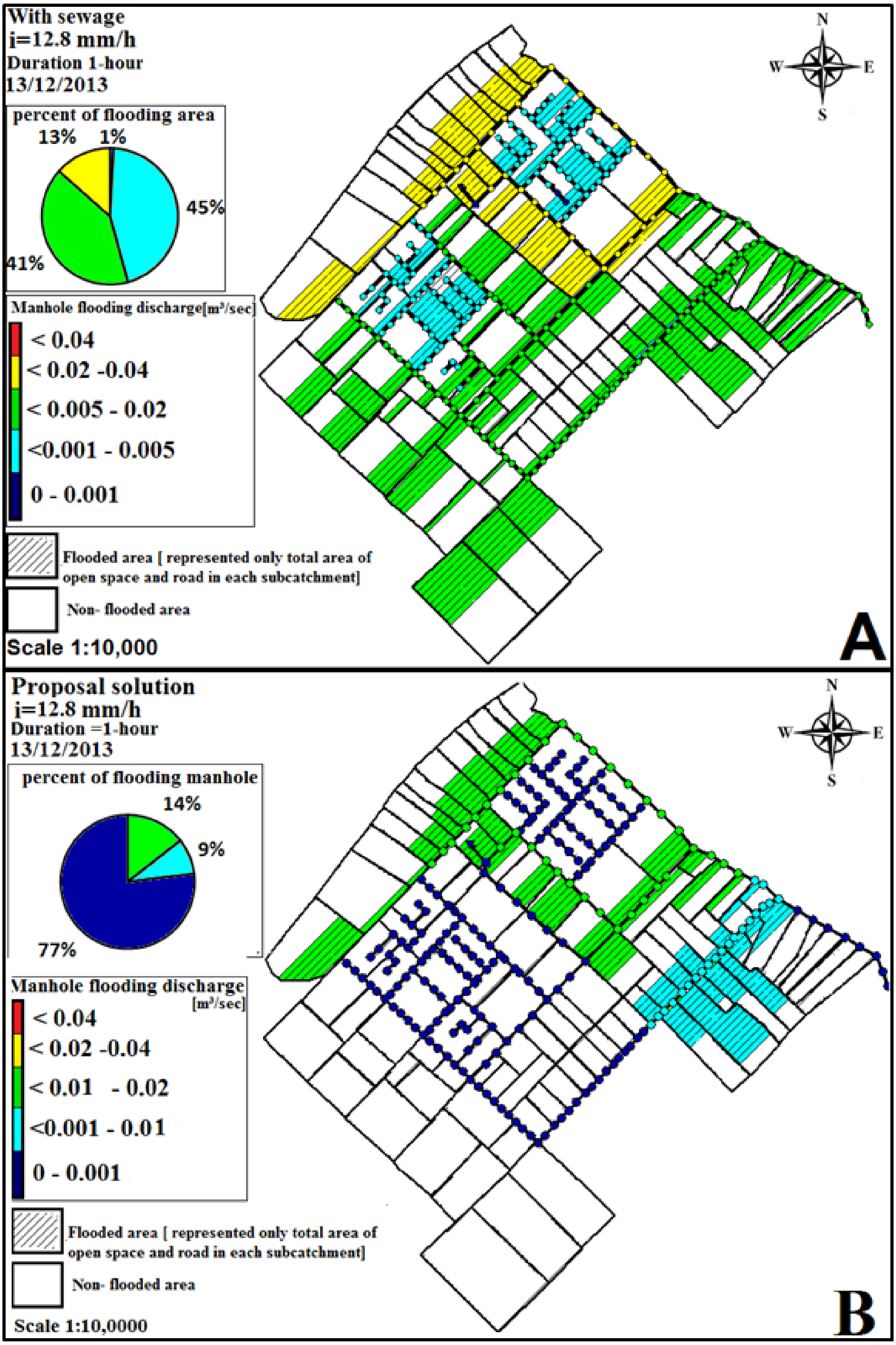

Fig. A.24: Performance of storm drainage (A-with sewage,B-proposal solution) under 12.8 mm/hr

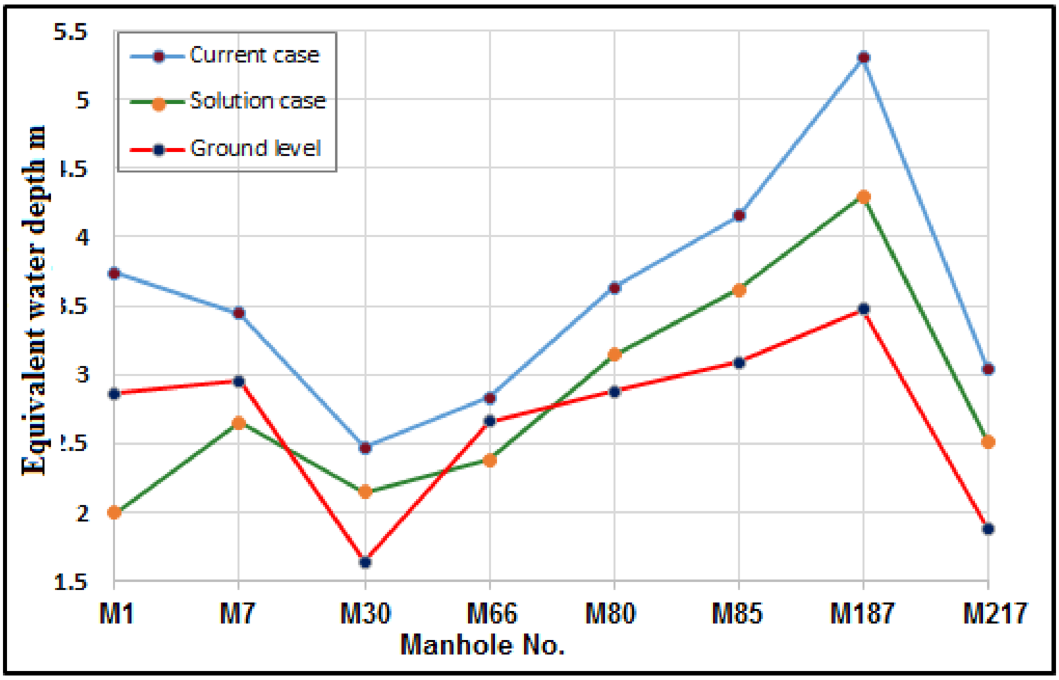

Fig. A.25:Water depth in selected manholes (current case+proposal solution) under 12.8 mm/hr

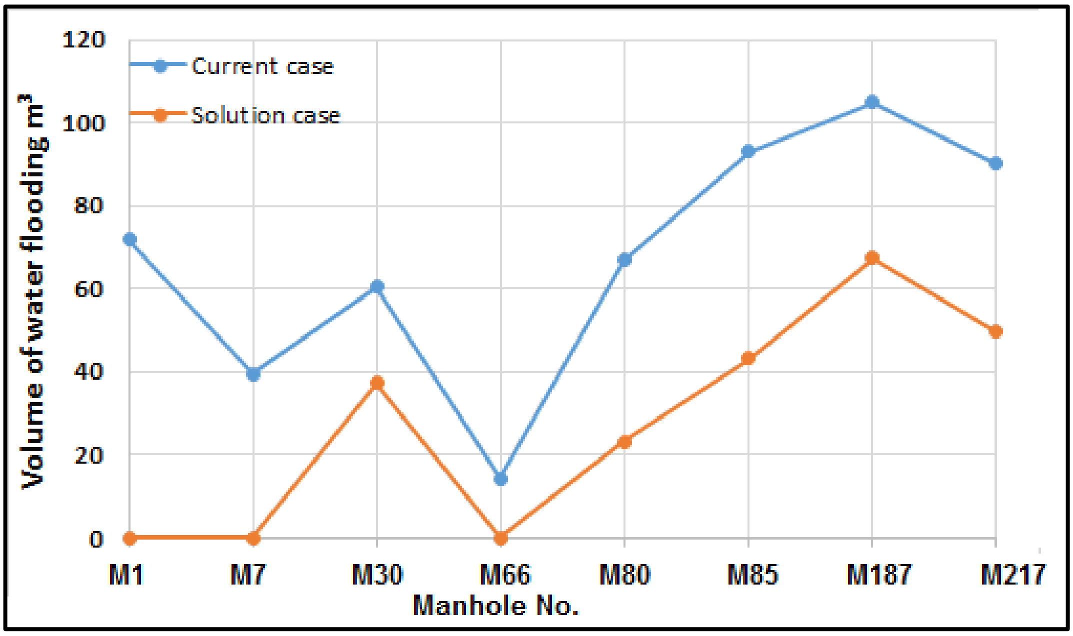

Fig. A.26:Volume of water flooding in a selected manholes (current case+proposal solution) under 12.8 mm/hr

جمهوريه العراق

وزارة التعليم العالي والبحث العلمي

جامعة كربلاء

كلية الهندسة – قسم الهندسة المدنية

تقييم تأثيرات الشده المطرية على شبكة الامطار ومشاريع البنى التحتية

[منطقة الدراسة الحر –كربلاء كحالة دراسيه]

رساله مقدمه الى كلية الهندسة جامعة كربلاء

كجزء من متطلبات نيل درجة الماجستير في

علوم الهندسة المدنية (البنى التحتية)

من قبل

بتول عبد الله إسماعيل المسعودي

بكالوريوس هندسه مدنية 2013م

بأشراف

أ.م.د. باسم خليل نايل السعدي

أ.م.د. واقد حميد حسن الموسوي

2017م-1437هـ

مستخلص

ان تباين الشدات المطرية بمستوى اعلى من المعايير التصميمية نتيجة التقلبات المناخية و ظاهرة الاحتباس الحراري يعد واحدا من أكبر المشاكل التي تواجه المناطق الحضرية. ادى التحضر في المدن الى زيادة نسبة الأراضي غير المسامية وبالتالي نقص نسبة الارتشاح للتربة وزيادة كمية السيح السطحي. ان سوء استخدام شبكة تصريف الامطار بواسطة السكان أدى الى تقليل سعة التصريف وحدوث الفيضانات بالمناطق الحضرية. تعد نمذجة الفيضانات طريقة مهمه تساعد في اكتشاف وإدارة مشاكل الفيضان.

تم في هذا الدراسة الاستعانة ببرنامج لتكوين انموذج لإدارة مياه الامطار يعرف ب[SWMM] لمحاكاة فيضان شبكه تصريف الامطار في الحي العسكري – كربلاء –العراق. جمعت البيانات الهيدرولوجية مثل (الشده المطرية – درجات الحرارة وسرعة الرياح) خلال الفترة من 2008 – 2016.وتم توفير البيانات الهيدروليكية لشبكة تصريف امطار الحي العسكري المتعلقة بأحواض التفتيش (منهولات) والانابيب والمضخات. وقد كانت البيانات المطلوبة لمعايره النموذج لشدتين مطريتين الأولى في 2016/1/18 بشده مطريه مقدارها 2 ملم/ساعة والثانية في 2016/1/22 بشده مطريه 4 ملم/ساعة.

الهدف الأساس من هذه الدراسة هو تطوير نموذج لتقييم تأثير تغير المناخ وسوء استخدام نظام تصريف الامطار من خلال إضافة كميات غير قانونية من مياه المجاري اليها بواسطة السكان. إضافة الى تحديد كلفة الضرر الواقعة على البنى التحتية الناتج عن الفيضان. حاولت هذه الدراسة توفير دعم تقني لصانعي القرار من خلال تطوير حل لتقليل حدة الفيضان في منطقة الدراسة.

اشارت نتائج الدراسة الى ان شبكه تصريف مياه الامطار ملائمه للشده المطرية المصممة لها (9.6 ملم/ساعة مدتها ساعة واحده) لفتره تكراريه مدتها سنتين بدون اعتبار تأثير كميات مياه المجاري غير القانونية. ان التقلبات المناخية لعبت دورا رئيسيا في حدوث الفيضان في منطقة الدراسة حيث ان الشده المطرية في أحد الاحداث المطرية وصلت الى ثلاث اضعاف الشده التصميمية (33.54 ملم/ساعة مدته ساعة واحده). حيث ان الشبكة وعند هذه الشده وبدون اعتبار كميه مياه المجاري يصل عمق الماء المكافئ في اخر حوض تفتيش في الشبكة الى 5.55 م مقاسه من قاعدة حوض التفتيش وان47% من احواض التفتيش طفحت بتصريف أكبر من 0.04 م^3/ثا بمدة فيضان مقدارها 43 ساعة.

وجود مياه المجاري في شبكة تصريف الامطار زاد من مشكلة الفيضان في منطقة الدراسة نتيجة للانسدادات التي تحدث في النظام حيث ان النظام يبدأ بالفيضان عند شده مطريه اقل من الشده التصميمية. عمق الماء المكافئ في اخر حوض تفتيش وباعتبار وجود كميات مياه المجاري عند تأثير أكبر شده مطريه

الى 5.62م و48% من احواض التفتيش في نظام تصريف مياه المجاري كانت طافحه بتصريف أكبر من 0.04 م^3 /ثا وبمده فيضان مقدارها 72 ساعة حيث ان كميه مياه المجاري ضاعفت مده الفيضان.

ادى حدوث الفيضان وبمدد طويله الى حدوث ضرر في البنى التحتية خصوصا للطرق والمباني. بلغت الكلفة الكلية للأضرار الناجمة عن الفيضان في منطقه الدراسة (277200000 دينار عراقي) وهي كلفه متكررة إذا لم تتوفر الحلول لتقليل الفيضان. في هذه الدراسة تم اقتراح حل لتقليل حدة الفيضان بواسطة إضافة خط ناقل بكلفه إنشائية مقدارها (1500000دينار عراقي) لكل متر طول من الانبوب. كان أداء الانبوب المقترح عند أكبر شده مطريه ومع اعتبار وجود مياه المجاري قد وصل 5م وحجم المياه الفائضة وصلت 129م^3 و ان 33% من احواض التفتيش طفحت بتصريف وصل الى 0.04 م^3/ثا مع مده طفح مقدارها 20 ساعة. ان نتائج إضافة أنبوب ناقل كانت واعده وقللت نسبة الفيضان في منطقة الدراسة بدرجة كبيره. نتائج المحاكاة تستطيع توفير دعم تقني جيد للتحكم والتخطيط لشبكات التصريف الحضرية.