KWAME NKRUMAH UNIVERSITY OF SCIENCE AND TECHNOLOGY,

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REVIEW OF DESIGN OF CULVERTS ON ROADS WITHIN FLOOD PRONE AREAS OF ACCRA EAST AND ASSESSING THEIR IMPLICATIONS ON MAINTENANCE COST.

BY

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DECLARATION

I hereby declare that this submission is my own work towards the MSc Roads and Transportation Engineering and that to the best of my knowledge, it contains no material previously published by another person , nor material which has been accepted for the award of any other degree of the university, except where due acknowledgment has been made in the text.

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DEDICATION

I would like to dedicate this research to my family, especially my son Jeremiah Peter Kwabena Amoako who used to ask when I was going to finish so that I could be available at home.

I thank them for their prayers and support in diverse ways and may the Almighty Lord richly bless and protect them.

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ABSTRACT

Sections of the urban road network in Accra East deteriorate after almost every raining season notwithstanding the preceding routine maintenance activities. Most of those sections are culvert approaches of roads that cross Songor stream at Teshie Nungua and Onukpa Wahe stream at Tema Communities 19 and 20. Homes of residents along the two streams are not spared of seasonal floods. Huge maintenance costs are incurred by both Road Agencies and residents along the streams. Rapid urbanization has increased the imperviousness of the catchment areas translating into increases in runoff beyond what the culverts could safely discharge. Review of the culverts designs was therefore, necessary in establishing if they contribute in any way to the flooding and washout of the roads.

A detailed study was undertaken at the Songor catchment area which has 14 road culverts of different sizes and 1 rail culvert. HEC-RAS hydrological software was used to model the flood using ten (10) years return period. Survey instruments in the form of two questionnaires were administered, one to residents of the flood modelled area and the other to the LEKMA roads department mainly to illicit domestic and road maintenance cost respectively. Land use land cover maps of 1991, 2001, 2011 and 2016 were downloaded using GIS application to assess the percentage changes in settlement over those years. The area without vegetation cover was considered settlement area and impervious, together with the runoff co-efficient model; $C = 0.74IM + 0.132$, runoff coefficients of construction and study years could be computed. Modified Rational Method was used to compute the flows at both construction and study years. Civil 3D drainage software was used for the hydraulic analysis.

The runoff co-efficient (C) against year graph shows equilibrium condition of 0.82 at 92% impervious for the catchment. Apart from the rail culvert, all the fourteen road culverts are of inadequate capacity. The maintenance costs obtained indicated that ten (10) years of combined maintenance costs by Road Agencies and residents can address the cost of culvert inadequacies except the proposed bridge. The culverts could be replaced using NPV/C in their selection due to budgetary constrain.

Regular removal of silt in the stream channel is recommended to improved flow. Wire fencing of the Waste Transfer Station located along the stream should be undertaken to prevent siltation of the stream channel. The study can be replicated along other streams with similar challenges.

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4.0 Background

Flooding of road networks occur when the road networks which are normally dry are inundated usually after rainfall. Buildings are sited on water causes which reduce buffer zones meant for free flow of water and in some cases, the blocking of flow of water passage entirely leading to the flooding of communities and road networks upstream of the restricted sections of the catchment areas. Culverts and other drainage structures that are constructed on roads for safe channeling of water at stream/ water crossings are often found to be the bottleneck, limiting flow and contributing to flooding and washout of road pavements. Flooding of Accra after medium to heavy rainfall has become routine annual events with undesirable consequences which are mostly felt by the urban poor (Okyere, et al. 2013). Increasing urbanization within catchment areas that hither to had vegetation cover with enhanced percolation of surface water has continuously reduced those vegetative areas and no longer function as such. Instead the vegetation cover is replaced with concrete paving, tiles, buildings, roads and other facilities with various finishing that prevent surface water from infiltration. These increase the runoff whenever there is rainfall. The problem of road flooding is mostly a developmental issue in our cities.

Rainfall is a very common cause, but not the only cause of road floods. Road flooding could occur through a combination of hydrological, hydraulic and anthropogenic sources. Hydrological factors concern rainfall, its formation and the flow of water on the surface of the earth, hydraulic factors could arise out of inadequacy of drainage structures mostly culverts capacity to ensure free flow of water and debris leading to artificial damming of the stream upstream whiles anthropogenic factors are those human activities that cause flooding. These are compounded along the coastal belts where slopes of outfall of streams are very gentle and the floods coincide with periods of high tides with tidal current. In recent past climate change is impacting heavily on the hydrological factors. Major cities across the world have experienced varying degrees of flooding leading to destruction of roads and culverts especially those that fall within high rainfall belts. As more towns become urbanized as a result of increase in population, this leads to increasing imperviousness which increases flood risk globally.

As at 2008, half of the world‘s population started living in urban areas. Two-thirds of those in the urban areas are in low to middle income countries. This is estimated to rise to 60 percent in 22 years, and 70 percent in 42 years to a total of 6.2 billion, which is projected to be twice the rural population by then (Jha et al., 2013).

In the very recent past, specifically on 2nd June 2016, there was a reported rainfall in Germany which led to the death of three people in the town of Simbach am Inn. Rescue teams found the bodies of the three in a house which was flooded, in addition to a female body was seen along a stream in the vicinity. The area most affected by the flood in southern Germany was Rottal Inn district. The impact was such that a disaster center had to be set up. In some of the towns especially Triftern, water bodies overflowed their banks. The extent of the flood was such that cars were dragged along, timber products in the form of cabinets and furniture were seen floated in flooded homes. In many places water levels were such that emergency rescue teams had to intervene using helicopter. In France, inhabitants of Nemours who live close to their capital had to be evacuated. The flooding was the worse in the century. Some areas received their six weeks of rainfall in three days. The aged were venerable as the body of an old woman was retrieved from her own house. (http://www.bbc.com/news/worldeurope-36429381, 2 June 2016).

The 2016 flooding affected parts of Asia as reported flood casualties stood at least 300 people dead in eastern and central India. More than 6 million others were affected by floods that had submerged homes, washed away crops, destroyed roads and disrupted power and communication networks. Heavy monsoon rains had caused rivers, including the mighty Ganges and its tributaries, to overflow their banks forcing people to seek refuge in relief camps in the states of Madhya Pradesh, Bihar, Uttar Pradesh, Rajasthan and Uttarakhand (Bhalla and Dash, 2016). On 19th August 2016 the reported Louisiana floods was one of the worst recent US disasters. The downpour continued for more than 72 hours, leading to widespread and dramatic flooding in the affected area and downstream along the Amite river. The impact was extensive and destructive.

There were reported deaths of not less than13 people with over 40,000 homes destroyed (Taylor, 2016). The above examples of floods across the world are ample evidence that every part of the world experience flooding and it is a major issue that requires the needed attention.

The 3rd June 2015 rainfall in Accra, Ghana's capital was reported to have caused flush flood event with extremely huge destruction to lives and property never to have been witnessed before. The high casualty level was due to an explosion at a fuel station where both the workers of the station and some members of the general public were seeking refuge from the flood which resulted in 152 deaths. It is therefore necessary to probe for more creative ways that can be integrated into the existing structures to manage the floods (Asumadu-Sarkodie et al., 2015). The 3rd June 2015 Accra flooding amidst the burning of a filling station with estimated casualties of 152 bodies is ample testimony that Ghana has its share of the global situation. Heavy and continues downpours during some rainfall seasons coupled with building of unauthorized structures on water courses worsen the situation. When hydraulic issues also arise, worsening conditions emerge and our road networks are not spared. This leads to a situation where sections of our road network that fall within the catchment areas of the various water bodies are washed away worsening their condition. In some situations, some of such roads are closed to vehicular traffic on the basis of public safety. Funding would have to be sort for from the limited budget to be able to reinstate those sections of the road networks. The floods also come with high casualty levels in some instances as was the case of June 3rd disaster in the country. Properties worth millions of Cedis are destroyed with huge implications on increasing poverty levels. A case in mind was the Secaps hotel near Tetteh Quarshie interchange where a three-star hotel situated in a prime area of Accra and along a stream channel was inundated with water after a major down pour.

The Municipal Assemblies through the Town and Country Planning and Works Departments are to ensure the proper planning and compliance to physical development. The Ghana Highways Authority Road Design Guide and Urban Roads Design Manual are official guiding documents for the sizing of road side drains, culverts and other drainage structures in Ghana, however, a lot of subjective assumptions will have to be made in the choice of design parameters which will be based on the designer‘s experience. In the processes of designing and selection of culverts, various parameters of varying soil characteristics, catchment type and shape, topography, selection of the appropriate return period, run off co-efficient etc. have to be carefully considered with some level of subjectivity based on experience.

Most of the designs and constructions of culverts on roads are not carried out around the same period. This often leads to non-coordination of activities leading to choice of varied factors that culminate into varying sizes along the same stream in some cases.

The sizing based on an engineer‘s experience also comes with its challenges. The area under study falls within the Accra East road network. Sections of the road network in the area get flooded during rainfall. The flooding also affects some of the communities within the catchment area.

1.1 Problem Statement

Flooding and wash out of sections of road networks with high maintenance cost to road agencies and road users in many parts of Accra is a common occurrence resulting in the following challenges.

Within the Accra East road network area, two notable stream catchment areas normally experience varying magnitudes of flooding and on few occasions with loss of lives and properties.

1. The capacities of culverts provided at the road crossing locations vary from one crossing point to the other, a smaller sized culvert could even be downstream of a bigger one. The inadequate culvert sizes are believed to be contributing to the flooding in those areas and need to be investigated.
2. Sections of the road network that get flooded get washed out with a lot of defects on the network.
3. This leads to high road maintenance costs to the road agency and high vehicle operating cost to the vehicle owners.
4. Increased travel time on the bad sections of the road are recorded which develops into long queues at the culvert approaches.
5. The road agency is not able to timely respond to maintenance works obviously due to budgetary constraints. It is, therefore, evident that financial constraints exist and instead of routine maintenance, a review of the culverts and possible replacement when found to be under capacity is a better option with more benefits to the road agency and the road users (bad roads leads to high Road Agency Cost, high Road User Cost and NPV/C ratio appraisal is required to address them when limited resources are available).

1.2 Objective of the Study

The main objective of the study is: To investigate the causes of the perennial flooding and washout along some sections of road networks in Accra East. Based on the above information, this study seeks to;

1. To identify roads in the study area vulnerable to persistent floods and the courses for the persistent floods.
2. Assess the existing culvert sizes for adequacy or otherwise.
3. Propose new designs (sizes) if any of the existing is found to be inadequate.
4. Make comparative assessment of cost of replacement of inadequate culverts with cost of perennial maintenance works on the roads after every rainy/flooding season and estimated losses of the residents through the floods.
5.

1.3 Significance of the Study

There is a saying which attributes the three most important elements in road maintenance to drainage, drainage, and drainage. Water, be it on the road or under the roadway through capillary action is a major cause of damage to the roadway. Water related road defects include rutting, cracking, potholes, erosion, washouts, heaving, flooding, and premature failure of the roadway. (http://www.ctre.iastate.edu/pubs/maint_worker/chap5.pdf). The study seeks to address the challenges of parts of road networks getting flooded in Accra East for hours with loss of lives and property when there is continuous heavy downpour. This could be achieved through reviewing the existing culverts and the providing appropriate designed hydraulic capacities of culverts for those that will be found inadequate for safe passage of water at road crossings. It will make the road networks free of floods and improve on safety at all times.

The sections of the network that often deteriorate in the raining season with high maintenance cost due to flooding and wash outs will also be addressed with the implementation of the appropriate designed hydraulic capacities. High maintenance budget expected to be used in restoring deteriorated roads can be used to improve the road condition mix. Invariably homes that get flooded with loss of property in the study area will cease. The methodology that will be used in addressing the challenges of hydraulic capacity of the culverts can be applied to other road networks that are experiencing similar conditions.

1.4 Scope of Work

From the problems stated earlier, it is clearly evident that the problems cut across most of the municipalities in the country. Issues of road flooding in Accra alone are so widespread that they cannot be addressed within the time for which this research has to be completed. Consequently, for the sake of time constrain, this research is limited to road flooding areas in Accra East comprising the area bounded by the Independence Avenue, the Accra – Tema motorway, the Tema Hospital road and the Atlantic Ocean in the south. The specific locations are from GREDA Estates through Teshie Nungua Estates to the Atlantic Ocean at Coca Beach in the south and from Community 20 through Communities 19 before joining the Sakumono lagoon. It is expected that the outcome of the research findings will be applicable to areas of similar characteristics and challenges.

1.5 Organization of the Thesis Work

This research work has been structured with the following contents,

Chapter one comprises the background to the study, the introduction of the issues to study, the Problem Statement necessitating the research, Aims and Objectives to achieve and a description of the study area. Chapter two follows with literature review of relevant literature on the subject to see how far the subject has been treated and where to continue from to ensure that the study is novel. The third chapter is the methodology adopted in the research. Chapter four brings out the results and discussions with chapter five ending with conclusion and recommendations.

CHAPTER 2: LITERATURE REVIEW

5.0 Introduction

Flooding is a global phenomenon and extensive research have been undertaken that could be found in literature with diverse objectives that vary from establishing the causes of flooding to deriving parameters, models and theories that help in detailed hydrological and hydraulic computations. To ensure that the study is novel, this chapter reviewed related literature relevant to the study to find out if any research has been undertaken in the identified area and if any, to what extent. This was to determine where this could commence from for new contribution to knowledge on the subject.

2.1 What is Floods

Floods occurs when a water body bursts or overflows its basin when carrying so much water that cannot be confined within its usual channel. Flooding is not a normal condition for the river, but it is seen as an extreme situation due to high levels of flow. The volume of runoff that goes into the water body determines the extent to which it exceeds its banks and the severity of the flood. Frequency of occurrence is how often flooding occurs. Encroachment of water channels obstruct flow and compound the effects of floods. Normally, the frequency of more severe floods is low while that of the less severe flood events is high. Floods are common events. (http://www.acegeography.com/causes-of-flooding.html). There is an aspect of flooding that occurs along the coast as a result of high tides and tidal currents.

2.2 Flood Types

Different flood types can have varying magnitude of impact, damage and cost, by the people who experience them (Hubbard, 2012).

2.2.1 Coastal Floods

Coastal flood is when the coastal area is flooded by the sea or a combination of the sea and stream flow. The cause of such a surge is a severe storm. The storm wind pushes the water up and creates high waves. The rise and fall of tide is accompanied by horizontal movement of the water called tidal current. Tide is the vertical rise and fall of the water and tidal current is the horizontal flow. The tide rises and falls and the tidal current floods and ebbs. (http://msi.nga.mil/MSISiteContent/StaticFiles/NAV_PUBS/APN/Chapt-09.pdf).

Flooding can occur whenever a large amount of rain falls over a relatively short period of time or moderate amount over a very long time. The risk increases if the rain coincides with high tide, or if the ground is already saturated by consecutive days of rain fall (http://www.hollywoodfl.org/DocumentCenter/View/7450). The tidal current contributes to coastal flooding by increasing the water available for flooding and rapid saturation of the coastal soil leading to increase excess run off. This prevents fluvial/pluvial floods from discharging into the sea to reduce the flooding time and the impact on communities. In some of the coastal flooding, tides and tidal currents are purely responsible for the floods as it is sometimes the case of Odaw river floods in Accra, the Sango stream that flows through Teshie-Nungua and joins the sea at Old Kasapreko area, the Brekese stream through Nungua and the Sakumono lagoon that enters the sea at Tema. Some of these streams and lagoons have their entry points into the sea specially constructed to reduce the effects of the high tides and tidal currents but its effects appear very marginal as flooding persist. Coastal flooding could also be caused by extreme tidal conditions that occur because of high tide levels, storm surge and wave action (Ballet al., 2008).

2.2.2 River or Fluvial Floods

Fluvial flooding occurs when excessive rainfall over an extended period of time causes a river to exceed its capacity. The damage from a river flood can be widespread as the overflow affects smaller rivers downstream. Pluvial flooding is a characteristic of urban areas where large areas of impervious ground exist and inadequate drainage systems abound. As urban growth increases, the impervious surface area also increases; thereby rendering populations vulnerable to water inundation as natural streams and humanmade drainage fails to cope with increased runoff subsequent to heavy rainfall (Youssef et al., 2011). Pluvial flooding often occurs unexpectedly in locations not obviously prone to flooding and with minimal warning and is not well understood by the general public – hence the term „invisible hazard" (Houston et al., 2011).

Basically, fluvial flooding is of two types;

1. Overbank flooding; this occurs when water rise overflows the edges of a river or stream. This is the most common and can occur in any size channel, from small streams to large rivers.
2. Flash flooding; this type is associated with intense, high velocity torrent of water that occurs in an existing river channel with little or no notice. Flash floods are very dangerous and destructive not only because of the force of the water, but also the hurtling debris that is often swept up in the flow (Ivan Maddox, 2014).

2.2.3 Urban Floods

Urban flooding is the inundation of land or homes in a built environment, especially in more densely populated areas, caused by rainfall overwhelming the capacity of drainage systems such as storm sewers. This can also happen in the absence of good drainage system to channel flow. Although it is sometimes triggered by events such as flash flooding, urban flooding is a condition, repetitive in nature and systemically impacts on communities regardless of location, whether it is within designated floodplains or near any body of water (The Center for Neighborhood Technology, May 2014 - Revised Version).

Urban flooding is due to the absence of good drainage network in an urban environment. Settlement reduces the available area for infiltration which leads to increased runoff that has to be transported as surface water or through sewage system. Medium to high intensity rainfall can cause flooding when the city sewage system and drainage channels are inadequate to drain away the amounts of rain that are falling. Water may even enter the drainage system at one place and runs over a street at another place due to sectional break down of the drain. (http://www.floodsite.net/juniorfloodsite/html/en/student/thingstoknow/hydrology/floodtypes.html). Urban flooding can be either coastal, fluvial or pluvial or even a combination of these types of floods (Ballet al., 2008).

The primary cause of urban flooding is severe thunderstorm or rainstorm preceded by long lasting moderate to heavy rainfall that saturates the soil (Andjeikovic, 2001). The runoff from storms attributed to roofs and paved surfaces in urbanized areas are very high owing to the associated imperviousness and very often than not, drainage structures cannot contain the runoff resulting in localized flash floods. These flash floods occurrence is very sudden, with little or no advance warning signal and are swift and violent. This sometimes results in life threatening conditions and extensive damage to property and infrastructure. The geographical area of impact is normally small in size (Douglas *et al.*, 2008).

2.3 Types of Urban Flooding Associated with African Towns and Cities

Urbanization in Africa may trigger any of the following four flood types;

1. Local area floods owing to absence of drainage system or are inadequate when present.
2. Flooding of communities from streams that have their watersheds settled upon.
3. Urban areas that got flooded as a result of settlement on major river banks.
4. Coastal flooding which occurs along the coast when flooded streams from the hinterland flowing into the sea coincide with high tides.

The frequency of occurrence of the local area and communities flooding is higher compared to the urban areas that get flooded from major rivers. The communities along the coast get flooded when there are encroachments on low lands along the coasts. (Douglas et al., 2008).

Localized flooding

This type of flooding occurs more frequently in the city in communities that lack adequate drainage system especially the slums. The ground in many parts of urban areas is highly compacted and reduces infiltration when it rains. These results in pathways between dwellings become areas available to channel the runoff after rainfall which sometime create gullies due to absence of drains. The few drains are choked with plastics and other waste due to lack of efficient municipal garbage collection and cleaning services which prevent free flow of water.

Streams flooding

Many of the flood plains of stream channels in the urban areas have been encroached upon and this contributes to flooding when there is heavy downpour. The water levels rise quickly due to blockage of stream channels which impede flow. When culverts are used to channel flow across road network, they could compound the situation if they are inadequate to pass peak flow. Usually the culverts are adequate at the initial years after construction for access to communities, however with increased urbanization the runoff increase and may be rendered inadequate if rate of urbanization is not anticipated in sizing the culverts. Sometimes garbage is disposed off in drains when there is rainfall together with silt deposition. When these prevail for long duration without routinely removal of silt from the stream channels, channel capacity eventually reduces and become inadequate to contain the flow leading to frequent flooding.

Rivers flooding

Rivers in urbanized communities are also not spared with encroachment. Human activities within the flood plains lose the soil leading to a lot of silt deposition which results in reduction in the capacity of the river channel. Urbanization within the river catchment also increases imperviousness resulting in higher volumes of discharge into the river. Invariably many streams discharge into major rivers and all the filth deposits in streams are emptied into the rivers. These activities often lead to the overflow of the river beyond its banks in the raining season. The creation of dams on rivers for hydroelectric power in urban areas certainly come with its consequences of flooding and when the dams are also constructed in rural environments for irrigation purposes, it comes with flooding of upstream communities.

2.4 Causes of flooding in Accra

Latest research on flooding in Ghana came out with several causes. The combined factors lead to severe consequences in certain areas of the country. There are several associated factors that cause flooding in Ghana. This is an indication that inadequate culvert sizes alone are not responsible for the flooding and whiles efforts are made to address hydrological and hydraulic challenges, similar efforts should be made to address the other factors especially anthropogenic ones in a coordinated manner.

Lack of drainage system or inadequately sized drains in some communities also contribute to flooding. Visual and in some cases with hydraulic assessments of existing drainage structures concluded on the presence of under sized drainage system at Mataheko and Kaneshie area (Sam Jr., 2015). This appears to be the situation along the Songor stream. However, they have to be assessed in consideration with their design parameters for the years of construction.

In the past decades, flooding has become a global pandemic, which hampers economic and social development. This global phenomenon has led to loss of many lives and economic damages in many countries worldwide including Ghana. In Accra, low-lying areas are usually subjected to severe flooding, which is generally attributed to inadequately sized culverts and drains, blockage of the major drainage structures by accumulated silt caused by years of neglect and lack of maintenance (Sam Jr, 2015). As reported by Asumadu-sarkodie *et al*., 2015, flood is ranked as second national disaster in Ghana occurring about eighteen (18) times, and killing over four hundred (400) people.

Table 2.1: List of natural disasters in Ghana from 1900 to 2015/16

Disaster	No. of Events	Killed	Total Affected	Damage**(US$)**
Drought	3	0	12,512,000	100,000
Flood	18	415[1]	3,885,695	108,200,000
Epidemic	19	875	33,799	-
Wildfire	1	4	1500	-
Earthquake	1	17	-	-

**1 Does not include 3rd June, 2015 floods and fire explosion at the Goil filling station in Accra leading to loss of over 152 lives. (Source: Asumadu-Sarkodie et al., 2015)*

From the table, an estimated total of three million eight hundred and eighty-five thousand, six hundred and ninety-five people have since been affected and recorded damages worth over one hundred and eight (108) million dollars. These statistics excludes the 3rd June 2015 and 10th June 2016 floods. These are the officially reported figures (Asumadu-sarkodie *et al*., 2015). These floods washed out sections of the road networks in the flood prone areas of Accra and worsened their condition. This resulted in public outcry on the state of our roads in 2015 and early part of 2016. The government of the day responded to the situation through the Department of Urban Roads to undertake asphalt overlay on the minor arterials and some of the distributor networks at a huge cost.

There are many more disasters in some of the municipalities that are not either reported or captured or both. Sometimes for political expediency, the figures are kept low. The floods and the casualties that occured at Teshie and Nungua (LEKMA) and Communities 19 and 20 of Tema Municipality were mostly not captured. The least talk about the effects of these floods and damages cause to properties especially road networks that go unrecorded, the better. However, these are the very data that are needed to be able to make economic justifications on why urgent replacements of culverts are very necessary. The Author of this research lives in the area under research and witness high casualty levels that are only heard talked about on radio or published in the dailies. These are not added to the official figures that are documented as was evident in the 3rd June 2015 floods. Similar situations also take place in other suburbs and are not captured

2.4.1 Urbanization

Globally, rural - urban migration in search of better job opportunities exist and sometimes intra city migration. Pressure on housing leading to high cost often compel people to settle in areas that are highly exposed to flooding thereby making them highly vulnerable if there is no flood defence mechanism (Jha, et al., 2013). This is due to the urban poor's inability to afford prime areas of the city. In some cases, they settle in uncompleted buildings at the outskirts of the city leading to faster than usual urban sprawling rate.

In the course of urbanization, the cities and towns are enlarged with increased imperviousness, consequently, volume of runoff goes up. Besides, pervious areas get reduced by the physical developmental projects that are undertaken. Flow volumes then increase leading to reduction in time of concentration of flow. Consequently, runoff volumes and peak flows are on the increase in catchments experiencing rapid urbanization (Gardiner *et al.*, 1995; Rosenthal and Hart, 1998; Ahmed, 1999). However, in many cases there is no corresponding expansion of drainage facilities to accommodate the increase. In due course, the existing drainage system becomes inadequate to take care of the flow leading to higher exposure to floods. Urbanization aggravates flooding by restricting where floodwaters can go, covering large parts of the ground with roofs, roads and pavements, thus obstructing natural channels, and by building drains that ensure that water moves to rivers more rapidly than it did under natural conditions (Action aide, 2006).Thus urbanization invariably increases the flood risk as a result of heightened vulnerability to floods due to concentration of population, wealth and infrastructure to smaller areas (Huong and Pathirana, 2013).

The increase in artificial surfaces of compounds of homes in urban areas results in a rise in flood events arising from poor infiltration and reduction of flow resistance. The hydrometeorological changes driven by urbanization, and resulting impacts on extreme rainfall, are also being established through research: a significant amount of studies over the last twenty years has shown a strong relationship between urban areas and local microclimate. The ―urban heat island‖ (UHI) effect is now well established, this is a situation where a lot of heat is generated from the activities of areas than surrounding regions (Seto and Kaufmann, 2009)

Urbanization invariably increases the flood risk as a result of heightened vulnerability stemming from population concentration even along encroached stream banks, wealth, and infrastructure to smaller areas (Taisuke et al., 2009). In densely populated cities, high volumes of anthropogenic waste heat is emitted from the activities of urban dwellers, and the increase of energy consumption through industrialization are posing environmental challenges, including the temperature rise in the urban atmosphere (Aikawa *et al.*, 2008). Cities can facilitate the buildup of rainstorms by means of the urban heat island effect: with industrialization in some cases, the temperature rise becomes significant facilitating increases in rainstorm activity. Obviously, higher intensities of rainfall activities may be recorded independent of climate change with its consequences on people (Eigege, 2014)

In urban areas, the percent imperviousness is largely governed by the population density; the larger the population density, the larger will be the percent imperviousness. Stankowski(1974) correlated percent imperviousness, I with the population density, PD_d, as $I = 9.6PD_d^{(0.573 - 0.0391 \log_{10} PD_d)}$ where PD_d is in persons per acre. In metric units, this is expressed as (Pandit and Gopalakrishnan, 1996); $I = 9.6PD_d^{(0.573 - 0.0391 \log PD - 0.398)}$ where PD_d is in persons per hectare (Mishra and Singh, 2003). $_{10\ d}$

Urbanization increases the surface area of imperviousness which results in increased runoff and rendering culverts provided along streams at road crossings inadequate with time if the level of urbanization was not anticipated. Flooding leads to overtopping and washout of road networks and results in devastating consequences on lives of communities, the very recent one occurring at Tamale where officials of NADMO have disclosed that a car was washed off the road resulting in the death of a former teacher, Hajia Fati Bapuni Amams, and three minors while hundreds of residents were displaced after a reported heavy rainfall that lasted for less than two hours on 25th Jul 2017 (https://yen.com.gh/96255-video-woman-4x4-car-washed-by-floods-tamale.html).

2.4.2 Inadequate Hydraulic Capacities of drainage System

Culverts are commonly used as relief culverts on slopes to channel water across a road and at natural stream crossings. They need to be properly sized and installed, and protected from erosion and scour. Natural drainages need to have culvert openings large enough to pass the expected flow plus extra capacity to pass debris without plugging (CARE, 2012). The Department of Urban Roads drainage manual allows for between 20% and 30% of flow as provision for debris and silt build up.

Inadequate hydraulic capacity of culvert results in overtopping of culvert and erosion of embankments, this could arise from insufficient culvert capacity and/or inefficient end sections. The inadequate capacity may be a result of inappropriate hydrologic analysis of flood peaks and volumes, and/or application of inappropriate culvert design criteria (http://www.conservationtech.com/FEMA-WEB/FEMA-subweb-flood/01-06FLOOD/2-Culverts/2A-Capacity.htm).

Urbanization results in increase run off due to changes in land use among others. This often renders culverts that were correctly sized decades ago inadequate in contemporary times. None maintenance of the culverts also results in reducing the capacity. It is, therefore, important that routine maintenance activities are carried out to clear the channels and culvert approaches of weeds and debris to ensure that they function at full capacity as designed.

Inconsistency of culvert dimensions at water crossings on road networks also creates its own type of flooding similar to flush flooding. Smaller sized culverts downstream of a water body with bigger sized upstream will allow faster passage upstream to rush to the restricted sections downstream where smaller sized culvert are placed. This creates flooding upstream of the smaller culvert with overtopping and wash out of parts of the road pavement. An example of this situation is the culvert on the Cocoa Street near Busy Mouth restaurant at Teshie -Nungua estate. On 3rd June 2015, the overtopped flood on this culvert was able to carry a car with it‘s two occupants into the main channeled drain. That was the unfortunate end of the two.

2.4.3 The Changing Climate in Africa

Climate change refers to any change in climate over time, whether due to natural variability or as a result of human activity (IPCC, 2001).

―Africa is not a driver of climate change, but a victim.‖ Commission for Africa (2005), *Action for a Strong and Prosperous Africa, London, page 249*. ―The weather is becoming increasingly volatile in Africa.‖ Commission for Africa (2005), *Action for a Strong and Prosperous Africa, London, page 51*.

Urban flooding has become more frequent due to a number of factors including climate change with the different patterns of precipitation, urban growth and an increase in paved surfaces (Eigege, 2014). Climatic changes seem to be increasingly changing flood trends in Africa. Research or studies revealed trends of rare large floods will change significantly than long-term average river flows. More hours of rainstorms may be experienced with changing climate (Mason and Joubert, 1997), expected rise in temperature are estimated to be accompanied with extreme weather events more often like rainstorms, floods, fires, hurricanes, tropical storms and El Niño events. There is also the possibility of strong winds in the tropics (typhoons and hurricanes) increasing in intensity, resulting in higher peak wind speeds and rainstorm events (IPCC (2007). West Africa has not been spared of the consequence. From 1995, effects of floods have been devastating in Ghana, especially in the national capital, Accra. Kumasi and recently Tamale have been spared of the unfortunate events, with many abandoning their houses for weeks or at times throughout the season of floods (Douglas, et al., 2008). Studies

show that more flooding should be expected due to climatic changes and variability as well as the population increase and growth in human settlement (Okyere *et al*., 2012; Ahadzie, and Proverbs, 2011). Integrated solution is therefore, necessary for effective solution to the annual events that have long lasting effect on people
(Okyere et al., 2012).

2.5 Effects of Urban Development on Floods

Land-use also influences the interception rates, the infiltration and soil water redistribution process, because agricultural practices and plant roots influence soil‗s saturated hydraulic conductivity and it also partly controls surface roughness, either by the vegetation structure or by the tillage of arable fields (Camorani et al., 2005). Land use changes also affect time of concentration.

Removing vegetation and soil for construction works, grading the land surface, and constructing drainage networks increase runoff to streams from rainfall. As a result, the peak discharge, volume, and frequency of floods increase in nearby streams. Encroachment on stream channels during urban development can limit their capacity to convey floodwaters. Roads and buildings constructed in flood-prone areas are susceptible to increased flood hazards, including inundation and erosion, as new development continues. Information about stream flow and how it is affected by land use can help communities reduce their current and future vulnerability to floods (Konrad, 2014).

The inundations that occurred are considered to be caused by man-induced land use, land cover changes. They are mainly associated with increasing urbanization and change of the agricultural practices (Camorani, et al. 2005). However, the different regional climatic conditions usually do not allow universally valid statements to be made about the land use impacts on the flood risk (Klo¨cking and Haberlandt, 2002). Urbanization levels are directly related to the level of economic development/activities and in some cases natural resources of a region, but it also changes water cycle in many aspects. Urbanizing is a major land use, land cover (LUCC) change trend for most regions of China in recent decades. It modifies hydrological process by affecting runoff generation and concentration which has been research focus currently (Amini et al., 2011; Delgado et al., 2009; O'Donnell et al., 2011; Wu et al., 2007). Rapid urbanization as evident from land use changes map results in increase in impervious areas within the catchment of Songor stream and has contributed to a corresponding increase in runoff. It is increasingly becoming obvious that the land-use change, especially urbanization has influenced hydrological attributes intensely. Flood characteristics variation could likewise increase flood risks and pose higher demand on water management (http://www.proc-iahs.net/370/33/2015/piahs-370-33-2015.pdf).

A study undertaken in China used event-based hydrological model (HEC-HMS) to investigate the impacts of land use change on runoff responses in Xiaoqing catchment. The catchment went through rapid urbanizing expansion between 1995 and 2008 and it was concluded that the relationship between urban development and hydrologic responses are complex at sub-basin scale. In general, more runoff was generated in the area with higher increase rate of build-up area but the increase magnitude of runoff did not present a simple linear relationship with the urban area expansion rate. It was also concluded that some sub-basins also showed opposite effect of urbanization on runoff. (http://www.proc-iahs.net/370/33/2015/piahs-370-33-2015.pdf).

Many studies have discussed the influence of land use changes on hydrology and sediment build up at different spatial and temporal scales (Van Rompaey et al., 2002; Siriwardena et al., 2006; Bakker et al., 2008; Casali et al., 2010; Tang et al., 2011). Multivariate statistics are commonly used to relate land use changes to the dynamics of stream flow or sediment yield (Hao et al., 2004; Zhang and Schilling, 2006; Zhang et al., 2007; Nie et al., 2011). We must therefore, investigate further how changes in each land use type influence stream flow and sediment build up to achieve a more effective and more accurate means of conducting watershed management and of predicting hydrological consequences of land use changes (Yan et al., 2013).

2.6 The Contribution of Urbanization and Climate Change to Future Urban Flooding

One major factor that influences flood is the climatic condition of a particular geographic location, and this manifests in the form of amount, duration and intensity of precipitation (rainfall). A combined precipitation and high temperature affect the soil moisture content (percentage saturation), liquid limit, infiltration rates etc. One of the consequences of global warming in humid environments is increases and changes in rainfall patterns (O‟Hare, 2002). Urban pluvial flooding frequency is expected to increase not only due to urbanization but also to expected climate changes (Ugarelli *et al.*, 2011; Simes *et al.*, 2014).

Climate change will likely increase the effects of urbanization on storm water runoff and further increase the quantity of runoff. Besides, climate change predictions point to scenarios for heavy precipitation events to increase in frequency and intensity (Bates *et al.,* 2008). Climate change is therefore likely to increase flood risk significantly and progressively over time. The areas with increased risk will be low-lying coastal areas. As sea levels rises and areas not currently prone to fluvial or tidal flooding experience more intense rainfall, it leads to significantly higher risk of flooding from surface runoff and overwhelmed drainage systems. This appears to be the reason behind flooding of parts of the coastal cities where settlements have extended onto river and stream basins, examples being parts of the Old Kasapreko community of Teshie – Nungua and Gleffe, all in Accra.

There is no doubt of the effects of climate change in adjusting the precipitation patterns in terms of distribution, intensity and duration of extreme rainstorm events and a higher frequency of higher intense precipitation. Due to higher temperatures and drought, lands have become more susceptible to runoff, exacerbating floods intensity. Changes in rainfall intensity and distribution influence river morphology (erosion of banks, fast sedimentation in riverbeds) which introduces more dynamic changes in flood patterns (Amangabara and Gobo, 2007). The sedimentation in river beds reduces the channel capacity to hold more runoff that flows into the stream channel leading to more frequent flooding.

Urban pluvial flooding frequency is expected to increase not only due to urbanization but also to expected climate changes (Ugarelli et al., 2011; Simes et al., 2014). This type of flooding can happen virtually anywhere and has the potential to cause significant damage and disruption in highly urbanized areas, where the density of properties, critical infrastructure and population is usually high. The volumes involved and the risk related to pluvial flooding often result in consistent economic losses and consequent damage in the long term due to the high frequency of this kind of event (Freni et al., 2010).

Studies showed that for the first time in human history, more than half of the world‘s population was leaving in cities as at 2008. The urban population is projected to reach 5 billion in 2030, which is 60% of the world‘s population (UN, 2006). The increase in population in urban areas occurs more in developing countries than in developed countries. Many cities in the developing world are growing rapidly due to real population growth, but to a much larger extent due to rural urban migration and transformation of rural settlements into cities. The result is an uncontrolled urban sprawl with increasing human settlements, industrial growth and infrastructure development (UN, 2006). Flood hazard also increases by hydrological and hydro climatological changes brought about by the land use and microclimatic changes driven by urbanization (WMO/GWP, 2008).

2.7 Effects of Urbanization on Time of Concentration

2.7.1 Land Surface Roughness

One of the most significant effects of urban development on overland flow is the lowering of retardance to flow enhancing higher velocities. Undeveloped areas with very slow and shallow overland flow (sheet flow and shallow concentrated flow) through vegetation become modified by urban development. Flow is then delivered to streets, gutters, and storm sewers that transport runoff downstream more rapidly. Travel time through the watershed is generally decreased (National Engineering Handbook, May 2010). Urbanization affects time of concentration as time runoff is held by areas with vegetation cover (lag time) is reduced.

The Soil Conservation Service (SCS) method (equation 2.1) for watershed lag was developed by Mockus in 1961. It spans a broad set of land surface conditions ranging from heavily forested watersheds with steep channels and a high percent of runoff resulting from subsurface flow, to meadows providing a high retardance to land surface runoff, to smooth surfaces and large paved areas.

$$Tc = \frac{l^{0.8}(S+1)^{0.7}}{1140Y^{0.5}} \qquad 2.1$$

where:

Tc = time of concentration, hr.

l= flow length, ft

Y = average watershed land slope, %

S = maximum potential retention, in

$$S = \frac{1,000}{cn'} - 10 \qquad 2.2$$

where: cn′ = the retardance factor, (National Engineering Handbook, May 2010). This method was used in estimating time of concentration at the time of constructing the culverts.

2.7.2 Retardance Factor

The retardance factor, cn , is a measure of surface conditions relating to the rate at which runoff concentrates at some point of interest and it depends on the land use and type of soil.

The retardance factor is approximately the same as the Curve Number (CN) as defined in NEH630.09, Hydrologic Soil-Cover Complexes. A CN between 50 and 95 is acceptable in the solution of equations 2.1 (Mockus 1961).

2.7.3 Applications and Limitations

The watershed lag equation was developed through a study that used data from 24 watersheds ranging in size from 1.3 acres to 9.2 square miles, with the majority of the watersheds being less than 2,000 acres in size (Mockus 1961). Folmar and Miller (2000) revisited the development of this equation using additional watershed data and found that a reasonable upper limit may be as much as 19 square miles (National Engineering Handbook, May 2010).

2.8 Effects of Flooding on Roads (Road Defects)

Urban and periphery urban communities will have to be connected with road networks together with other social infrastructure to be able to access schools, hospitals, market, etc. These road networks are exposed to flooding that takes place in the area if the community is within a flood prone environment. Susceptibility to floods often raises the water table of the area which sometimes comes with capillary action to the road pavement. The consequent defects are major and expensive to reinstate. This is the fate of road networks that are constructed at flood prone environment. They are either expensive to construct or they come with high maintenance cost or both due to high water tables, possible capillary actions and high possibilities of flooding and wash out.


Plate 2.1 Plate 2.2

Plate 2.1: A section of E Street at GREDA Estates showing a lot of road defects at culvert approach.

Plate 2.2: This inadequate culvert capacity and lack of regular routine maintenance are responsible for the road defects showing in Plates 2.1and 2.3; pictures were captured in 2017.


Plate 2.3 Plate 2.3A

Plates 2.3 and 2.3A are showing the other approach side of the culvert in plate 2.2 on the E Street at GREDA Estates before and after a downpour respectively.

Plate 2.3B: Trucks and cars were lined up on both approaches of the flooded section on the E Street waiting for the water level to subside for safe crossing at GREDA Estates.

Plate 2.

4: A ponded Cocoa Street at Teshie Nungua Estates before and after a downpour, vehicles and pedestrians had to abandon the otherwise busy street for their own safety during the period of flood.

Plate 2.5A: Section of road showing a lot of defect on Jasmine Street at Community 20, Tema. **Plate 2.5B**: This culvert‘s inadequacy is largely responsible for the defects on the Jasmine Street above.

2.9 Other Effects/ Economic Costs

Recent floods were reported to have cause loss of human lives. In some cases, they were the bread winners of families that died and the affected families grieved for long periods. This certainly impacted on both the immediate and extended families‘ social and economic well-being. It also results in outbreak of diseases especially cholera in affected areas which could be attributed to floods. Therefore, holistic solution to floods will impact on the health of the citizenry and increase productivity all year round (Okyere *et al.*, 2012).

Some water bodies also get contaminated during flooding and those who consume from such sources get exposed to cholera. The continuous annual road agency costs in reinstating the roads but only for a while and vehicle operating costs by road users on cost of repairs, wear and tear are very high. Where maintenance budgets are not immediately available, it leads to prolonged use of deteriorated roads before maintenance. Road agency cost and road user cost subsequently increase when timely maintenance is not undertaken.

2.10 ACCRA DRAINAGE BASINS

Rainstorm events in eight drainage basins in Accra namely; Densu, Lafa, Korle, Osu, Kpeshie, Songor Mokwe, Sakumo and Chemu results in floods which are facilitated by lined drains resulting in short response time with volumes of peak discharges. Rainfall patterns in Accra like other part of the country have changed within the past decades. According to IPCC scenario B1, A1B, and A2 forecasted an increase in the average monthly rainfall values from 160 mm in 1991-2010 to 200 mm in 2011-2020. A ten (10) year return period was employed to model the rate of urban damage which is more conservative compared to 25 and 50 years return period estimates. Analysis shows impact on Gross Domestic Product (GDP) and the areas to be affected in Accra, (Asumadu-Sarkodie et al., 2015). The change in precipitation is confirmed by the revised Intensity Duration Frequency (IDF) curves by Ministry of Roads and Highways of Ghana in 2016. A report by United Nations Regional Coordinator in Dakar (October: 2007), put the July 2007 floods in West Africa as the worse in the last thirty years. The evaluation was based on over 210 deaths recorded with more than 785,000 people affected **(**Asumadu-Sarkodie et al., 2015).

2.11 Road Drainage System

The side (longitudinal) and cross drains (culverts and bridges) constitutes the fundamental requirement of any urban drainage network. The road drainage system has four main functions:

i. It is to intercepts ground and surface water flowing towards the road; ii. It is to convey storm water from the surface of the carriageway to outfalls; iii. It is to control the level of the water-table in the subgrade beneath the carriageway; iv. It is also to convey water across the alignment of the road in a controlled fashion.

The first three functions are performed by longitudinal drainage, in particular side drains, while the fourth function requires cross-drainage structures, such as culverts, fords, drifts, and bridges. (Camorani et al., 2005). Bigger sized underground storm drains are often provided in a built-up environment in the urban areas with restricted space and pedestrian safety has to be protected. The smaller side drains and cross drains are then made to discharge into the storm drain for further conveyance. Culverts were used as cross drains at the study area therefore, for the purpose of this study issues of culverts are considered.

Roads will affect the natural surface and subsurface drainage pattern of a watershed as the cut across individual hill slopes. Road drainage design has as its basic objective of reducing or eliminating the energy generated by flowing water. When water is allowed to develop sufficient volume or attain erosive velocity, it could cause excessive wear along ditches, below culverts, or along exposed running surfaces, in cuts, or fills. Provision for adequate drainage is of paramount importance in road design and cannot be overemphasized (Mukherjee, 2014).

2.12 Hydrology

Hydrology is the scientific study of the properties, distribution, and effects of water on earth's surface, in the soil, underlying rocks, and in the atmosphere (AHD, 2000). Hydrologic analysis is needed in designing highway drainage in order to determine the amount and frequency of flows. Drainage facilities must not only be hydraulically efficient, but must be taken into consideration in relation to the importance of safety, roads, cost, environmental impacts, maintenance, aesthetics, and legal responsibilities

(CDOT, 2004). In the design of highway drainage, floods are usually considered in terms of peak runoff or discharge in cubic meters per second (m3/s). The peak discharge is used to design storm drain systems, culverts, and bridges. The analysis of peak rate of runoff, volume of runoff, and time distribution of flow are important for highway drainage design.

Any errors in the estimation of these data would result in related errors in the design of the structure, which could be either undersized, causing drainage problems, or oversized, resulting in an unnecessarily large cost for the structure. Because hydrology is not a precise science, different hydrologic methods were developed for determining flood runoff. Engineering judgment and certain federal or state agencies may require (or local agencies may recommend) different techniques to select the proper method for computing the runoff (Tawfiq, 2012). Some hydrological factors that impact on flooding are; water cycle, water storage capacity of a basin, drainage capacity, rainfall intensity and catchment area and characteristics.

2.13 Culverts

A culvert is a conduit which conveys stream flow through a roadway embankment or past some other type of flow obstruction. A culvert can also be used to discharge at a controlled flow rate when a detention pond is constructed upstream to regulate flow which will reduce a storm runoff peak. In such situations, the embankment that the culvert penetrates should effectively be designed as a dam. Culverts are constructed from a variety of materials and are available in many different shapes and configurations. Culvert selection factors include roadway profiles, channel characteristics, flood damage evaluations, construction and maintenance costs, and estimates of service life. (Hydraulic Design of Highway Culverts Third Edition, April 2012).

2.13.1 Shapes of Culverts

Numerous cross-sectional shapes are available for both closed conduit and open-bottom types of culverts. The most common closed conduit shapes are circular and box (rectangular) which are mostly used in Ghana, elliptical and pipe-arch. These typically manufactured culvert shapes have the same material on the entire perimeter. Shape selection is based on the volume of flow, limitation by the roadway embankment height, the limitation on upstream water surface elevation, hydraulic performance and cost of construction.

2.13.2 Materials

The selection of a culvert material may depend upon structural strength, hydraulic roughness, durability (corrosion and abrasion resistance), and constructability. The most commonly used culvert materials are concrete (both reinforced and nonreinforced), corrugated metal (aluminum or steel) and plastic (high-density polyethylene (HDPE) or polyvinyl chloride (PVC). Under harsh environmental conditions, Sulphur resistance cement may be recommended to be used for the concrete.

2.14 General Planning Considerations

2.14.1 Drainage Master Plan

This is to ensure that proposed structures fit into the relevant major drainage way master plan. Care should be taken to ensure that residents along the stream are not adversely affected by proposed hydraulic conditions and this can be achieved through scenario analysis modeling.

2.14.2 Safety Concerns

Specific public safety issues related to the culvert location should be addressed. Culverts are often located in valley and in sags of road designs. Large box culverts in embankments can create conditions where there is a significant falling hazard, which poses risk to the public. In such cases, fencing or guardrails for public safety is recommended.

2.15 Specific Design Considerations

2.15.1 Siting of Culvert

Identifying areas to introduce a culvert is an integral part of roadway design. After the geometric design, the Drainage Engineer or responsible officer should identify all live stream crossings, springs, low areas, gullies, and impoundment areas created by the new roadway embankment for possible culvert locations. The culvert should be located as not to change the prevailing stream alignment and be aligned for ease of entry and exit by the stream. Sudden changes in direction at either end may reduce capacity and encourage siltation which has to be compensated for with a larger structure. In some situations, smooth entry and exit can be achieved by channel realignment, skewing the culvert, or a combination of the two.

2.15.2 Design Flood Frequency and Discharge

The design flood frequency for culverts is closely related to the pavement encroachment and road overtopping criteria. The Ministry of Roads and Highways adopts Design Storm Selection Guidelines (AASHTO, 1999) minimum design

frequencies as presented in table 2.1 below. This agrees with the Department of Urban Roads Highway Drainage Manual, Second Edition (August 2006). The functional classification of the road determines the design frequency to apply. The required hydraulic capacity (design discharge) is based on the time of concentration, the selected return period and the resulting design flow rate calculated for the watershed to the proposed culvert.

2.15.3 Time of Concentration

For natural catchments and mixed flow paths, the time of concentration can be found by using the Bransby-Williams' Equation which includes the time of overland flow and channel flow:

$$t_c = \frac{F_c.L}{A^{1/10} S^{1/5}} \qquad 2.2$$

where;

t_c = Time of concentration (minutes)

F_c = Conversion factor = 92.5

L = Length of flow path from catchment divide to outlet (km)

A = Catchment area (ha)

S = Slope of stream flow path (m/km)

http://msmaware.com/blog/upcoming-msma2-workshop-at-wisma-iem-pj/

Thus, $$t_c = \frac{92.5\,L}{A^{0.1} S^{0.2}} \qquad 2.3$$

The correct intensities at the various pour points along the channel were determined, using the ten (10) years return period mmaximum rainfall intensities (mm/hr) IDF curve for Accra, (Ministry of Roads and Highways consultancy services for coordination of the review of the Intensity Duration Frequency (IDF) of The Rainfall Intensity Graph Draft Report, July 2016). The 10 years return period gives a probability of flooding in any particular year to be 0.1 or 10%. Rainfall Intensities are inversely related with time of concentration. As the t_C goes up, ―I‖ goes down.

2.15.4 Estimation of Runoff

The two methods recommended by the Ministry of Roads and Highways Standard Technical Specification for road works are the Soil Conservation Service Curve

Number (SCS-CN) and Rational Method. The Soil Conservation Service Curve Number (SCS-CN) is a well-established loss-rate model to estimate runoff. It combines watershed parameters and climatic factors in one entity curve number (CN). The CN exhibits an inherent seasonality beyond its spatial variability, which cannot be accounted for by the conventional methods (Gundalia, and Dholakia, 2014).

The Natural Resources Conservation Service (NRCS) curve number (CN) procedure is widely used to estimate runoff resulting from rainfall. The primary reason for its wide applicability and acceptability lies in the fact that it accounts for major runoffgenerating watershed characteristics, namely, soil type, land use/treatment, surface condition and antecedent moisture condition. In contrast, the main weaknesses reported in literature are that the SCS-CN method does not consider the impact of rainfall intensity; it does not address the effects of spatial scale. It is highly sensitive to changes in values of its single parameter CN, and it is ambiguous considering the effect of antecedent moisture conditions, (Gundalia, and Dholakia, (2014).

2.15.5 Rational Method

The Rational Method was originally developed for estimating peak discharge for sizing drainage structures such as culverts when the total upstream catchment area contributing to runoff at the point of consideration is less than 100 acres. The Rational method has been modified to match the results obtained from HEC-1 in the Clark County area for areas greater than the recommended 100 acres. (*Clark County Regional Flood Control District Hydrologic Criteria and Drainage Design Manual Section 600, Storm Runoff; Adopted August 12, 1999*).

The Modified Rational method (MRM) is a method to parameterize simple runoff hydrographs. The MRM produces a runoff hydrograph (and volume) while the original rational method produces only the peak design discharge. It is also used to size detention and retention facilities for a specified recurrence interval and allowable outflow rate. The MRM which was used in computing flow at the various pour points of the culverts is stated below:

$$Q = 0.278\, C_R C I A \qquad 2.4a$$

Where;

Q = Peak rate of design discharge (m^3/s)

C = Run-off coefficient (dimensionless)

I = Rainfall intensity (mm/hr)

C_R = Area Reduction Factor (ARF) (dimensionless)

A = Catchment area (km^2)

The ARF, C_R, is determined using Chow (1964) relation:

$$C_R(ARF) = \left[1 + \frac{1.93A}{10^3}\right]^{-1} \qquad 2.4b$$

The Modified Rational Method was, therefore, opted for in this research.

2.15.5.1 Assumptions of The Rational Method

The following are the basic assumptions made when applying the Rational Formula method;

1. The computed maximum rate of runoff to the design point is a function of the average rainfall rate during the time of concentration to that point.
2. The maximum rate of rainfall occurs during the time of concentration, and the design rainfall depth during the time of concentration is converted to the average rainfall intensity for the time of concentration.

3. The maximum runoff rate occurs when the entire area is contributing flow. However, these assumptions are modified from time to time when local rainfall/runoff data was used to improve calculated results.

2.15.5.2 Limitations on Methodology

The Rational Formula method adequately approximates the peak rate of runoff from a rainstorm in a given basin based on the assumptions. The critics of the method usually are unsatisfied with the fact that the answers are only approximations. A shortcoming of the Rational Formula method is that only one point on the runoff hydrograph is computed (the peak runoff rate). A local factor (Area reduction factor, C_R) has been developed to match the results from the Rational Method to those obtained using HEC1. A disadvantage of the Rational Formula method is that with typical design procedures one normally assumes that all of the design flow is collected at the design point and that there is no "carry over water" running overland to the next design point. However, this is not the fault of the Rational Formula method, but of the design procedure.

2.15 .5 .3 Rainfall Intensity

The rainfall intensity, I, is the average rainfall rate in millimeters per hour for the period of maximum rainfall of a given frequency having a duration equal to the time of concentration. After the design storm frequency has been selected, a graph should be made showing rainfall intensity versus time (*Clark County Regional Flood Control District Hydrologic Criteria and Drainage Design Manual*, *Section 600, Storm Runoff; Adopted August 12, 1999*).

2.15.5.4 Runoff Coefficient

The runoff co-efficient, C, represents the integrated effects of infiltration, imperviousness, evaporation, retention, flow routing, and interception, all which affect the time distribution and peak rate of runoff. Determination of the coefficient requires judgment and understanding on the part of the engineer. The rate of urbanization in the various sub catchments were also determined through the use of land use, land cover change maps of January 1991, January 2001, January 2011 and January 2016 successive years of the area. Urbanization is believed to be a major factor for increased run off. Experience of the soil texture of many parts of Accra showed that clay content is more than 20% and therefore, is classified in Groups C and D of Hydrologic Soil Groups (HSG). This therefore, was applied to the catchment area of the Songor Stream. For storm return period of 10years, C of the catchment area is given by;

$$C = 0.74IM + 0.132 \qquad 2.5$$

For storm return period of 25 years C of the catchment area is given by;

$$C = 0.56IM+0.319 \qquad 2.6$$

Where:

IM = % imperviousness (divided by 100) (Urban Storm Drainage Criteria Manual Volume 1, March 2017). The 10 years return period equation for C was used in the estimation of discharge, equation 2.5.

Thus,

$$C = 0.74IM + 0.132$$

This also satisfies the provision in the Ghana Highway Authority Design Guide Table 6.4.3. Runoff Coefficient (C) in town is between 0.6 – 0.9. Combining the outcomes of equations 2.4b and 2.5 into 2.4a gives;

$$Q = 0.278\ C_RCIA \qquad 2.7$$

For HSG B soils with clay content between 10% and 20%, the following models can be applied for return periods of 10 years and 25 years respectively;

$$C = 0.81IM+0.057 \qquad 2.8$$

$$C = 0.63IM+0.249 \qquad 2.9$$

Graphs of various runoff co-efficient (C) through land use changes were plotted with their corresponding years for the various sub catchments and it is the polynomial which accurately describes the imperviousness of the sub catchments with regression coefficient (R^2) between 0.96 and 1. From the polynomial models, the C's for the design year of the existing culverts were determined.

Where drainage areas are composed of parts having different runoff characteristics, a weighted coefficient for the total drainage area is composed by dividing the summation of the products of the area of the parts and their coefficients by the total area.

ie $C_w = C_1A_1 + C_2A_2 + \ldots C_nA_n / A_t$ (Johnson and Chang, 1984).

A disadvantage of the Rational Formula method is that with typical design it is assumed that all the flow is channel through a drainage structure, example been culvert at the design point and that there is no over flow or "carry over water" running overland to the next design point. However, this is not the fault of the Rational Formula method, but of the design procedure. The problem becomes one of routing the surface and subsurface hydrographs which have been separated by the storm sewer system. (*Clark County Regional Flood Control District Hydrologic Criteria and Drainage Design Manual, Section 600, Storm Runoff; Adopted August 12, 1999*)

2.15.6 Permissible Headwater Depth for Culverts

Culverts normally constrict/ limit the original stream flow, which causes a rise in the upstream water level, the headwater level. The elevation of the upstream water surface is termed *headwater elevation*. The depth of headwater is measured from the invert of the culvert inlet to the headwater elevation for a known event. In selecting the design headwater elevation, the following should be considered:

1. The headwater depth /culvert diameter ratio (*HW/D*) should not exceed 1 - 1.5 range for the event peak flow unless there is justification and sufficient measures are taken to protect the culvert inlet. In urban environment where embankments are usually low, HW/D ratio of more than 1.2 should not be entertained as this will lead to culvert overtopping. Piping failure can be of concern for deep headwater depths, especially if there are animal burrows in the embankment.
2. Assess the impacts as a result of exceeding the design headwater depth, including:
    a. Hazard to human life and safety.
    b. Potential damage to the culvert, embankment stability and roadway.
    c. Traffic interruption in the event of roadway overtopping.
    d. Anticipated upstream and downstream flood risks, for a range of return frequencies.
3. The elevation of the watershed divides should be higher than the design headwater elevations in order to prevent the headwater from spilling into adjacent watersheds. In flat terrain, watershed boundaries are often poorly defined, and culverts locations should be carefully identified and sized to prevent disruption to the existing flow paths together with spillover into adjacent watersheds due to culvert backwater effects.

2.15.7 Tailwater Depth of Culverts

Tailwater depth constitutes the flow depth downstream of the culvert which is measured from the invert of the culvert outlet to the water surface (assuming normal flow downstream. Knowledge of tailwater depth is critical for culvert design for efficient functioning, thus submerged outlet may cause the culvert to flow full instead of partially full.

2.15.8 Acceptable Outlet Velocity for Culverts

The outlet velocity of a culvert is usually higher than the maximum velocity allowable velocity that the original basin can withstand without experiencing significant erosion of the bed and/or banks. Culverts should be designed to have adequate outlet protection (typically riprap or a stilling basin) and should not be overlooked at design stage. Allowable velocities should be such that the stream channel is protected from erosive velocities at all times. As a general rule, the velocity at the downstream edge of a project right-of -way should not be greater than the pre-construction velocity. High outlet velocities may be reduced by increasing the barrel roughness and choosing acceptable slope since slope and roughness are the principal factors affecting the outlet velocity. Aslope of 1% is normally recommended. The acceptable range of velocity is

0.6 – 3m/s (Ghana Highway Authority Road Design Guide Table 6.5.1).

2.15.9 Environmental Permitting

Environmental permitting constraints often prevail when applications are made for new culverts or retrofits as well as for construction of bridges. An environmental permit is often required when a new culvert is installed.

2.15.10 Fish Passage and Culverts

Aquatic life in some of the water bodies is an important factor to consider. At some culvert locations, the ability of the structure to accommodate migrating fish is an important design consideration. For such sites, fish and wildlife agencies should be consulted early in the planning process. Some situations may require the construction of a bridge to span the natural stream. However, culvert modifications such as over sizing the diameter to allow for fish movement or rise of the culvert, constructing it to allow for native stream bed material desired the fishes to meet the design criteria established by the regulatory agencies such as the fish and wildlife agencies.

2.16 Other Design Considerations

Other design considerations include the following:

1. The impacts of various culvert sizes, dimensions, and materials on upstream and downstream flood risks, including the implications of embankment overtopping
2. Sediment load and bed load can be anticipated for the culvert. For streams with a heavy bed load, abrasion and debris blockage can be of concern. For culverts with milder slopes or sudden changes in slope within the culvert, filling in of culverts with sediment can be problematic and lead to increased maintenance frequency.

2.16.1 Deposition Considerations

Deposits in culverts may also occur due to the following conditions:

When moderate flow rates prevail, the culvert cross section may be larger than that of the stream so the flow depth and sediment transport capacity is reduced. Placing of culverts in curves of stream channel will be subjected to debris deposition. This effect is most pronounced in multiple cells culverts with the cell in the inner part of the curve and the dividing walls trapping garbage and other waste material that flows in the stream. Structural and geotechnical considerations are very necessary for project success.

2.17 Hydrologic Considerations

In highway designing, the fundamental objective is the runoff and most importantly the water bodies that ultimately cross transportation arterials. In every hydraulic design to size culverts, the flow that can safely be channel through the culvert should be estimated first through hydrological computations.

2.18 Parts of Hydrologic Cycle Relevant to Stream Crossings

The main elements of hydrology to drainage design are:

1. rainfall,

2. infiltration,
3. storage,
4. runoff

2.18.1 Surface Runoff Process

Runoff entail processes of rain falling into the catchment, abstraction through infiltration and storage and the excess moving on the surface into stream channels. It is obvious that it is a fraction of the rainfall that flows as runoff.

2.18.2 Aspects of Precipitation

The aspects of precipitation that are important to road drainage are the;

1. intensity (rate of rainfall)
2. duration
3. time distribution of rainfall
4. storm shape, size, and movement and
5. Frequency (of occurrence).

The intensity determines the rainfall rate to use and makes it the most important aspect of precipitation. All things being equal intensity is directly related to discharge. Rainfall-time relationship can be shown in the form of intensity hyetographs. The duration of rainfall determines the runoff to be generated. High initial intensities of rain lead to a sudden increase in runoff with a long recession of the flow. Initial high intensity and prolonged rainstorm is a recipe for flush flood conditions. On the other hand, if the high intensity falls close to the end it takes much longer time to peak followed by a sudden falling time.

Storms that are close to pour points in a catchment will have shorter time of concentration than when it‘s located further away. Flood events easily occur under this situation and are worsen with prolong rainstorms. Similar rainstorm further away from the pour points will have a longer time of concentration with lower peak flow due to infiltration and storage in the catchment.

Catchment shape also has a comparable effect on the runoff distribution especially if the catchment is long and narrow compared to short and circular catchments. Besides, storm movement affects rainfall distribution especially long and narrow catchments, rainstorms moving up a in a catchment from the pour points give distributions that is relatively symmetrical while reserve movement will normally give a higher peak flow and an unsymmetrical distribution with the peak flow occurring later in time.

The frequency is equally important as it provides guide to how computed flows based on selected return periods are likely to be equal or exceeded. The main purpose is to establish the frequency of occurrence of significant runoffs especially the frequency of the peak discharge. The traffic volume hence the importance of a road is a factor in selecting a particular frequency of occurrence.

2.18.2.1 Relationship Between Rainfall, Intensity, Duration and Frequency

The important aspects that are interrelated in any drainage design are;

(i) rainfall intensity

(ii) duration and

(iii) frequency

For good interpretation and design analysis they are combined graphically into the intensity-duration-frequency (IDF) curves. For every selected frequency, there is its corresponding intensity and duration curve. There are tabular forms that could be read. IDF curves are location related due to differences in rainfall. The differences, slight though they may be, are reflection of different elements of rainfall at different locations.

Due to the variations at different areas, local IDF curves reflecting the area rainfall elements must be used for hydrologic design work (Johnson and Chang, 1984).

Both the Ghana Meteorological Services Department Departmental Note 23(Maximum Rainfall Intensity - Duration Frequencies in Ghana) by Dankwa (1974) and Ministry of Roads and Highways Consultancy Services for Coordination of The Review of The Intensity Duration Frequency (IDF) of The Rainfall Intensity Graph, (July 2016) were used for the determination of intensity at construction year and for current assessment respectively.

2.18.3 Peak Flow

Peak flow, also called peak discharge, is the maximum runoff in cubic meters per second that passes a pour point in a catchment as a result of rainfall events. In designing any drainage system, the interest should be the peak flow at a point in the catchment owing to the fact that the size of the structure is depended upon the peak flows.

Accuracy of judgment of peak flow, to a large extent results in accurate sizing.

2.18.4 Frequency of Occurrence

The frequency of occurrence or exceedence frequency is indicative of the frequency at which flood of certain magnitude is expected to occur in a time period which could be expressed in percentage or ratio. The frequency of occurrence relates to the probability of attaining or exceeding the specified flood limit in any year. Once again the traffic volumes, hence importance of a road will determine the acceptable level of risk of exposure when designing a culvert.

2.18.5 Return Period

This is the average time interval whose maximum rainstorm event or flood is used in drainage design. The probability of occurrence or exceedence probability (p) and return period (T) are inversely related as shown in equation 2.10.

$$T = \frac{1}{P} \qquad 2.10$$

2.19 Channelization

Channelization creates improved access to streamline flow in a stream basin. It increases the hydraulic efficiency of the main channel flow thereby, impacting on that of the tributaries. Channelization therefore, prevents or minimizes localized floods challenges. Many related studies attest including the one by Liscum and Massey (1980) to the fact that the benefits of channelization may be comparable to the increase of imperviousness. This implies that channelization can be used to minimize on the effect of increase in imperviousness. (FHWA, October 2002).

2.20 Detention Ponds

Detention ponds are temporal storage facilities usually meant to reduce the effects of peak flows. However, there is no simple means of determining the effects of the ponds at a specific location. Open spaces along stream channels could be assessed for the possibility of constructing storage ponds to alleviate the effects of floods especially on coastal communities.

2.21 Diversions, Existing and Proposed Dam Construction

The highway or drainage engineer needs to investigate on previous water diversions, existing and proposed dams to be constructed on the watershed owing to the fact that these facilities will significantly impact on the quantum and nature of the runoff reaching the highway crossing. Consideration should be given to proposals under study by the various water related agencies working in that area and how it can impact on the catchment. Effects of decommissioning of dams on drainage of highways downstream should also be considered.

2.22 How Basin Characteristics Affect Runoff

2.22.1 Catchment Area

The size of a catchment determines the amount of water that it can collect as runoff which makes it the most significant factor when considering runoff. The bigger the catchment area, the more the runoff that can lead to flooding will be. In the entire hydrological models, peak flow and catchment area are directly related.

2.22.2 Slope

Steep slopes increases the speed of runoff compared to gentle slopes, resulting in rapid runoff responses and consequently higher peak discharges. When slopes are steep, runoff is quickly removed from the watershed and could create a supercritical flow condition. The steep slopes do not enhance infiltration resulting in increased volume of runoff compared with gentle slopes. Mild and flat slopes allow more time for abstractions as the surface water takes much longer time to flow which reduces the volume of runoff.

2.22.3 Hydraulic Roughness

The roughness of the ground surface has an influence on the amount of rainwater that is lost through infiltration, it also determines the depth and speed of flow on a natural ground, in lined drains, through a culvert or channelized earth drains. Smooth surfaces reduce the response time of a drainage channel which therefore, affects the channel storage capacity. The higher the roughness of the surface, the longer it takes to peak and the lower the peak volume of a rainstorm.

2.22.4 Storage

Storage features impacts on the response time of a given rainfall event. They can hold a lot of water back in the catchment which reduces peak volumes considerably. Some of the storage features are marshes areas, mangroves, lakes, dams, storage in floodplains of large, wide rivers and streams and manmade constrictions in the watersheds that retain or cause backwater.

2.22.5 Channel Length

The length of a channel is an important factor to consider in a catchment. Longer channels require more time for water to flow from the remotest point of the catchment to the inlet of designed drainage structure example culvert, all other factors remaining the same. This relates directly to time of concentration. Consequently, a catchment with shorter channel length usually has faster response time compared with one with longer channel length.

2.23 Peak Flow for Ungauged Sites

The use of frequency approach in determining peak flow is the most appropriate option. However, there is lack of sufficient stream gauging data or often times no records available for flood frequency analysis. Empirical methods like the rational method and the NRCS (formerly the SCS) graphical method are used in the absence of gathered data. Both methods use empirical models that relate rainfall with runoff and are used in the computation of design discharges of ungauged catchments using parameters that replicate the catchment.

2.24 Selection of Design Flood

There are two main ways by which the return period (probability of exceedance) in hydraulic design of bridges and culverts can be selected;

1. Using policy of implementing agency - permissible preselected recurrence interval.
2. Situation analysis - choosing recurrence interval that is most cost effective and best satisfies the specific site conditions and associated risks.

Either of these alternatives may be used exclusive of the other however, in practice both alternatives are jointly considered in selecting the flood frequency for hydraulic design. The AASHTO (1999) recommended table below is adopted by the Ministry of Roads and Highways and its agencies. This is also collaborated by the Department of Urban Roads of Ghana‘s Drainage Manual, 2006.

Table 2.2: Guidelines for the Selection of Design Storm

Roadway Classification	**Probability of Exceedence**	**Return Period**
Rural Principal Arterial System	2 %	50 – years
Rural Minor Arterial System	4% - 2%	25 – 50 years
Rural Collector System, Major	4%	25 – years
Rural Collector System, Minor	10%	10 – years
Rural Local Road System	20% - 10%	5 – 10 – years
Urban Principal Arterial System	4% - 2%	25 – 50 years
Urban Minor Arterial Street System	4%	25 – years
Urban Collector Street System	10%	10 – years
Urban Local Street System	20% - 10%	5 – 10 - years

*Adapted from AASHTO, 1999.

The selection of a design flood with a lesser or greater peak discharge may be warranted and justified by economic analysis. The road networks that the Songor stream crosses are mostly urban collector roads and urban local streets. The Tema Beach road is, however, a minor arterial. In selecting the probability of exceedence, the importance of the road to traffic distraction is considered.

2.25 Culvert Hydraulics

2.25. 1 Culvert Selection

Factors influencing type of culvert to select include road profile, nature of the channel, ease of construction, construction and future maintenance costs and design life. In rural environments where concrete pipes are not easily available, the use of U culverts is prevalent.

2.25.2 Hydraulic Design of Culverts

Having estimated the design discharge (Q), the safe conveyance of the flow across the road has to be assessed and it is referred to as hydraulic design. A culvert is a type of hydraulic structure whose flow is complex to analyse because it is usually non-uniform. Hydraulic jumps are sometimes experienced within the barrel or beyond the outlet of the culvert. The type of flow within the barrel changes with changing flow rate and tailwater elevations. A thorough hydraulic analysis therefore, involves considering all the factors identified as relevant in the study of hydraulics.

2.25.3 Types of Culvert Flow

The flow and control types considered when designing culverts are;

1. Inlet Control – A lot of the culverts operate under this, a situation where the culvert barrel can pass more flow than the inlet can take. This condition results in supercritical flow within the culvert barrel where the flow is swift. Under this condition, a submerged outlet creates a hydraulic jump situation in the barrel.
2. Outlet Control – A culvert is said to be flowing under this condition when the barrel is not able to convey all the flow that the inlet will accept. This normally happens when the culverts outlet is submerged resulting in subcritical flow within the culvert barrel, thus flow rate varies from slow to gentle. Outlets control conditions can also be experience when the outlet is not submerged.

Each control type has its formulas and factors to apply in computing the hydraulic capacity of the structure. Table 2.3 below shows the factors relevant under each of the controls (FHWA Hydraulic Design Series No. 5, "Hydraulic Design of Highway Culverts."). The flow through the culverts in Songor stream catchment area is under inlet control.

Table 2.3 Factors affecting inlet and outlet controls

Factor	Inlet Control	Outlet Control
Headwater	X	X
Area	X	X
Shape	X	X
Inlet Configuration	X	X
Barrel Roughness	-	X
Barrel Length	-	X
Barrel Slope	X	X
Tailwater	-	X

It should be noted that for inlet control the area and shape factors relate to the inlet area and shape and for outlet control they relate to the barrel area and shape.

2.25.4 Key Hydraulic Principles

The discharge capacity of the culvert is ascertained using the Manning‘s equation below.

From the Manning‘s Equation;

$$Q = \frac{A}{n}\ x\ R^{2/3} x\ S^{1/2} \qquad 2.11$$

Where:

Q = flow rate or discharge (m^3/s) n =

Manning roughness coefficient below (Table 2.2)

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

R = hydraulic radius (m)

S = longitudinal slope (m/m)

Also from the Continuity Equation;

$$Q = v_1 A_1 = v_2 A_2 \quad 2.12$$

Where:

Q= flow rate or discharge (m^3/s)
v = velocity (m/s)

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

Subscripts refer to two different locations within a culvert or channel between which flow is constant. The continuity equation will ensure that the capacity of the discharge in the stream channel is catered for in the culvert. Also from the Energy Equation,

$$\frac{v^2}{2g} + \frac{P}{\gamma} + z + \text{losses} = \text{constant} \quad 2.13$$

Where:

v = velocity (m/s) g = gravitational acceleration ($9.8 m/s^2$) p = pressure (N/m^2) γ = specific weight of water ($9.807 kN/m^3$). (Note: p/γ = pressure head or depth of flow [m]) z = height above datum (m).

Table 2.4: Manning's roughness coefficients

	Reinforced Concrete Pipe (RCP	Aluminized Steel Pipe (ASP	Polymer Coated Steel pipe	Corrugated Aluminum Pipe	Polyvinyl Chloride Pipe (PVC)	High Density Polyethylene Pipe (HDPE)

Manning's Roughness Coefficient	0.013	0.013	0.013	0.013	0.013	0.011

For concrete culverts, the n = 0.013

2.25.4.1 Outlet Control Flow Conditions

The outlet control flow conditions can be calculated based on energy balance. The total energy (HL) required to pass the flow through the culvert barrel is made up of the entrance loss (He), the friction losses through the barrel (Hf), and the exit loss (Ho). In addition bend losses (Hb), losses at junctions (Hj), and loses at grates (Hg) are included. Therefore,

$$H_L = H_e + H_f + H_o + H_b + H_j + H_g \qquad 2.14$$

The velocity in the barrel is calculated as follows:

$$V = \frac{Q}{A}$$

V = Average velocity in the barrel of culvert, (m/s), Q = Flow rate, (m3/s), A = Full cross sectional area of the flow or culvert (m2). The velocity head is:

$$H_v = \frac{V^2}{2g} \qquad 2.15$$

g is the acceleration due to gravity (9.8 m/s2). The culvert entrance loss is a function of the velocity head in the barrel, which can be expressed as a coefficient times the velocity head.

$$H_e = k_e \left[\frac{V^2}{2g}\right] \qquad 2.16$$

Values of k_e based on various inlet configurations. The friction loss in the barrel is also a function of the velocity head. Using the Manning equation, the friction loss is:

$$H_f = \left[\frac{K_u n^2 \; L}{R^{1.33}}\right] \frac{V^2}{2g} \qquad 2.17$$

K_u = 19.63 in SI units n = Manning roughness coefficient (for composite roughness n_c)

L = length of the culvert barrel (m)

R = Hydraulic radius of the full culvert barrel A/ p (m) A = Cross sectional area of the culvert barrel (m^2)

p = Perimeter of the culvert barrel (m)

V = Velocity in the culvert barrel (m/s)

$$n_c = \left[\sum_{i=1}^{G}(pn_i^{1.5})\right]^{0.67} \quad 2.18$$

n_c = Composite or weighted Manning‘s roughness coefficient, n

G = Number of different roughness materials in the perimeter.

P_1 = Wetted perimeter in meters influenced by the material 1

P_2 = Wetted perimeter in meters influenced by the material 2 etc.

n_1 = Manning‘s roughness coefficient n for material 1, n_2 is for material 2 etc. p = Total wetted perimeter in meters.

The exit loss is a function of the change in velocity at the outlet of the culvert barrel. For a sudden expansion such as an endwall, the exit loss is:

$$H_e = 1.0\left[\frac{V^2}{2g} - \frac{V_d^2}{2g}\right] \quad 2.19$$

V_d is the channel velocity downstream of the culvert in (m/s). A factor of less than 1.0 is used to normalize the overestimation of the exit losses (FHWA 2006a) for a transition loss. The velocity downstream is usually neglected hence equation 2.20 below.

$$H_o = H_v = \frac{V^2}{2g} \quad 2.20$$

Equation 2.20 is the standard option used in computer programs. The Utah State University (USU) Method which is the alternate to equation 2.20 is showed in equation 2.21 below.

$$H_o = \frac{(V - V_d)^2}{2g} \qquad 2.21$$

Other losses are often avoided through careful selection of appropriate locations for the culvert. When unavoidable, they are added to the total losses using equation 2.14.

Inserting the above relationships for entrance loss, friction loss, and exit loss (Equation 2.19) into Equation 2.1 4, the following equation for barrel losses (H) is obtained:

$$H = \left[1 + k_e + \frac{k_u n^2 L}{R^{1.33}}\right] \frac{V^2}{2g} \qquad 2.22$$

Figure 2.1 below depicts both the energy and the hydraulic grade lines for full flow in a culvert barrel. The energy grade line represents the total energy at any point along the culvert barrel. The headwater depth, HW_o, is the depth from the inlet invert to the energy grade line. The hydraulic grade line is the height to which water would rise in vertical tubes connected to the sides of the culvert barrel.

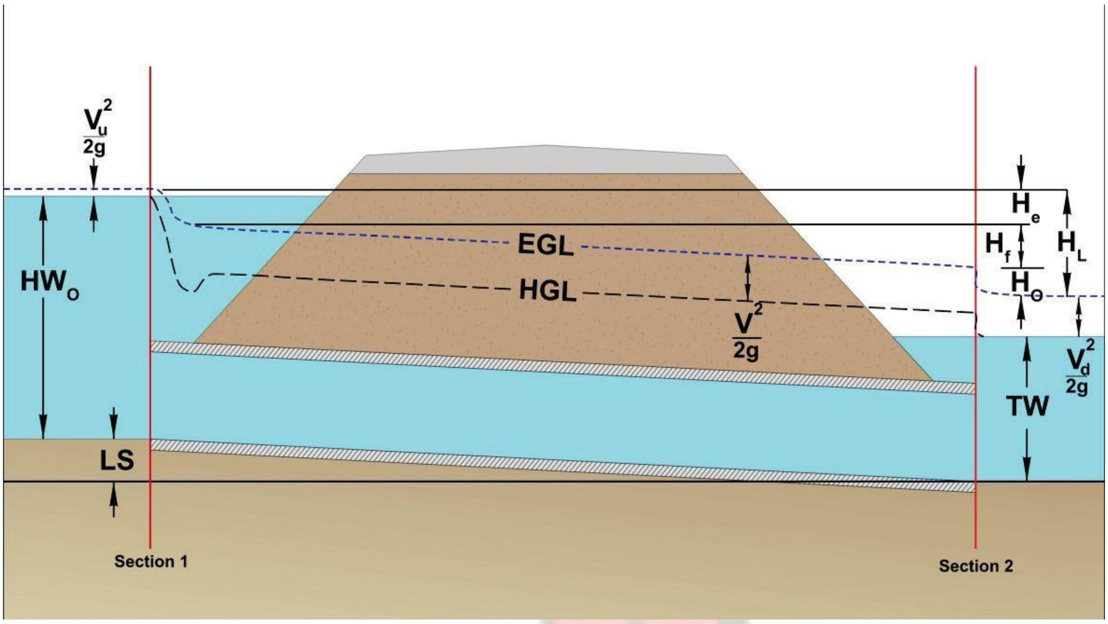

Figure 2.1: Culvert with full flow energy and hydraulic grade lines (*Source FHWA Hydraulic Design Series, April 2012)*

The headwater and tail water conditions as well as the entrance, friction, and exit losses are shown in Figure 2.1 above. Equating the total energy at sections 1 and 2, upstream and downstream respectively in Figure 2.1, the following relationship results:

$$HW_o + LS + \frac{V_u^2}{2g} = TW + \frac{V_d^2}{2g} + HL \qquad 2.23$$

HWo = Headwater depth above the entrance invert in outlet control (m)

Vu = Approach velocity (m/s)

TW = Tail water depth above the outlet invert (m)

Vd = Downstream velocity, m/s

HL = Sum of all losses including entrance (H_e), friction (Hf), exit (Ho) and other losses, (H_b), (H_j), (m)

LS = Drop through the culvert,(m).

Neglecting the approach and downstream velocities, equation2.23 becomes:

$$HW_o = TW + H_L - LS \qquad 2.24$$

The acceptable HW/D is 1 – 1.5 (DUR Drainage Manual, 2006)

2.26. Related Literature of the Study Area

Literature reviewed showed that extensive research has been undertaken in the field of urban flooding both internationally and locally. However, from the literature reviewed urban flooding due to inadequate hydrological and hydraulic capacities of existing culverts in flood prone areas of Accra East has not previously been studied. Enquiries made at the Hyrological Services Department indicate that no assessment had been undertaken on the culverts. This study seeks to review the hydrological and hydraulic capacities of culverts in the flood prove areas for their adequacy and where they are found inadequate, appropriate sizes are proposed. The effects/impacts of washouts of road pavements and other defects on portions of the affected roads will also be assessed. The literatures that have been perused indicate that there are several courses of flooding, however this study seeks to review and address the hydrological and hydraulic inadequacies that contribute to flooding at the research area.

2.27 Waste Transfer Stations

There used to be a waste compost plant between GREDA and Teshie Nungua estates sited along the Songor stream. The compost plant was decommissioned in 2008 and now being used as Waste Transfer Stations (WTS). There are two WTS, one engineered being operated by Zoom Lion and the other not engineered which are under the supervision of LEKMA. The un engineered WTS uses skips for the collection of the waste generated in the community that are not taken away by any of the waste contractors operating under LEKMA.

The Environmental Protection Agency (EPA) has provided guidelines to regulate the management of WTS. The guidelines is to limit sanitary nuisances, vector breeding and the physical hazards of flooding and compel the District Assemblies to provide the communities with adequate and consistently functioning drainage works in accordance with nationally defined design standards. The Assemblies shall ensure, through appropriate bye-laws and control mechanisms that faecal and solid wastes are not discharged into storm water drainage systems (Environmental Sanitation Policy, April 2010). Fencing is important at all dumping of landfill and WTS sites to prevent encroachment, control waste inputs to the site and reduce windblown litter. Chain-link fencing is the preferred type, but for smaller dumps in rural areas, bamboo or other locally available materials may be used, (Item 6.3.6 of Ghana Landfill Guidelines: July, 2002).

Contrary to the standard provisions, more often than not, waste spill over from the skips or are dumped at the un engineered transfer station at night to avoid payment. The WTS is not fenced and when it rains, the runoff washes a lot of plastic waste into the stream channel which go to block the already inadequate culverts and compound the problem. The spillovers especially the plastic rubbers and bottles are also blown away into the stream creating significant blockage and silt in the stream channel when regular routine maintenance is not carried out. Some of the plastics eventually end up in the sea creating lots of challenges for the fishermen and those that are washed ashore litter the coast as was reported by Portia Solomon, a TV3 news reporter on 5th September 2017.

The literature reviewed considered urban flooding, causes and their effects but particular emphasis was given to flooding caused by culverts on road networks at water crossings with particular reference to the problem areas of Accra East. Chapter three follows with the methodology employed in this study in addressing the issues.

CHAPTER 3: METHODOLOGY

3.0 Study Area

This chapter looks at methods and materials used in achieving the objectives of the study. However, before the methods and materials, a brief description of the study area is given. The flood prone areas in Accra East with culverts having hydraulic capacity issues/challenges are;

1. Road networks that cross the Sango stream within the GREDA and Teshie Nungua estates in the LEKMA municipality.

2. Road networks that cross the Onukwa Wahe stream that flows through Communities 19 and 20 in Tema municipality, a tributary of the Sakumo stream. This stream has been diverted to flow across a bridge in the course of the study to relieve the community of the effects of floods.

Accra East falls between longitudes 000 and 001 degrees west of the Meridian. It also lies between latitudes 004.5 and 005 degrees north of the equator. The location is a tropical region that adjoins the Atlantic Ocean. It is part of the Dry Equatorial (Accra coastal plains) of Ghana and receives a lot of moisture from the Atlantic Ocean. The communities that get flooded as a result of hydraulic inadequacies on road networks are in the Ledzokuku and Krowor Municipality, LEKMA (Teshie and Nungua) and Tema Municipality, all in the Greater Accra region.

This chapter outlays the processes used to identify two major flood prone areas in Accra East, the Songor and Onukwa Wahe basins. Initial observation was made of how the culverts contribute to flooding of the road networks in those basins and how the floods in turn impact on road networks leading to road defects. Owing to the limited time available for this research one of the stream basins, the Songor stream basin, was selected for detailed study and its results can be applied to other basins with similar challenges. Fig. 3.1 is the location map of the catchment of Songor stream.

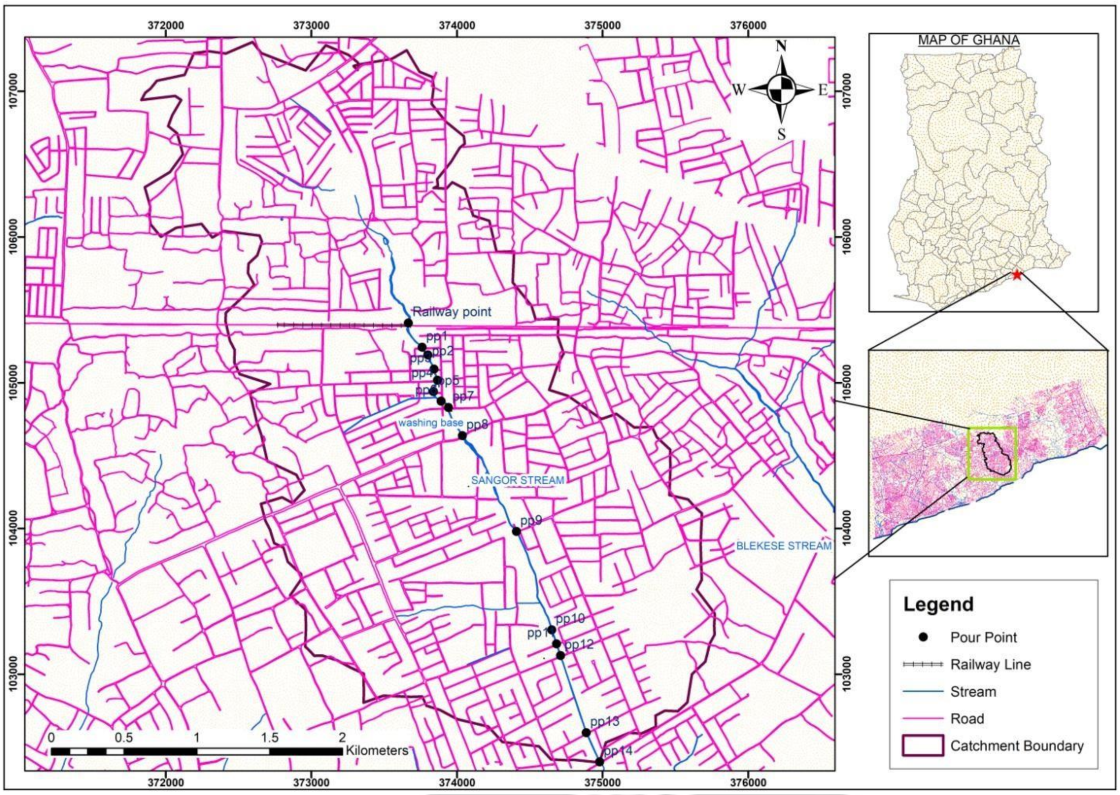

Figure 3.1: Map showing study area in Accra East and the main catchment with pour points.

The Naaplajor stream is located at the northern part of Teshie as a tributary of the Kpeshie stream. According to officials of Town and Country Planning Department (2008), the cause of floods in the area is due to inadequate culvert for safe passage of the stream. Two people were reported to have lost their lives in recent years, through flooding (Karley, 2009). The problem of the area was the result of urban sprawling beyond what the central government/municipalities could cope at the time. Currently, the needed culvert interventions have been made. The Sango stream basin falls in the central part of the Teshie-Nungua municipality (LEKMA) in Accra. The Sango stream has three sections that experience severe floods that require urgent attention. The problem of the three floods areas is inadequate culverts for safe passage of the stream when there are major rains. The worst affected of the three areas is the community that live around Old Kasapreko area, between the Tema Beach road and the coast. Removal of silt deposits through routine dredging in the area does not stop the flooding, (Karley, 2009).

The problem of the Old Kasapreko area is the combination of high tides with flash floods. Urbanization is forcing people to settle on low lying areas of the stream basin. It therefore, appears that the issue of coastal floods in communities that have settled on stream basins along the coast will continue to prevail for a long time to come. Construction of detention ponds to delay some of the runoff from reaching the community during high tides could be the solution. The remaining two of the three affected communities are the GREDA and Teshie –Nungua Estates. This research will review the hydrology and hydraulic requirements of the culverts on the road networks that cross the the stream in addressing the flooding issues.

The Blekese stream also starts from Batsona through Buade community. It crosses the Accra – Tema road at Kantamanto and continues through Nautical college and it's environs before entering the sea. It is a collection of most of the minor streams from Batsona area to Nungua and the basin opens up at the lower sections at Nungua. For many years in the past, residents along this stream used not to experience floods. However, in recent past rapid developments/settlements very close to the stream has changed the situation to the extent that the major culvert on the Accra – Tema road which ensures safe passage of the Blekese steam now encounter washout of portions of the embankment and pavement. Some of the communities along the stream also now experience flooding when there was heavy downpour.

The chapter also examined some of the effects of flooding on road networks due mainly to hydraulic inadequacies of culverts and, therefore, the importance of this study. The methodology used to establish the fact that perennial flooding occurs along the Songor stream in Accra East is outlined below.

3.1 Introduction to Methodology

The study was started with identifying the road networks in the study area that develop defects as a result of the flooding and to what extent the culverts that are meant for safe passage of peak flow rates contribute to the flooding. The study area has in all 14 road culverts of different sizes and 1 rail culvert. A hydrological software HEC-RAS was used to model the flood using ten (10), twenty-five (25) and fifty (50) years return periods to ascertain the extent.

Each of the fourteen (14) culverts identified in the flood prone area together with the one constructed by Ghana Railway cooperation in 1953 were considered for the study and became the study units for this research. Even though all the culverts are within one catchment, for hydrological computation purposes, the area that contributes to each of them was considered a sub catchment. For the fourteen road culverts together with the rail culvert, fifteen sub catchments were generated.

Land use land cover maps of 1991, 2001, 2011 and 2016 were downloaded using GIS application to assess the percentage changes in settlement over those years. The area without vegetation cover was considered settlement area and impervious. Graphs of percent impervious were plotted against the years for the various sub catchments to enable computation of runoff, C, using equation 2.5. By plotting runoff coefficients C against years, one could determine the value of the runoff coefficients within the study area for years of construction of culverts under study. The runoff co-efficient C, together with the catchment area and rainfall intensity based on time of concentration was able to give the flood discharge using equation 2.4. This formed the bases for assessing the adequacy of the culverts even at construction years. By the GHA design guide, the minimum design life of culvert is 25 years. The runoff co-efficient, C, at 25 years after the construction years were used to compute the flows to size the culverts which should have been the appropriate size to have been provided during the construction period. The discharges at the study year were also computed, this together with the discharges at construction year were used to assess the performance of the culverts from construction year to the study year. Additional 30% of the discharges at the study year were added to take care of silt deposits which were used to size the proposed culverts.

3.2 Methodology

There were series of complaints on radio, in print and electronic media on flooding of the communities along the aforementioned Songor and Onukwa Wahe basins during and hours after moderate to heavy downpour. This was confirmed by a research undertaken by Karley (2009) on issue of flooding in the area. For the purpose of this study, the occurrence of flooding with moderate to heavy rainfall in the catchment has to be confirmed by both residents and the road agency responsible and therefore, two different questionnaires were developed to elicit relevant information from them.

3.2.1 HEC-RAS Modelling of Flood

There was the need to know the extent of impact of the floods in the Songor stream catchment. A hydrological software, HEC-RAS was employed to model the extent of flooding for ten years, twenty-five years and fifty years return periods. The number of affected houses within the modeled area was obtained by comparing the modeled area to the Town and Country planning layout of the area. An approximate number of 160 houses were found to be within the affected area and did validate it with actual field account. The affected area covered mostly first four houses away from the stream on either side of the stream banks. This helped in limiting the questionnaires to the residents to the first four houses from the stream.

3.2.2 Survey Instruments

Two questionnaires were prepared, one to residents and the other to the road agency responsible for the area. The questionnaires to the residents were mostly close ended questions meant to establish the fact that flooding occurs in the area with moderate to heavy rainfall. Stratified Random Sampling method was used to administer the questionnaires to the residents. It was also to obtain routine maintenance costs incurred in the past one year in their houses through flooding and number of lives lost through flooding in the past five years. Additionally, there were few open-ended questions to explore the answers of the respondents on solution to the flooding apart from hydraulic capacities of the culverts which the study is to address, copy attached in Appendix 3. Stratified Random Sampling method was used to administer to residents.

The other questionnaire to the LEKMA Roads Department was also to confirm the issues of flooding from them and obtain the year's routine maintenance cost incurred on maintaining the defects caused by the floods at the approaches of the culverts. Their contribution and suggestions were also captured under the recommendation, copy attached in Appendix 3. Tables and charts were generated from the Scientific Package for Social Sciences (SPSS) in the analysis of the data collected with the significant ones shown in Figures 4.1, 4.2 and Table 4.1.

3.2.3 Inventory and Condition Survey of Study Units

With issues of flooding confirmed or established through the survey and initial visits to the sites to observe the flooding on rainy days, indications were that the culvert sizes were too small to safely pass peak discharges. The study then settled on the culverts as the study units of the research.

Inventory and condition survey of all the fourteen (14) culverts at road crossings were undertaken, in addition the details of the culvert at rail crossing were captured. Relevant field measurements of sizes, number of cells and lengths were undertaken of the study units. The X and Y coordinates at the inlet points of the various culverts were obtained using a base map of roads and streams of Accra East obtained from the Department of Urban Roads whose positional accuracy was confirmed to be acceptable using Global Position System (GPS) coordinates. All the field measurements were tabulated in Table 3.1 shown below.

Table 3.1: Summary of inventory of road names, culverts identified and their details, catchment areas, stream lengths, runoff coefficient at design year and computed discharges at design year

Pour Point No	Culvert location	Size and culvert type (existing)	Culvert lengt	Coordinates of pour point	Year of construction of culvert	Runoff Coefficient(C) at construct. year	Q (m^3/s) at construct. year	Catchment Area for Pour Point (Km^2)	Stream length only (m) to culvert	Total length from remotest point to pour point
RAIL C1	Under Railway	5 X 1.8 PC	28m	X:373629.60 Y: 105135.74	1953	0.14	2.28	2.88713	1,928.00	3,151.755
1	9th Avenue	3 x 0.9 PC	11m	X:373729.09 Y:104970.34	1994	0.23	3.82	2.94481	3,564.694	4,788.449
2	8th Avenue	3 x 0.9 PC	10.5m	X:373769.00 Y:104917.36	1994	0.23	4.22	3.25960	3,692.699	4,916.454
3	7th Avenue	3 x 0.9 PC	10.5m	X:373810.01 Y:104820.41	1994	0.24	5.14	3.81083	3,859.932	5,083.687
4	6th Avenue	3 x 0.9 PC	10.5m	X:373834.86 Y:104744.32	1994	0.24	5.18	3.84055	4,109.859	5333.614
5	4th Avenue Extension	3 x 0.9 PC	14.5m	X:373810.00 Y:104665.50	1994	0.26	6.49	4.44512	4,347.458	5,571.213
6	E - Street	3 x 0.9 PC	9m	X:373859.08 Y:104600.52	1994	0.29	7.44	4.57046	4,470.346	5,694.101
7	3th Avenue Extension	2No. x 3m x 2m BC	12m	X:373908.98 Y:104557.44	2015	0.85	37.33	4.59470	4,609.555	5,833.31
8	Tsui Bleo road	3No. x 3m x 1.5m BC	15m	X:374005.35 Y:104362.90	2003	0.61	29.50	5.06117	5,037.433	6,261.188
9	Volta Street	2No. x 1.8m PC	11m	X:374376.69 Y:103705.10	2003	0.64	38.03	6.23022	6,576.929	7,800.684
10	Coffee Street	2No. x 3m x 2.2m BC	10m	X:374620.51 Y:103028.14	2014	0.82	64.58	8.28888	8,008.143	9,231.898
11	Moon flower street	1No. x 4m x 1.8m BC	14m	X:374651.80 Y:102932.79	2003	0.67	53.08	8.33868	8,238.661	9,462.416
12	Cocoa street	1No. x 3m x 1.3m BC	12m	X:374680.34 Y:102850.26	1969	0.17	7.87	8.35664	8,449.273	9,673.028
13	Life Street	1No. x 3m x	12m	X:374856.21				9.93468	9,564.593	10,788.348

		1.3m BC		Y:102324.74	15/5/ 1969	0.17	9.32			
14	Tema Road	2No. x 3.5m x 1.5m BC	10m	X:374948.48 Y:102124.23	1965	0.14	8.19	10.62131	10,155.357	11,378.112

*Where: BC is box culvert and PC is pipe culvert. *Length of the remotest point of the catchment to the beginning of the stream = 1,223.755m

Topographical maps of the sub catchments and the main catchment were generated from Goggle Earth using GIS at the Geomatic Engineering Department of KNUST, Kumasi which helped in determining the following;

i) Catchment area for each of the culverts, ii) Overland flow length of the runoff, iii) Length of stream to the various pour points.

iv) Elevation differences between the remotest point in the catchment and the various pour points (inlet points).

These helped in the computation of slopes of flow and time of concentration to the various culverts. Relevant field data were gathered; example soil types, characteristics and any other data that were found necessary to the research.

3.2.4 Land Use Land Cover Satellite Images

The percentage impervious of the catchment impacts on the runoff of the area. Consequently, it was imperative to obtain land use land cover change images of the catchment and sub catchments. Once again, the Geomatic Engineering Department of KNUST was very helpful in providing the images of 1991, 2001, 2011 and 2016. These were used to determine the percentage of areas without vegetative cover and those with vegetative cover for the years. The percentage areas without vegetative cover were considered percent impervious. The percent impervious against years graphs were plotted from which percent impervious of years culverts were constructed were obtained.

The percent impervious equation, equation 2.5 was used to compute the runoff coefficients of the aforementioned years for the various sub catchments.

$$C = 0.74IM + 0.132 \qquad 2.5$$

Where IM = % imperviousness (divided by 100). The runoff co-efficients were plotted against years to generate models for the various sub catchments from which the runoff co-efficients of culverts construction years were computed.

3.3 Time of Concentration

The time of concentration, t_c, was computed using the Bransby-Williams' Equation for natural catchments and mixed flow paths which is a combination of the times of overland flow and channel flow as shown in equation 2.3.

$$t_c = \frac{92.5\,L}{A^{0.1}S^{0.2}} \qquad 2.3$$

where;

t_c= Time of concentration (minutes)

L = Length of flow path from catchment divide to outlet (km

A = Catchment area (ha)

S = Slope of stream flow path (m/km)

The time of concentration for catchment no. 1 was assumed for all. This was to cater for flush flood and worse scenario conditions, it will also ensure that the same intensity was used for all the sub catchments. The ten years Maximum Rainfall Intensity for Accra deduced from the Ministry of Roads And Highways Consultancy Services for Coordination of The Review of The Intensity Duration Frequency (IDF) of The Rainfall Intensity Graph, (July 2016) was used for the intensity of the study year. This is a revision of Ghana Meteorological Services Department Departmental Note 23(Maximum Rainfall Intensity – Duration Frequencies in Ghana) by J. B. Dankwa (1974).

For the culverts that were constructed between 2003 and 2015, the same time of concentration was applied retrospectively to the Ghana Meteorological Services Department Departmental Note 23(Maximum Rainfall Intensity – Duration Frequencies in Ghana) by J. B. Dankwa (1974). Settlement affects time of concentration and therefore, in obtaining the time of concentration for culverts constructed between 1965 and 1969 when there were on settlement, the time lag equation, equation 2.1 was applied.

$$T_c = \frac{l^{0.8}(S+1)^{0.7}}{1140Y^{0.5}} \qquad 2.1$$

where:

T_c = time of concentration, h

l= flow length, ft

Y = average watershed land slope, %

S = maximum potential retention, in

$$S = \frac{1{,}000}{cn'} - 10 \qquad 2.1b$$

where: cn′ = the retardance factor. The retardance factor is approximate to be the same as the curve number (CN) as stated in NEH630.09. A CN in the limits of 50 and 95 is acceptable to be used in solving equations 2.1b (Mockus 1961).

3.4 Rainfall Intensities (I)

The rainfall intensities for the culvert construction years were obtained using the computed times of concentration and the 10 years return periods of the various IDF curves that prevailed at the culvert construction years. The intensity for the study year was obtained using the time of concentration of the study year and the 10 years return period of maximum rainfall intensity for Accra deduced from the Ministry of Roads and Highways Consultancy Services for Coordination of The Review of The Intensity Duration Frequency (IDF) of the rainfall intensity graph, (July 2016).

3.5 Estimation of Peak Discharge

The parameters obtained for C, I and the area, A, were used to compute the peak discharges at the culvert study year and construction years using the modified rational method of equation 2.4a.

$$Q = 0.278\, C_R C I A \tag{2.4a}$$

Where,

Q = Peak rate of design discharge (m^3/s)

C = Run-off coefficient (dimensionless)

I = Rainfall intensity (mm/hr)

C_R = Area Reduction Factor (ARF) (dimensionless)

A = Catchment area (km^2)

The ARF, C_R, is determined using Chow (1964) relation:

$$C_R\ (ARF) = \left[1 + \frac{1.93A}{10^3}\right]^{-1} \tag{2.4b}$$

The discharges at the culvert construction years were used to ascertain the adequacy of the culverts at time of construction. The discharges at the study year together with those of the construction years were used to assess the performance of the culverts from construction till the study year.

Figure 4.4 was used to determine the C values after 25 years of construction to calculate the flows and ascertain the culvert sizes to have been provided assuming a design life of 25 years. With figures 4.4 and 4.5 showing equilibrium runoff coefficient (C‘s), additional 30% of the study year‘s discharges were added to the computed ones to allow for silt build up and the plastic spillovers from the WTS in accordance with the Department of Urban Roads drainage design manual. The resultant discharges were used to propose the various sizes for the culverts which were found to be inadequate except the rail culvert.

3.6 Estimation of Cost Due to Flooding in Study Area

The cost of routine maintenance on roads at stream crossings and approaches to culverts in the study area were considered together with the costs incurred by residents to justify the need for urgent replacement to save lives and costs on maintenance to the road agency, the road users and residents in the community. The channel capacities at various sections were determined and recommendations were made for sections of the drain that are lined but have smaller capacities to be changed.

Further recommendations were for the construction of detention ponds at a specified location between culverts 12 and 13 to save the communities along the coast near Coco Beach of flooding during high tides. The urgent need for action to be taken on the existing culverts was stressed for immediate response from the appropriate agencies.

CHAPTER 4: RESULTS AND DISCUSSION

4.1 Summary of Results on Survey and Hydrological Analysis

As part of the study, two different interviews were conducted in a form of survey. The instruments used were questionnaires with most of the questions being close ended and a few open-ended ones to explore residents‘ views on the subject matter. Hydrological software, HEC-RAS was used to model the extent of flooding for ten years, twenty-five years and fifty years return periods in the catchment to determine the extent of the impact under those return periods. The modeled area was compared to the Town and Country planning layout of the area to determine the number of affected houses. The affected area covered mostly first four houses away from the stream on either side of the stream banks. An approximate number of 160 houses were identified to be affected. This was collaborated by a topographical map of the area which showed higher elevations beyond four houses from the stream and limits the floods to the four houses on either side of the stream, this was again confirmed during the interview. The interview was therefore, meant to cover about four houses on either side of the stream channel from GREDA estates to the end of Teshie Nungua estates.

Out of the houses to be affected, one hundred houses were sampled through stratified sampling among the two estate communities separated by a compost plant at Teshie. The randomness was based mostly on availability to respond to the questionnaires. The method of house survey was found to be most appropriate which also gave the researcher firsthand information of affected residents along the stream. A high response rate of 0.95 (100/105) was attained due to the fact that with the exception of few who were not available and the questionnaires were left behind for later collection, responses were elicited from one respondent/house before moving onto the next house. Tables and charts were generated from the responds using Scientific Package for Social Sciences (SPSS) in the analysis of the data collected. Table 4.1 and Figures 4.1 and 4.2 below are summaries of the close ended information collected with some of the valuable contributions in the open-ended questions captured in the recommendation.

The application of GIS was explored in obtaining land use land cover changes maps of the catchment area for the years; 1991, 2001, 2011 and 2016, all in the month of January. The main catchment area was divided into fourteen sub catchments for every culvert to have its own separate catchment. In addition, the catchment for the railway culvert was also processed. However, catchment no. 14 is the totality of the entire catchment. The land use changes for the sub catchments were done similar to the one shown in Fig. 4.3 for catchment no. 14 and are attached in Appendix 4.

The land use changes over the years showed corresponding percent impervious. Graphs of percent impervious were plotted against the years for each catchment which gave polynomial equations of curves from which percent impervious of years existing culverts were constructed were computed. The percent impervious were substituted in equation 2.6 to compute for the runoff co-efficients, C's, for the aforementioned years for the fourteen culverts and that of the rail culvert. The runoff co-efficients C for the various years were plotted against the years which produced polynomial curves with equations from which runoff co-efficient of the years the culverts were constructed were computed to assess the adequacy of the culverts at the construction year.

GIS was explored in obtaining the various sub catchment areas, stream lengths to the various culvert points, topography of the catchment for the purpose of obtaining relevant elevations for slope computations. Time of concentration using BransbyWilliams' Equation, equation 2.3, for mix flow (overland flow and stream flow) was computed for catchment no. 1 which was assumed to be the same for all the other catchments. This was to cater for flush floods situations and also use the same intensity for all the sub catchments. The ten years maximum rainfall intensity for Accra was deduced from the Ministry of Roads and Highways Consultancy Services for Coordination of the review of the Intensity Duration Frequency (IDF) of the rainfall intensity graph, (July 2016).

Settlement affects time of concentration so the times of concentration at the various years of constructing the culverts were computed. For those culverts that were constructed when there was no settlement, the lag time equation, equation 2.1, was used to compute their time of concentration. This was used to compute the discharge at the years of construction of the culverts. All the catchment were bigger than 2km^2 which is the limit for the use of Rational Method and therefore, the Modified Rational Method was adopted for the computation of the discharge using the Modified Rational Method of equation 2.4. Due to the fact that the equation used in the computation of runoff coefficients, C, was generated using the NRSC curve number approach, the flow was compared with a HEC-HMS hydrograph output for one of the catchment which gave a comparable flow. The Modified Rational Method was then used to compute the peak flow for the remaining culverts and additional 30% of flow was allowed for silt build up. Civil 3D was then used to assess the adequacy of the culverts at the construction year, after design life of twenty-five years and their performances from construction year to the study year for adequacy or otherwise. Proposals were made for replacement of those found inadequate.

4.2 Results and Discussion

The results of the various aspects are as follows.

4.2.1 Data Analysis of Survey Results from Residents

Table 4.1: Statistical summary of close ended questions of questionnaires to residents in the area that gets flooded using SPSS. Q2, Q4, Q5 etc are the question numbers in the questionnaire. The statistical results on responses to close ended questionnaires to the residents have been summarized in Table 4.1. Sample of the questionnaire are attached in Appendix 3.

Age group

The number of respondents is 100. The mean age of the respondents lies in the 41 – 50 years group which also represents the median group and the 50 percentile. The most frequently occurring year group that responded is the 31 – 40 years group. This is understandable because the days of the interviews were weekends and holidays and therefore the working class within which this age group fall, was available. The standard deviation is 1.47 from the mean which implies that the 31 – 40 years group and that of the 51 – 60 years together with the modal age group were the main respondents. The high standard deviation of 1.47364 is explained by the fact that some of the respondents were 70 years and above which is a major deviation from the mean. The mature age of respondents is an indication that data collected were from responsible residents within this catchment that get flooded. This also is an indication of the suffering they go through with the annual ritual of flooding and their readiness to support the study with the hope of finding solution to the problem.

		Q2	Q4	Q5	Q6	Q7	Q8	Q9	Q10	Q11	Q12	Q13	Q14	Q16
N	Valid	100	100	100	100	100	100	100	100	100	100	100	100	100
	Missing	0	0	0	0	0	0	0	0	0	0	0	0	0
Mean		2.9900	1.5500	3.5200	2.1100	1.0000	4.0000	1.7200	2.3800	1.3100	1.7300	1.2100	3.4100	2.7800
Std. Error of Mean		.14736	.07300	.17893	.10337	.00000	.06513	.07396	.08502	.04648	.05478	.04094	.15769	.13896
Median		3.0000	1.0000	4.0000	2.0000	1.0000	4.0000	2.0000	2.0000	1.0000	2.0000	1.0000	3.0000	2.0000
Mode		2.00	1.00	4.00	1.00	1.00	4.00	1.00	2.00	1.00	2.00	1.00	3.00	4.00
Std. Deviation		1.47364	.72995	1.78931	1.03372	.00000	.65134	.73964	.85019	.46482	.54781	.40936	1.57695	1.38957
Variance		2.172	.533	3.202	1.069	.000	.424	.547	.723	.216	.300	.168	2.487	1.931
Skewness		.307	1.413	-.174	.447		-.224	.502	1.391	.834	.316	1.446	.051	.105
Std. Error of Skewness		.241	.241	.241	.241	.241	.241	.241	.241	.241	.241	.241	.241	.241
Kurtosis		-.930	2.078	-1.260	-1.005		.127	-1.015	2.162	-1.331	1.782	.092	-1.087	-1.481
Std. Error of Kurtosis		.478	.478	.478	.478	.478	.478	.478	.478	.478	.478	.478	.478	.478
Range		5.00	3.00	5.00	3.00	.00	3.00	2.00	4.00	1.00	3.00	1.00	5.00	4.00
Minimum		1.00	1.00	1.00	1.00	1.00	2.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Maximum		6.00	4.00	6.00	4.00	1.00	5.00	3.00	5.00	2.00	4.00	2.00	6.00	5.00
Percentiles	25	2.0000	1.0000	2.0000	1.0000	1.0000	4.0000	1.0000	2.0000	1.0000	1.0000	1.0000	2.0000	2.0000
	50	3.0000	1.0000	4.0000	2.0000	1.0000	4.0000	2.0000	2.0000	1.0000	2.0000	1.0000	3.0000	2.0000
	75	4.0000	2.0000	5.0000	3.0000	1.0000	4.0000	2.0000	3.0000	2.0000	2.0000	1.0000	5.0000	4.0000

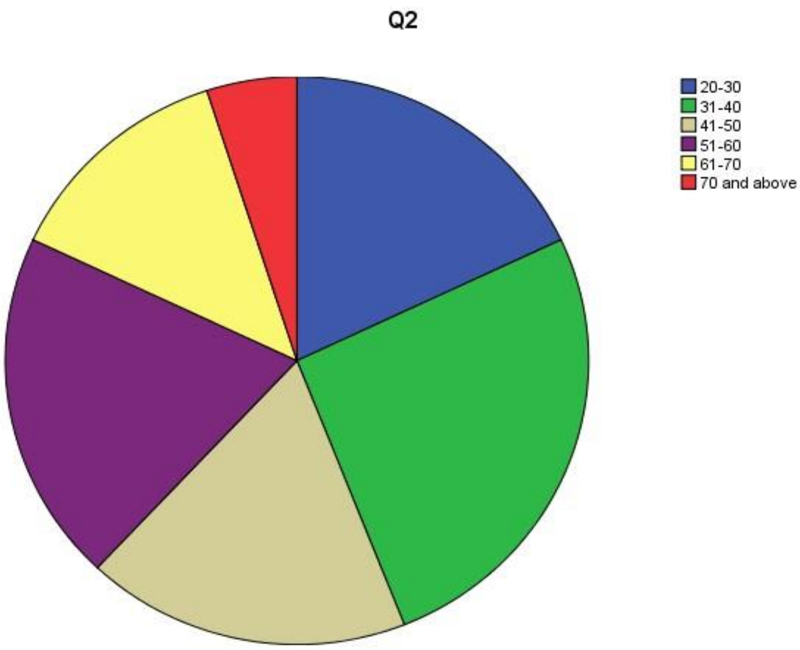

Figure 4.1: Age group of respondents. The Q2 is the questionnaire number for age group of respondents

Number of occupants in a house

The mode and median and the 50 percentiles are all in the 1 – 5 number per house group. The mean is 6 – 10 occupants per house. Being mostly estate houses, they were mostly single households per house. Population density is therefore not very high in some houses in the area. This allowed them to have a lot of green areas to reduce runoff after construction of their houses. This is the case where the 2016 percent impervious is slightly lower than that of 2011, example been 90.15% of 2016 is lower than that of 91.2% of 2011for catchment number 14.

Types of finishing of compound

The mean, median, mode and 50 percentiles are all indicating earth (natural ground)/gravel finishing. These finishing reduce runoff as they allow for infiltration.

The high standard deviation of 1.78931 is explained by the fact that other finishing like terrazzo/concrete and green grass, use of pavement block and green grass were recorded which are different from the earth/gravel finishing. Some of the finishing mostly allow for infiltration thereby reducing runoff of the area.

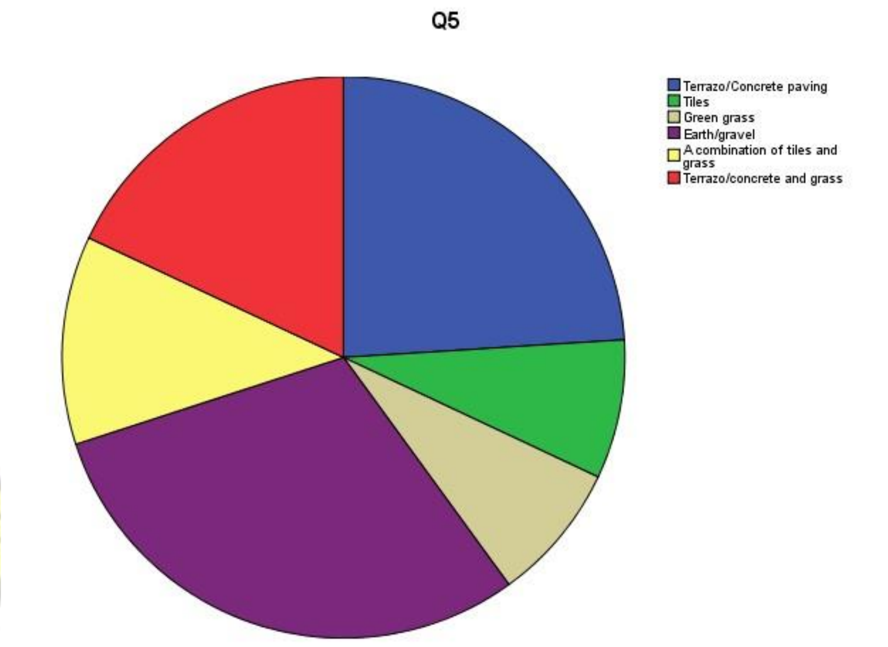

Fig 4.2: Types of finishing of compound. Q5is the questionnaire number for types of finishing of compound.

Proximity to the stream

The first four houses on either side of the stream were targeted for the survey, the mean, median and 50 percentile of houses interviewed indicate two houses from the stream, the mode is next to the stream. This is an indication that the most affected persons were interviewed.

Does the stream flood during medium and heavy rainfall

The mean, median and mode all answered yes with the standard deviation being zero, an indication that all the residents answered that the stream floods during medium and heavy rainfall. All the percentiles confirmed the yes answer.

Level of intensities of rainfall that cause flooding

The mean, median and mode showed both medium and heavy intensities of rainfall over long period causes flooding. All the percentiles confirm that both medium and heavy intensities of rainfall over long duration of time cause flooding.

Frequency of flooding during the main rainy season (April – July)

The mean, median mode and 50 percentile all indicate 6 – 10 times of flooding per season, however individual proximity to the stream was a key factor to the frequency per house. This was realized during the survey. The high frequency of flooding was responsible for the washout of the road at culvert locations.

Frequency of flooding during the minor rainy season (September – November) The mean, median, mode and 25 and 50 percentiles indicate 4 – 6 times of flooding in the months of September - November. However, once again individual proximity to the stream was a key factor to the frequency experience per house and sometimes the level of impact.

Fatalities/loss of lives during flooding

There were two possible answers here, yes and no. The mean, median and mode all indicated yes. Both the 25 and 50 percentiles indicate yes, thus the flooding comes with fatalities and the 75 percentile indicates otherwise. Most of those who responded no were people who relocated to those communities under two years ago. The high response on fatalities goes to confirm the need to find a lasting solution to the problem.

Number of fatalities in the last 5 years

The mean, median and mode each indicates between 6 - 10 lives were lost. This is collaborated by the 50 and 75 percentile. For even a life to be lost through flooding is unacceptable let alone between 6 – 10 persons in the last 5 years as indicated by the respondents. Those who lost their lives were unsuspecting road users driving through the community in vehicles and some children in the community.

Loss of property/ damages

The mean, median and mode each indicates that people get their properties damaged through flooding. All the percentiles collaborate with the fact that people had suffered some form of loss of property. Basically, almost all those interviewed had suffered some form of loss of property owing to the flooding.

Estimates of extent of annual damages

The following are the categories;

1. None
2. Ghs 1 – Ghs 1000.
3. Ghs 1001 – Ghs 2000.
4. Ghs 2001 – Ghs 5000.
5. Ghs 5001 – Ghs 10,000.
6. Above Ghs 10,000.

The mean, median, mode and 50 percentile estimate of annual damage is in Ghs 1001 – Ghs 2000 range. The standard deviation is 1.57695 which is high. The reason was that some homes spent above Ghs 10,000 owing to the severe impact of the flood which is far above the mean. These expenditures together are huge. The estimated total is (1001 +2000)/2 x 160 amounting to Ghs 240,080.00 per annum. This together with maintenance cost incurred by the road agency would justify why the problems should be solve now no matter the cost since the cost of replacing the culverts is lower than all other costs together for many years.

Communities' contribution to factors causing the flooding

The following are the categories;

1. Encroaching on the stream channel
2. Throwing of garbage into drains and the stream when it rains
3. Others
4. Both encroachments on stream channel and throwing of garbage into drains and stream when it rains.

The mean, median and mode indicate that the combine effect of encroachment on stream channel and throwing of garbage into drains and stream when it rains are contributing factors to the flooding. This is collaborated by the percentiles.

The high standard deviation of 1.38957 is explained by the fact that most of those who built close to the stream channel did not think that encroachment was a contributing factor and others thought because they live in estates with good waste collection system, throwing of garbage into drains and stream was not a factor forgetting that the catchment stretches beyond the estates.

4.2.2 Analysis of Survey Results from LEKMA Roads Unit

The LEKMA Roads Department responds to the questionnaire have been summarized below.

Catchment lies in the municipality with road crossings

The roads department answer was affirmative to the catchment being in the municipality and that the following streets/roads cross the stream by means of culverts;

(i) Ninth Avenue Street

(ii) Eighth Avenue Street

(iii) Seventh Avenue Street

(iv) Sixth Avenue Street

(v) Fourth Avenue Extension

(vi) Road E

(Vii) Third Avenue Extension Street

(viii) Tsui Bleo Road

(ix) Volta Street
(x) Coffee Street

(xi) Moon flower Street

(xii) Cocoa Street

(xiii) A – Life Street

(xiv) Tema Beach Road

The roads department also indicated that sections of the road experience flooding and wash out together with other defects which are flood related

Annual maintenance cost at culvert approaches

The roads department could not provide the cost related to maintenance of all the above listed roads. However, the following were provided; Maintenance of Road E culvert approaches Ghs 42,111.00

Maintenance of Volta Street culvert approaches after wash out was Ghs 39,482.31.
Replacement of Third Avenue Extension culvert in the year 2014 - Ghs 366,607.15.

The above costs give an average maintenance cost per culvert approach to be Ghs 40,796.66. Comparing the average maintenance cost to cost of replacement shows that the cost of nine times maintenance of each culvert approach is enough to replace it. Replacement will reduce road agency cost as maintenance cost will reduce, reduce vehicle operating cost of those that ply the roads and save lives and property of the residents in the community. New culverts can have design lives of fifty years which with early replacement make the road agency save the scarce resources for other important projects. Therefore, apart from human lives that cannot be valued under this study, the total amount that goes into road maintenance per annum due to flooding and washout given that the culvert approaches are maintained once in a year is;

$$N \times \text{Ghs } 40{,}796.66 + \text{Ghs } 240{,}080.00 \tag{4.1}$$

where N is the number of undersized culverts that contribute to flooding.

From the hydraulic analysis, all the fourteen culverts are inadequate for safe passage of the runoff hence N is 14 and therefore, the total cost per annum is;

$$14 \times \text{Ghs } 40{,}796.66 + \text{Ghs } 240{,}080.00 = \text{Ghs } 811{,}233.24 \tag{4.2}$$

The LEKMA roads unit was challenged with budgetary approval but this analysis can give the authorities responsible for budgetary allocation a better understanding of the situation for prudent allocation of resources. This study can also be applied to other areas with flooding challenges for prudent budgetary allocation.

4.2.3 Analysis for Objective No. 4

The breakdown of the replaced culvert of Ghs 366,607.15 is as follows;

(i) Cost of construction of the 2no. 3m x 2m culvert is Ghs 205, 425.45

(ii) Cost of approach drains is Ghs 79,782.80 and

(iii) Cost of culvert backfilling is Ghs 81,398.90 totaling the Ghs 366,607.15

Apart from the recommended bridge for catchment number 14 and addition of a cell to catchment numbers 7 and 8 and two cells to catchment number 10, the average culvert for the remaining ten sub catchments is 4no. 4m x 2m. This is approximately three times the cost of item (i). The total cost of replacing the ten culverts therefore, is

(10 x 3 x Ghs 205, 425.45) + (10 x (Ghs 79,782.80 + Ghs 81,398.90))

= Ghs 6,162,763.5 + Ghs 1,611,817.00

= Ghs 7,774,580.5 4.3

Approximately, the cells to be added are equivalent to 4no. 4m x 2m box culvert which is about three times item (i)

= 3 x Ghs 205, 425.45 = Ghs 616,276.35 4. 4

Summing up equations 4.3 and 4.4; the total cost is Ghs 8,390,856.85. Dividing the total cost of replacement by the combined cost incur by residents and road agencies on maintenance shows that about ten years combined cost by residents and road agencies is sufficient to address all the hydraulic inadequacies of the culverts except that of the proposed bridge.

4.3 Data Analysis of Hydrological and Hydraulic Results

The catchment area under study has fourteen road culverts and a rail culvert, which form the study units. They are within the GREDA and Teshie Nungua estates and experience flooding whenever there is moderate or heavy rainfall. The study units were made mainly of concrete culverts however, some are pipe culverts and others are box culverts. The culverts were sized and constructed at different times leading to inconsistency in judgments hence inconsistent sizes along the stream channel. The culverts of the study units consist of headwalls, wing walls and toe walls which are attached to the main unit that allows safe passage of water across the road.

All the culverts are under the jurisdiction of LEKMA roads unit. The LEKMA roads unit was created in 2008 by an act of Parliament. It implies that apart from two culverts that were constructed in 2014 and 2015, all the culverts were in place before the creation of the municipality and therefore, access to design flow data was difficult.

Those culverts within the Teshie Nungua estates were constructed by State Housing Corporation as part of the housing project. Those within the GREDA estates were also constructed by Modern Ghana Builders Company as part of the estates project. The rest were constructed around 2003 under Accra Metropolitan Roads Department. None of the above-mentioned institutions could provide any design flow data to ascertain the discharges used for the design of the various culverts. Below is the procedure used in estimating the design flows at the various culverts for this research.

4.3.1 Land Use Land Cover Change Images

Images of land use changes were obtained for the years 1991, 2001, 2011 and 2016. This was to determine the percentage of land area not covered with vegetation and that covered with vegetation. The percentages of land area not covered with vegetation were obtained for the years aforementioned and were considered as percentage impervious while the corresponding areas with vegetation cover were considered as percent pervious. In the process, it was realized from the 2016 data that the percent impervious reduced marginally with respect to that of 2011, example is a reduction from 91.2% in 2011 to 90.15% in 2016 for catchment no. 14. Others show almost the same figures, indicative of an equilibrium state. The reduction could be as a result of compliance with the National Building Regulations 1996, (Li 1630) which according to Regulation 14, sub-regulations (1), (2) and (3) state that; (1) No dwelling house shall be constructed on a plot of land smaller than 450 square meters with a frontage of less than 15 meters except where there are roads on all sides of the which case the plot size shall be not less than 330 square meters and the frontage not less than 15 meters. (2) No dwelling house together with its out-buildings shall cover area more than the following; single storey detached .. .. 50% , two and three Storey detached .. .. .. 40%, single storey semi-detached .. .. .. 60%, two and three storey semi-detached.. .. 50%, two and three storey terrace .. .. .. 50% given that the total floor area of a residential building other than a block of residential flats shall not exceed 80 percent of the total area of the plot. (3) No business premises together with its out-buildings shall cover an area more than 75 per cent of the plot and such provision shall be made as will be required by the District Planning Authority for loading, accommodation and car parking, provided that in areas zoned for residential use, no building shall cover more area of the plot than that provided in sub-regulation (2) of this regulation. Parts of open areas were converted into lawns and tree planting zone as wind breaks after the clearing and construction of the buildings and that reduced the runoff co-efficient.

A deduction therefore, is that areas without vegetative cover were considered as part of the percent impervious which includes bare ground not covered with building, roads, concrete etc. Such areas bare though they may be, allow infiltration to some extent and therefore, percentages given by satellite images as impervious in urban environment with such open spaces are over estimated. This leads to the over estimation of the runoff co-efficient, C. This is again reinforced by the results of the survey conducted with residents, which indicate that majority of houses have the natural ground (earth) or gravel surface as the finishing. These though were considered as impervious in the images, allows infiltration and therefore, the runoff co-efficient C is always over estimated in urban areas. For example, the satellite images of the land use land cover changes images downloaded have been summarized and tabulated below in Table 4.1. An example of the land use changes of catchment number 14 is shown in Figure 4.3 and the rest of the catchments are attached in Appendix 4. Using catchment number 14 as an example, a plot of percent impervious on the y-axis against years on the x- axis shows changes in percent impervious over the years as shown in Figure 4.4. The regression coefficient $R^2=1$ is an indication of urbanization over the years with its effects been responsible for the changes in imperviousness.

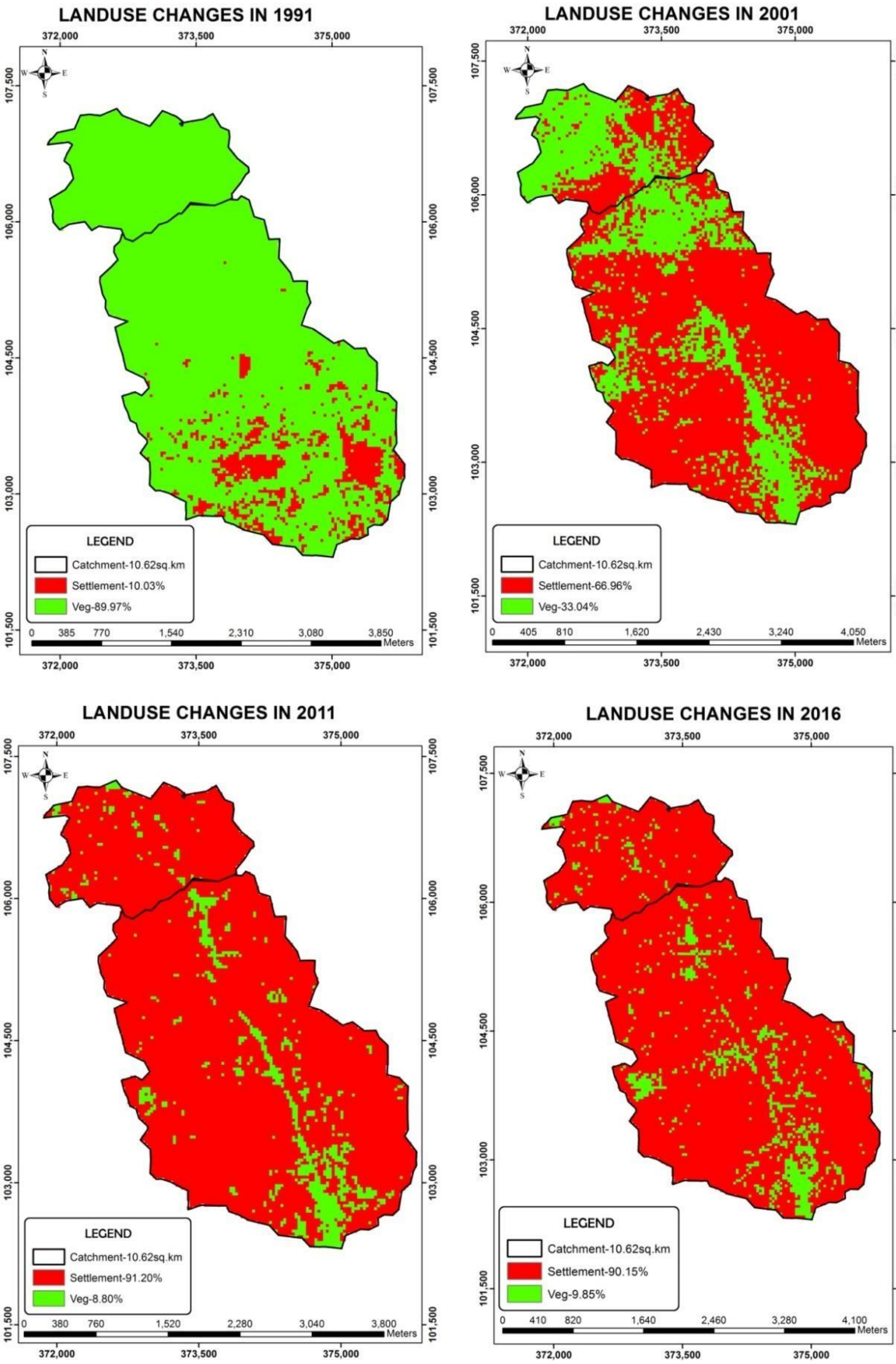

Figure 4.3: Catchment no. 14 land use, land cover changes map of Songor stream

Table 4.2: Summary of Land Use Land Cover Changes between 1991 and 2016

CATCHMENT NO.	TOTAL AREA Km^{2})	SETTLEMENT) 1991	% VEGITATION (%) 1991	SETTLEMENT) 2001	% VEGITATION (%) 2001	SETTLEMENT) 2011	% VEGITATION (%) 2011	SETTLEMENT) 2016	% VEGITATION (%) 2016
Railway C	2.88713	0	100	40.7	59.3	94.89	5.11	99.75	0.25
1	2.94481	0	100	37.8	62.2	92.8	7.2	92.64	7.3
2	3.2596	0.028	99.97	38.12	61.88	92.91	7.09	92.55	7.45
3	3.81083	0.024	99.98	43.07	56.93	93.58	6.42	92.75	7.25
4	3.84055	0.02	99.98	43.41	56.59	93.63	6.37	92.69	7.31
5	4.44512	0.06	99.94	49.17	50.83	94.21	5.79	93.2	6.8
6	4.57046	0.09	99.91	54.35	45.65	94.38	5.62	93.48	6.52
7	4.5947	0.08	99.92	50.48	49.52	94.37	5.63	93.31	6.69
8	5.06117	0.06	99.94	50.74	49.26	94.4	5.6	93.32	6.68
9	6.23022	1.03	98.97	58.19	41.81	93.75	6.24	92.32	7.68
10	8.28888	4.53	95.47	64.76	35.24	93.95	6.05	92.08	7.92
11	8.33868	4.57	95.43	64.62	35.38	93.82	6.18	91.93	8.07
12	8.35664	4.56	95.44	64.52	35.48	93.72	6.28	91.88	8.12
13	9.93468	8.63	91.37	66.84	33.16	92.55	7.45	90.98	9.02
14	10.62131	10.03	89.97	66.96	33.04	91.2	8.8	90.15	9.85

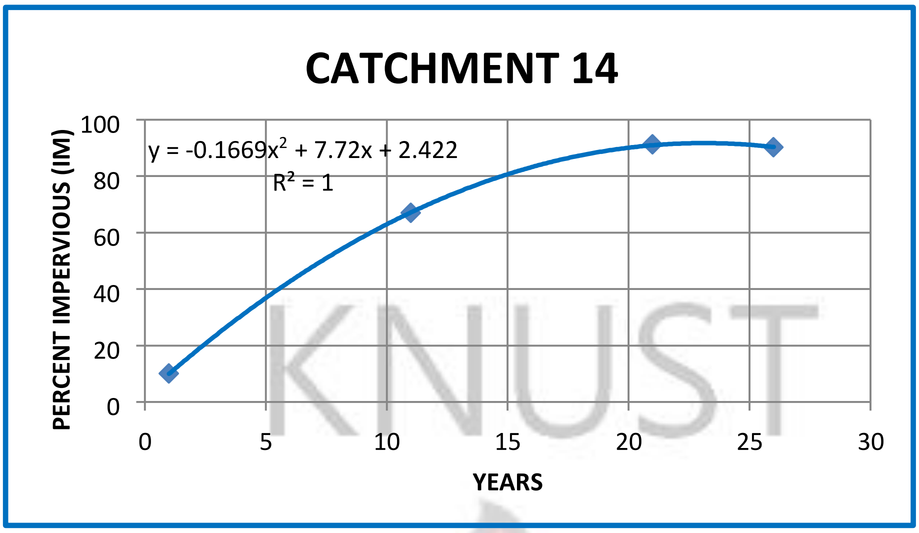

Figure 4.4: Percent impervious with years for catchment no.14.

In plotting, the objective was to generate an equation of the line of best fit to be able to determine the percent impervious of the construction years of the culverts. The lines of best fit are polynomials which indicate equilibrium state with respect to imperviousness. The generated model was not good enough to predict outcomes when the exact years were used but when 1990 was referenced as year zero; 1991, 2001, 2011 and 2016 were transform to 1, 11, 21 and 26 respectively. The percent impervious against the transformed years gave lines of best fit whose equations are very good predictive models with regression co-efficients varying from 0.96 to 1. Similar models for the other sub catchments were used to compute the percent impervious for the construction year of the culverts. This is demonstrated with figure.4.4 for catchment no.

14. The graph of percent impervious against year shows rapid urbanization within the catchment area. Thus area of settlement of 10.03% in 1991 increased to 90.15% in 2016, which shows that the area experienced very fast urbanization.

4.3.2 Determination of Runoff Coefficient

The10year return period equation for runoff co-efficient, equation 2.5 as shown below was used in computing the runoff co-efficient based on the percent impervious for all the catchments for the aforementioned years and summarized in table 4.2

$$C = 0.74IM+0.132 \qquad 2.5$$

Where:

IM = % imperviousness (divided by 100)

Table 4.3: Run-off co-efficient for the 14 road culverts sub catchments and the rail culvert sub catchment for the various years.

CATCHMENT NO.	TOTAL AREA (Km2)	C for 1991	C for 2001	C for 2011	C for 2016
Railway culvert	2.88713	0.13	0.41	0.82	0.82
1	2.94481	0.132	0.41172	0.81872	0.817536
2	3.2596	0.1322072	0.414088	0.819534	0.81687
3	3.81083	0.1321776	0.450718	0.824492	0.81835
4	3.84055	0.132148	0.453234	0.824862	0.817906
5	4.44512	0.132444	0.495858	0.829154	0.82168
6	4.57046	0.132666	0.53419	0.830412	0.823752
7	4.5947	0.132592	0.505552	0.830338	0.822494
8	5.06117	0.132444	0.507476	0.83056	0.822568
9	6.23022	0.139622	0.562606	0.82575	0.815168
10	8.28888	0.165522	0.611224	0.82723	0.813392
11	8.33868	0.165818	0.610188	0.826268	0.812282
12	8.35664	0.165744	0.609448	0.825528	0.811912

13	9.93468	0.195862	0.626616	0.81687	0.805252
14	10.62131	0.206222	0.627504	0.80688	0.79911

Once again in plotting, the objective was to generate an equation of the line of best fit to be able to determine the runoff co-efficient of the construction years and any other year thereafter of the culverts. The lines of best fit are polynomials which indicate equilibrium state with respect to runoff co-efficient, however, the generated model was not good enough to predict outcomes when the exact years were used but when 1990 was referenced as year zero with 1991, 2001, 2011 and 2016 transformed to 1, 11, 21 and 26 respectively, the models gave very good outcomes. The runoff co-efficient against the transformed years gave lines of best fit whose equations are very good predictive models with regression co-efficients varying from 0.96 to 1. The various models for the sub catchments were used to compute the runoff co-efficient for the construction years of the culverts. This is demonstrated with fig.4.5 of catchment no. 14. The runoff co-efficient for catchment fourteen (Catchment no. 14) for the year 2016 was found to be is 0.79911, refer to Table 4.2.

Another plot of runoff coefficient on y-axis against years on x-axis also gives changes in runoff co-efficient over the years and the C's for the years of construction of the various culverts were determined from the graph. In deducing the runoff co-efficient from the graph, 1990 will have to be subtracted from the year the culvert was constructed for the resultant to be entered on the year axis to obtain the C.

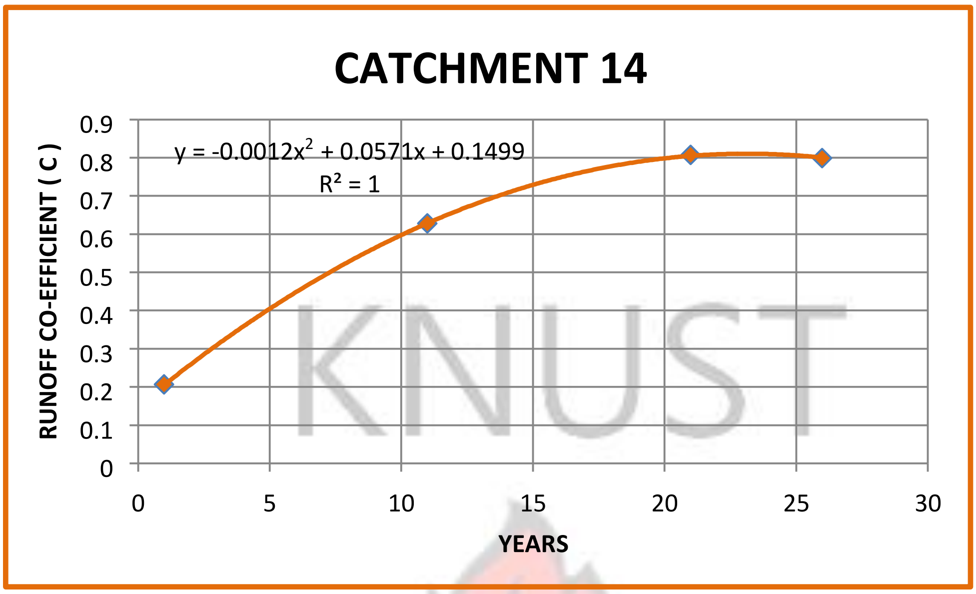

Figure 4.5: Runoff co-efficient against years for catchment no.14.

The rapid urbanisation certainly influenced rapid changes in the runoff co-efficient, C. The various values of percent impervious and C could be interpolated from the graphs. The graph of percent impervious shows a rapid increase in imperviousness till 91.15% in 2011 and a slight reduction in 2016. Similarly, the runoff co-efficient graph shows a rapid increase in C from 1991 to 2011 and a slight reduction in 2016. Percent impervious did not exceed the 2011 figure of 91.15. The marginal reduction or equilibrium state could be attributed to the adherence to the National Building Regulation1996 (Li 1630).

Lines of best fit gave polynomial equations and regression co-efficients that range between 0.96 and 1 which is an indication that increasing urbanization over the years is mainly responsible for the changes in the runoff co-efficient. In general, the catchment is in equilibrium state with runoff co-efficient (C) of 0.82 at an optimum imperviousness of 92%.

5.3.3 Time of Concentration Tc.

The time of concentration was computed using the Bransby-Williams' Equation for natural catchments and mixed flow paths which is a combination of times overland flow and channel flow as shown in equation 2.3 below;

$$t_c = \frac{92.5\,L}{A^{0.1}S^{0.2}} \qquad 2.3$$

where, t_c = Time of concentration (minutes)

L = Length of flow path from remotest point of catchment to outlet (km)

A = Catchment area (ha)

S = Slope of stream flow path (m/km)

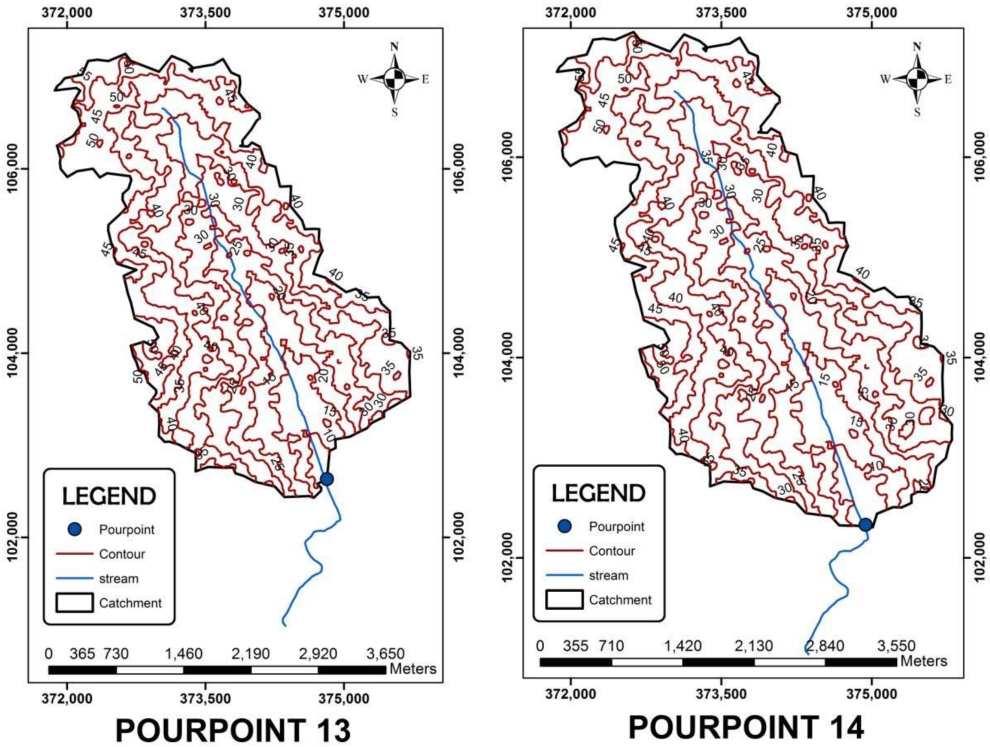

Figure 4.6: Topographical maps of sub catchment numbers 13 and 14 used for slope computations.

Table 4.4 Computation of time of concentration for catchment no.1

Catchment No.	Length to Pour point	Elevation H at pour point (m)	Elevation at remotest	Area of catchment	$\frac{92.5\,L}{A^{0.1}S^{0.2}}T_c =$

	(km)		point in catchment(m)	Km^2	ha	(min.)
1	4.788	27	59.5	2.9448	294.48	171.02

T_c = 171.02 min. = 2.85 hours

The T_c of 2.85 hours is the time it takes runoff to reach the pour point of catchment no.1. Beyond catchment no. 1, the flow is entirely channel flow with relatively less time to get to the other pour points. The various times it takes to get to the other culverts were therefore, assumed to be insignificant hence the T_c of the catchment no.1 was used for all the other culverts. This assumption was to ensure that the higher intensity of culvert no.1 is used for the others because intensity is inversely related to time of concentration. This also addresses the issues with flush floods which have the tendency to occur when flow is not impeded.

5.3.4 Determination of Intensity for Catchment No. 14

The ten years Maximum Rainfall Intensity for Accra deduced from the Ministry of Roads And Highways Consultancy Services for Coordination of the review of the Intensity Duration Frequency (Idf) of the rainfall intensity graph, (July 2016) for time of concentration of 2.85 hours is 39.77 mm/hr. as shown below;

I_{2hrs} = 51.53 mm/hr. and

I_{3hrs} = 37.70 mm/hr.

$$I_{2.85hrs} = I_{3hrs} + \left[\frac{I_{2hrs} - I_{3hrs}}{3-2}\right] \text{ x } (3 - 2.85)$$

$$= 37.70 + \left[\frac{51.53 - 37.70}{1}\right] \text{ x } 0.15$$

$$= 37.70 + 2.07$$

$I_{2.8hrs}$ = 39.77 mm/hr.

4.3.5 Computation of Discharge

The Modified Rational Method was used in computing the discharge for the various years.

$$Q = 0.278 C_R CIA \quad (m^3/s) \qquad 2.7$$

Table 4.5: Catchment No. 14

YEARS	IMPERVIOUS (%)	C= 0.74*i* + 0.132	$Q = 0.278C_RCIA$ (m^3/s)
1991	10.0	0.21	23.70
2001	67.0	0.63	72.26
2011	91.2	0.81	92.86
2016	90.2	0.80	91.94

$$\text{Where} \quad C_R = \left[1 + \frac{1.93A}{10^3}\right]^{-1} = 0.98 \qquad 2.4b$$

The use of equation 2.5 in the rational equation 2.7 to compute the peak flow was compared with the hydrograph using the HEC- HMS model. The results showed that the peak flow using rational is 91.9m^3/s and that of the HEC-HMS is 85m^3/s as shown in Fig. 4.5. These results are comparable but for the fact that this research is interested in sizing for the peak flow, the rational method was used in computing the flows.

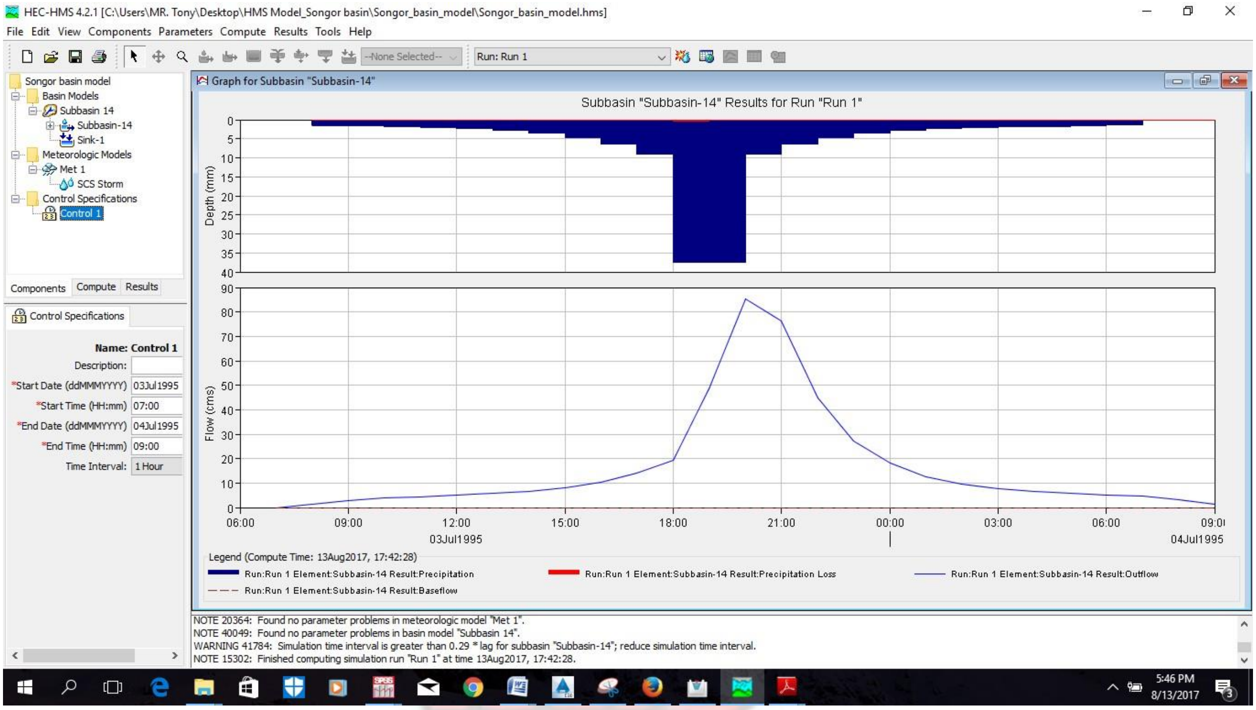

Figure 4.7: Output of hydrograph using HEC-HMS model for catchment number 14.

The flows for other catchments using the rational equation are shown in Appendix 2;

Table 4.6: Area reduction factors the various sub catchments

Catchment No.	Rail way	1	2	3	4	5	6	7	8	9	10	11	12	13	14
ARF (C_R)	0.994	0.994	0.994	0.993	0.993	0.991	0.991	0.991	0.990	0.988	0.984	0.984	0.984	0.981	0.980

Computed using equation 2.4b

4.3.6 Computation of Discharge at Construction years

In 1965 when the culvert for catchment no. 14 was constructed for access to Nungua and Tema, there was virtually no settlement within the catchment area. The Teshie Nungua estate was constructed in 1969/70 and land use changes images of 1991 even shows percentage settlement of 10.03. Land use changes affect concentration time and therefore, to obtain the time of concentration in 1965 the time lag equation, equation 2.1 below was applied.

$$T_c = \frac{l^{0.8}(S+1)^{0.7}}{1140Y^{0.5}} \qquad 2.1$$

where:

Tc = time of concentration (hr)

L = length of flow (ft)

Y = average catchment slope, %

S = maximum potential retention (in inches)

$$S = \frac{1{,}000}{cn'} - 10 \qquad 2.1a$$

where: cn′ = the retardance factor.

With clay content greater than 20% the runoff curve number is 80.

Substituting the parameters of *l* and Y of catchment 1 in equations 2.1 and 2.1a *l*=15704.64feet, Y = 0.679percent

$$S= \frac{1000}{80} - 10 \qquad S = 2.5 \text{ therefore,}$$

$$Tc = \frac{(15704.64)^{0.8}(2.5+1)^{0.7}}{1140(0.679)^{0.5}} = 5.82 \text{ hours.}$$

The earlier assumption of time of concentration at catchment number 1also applies to all the culverts in the various catchments hence Tc at time of constructing culvert for catchment number 14 is 5.82 hours. The time of concentration of was applied retrospectively to the Ghana Meteorological Services Department Departmental Note 23(Maximum Rainfall Intensity – Duration Frequencies in Ghana) by J. B. Dankwa (1974).

I_{3hrs} = 33.02mm/hr. and

I_{6hrs} = 19.56 mm/hr.

$$I_{5.82hrs} = I_{6\ hrs} + \left[\frac{I_{3hr}-I_{6hrs}}{6-3}\right] \times (6 - 5.82)$$

$$= 19.56 + \left[\frac{33.02-19.56}{3}\right] \times 0.18$$

$$= 19.56 + 0.81$$

$I_{5.82\ hrs}$ = 20.37 mm/hr.

The Tc for culverts that were constructed between 2015 and 2003 however, is 2.85hours because there were settlements in the catchment at that time and equation 2.3 applies to them. Consequently, their intensity is as follows;

I_{2hrs} = 44.45mm/hr. and

I_{3hrs} = 33.02 mm/hr.

$$I_{2.85hrs} = I_{3\ hrs} + \left[\frac{I_{2hr}-I_{3hrs}}{3-2}\right] \times (3 - 2.85)$$

$$= 33.02 + \left[\frac{44.45-33.02}{1}\right] \times 0.15$$

$$= 33.02 + 1.71$$

$I_{2.85\ hrs}$ = 34.73 mm/hr.

CATCHMENT No. 14

YEAR	IMPERVIOUS (%)	C= 0.74*i* + 0.132	Q =0.278C_RCIA (m^3/s)
1965	Assume 1.0	0.139	8.19

Table 4.7: Summary of runoff co-efficients at culvert construction year

Catchment	Rail

No.	Culvert	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Runoff co-efficient C	0.139	0.23	0.23	0.24	0.24	0.26	0.29	0.85	0.61	0.64	0.82	0.67	0.169	0.169	0.139

**An assumption of 1% impervious was made for catchment 14 and Rail Culvert; 5% for 13 and 14.*

Computation for the other catchments

The tables in Appendix 3 show runoff co-efficient and discharge for the various sub catchments at years of construction.

4.4 Assessment of the Existing Culverts at Construction Year

Civil 3D drainage software was used in determining the adequacy of the existing culverts by using the flows at the construction years. Culverts of catchment numbers 1, 2, 8, 12, 13 and 14 were found to be adequate at the year they were constructed including the one constructed by Ghana Railway cooperation in 1953 for rail transport. The rest of them were inadequate even at the construction year and it could be as a result of sizing the culverts based on one‛s experience and not through hydrological computations which turn out to be inadequate. When they were again assessed for 25 years design life, only culverts of catchment numbers 12, 13 and 14 together with the rail culvert were found to be adequate for that design life. These were culverts constructed before or in 1970. For the peak runoff using maximum runoff co-efficient C of 0.81 for the catchment, only the railway culvert is adequate for that. Table 4.6 below shows what was provided and what should have been provided for even a design life of 25 years.

Table 4.6: Table of existing and what should have been provided for design life of 25 years.

Catchment No.	Construction Year	Computed Discharge at Construction year(m^3/s)	Existing Culvert Size (meters)	Computed Discharge after 25years design life (m^3/s)	Minimum Proposed Culvert Sizes To have been constructed for 25 yrs. design life (meters)
Railway culvert	1953	2.28	5No. x 1.8m PC (Ok)	2.28	1No. x 1.8m PC
1	1994	3.82	3No. x 0.9mPC (Ok)	23.14	3No. 1.8m PC
2	1994	4.22	3No. x 0.9mPC (Ok)	25.65	3No. 1.8m PC
3	1994	5.14	3No. x 0.9mPC	29.95	4No. 1.8m PC
4	1994	5.18	3No. x 0.9mPC	30.19	4No. 1.8m PC
5	1994	6.49	3No. x 0.9mPC	34.91	4No. 1.8m PC
6	1994	7.44	3No. x 0.9mPC	35.86	2No. 3.2m x 2.0 m BC
7	2015	37.33	2no. 3m x 2m BC	36.01	2No. 3.2m x 2.0 m BC

8	2003	29.50	3no. 3m x 1.5m BC (Ok)	39.66	3No. 3m x 1.6 m BC
9	2003	38.03	2No. x 1.8m PC	48.73	2No. 4.0m x 2.0 m BC
10	2014	64.58	2no. 3m x 2.2m BC	64.58	3No. 4m x 2.2 m BC
11	2003	53.08	1no. 4m x 1.8m BC	64.97	3No. 4m x 2.0 m BC
12	1969	7.87	1no. 3m x 1.3m BC (Ok)	15.37	1No. 3.5m x 1.5 m BC
13	15/5/1969	9.32	1no. 3m x 1.3m BC (Ok)	19.31	1No. 4.5m x 1.5 m BC
14	1965	8.19	2no. 3.5m x 1.5m BC (Ok)	8.84	1No. 3.5m x 1.5 m BC

Note: BC is box culvert, PC is pipe culvert and Ok implies adequacy at construction year.

The flow of catchment no. 14 at construction year is demonstrated in Fig. 4.8 below.

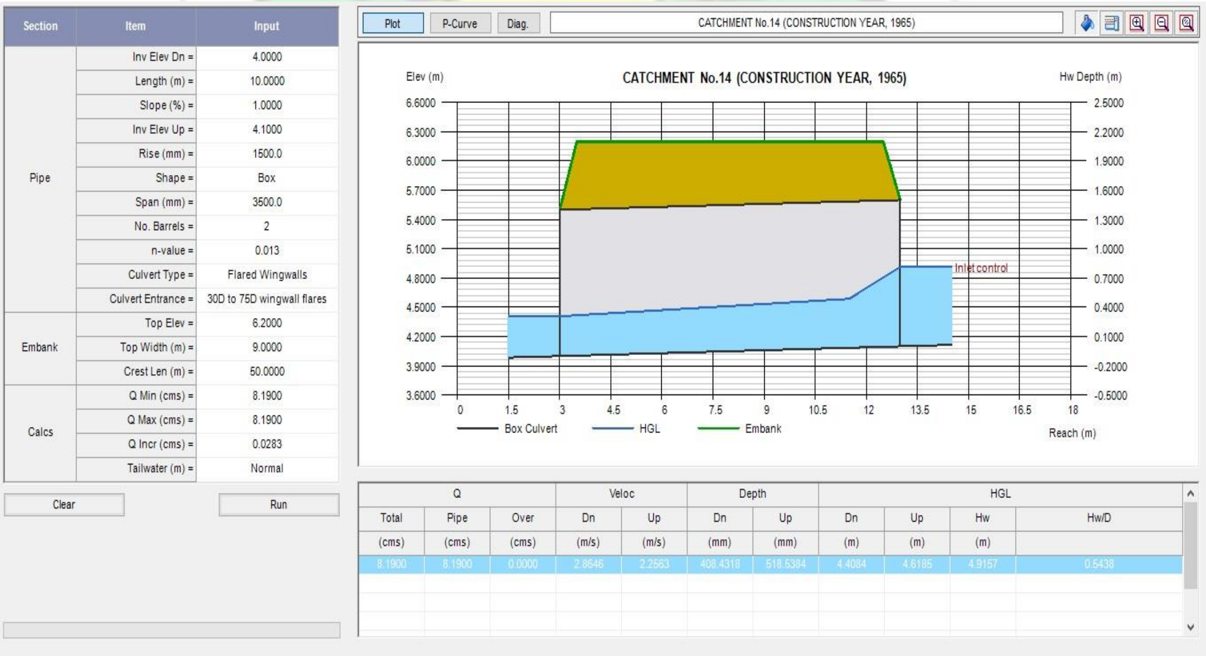

Figure. 4.8: Culvert in catchment no. 14 during construction year, 1965

The culvert was flowing under inlet control with no flow over and with Hw/D ratio of 0.5438. The acceptable upper range of the Hw/D is 1 – 1.5, according to the Department of Urban Roads drainage manual. The low Hw/D ratio at the construction year could be explained by the fact that it was constructed with the future urbanization of the catchment in mind. This was very good to prevent the inundation of the road since it is a minor arterial road with heavy traffic between Accra and Tema. The upstream velocity of 2.26m/s and downstream velocity of 2.87m/s were within the acceptable average velocity limits of 0.6 - 3.0 m/s, GHA design guide; Table 6.5.1. This is to prevent gutter scouring and material deposition through erosive velocities and low velocities respectively. Similar analysis of the remaining culverts were done, however, six out of the fourteen culverts across the road networks were adequate at construction year.

4.5 Analysis of the Performance of The Existing Culverts

The analysis of the culverts at construction year was followed with analysis of the performance of the culverts till the year the study is taking place which is 2017. With the exception of the Ghana Railway cooperation culvert, the six culverts that were adequate at the years of construction together with those that were under capacity even at the various years of construction showed high over topping implying that the culverts cannot pass the peak flow of the stream safely. This causes washout of the road networks and flooding with moderate to heavy rainfall. It is, therefore, established that the culvert sizes is a major contributing factor to the flooding and washout. This is illustrated using catchment no. 14 in Fig 4.9 below.

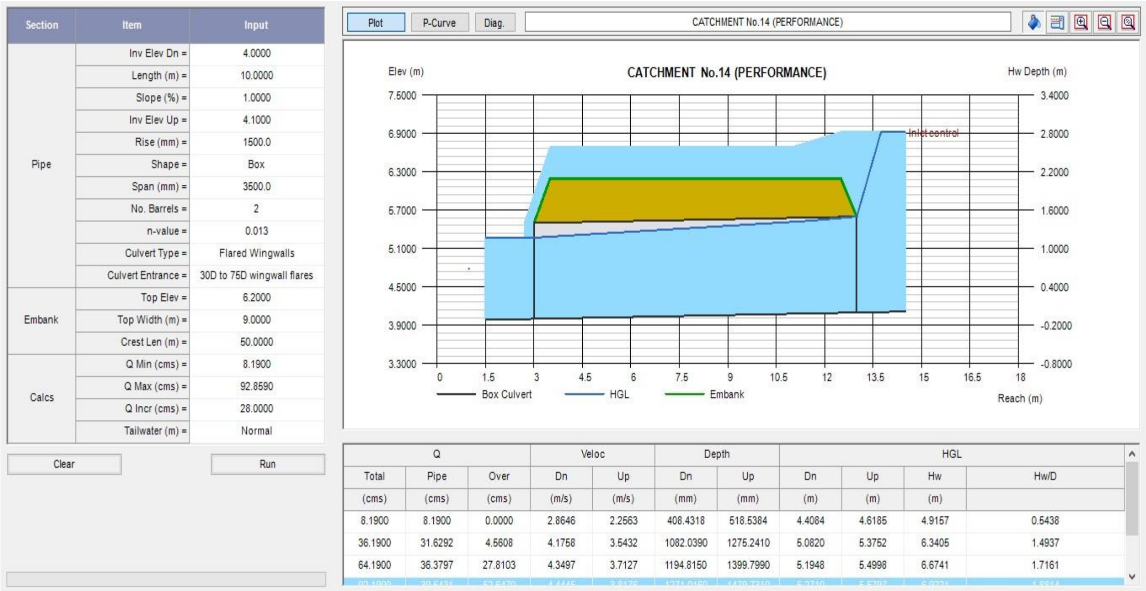

Figure 4.9: Performance of Culvert in catchment no. 14

The culvert for Catchment no. 14 above shows overtopping at a discharge of even 36.19m^3/s at Hw/D ratio of 1.49. The overflow is 4.60m^3/s which increases with increasing discharge. It also shows increasing velocities with increasing overtopping which energizes it to wash out sections of the road pavement. The Hw/D ratio also increases with increasing discharge from 0.54 at year of construction to 1.88 in the study year which exceeds the acceptable upper limit range of 1 – 1.5. This condition certainly facilitates overtopping and washout. At peak flow, the overflow alone is 52.65m3/s. The upstream velocity is 3.82m/s and the downstream is 4.44m/s. These velocities are beyond the acceptable average velocities and are erosive.

4.6 Analysis of the Proposed Culverts

Following from the analysis of the performance of the existing culverts at the study year, the study units were found to be inadequate for safe passage of peak flow except the culvert constructed by Ghana Railway Corporation in 1953. The graph of runoff coefficient C against years in fig. 4.5 shows that the maximum or equilibrium runoff coefficient C has been attained at a stable runoff co-efficient of 0.81. The C of 0.81 was therefore, used in the rational formula of equation 2.4 to compute for the study year‘s discharge to the culvert no. 14. 30% clearance was applied to the computed discharges to cater for silt build up due to the fact that the environment has Waste Transfer Stations (Department of Urban Roads Highway Drainage Manual Second Edition; August, 2006). The resultant outcomes were used in sizing of the culverts. This was to ensure that after reconstruction of the proposed culverts, their capacities are not exceeded beyond the ten years return period exceedence probability during their design life. The choice of ten years return period was primarily based on economic factors and it satisfies the MRH design specifications. The longer the return period, the bigger the size of the culvert has to be to minimize on exceedence which comes at a cost.

Proposed sizes of the culverts were made with Civil 3D drainage software with the objective of addressing the washout of road networks and flooding of parts of the communities within the catchment using the peak discharges. The use of the maximum runoff co-efficient C was to ensure that whatever investment that goes into the reconstruction of the culverts, it will have a long term positive impact on the road networks and the communities in the catchment.

The analysis showed that the 5No. 1.8m diameter pipe culvert constructed by Ghana Railway Cooperation in the year 1953, is the only culvert that is still adequate to pass the peak flow at maximum runoff co-efficient. This shows the foresight of the drainage designer which was sized to last for as long as the concrete can last. Comparing it to those that have to be replaced, obviously the railway culvert that was constructed with the future in mind appears cheaper. The culverts were sized with catchment no. 14 shown in Fig. 4.10 below and the remaining shown in Appendix 4

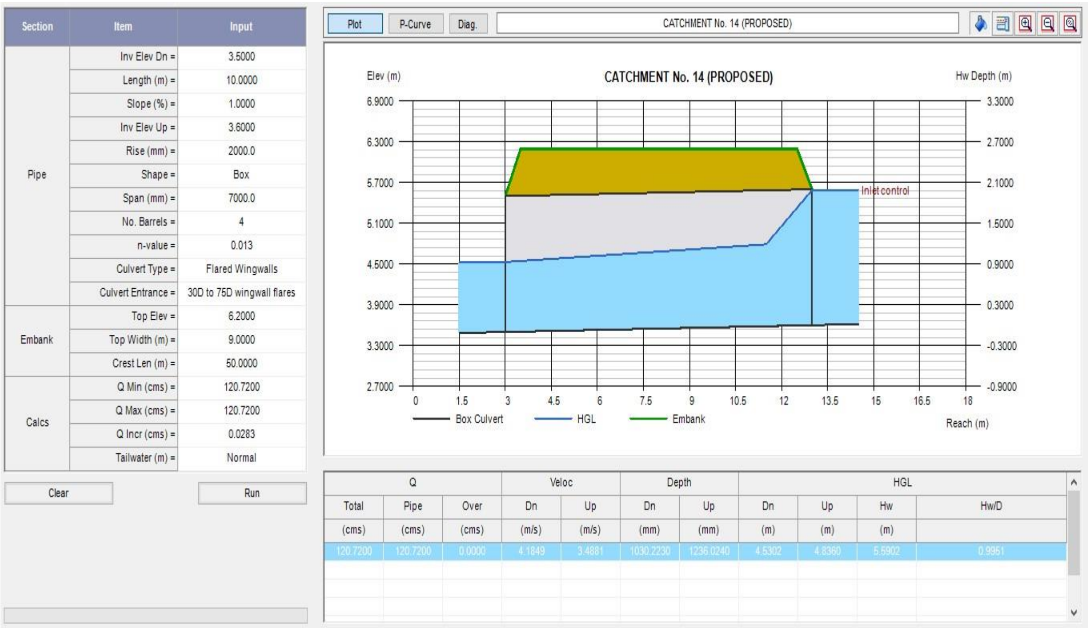

Figure. 4.10: Sizing of proposed culvert for catchment no. 14 for flow of 92.0m3/s

The above culvert in fig 4.9 will flow under inlet control with no flow over or overtopping and with Hw/D ratio of 1.05. The Hw/D ratio is within acceptable limits which will prevent the inundation of the road. The upstream velocity of 3.58m/s and downstream velocity of 4.27m/s fall outside the acceptable average velocity limits of

0.6 - 3.0 m/s. These velocities have high erosive tendencies that can cause gutter scouring and in some cases creating defects in portions of embankments. In such a situation, the culvert should be constructed with toes and scour checks at the inlet and outlet ends to address the challenges that the high velocities pose. Similar analysis of the remaining culverts was done to ensure that the proposed culverts are capable of solving the drainage problems of the catchment.

4.7 Justification for Implementation of Proposed Culverts

The combined cost of routine maintenance on roads at stream crossings in the study area per year approximating Ghs 811,233.24 (equation 4.2) and loss of lives through flooding were considered to justify the need for urgent replacement to save lives and reduce the road agency cost on maintenance and the road users cost. The community will also be free from flooding and abandoned houses will be available to help address housing needs in the community. Consequently, channel widening/improvement is recommended especially at the lined sections of the stream channel which restrict free flow. Recommendations and conclusions will therefore, be forwarded to the appropriate road agencies for the needed action to be taken.

Table 4.7: Table showing the existing culvert situation with what is adequate (proposed).

Catchment No.	Construction Year	Computed Discharge at Construction Year(m^3/s)	Existing Culvert Size (meters)	Designed flow at optimum C of 0.81(m^3/s)	Design flow with 30%	Proposed Culvert Size (meters)
Railway culvert	1953	2.28	5No. x 1.8m Ø PC	26.02	33.83	5No. x 1.8m Ø PC
1	1994	3.82	3No. x 0.9mPC	26.460	34.40	2No. 4.0m x 2m
2	1994	4.22	3No. x 0.9mPC	29.378	38.19	2No. 4.0m x 2m
3	1994	5.14	3No. x 0.9mPC	34.467	44.81	2No. 4.8m x 2m
4	1994	5.18	3No. x 0.9mPC	34.780	45.21	2No. 4.8m x 2m

5	1994	6.49	3No. x 0.9mPC	40.419	52.54	3No. 4.0m x 2m
6	1994	7.44	3No. x 0.9mPC	41.559	54.03	3No. 4.0m x 2m
7	2015	37.33	2no. 3m x 2m BC	41.741	54.26	3No. 4.0m x 2m
8	2003	29.50	3no. 3m x 1.5m BC	46.024	59.83	3No. 4.0m x 2.2m
9	2003	38.03	2No. x 1.8 n PC	56.212	73.08	3No. 5.0m x 2.2m
10	2014	64.58	2no. 3m x 2.2m BC	74.59	96.97	4No. 4.5m x 2.2m
11	2003	53.08	1no. 4m x 1.8m BC	74.944	97.43	4No. 4.5m x 2.2m
12	1969	7.87	1no. 3m x 1.3m BC	75.125	97.66	4No. 4.5m x 2.2m
13	15/5/1969	9.32	1no. 3m x 1.3m BC	87.991	114.39	4No. 6.0m x 2m
14	1965	8.19	2no. 3.5m x 1.5m BC	92.859	120.72	4no. 7.0m x 2m BC or 30m span bridge

Note: BC is box culvert and PC is pipe culvert.

From the above investigations, assessment and analysis, the objectives of the study has been achieved. The performance of the culverts as at the study year show a lot of overflows and the inadequacy of the road culverts create a dam upstream which is responsible for the flooding parts of the two communities.

4.8 Creation of Detention Pond

The stream certainly flows beyond the culvert at catchment no. 14 through the community around Old Kasapreko area at Nungua to the sea, the community live between the Tema Beach Road and the beach. The impacts of flooding on this community especially when peak flows coincide with periods of high tides are unimaginable. Certainly, the rich who can afford expensive building plots at prime areas in the city do not reside there. The community is made worse off on annual bases due to floods but they are compelled to live there probably for lack of resources to relocate. This community is the worst affected compared with the GREDA and Teshie Nungua communities confirmed by Karley (2009), which he referred to as Coco Beach.

The existing situation where the undersized culverts dam the runoff upstream even floods the Old Kasapreko community. The flooding will compound when the culverts are reconstructed to allow free flow, certainly flush flooding condition could prevail with even a moderate amount of rainfall at high tides. It is therefore, recommended that a detention pond be created to regulate the flow to this community during high tides.

This study also assessed possible areas of constructing a detention pond, fortunately a spacious area between culverts of catchments 12 and 13 along the western bank of the stream was identified.

Plate 4.1: Proposed site for construction of detention pond along the western bank of Songor stream.

The area is along the Songor steam which also gets flooded any time the stream floods however, it is not certain if it is part of the wetlands or has been sold out to developer(s). The time within which this study is to be undertaken will not allow for the hydraulic design of the proposed detention pond and it is recommended that another study takes it up.

CHAPTER 5: CONCLUSION AND RECOMMENDATION

5.1 Conclusion

The important outcomes of the study have been listed below; they are arranged in accordance with the stated objectives of the study.

1. Flooding of Songor stream catchment area from GREDA estates to Teshie - Nungua estates and along Onukpa Wahe stream at Communities 19 and 20 exist, these were observed through visits to affected areas soon after moderate to heavy rainfalls. Initial observation showed that the culverts were constricting flow. The Songor stream catchment was selected for detailed study. The flooding was confirmed through interview of both residents of GREDA and Teshie - Nungua estates and the LEKMA roads department.

2. The estimated annual cost of road maintenance given that culvert approaches are maintained once and the cost incurred by residents due to flooding is Ghs 811,233.24, all things being equal, ten years combined residents cost due to floods and road agency cost of road maintenance is enough to replace all the undersized culverts except the proposed bridge.
3. The existing culverts were constructed at different years between 1953 and 2015 by different agencies. Rapid urbanization has increased the imperviousness of the catchment translating into runoff beyond what the culverts could safely discharge. Only the railway culvert was found to be adequate at the study year. The runoff co-efficient against year graphs also show equilibrium state of the catchment with runoff co-efficient of 0.82 at imperviousness of 92%.
4. A runoff co-efficient model for the catchment is; $C = 0.74IM + 0.132$, this can be applied to other catchments with similar characteristics.
5. Civil 3D drainage software was used for the hydraulic analysis. Only seven (7) culverts of catchment numbers 1, 2, 8, 12, 13, 14 and the railway culvert, most of which were constructed before or in 1970, were adequate at the construction year. Again out of the seven culverts, only culvert for catchment number 14 and the Railway culvert were still adequate after 25 years design life. Eight (8) of the culverts were not adequate even at construction year and it appears that no hydrological and hydraulic assessment was undertaken, however, the railway culvert constructed in 1953 was sized to be adequate for the maximum flow of the catchment.

5.2 Recommendation

The following are recommended for the Ministry of Roads and Highways and Hydrological Services Department consideration;

1. The Ghs 811,233.24 (from equation 4.2) been the combined cost of road maintenance and residents maintenance cost in the flood prone area annually be arrested immediately by replacing the under sized culverts. This will certainly impact on socioeconomic lives of the community.
2. The concrete lined sections of the channel have become too small to allow free flow of the stream. This should be reconstructed with the varying width in mind to ensure free flow.
3. The study can be applied to other flood prone areas in Accra and other cities worldwide. It can also lead to projecting into the future to predict if the culvert will exceed capacity and when. This can help plan ahead of time for flood prevention.
4. Culverts at various catchments should be sized using the maximum runoff coefficient of their catchments. This will ensure that culverts are adequately sized to meet rapid urbanization demands of the catchment.
5. Regular removal of silt in the steam channel to improve free flow is recommended.
6. The LEKMA should fence the un engineered Waste Transfer Station to minimize on wastes from the station getting into the stream especially when it rains.
7. Detention ponds in open areas, under ECG pylons near flood prone streams should be explored to minimize the impact of flooding on communities.

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APPENDIX 1: QUESTIONNAIRES TO RESIDENTS AND LEKMA ROADS DEPARTMENT

<u>QUESTIONNAIRES TO RESIDENTS OF GREDA AND TESHIE NUNGUA ESTATES ALONG THE SONGOR STREAM</u>

1. Name of Resident.(optional) …………………………………………………

   Please tick in the appropriate boxes

2. Age group ; 20 – 30 ☐ , 31 – 40 ☐, 41 – 50 ☐, 51 – 60 ☐, 61 – 70 ☐ 70 and above

3. Name of the street that is close to your house/access road; …………………

4. Number of occupants in the house;1 – 5 ☐, 6 – ☐ 10 , 11 – 15 ☐, 16 -20 ☐

5. Type of finishing of compound; Terrazo/Concrete paving ☐, Tiles ☐ green grass ☐, earth/gravel ☐ , a combination of tiles and grass ☐, a combination of terrazzo/concrete and green grass ☐.

6. How close is your house from the stream; Next to the stream ☐, Two houses from the stream ☐ , Three houses from the stream ☐ , Four houses or more from the stream ☐ Others(specify)……………………………………

7. Does the stream floods with medium to heavy rainfall at your area? Yes ☐ , No

8. If no. 7 is Yes, what level of intensity of rainfall in your area cause the flooding to occur? (*please tick all that apply*). Light intensity of ☐ rainfall Medium intensity of rainfall ☐ , Heavy intensity of rainfall

9. If question 7 is Yes, on the average how often does the stream floods/ overflows its banks in the main rainy season (April - July) every year ; 1 – 5times ☐ , 5 – 10times ☐, 11times and more

10. Again if question 7 is Yes, on the average how often does the stream floods/ overflows its banks in the minor rainy season (September - November) every year; 1 – 3times ☐ , 4 – 6times ☐ , 7 – 10times ☐ , More than 10 times

11. Does the floods come with fatalities/loss of lives in your locality; Yes ☐ , No .

12. If No. 11 is yes, how many fatalities do you recollect in the last 5 years in the area; None ☐, 1 – 5 ☐, 6 – 10 ☐ , 11 – 15 ☐ , 16 and above ☐ .

13. Do you suffer from loss of property damages as a results of the flooding ; Yes , No

14. If No. 13 is Yes, if you are to estimate the extent of damages you suffer or amount spent in a year to protect lives and property, how much will that be? ;

None ☐,	Ghs1 – Ghs 1000 ☐ ,
Ghs 1001 - Ghs 2000 ☐ ,	Ghs 2001 - Ghs 5000 ☐ ,
Ghs 5001 - Ghs 10000 ☐ ,	Above Ghs 10000

15. Which year did you start noticing the flooding in your community? …………

16. In your own candid opinion, in what way(s) is the community contributing to the flooding of the Songor stream?;

Encroaching on the stream channel ☐, Throwing of garbage into drains and the stream when it rains ☐ , Others (explain)…………………………………………………………………………………
…………………………………………………………………………………………
…………………..

17. What do you advice on, other than the hydraulic capacities of the culverts, in addressing the flooding of parts of the community and sections of the road networks?
…………………………………………………………………………………………
…………………………………………………………………………………………
…………………………………………………………………………………………

QUESTIONNAIRE TO LEKMA MUNICIPAL ROADS ENGINEERS ON EXPENDITURE ON MAINTENANCE ON ROAD NETWORKS WITHIN THE CATCHMENT AREA OF THE SONGOR STREAM

1.Name of Municipal Roads Agency/Engineer:……………………………………

Please tick in the appropriate boxes, where multiply reasons apply please tick multiple.

2. The catchment of Sangor stream lies within your municipality; Yes ☐ , No ☐
.
3. The following streets/roads cross the stream within your municipality with culverts of various sizes;

(vi) Nineth Avenue Street Yes ☐ , No

(vii) Eigth Avenue Street Yes ☐ , No

(viii) Seventh Avenue Street Yes ☐ , No

(ix) Sixth Avenue Street Yes ☐ , No

(x) The Road E Yes ☐ , No

(xi) Third Avenue extension Street Yes ☐ , No

(xii) The Tsui Bleo Road Yes ☐ , No

(xiii) The Volta Street Yes ☐ , No

(xiv) The Coffee Street Yes ☐, No

(xv) The Cocoa Street Yes ☐, No

(xvi) The Tema Beach Road Yes ☐, No

4. Sections of the road networks listed under item no. 3 experience floods and wash out with other defects resulting from the floods of medium to heavy rainfall in the rainy seasons; Yes ☐, No ☐.

5. Does the Road Unit receive complaints of or observe flooding of the roads with medium to heavy rainfall? Yes ☐, No

6. Do you undertake maintenance activities after the rainy season? Yes ☐, No .

7. If no. 5 is yes, as part of maintenance measures to restore the road networks, how much was spent on maintenance on each of the road links listed below, which are within the stream basin, in your last maintenance activity?
   (i) Nineth Avenue Street Ghs …………………………….

   (ii) Eigth Avenue Street Ghs………………………………..

   (iii) Seventh Avenue Street Ghs……………………………….

   (iv) Sixth Avenue Street Ghs………………………………

   (v) The Road E Ghs………………………………

   (vi) Third Avenue extension Street Ghs……………………………….

   (vii) The Tsui Bleo Road Ghs………………………………..

   (viii) The Volta Street Ghs………………………………..

   (ix) The Coffee Street Ghs……………………………….

   (x) The Cocoa Street Ghs………………………………

   (xi) The Tema Beach Road Ghs……………………………….

8. Total maintenance cost on the networks listed under item no. 7; Ghs…………………..

9. What do you think are the causes of the floods

……………………………………………………………………………………
……………………………………………………………………………………
……………………………………………………………………………………
……………………………

10. I believe the unit made efforts as the Roads Unit to address the floods issues from hydraulic inadequacies view point ; Yes ☐ , No

11. If item No.10 is yes, what are some of the challenges that the Roads Unit was confronted with in addressing the flooding issues;

 Budgetary approval ☐ , Social (cooperation of the community for the necessary works to be undertaken ☐ , Continuous pollution of water body/stream ☐ Delayed payments of previously executed projects ☐
 .

12. Which year were the culverts and roads constructed in the GREDA estate enclave? ……………..

13. Which year were the culverts and roads constructed in the Teshie Nungua estate enclave? …………….

14. Which year did you start noticing or receiving information about the flooding of the GREDA community? ………………..

15. Which year did you start noticing or receiving information about the flooding of the Teshie - Nungua community? …………

16. In the opinion of the Road Unit, in what way(s) is the community contributing to the flooding of the Songor stream ?; Encroaching on the stream channel ☐ , Throwing of garbage into drains and the stream when it rains ☐,
 Others ☐
 explain……………………………………………………………………………
 …………………………………………………………………………………….

17. What do you advice on, other than the hydraulic capacities of the culverts, in addressing the flooding of parts of the community and sections of the road networks?
 ……………………………………………………………………………………
 ……………………………………………………………………………………
 ……………………………………………………………………………………
 …………………………….

APPENDIX 2: FLOWS FOR OTHER CATCHMENTS USING THE RATIONAL EQUATION

RAILWAY CULVERT

YEARS	IMPERVIOUS (%)	$C = 0.74i + 0.132$	$Q = 0.278C_RCIA$ (m^3/s)
1991	0	0.13	4.12
2001	37.8	0.41	13.01
2011	92.8	0.82	26.02
2016	92.6	0.82	26.02

CATCHMENT No. 1

YEARS	IMPERVIOUS (%)	$C = 0.74i + 0.132$	$Q = 0.278C_RCIA$ (m^3/s)
1991	0	0.13	4.27

2001	37.8	0.41	13.31
2011	92.8	0.82	26.46
2016	92.6	0.82	26.46

CATCHMENT No. 2

YEARS	IMPERVIOUS (%)	C = 0.74i + 0.132	Q =0.278C_RCIA (m^3/s)
1991	0.0	0.13	4.73
2001	38.1	0.41	14.83
2011	92.9	0.82	29.38
2016	92.6	0.82	29.38

CATCHMENT No. 3

YEARS	IMPERVIOUS (%)	C = 0.74i + 0.132	Q =0.278C_RCIA (m^3/s)
1991	0.0	0.13	5.52
2001	43.1	0.45	18.87
2011	93.6	0.82	34.47
2016	92.8	0.82	34.22

CATCHMENT No. 4

YEARS	IMPERVIOUS (%)	C= 0.74i + 0.132	Q =0.278C_RCIA (m^3/s)
1991	0.0	0.13	5.57
2001	43.4	0.45	19.10
2011	93.6	0.83	34.78
2016	92.7	0.82	34.49

CATCHMENT No. 5

YEARS	IMPERVIOUS (%)	C= 0.74*i* + 0.132	Q =0.278C_RCIA (m^3/s)
1991	0.1	0.13	6.44
2001	49.2	0.50	24.18
2011	94.2	0.83	40.42
2016	93.2	0.82	40.08

CATCHMENT No. 6

YEARS	IMPERVIOUS (%)	C= 0.74*i* + 0.132	Q =0.278C_RCIA (m^3/s)
1991	0.1	0.13	6.66
2001	54.4	0.53	26.74
2011	94.4	0.83	41.56
2016	93.5	0.82	41.26

CATCHMENT No. 7

YEARS	IMPERVIOUS (%)	C= 0.74*i* + 0.132	Q =0.278C_RCIA (m^3/s)
1991	0.1	0.13	6.69
2001	50.5	0.51	25.45
2011	94.4	0.83	41.74
2016	93.3	0.82	41.34

CATCHMENT No. 8

YEARS	IMPERVIOUS (%)	C= 0.74*i* + 0.132	Q =0.278C_RCIA (m^3/s)
1991	0.1	0.13	7.31
2001	50.7	0.51	28.08

2011	94.4	0.83	46.02
2016	93.3	0.82	45.58

CATCHMENT No. 9

YEARS	IMPERVIOUS (%)	C= 0.74*i* + 0.132	Q =0.278C_RCIA (m^3/s)
1991	1.0	0.14	9.53
2001	58.2	0.56	38.25
2011	93.8	0.83	56.21
2016	92.3	0.82	55.46

CATCHMENT 10

YEARS	IMPERVIOUS (%)	C = 0.74*i* + 0.132	Q =0.278C_RCIA (m^3/s)
1991	4.5	0.17	14.97
2001	64.8	0.61	55.11
2011	94.0	0.83	74.59
2016	92.1	0.81	73.32

CATCHMENT No. 11

YEARS	IMPERVIOUS (%)	C= 0.74*i* + 0.132	Q =0.278C_RCIA (m^3/s)
1991	4.8	0.17	15.06
2001	64.6	0.61	55.35
2011	93.8	0.83	74.94
2016	91.9	0.81	73.68

CATCHMENT No. 12

YEARS	IMPERVIOUS (%)	C= 0.74*i* + 0.132	Q =0.278C_RCIA (m^3/s)
1991	4.6	0.17	15.10
2001	64.5	0.61	55.39
2011	93.7	0.83	75.13
2016	91.9	0.81	73.85

CATCHMENT No. 13

YEARS	IMPERVIOUS (%)	C= 0.74*i* + 0.132	Q =0.278C_RCIA (m^3/s)
1991	8.6	0.20	21.11
2001	66.8	0.63	67.53
2011	92.6	0.82	87.99
2016	91.0	0.81	86.70

APPENDIX 3: COMPUTATION OF DISCHARGE AT CONSTRUCTION YEARS FOR VARIOUS SUB CATCHMENTS

RAILWAY CULVERT

YEAR	IMPERVIOUS (%)	C= 0.74*i* + 0.132	Q =0.278C_RCIA (m^3/s)
1953	0	0.14	2.28

CATCHMENT No. 1

YEAR	IMPERVIOUS (%)	C= 0.74*i* + 0.132	Q =0.278C_RCIA (m^3/s)
1994	12.9	0.23	3.82

CATCHMENT No. 2

YEAR	IMPERVIOUS (%)	C= 0.74*i* + 0.132	Q =0.278C_RCIA (m^3/s)
1994	13.1	0.23	4.22

CATCHMENT No. 3

YEAR	IMPERVIOUS (%)	C= 0.74*i* + 0.132	Q =0.278C_RCIA (m^3/s)
1994	15.0	0.24	5.14

CATCHMENT No. 4

YEAR	IMPERVIOUS (%)	C= 0.74*i* + 0.132	Q =0.278C_RCIA (m^3/s)
1994	15.1	0.24	5.18

CATCHMENT No. 5

YEAR	IMPERVIOUS (%)	C= 0.74*i* + 0.132	Q =0.278C_RCIA (m^3/s)
1994	17.4	0.26	6.49

CATCHMENT No. 6

YEAR	IMPERVIOUS (%)	C= 0.74*i* + 0.132	Q =0.278C_RCIA (m^3/s)
1994	19.3	0.29	7.44

CATCHMENT No. 7

YEAR	IMPERVIOUS (%)	C= 0.74*i* + 0.132	Q =0.278C_RCIA (m^3/s)
2015	95.4	0.85	37.33

I = 39.77mm/hr was used because in 2015, there were settlements and equation 4.4 was applied.

CATCHMENT No. 8

YEAR	IMPERVIOUS (%)	C= 0.74i + 0.132	Q =0.278C_RCIA (m^3/s)
2003	63.3	0.61	29.50

CATCHMENT No. 9

YEAR	IMPERVIOUS (%)	C= 0.74i + 0.132	Q =0.278C_RCIA (m^3/s)
2003	68.8	0.64	38.03

CATCHMENT No. 10

YEAR	IMPERVIOUS (%)	C= 0.74i + 0.132	Q =0.278C_RCIA (m^3/s)
2014	93.8	0.82	64.58

CATCHMENT No. 11

YEAR	IMPERVIOUS (%)	C= 0.74i + 0.132	Q =0.278C_RCIA (m^3/s)
2003	73.9	0.67	53.08

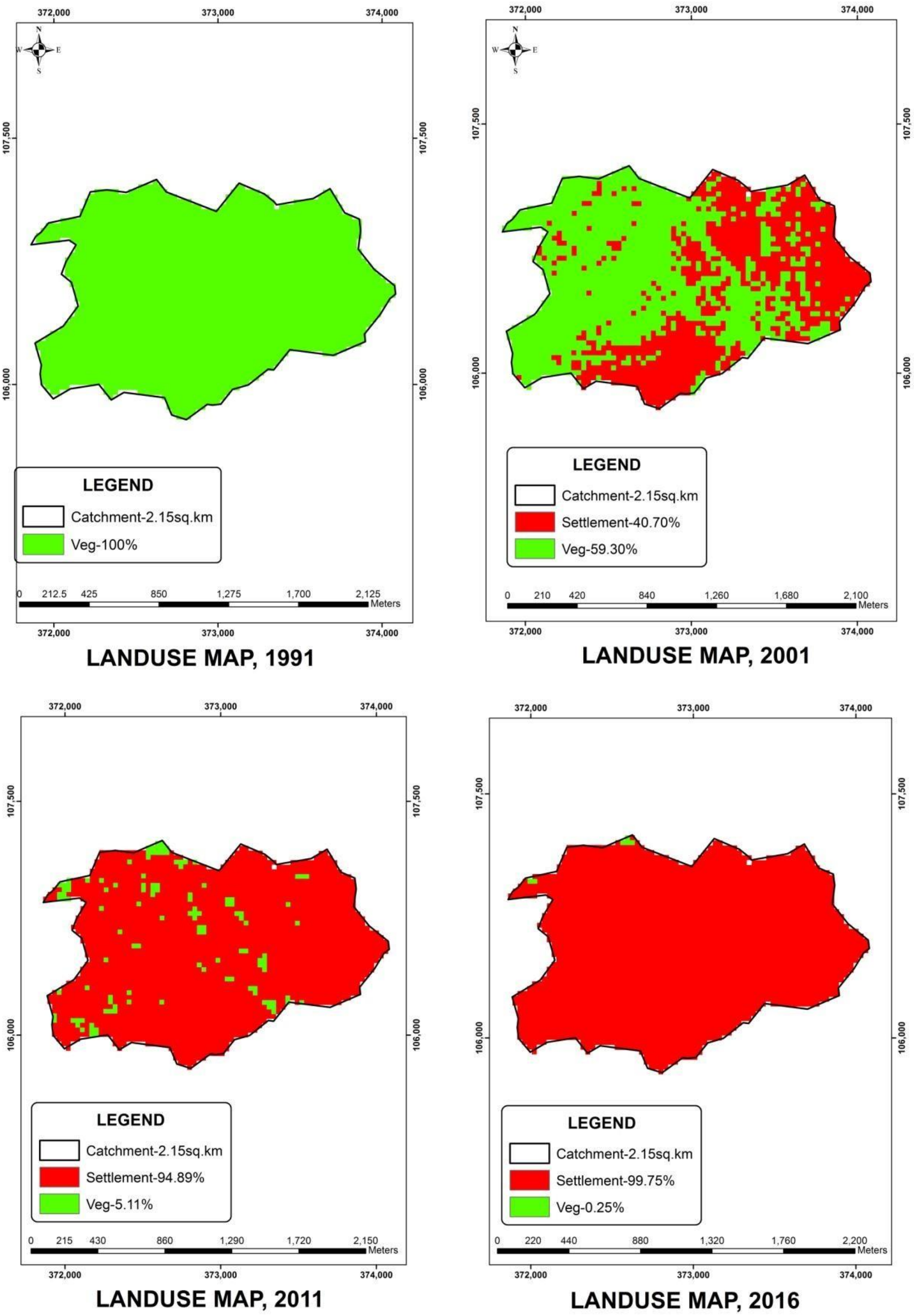
CATCHMENT No. 12

YEAR	IMPERVIOUS (%)	C= 0.74*i* + 0.132	Q =0.278C_RCIA (m^3/s)
1969	Assume 5.0	0.17	7.87

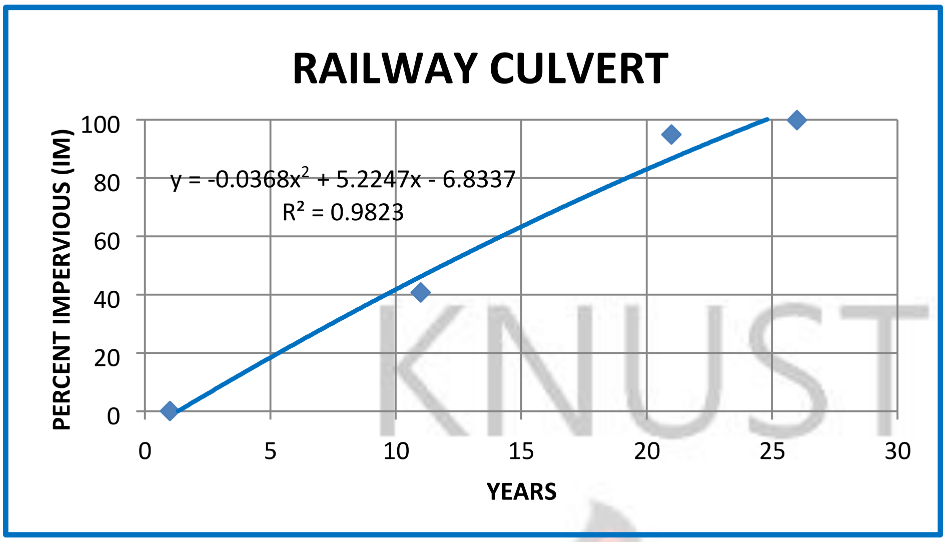
CATCHMENT No. 13

YEAR	IMPERVIOUS (%)	C= 0.74*i* + 0.132	Q =0.278C_RCIA (m^3/s)
15/5/1969	Assume 5.0	0.17	9.32

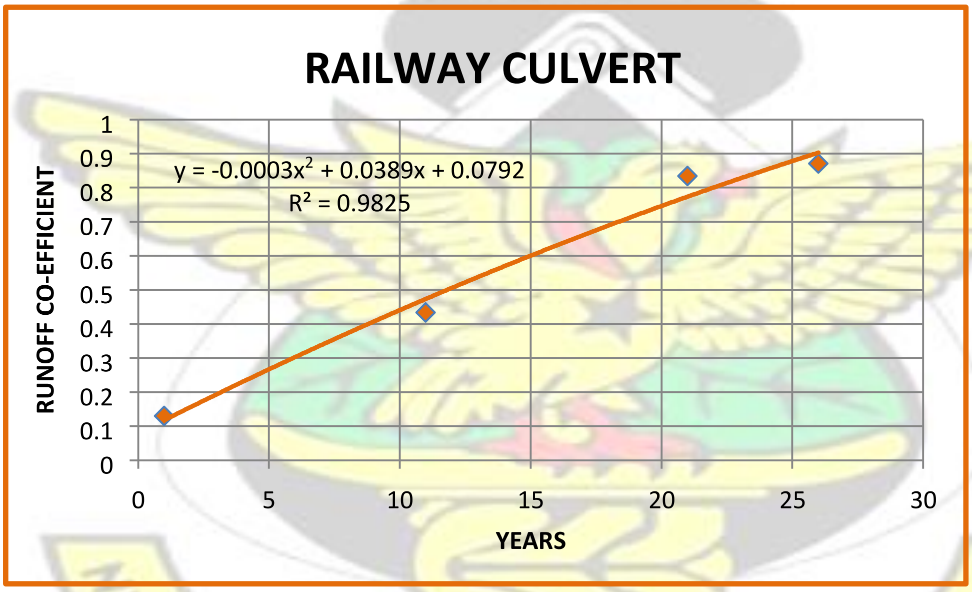
APPENDIX 4: ANALYSIS OF OTHER CATCHMENTS AND THEIR RESPECTIVE CULVERTS



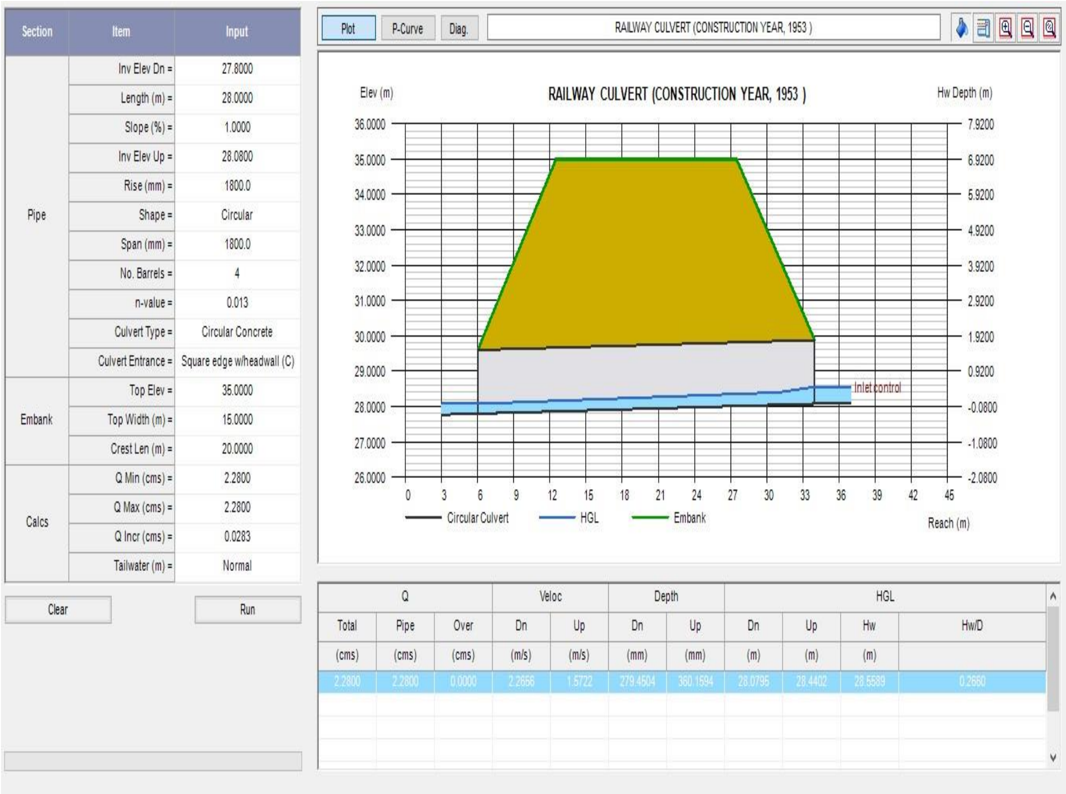
Appendix 4, Fig. 4.10: Land use, land cover changes for Railway sub catchment



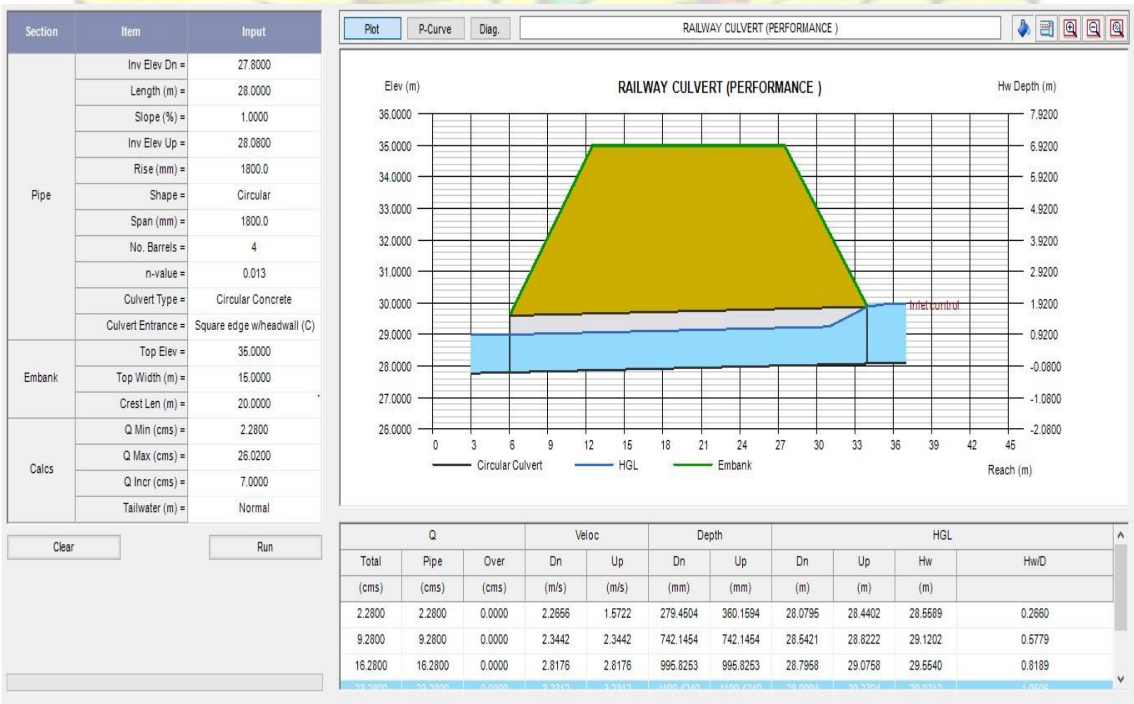
Appendix 4, Fig 4.11: Graph of percent impervious against years for catchment 1



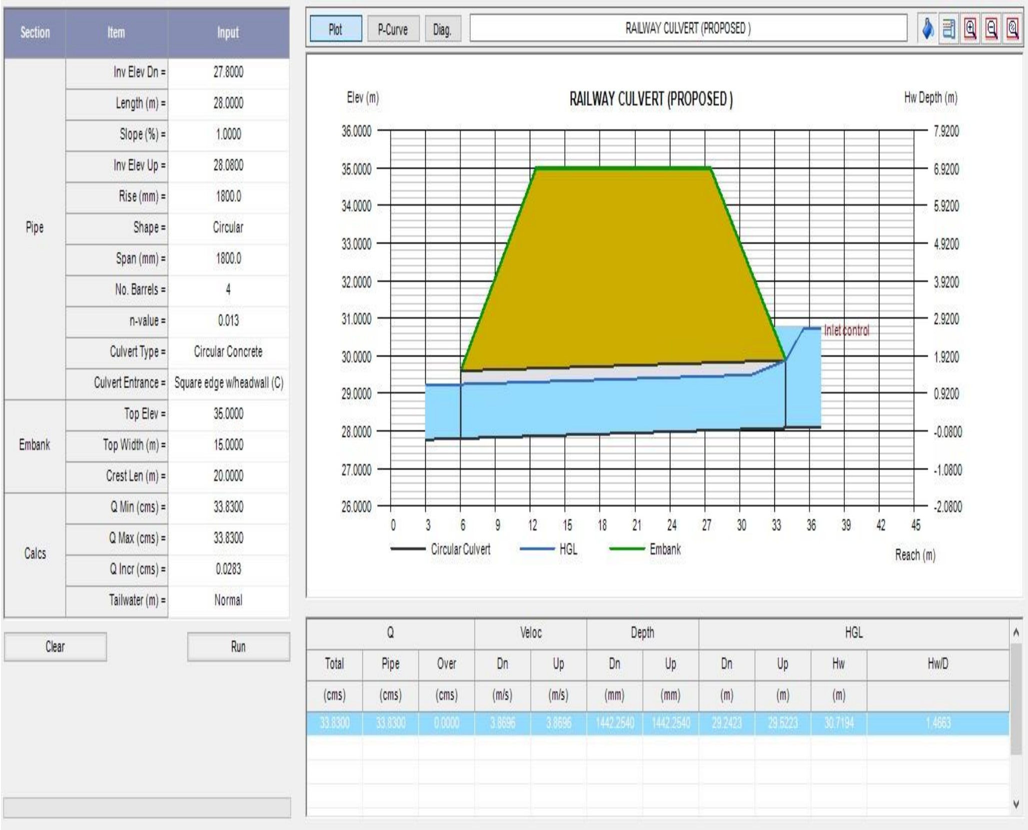
Appendix 4, Fig 4.12: Graph of runoff co-efficient against years for catchment 1



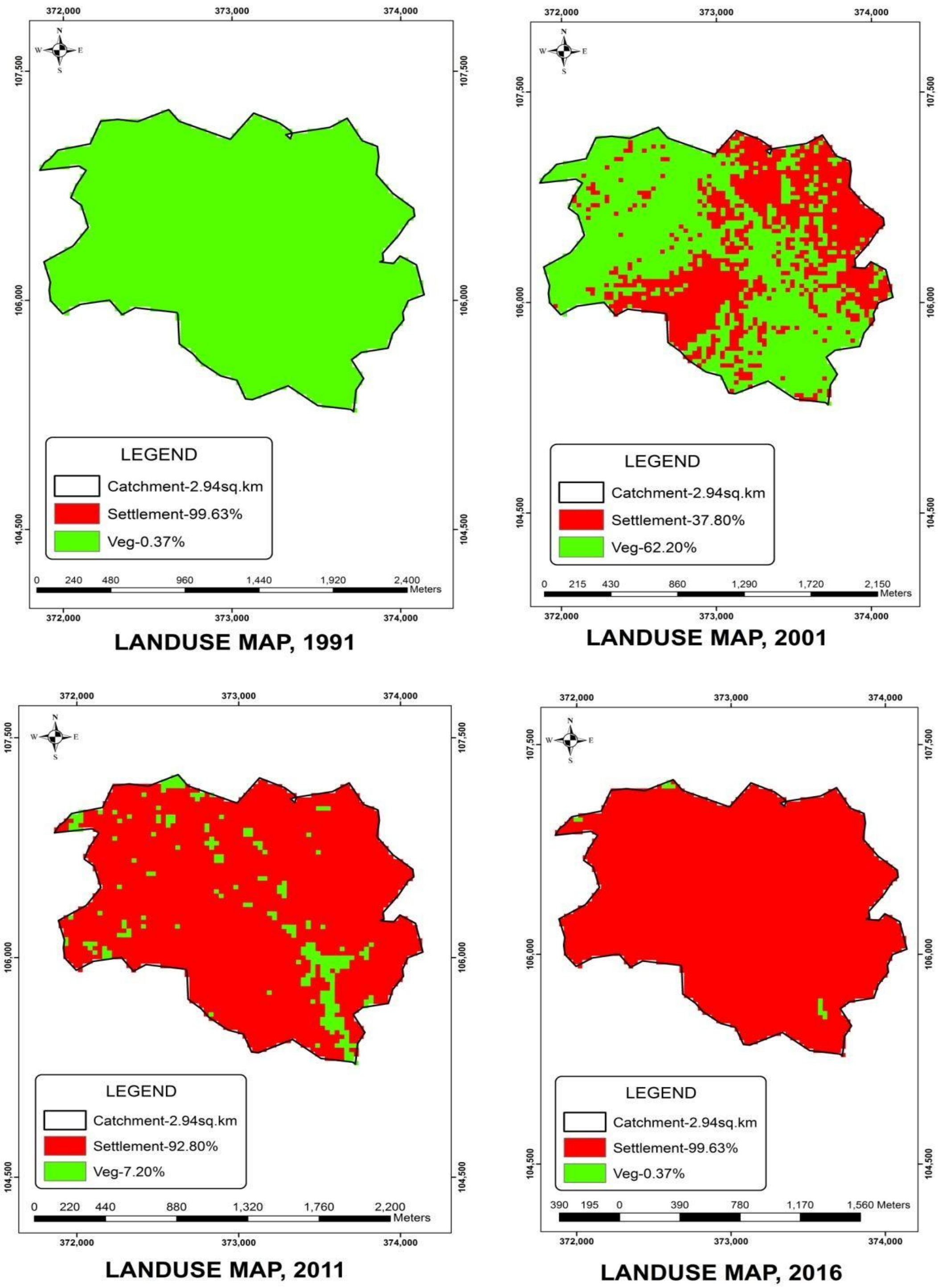
Appendix 4, Fig 4.13: Culvert at construction year 1953



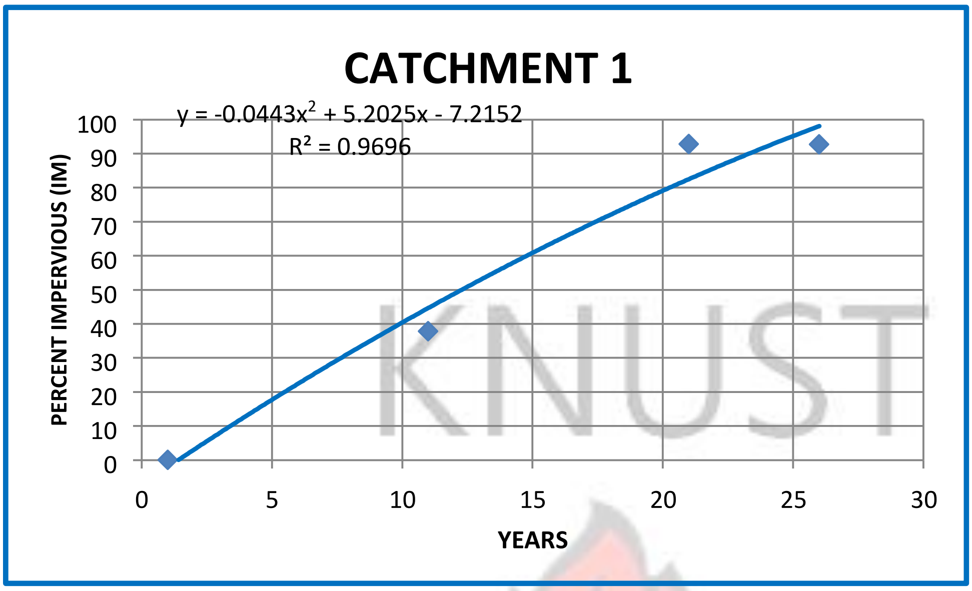
Appendix 4, Fig 4.14: Performance of culvert at 2016



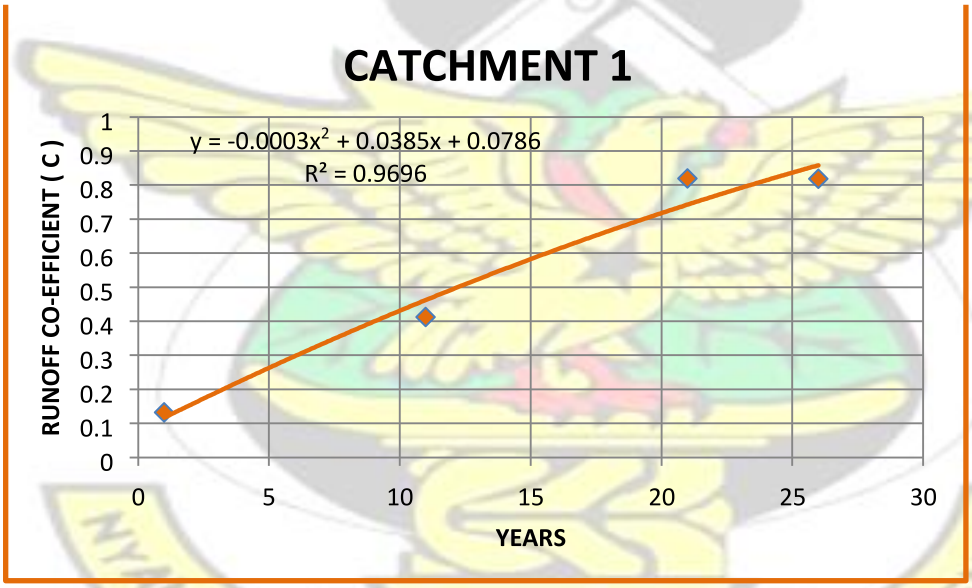
Appendix 4, Fig 4.15: Proposed culvert (existing adequate).



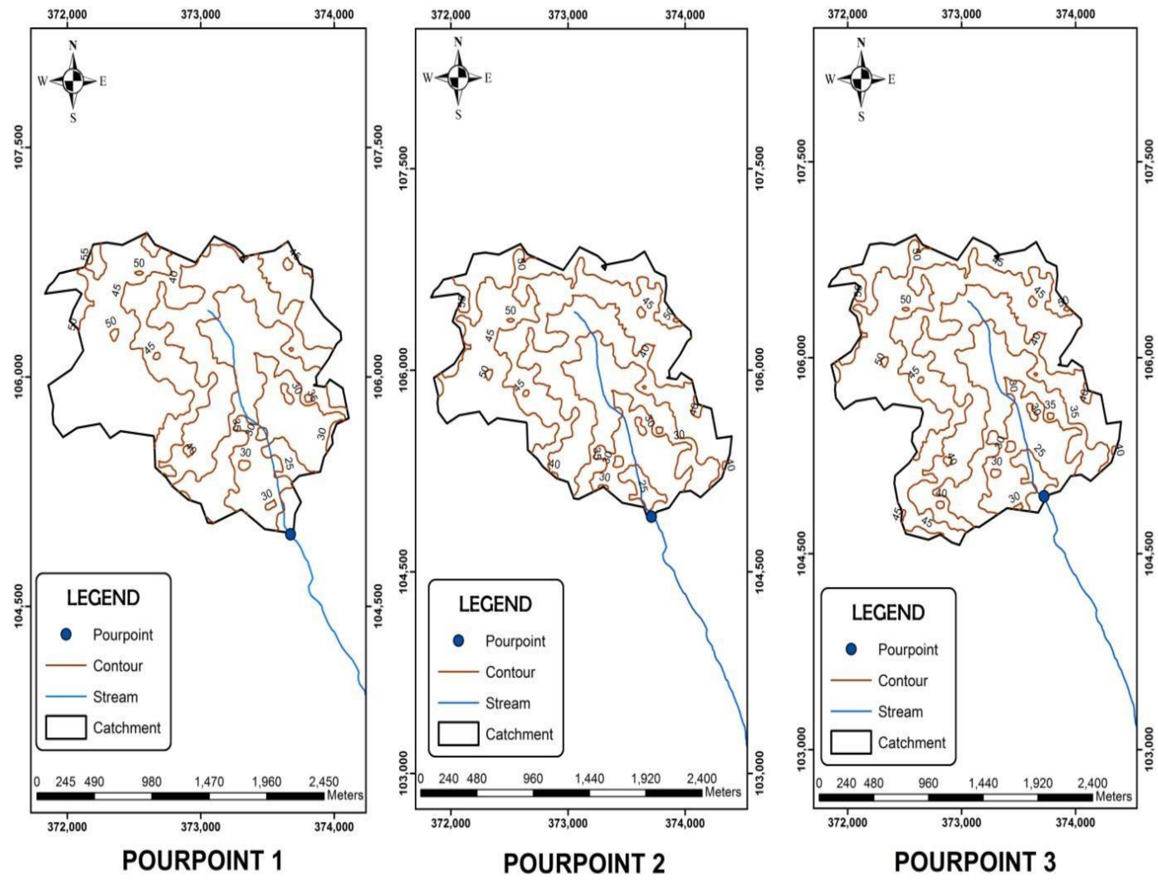
Appendix 4, Fig. 4.16; Land use, land cover maps of sub catchment no. 1



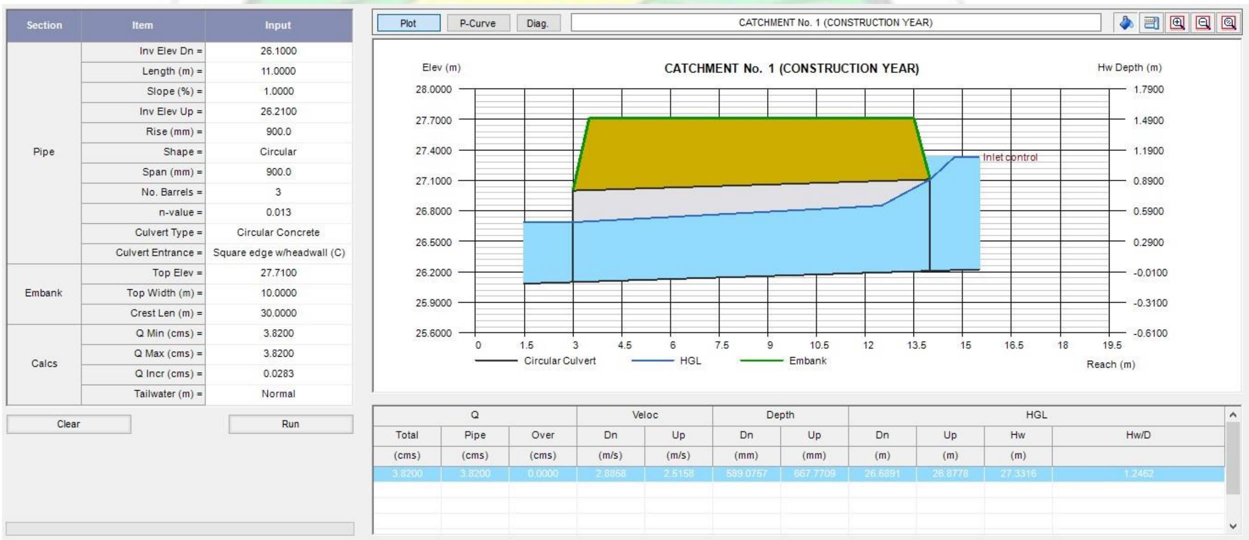
Appendix 4, Fig 4.17: Graph of percent impervious against years for catchment 1



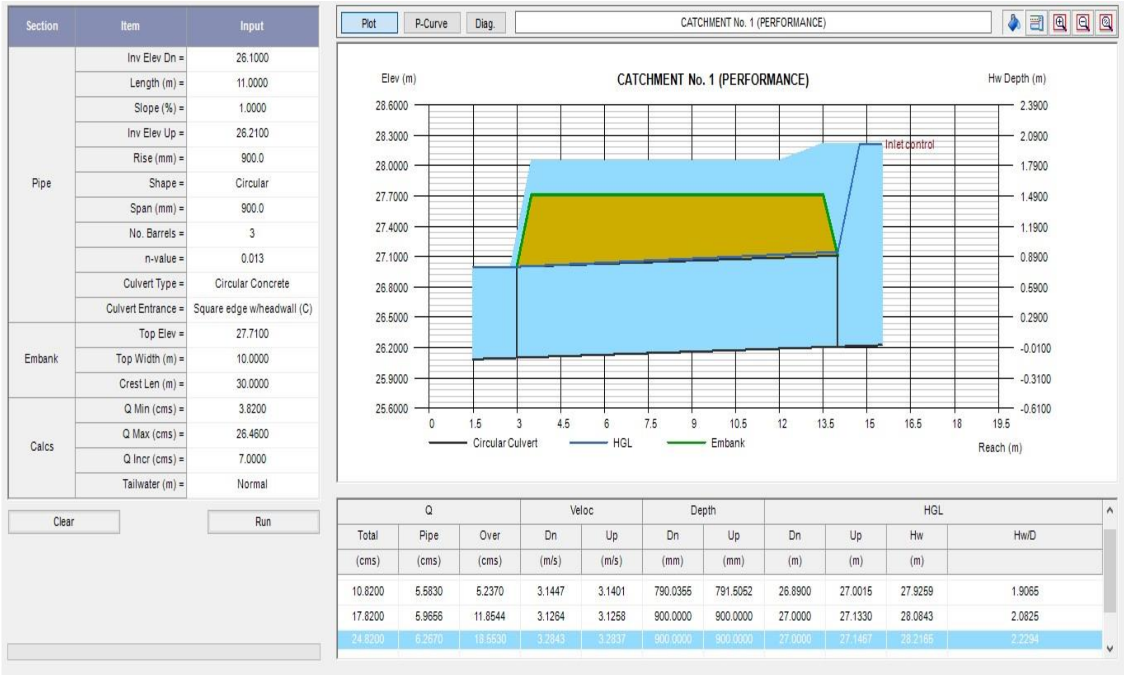
Appendix 4, Fig 4.18: Graph of runoff co-efficient against years for catchment 1



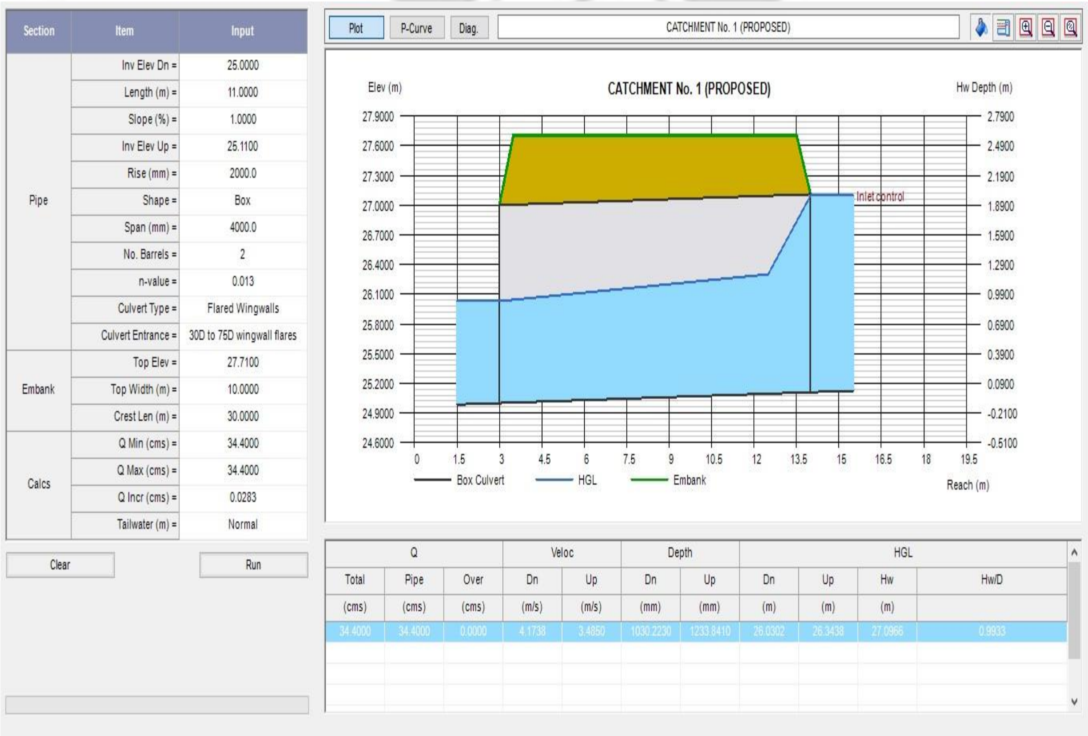
Appendix 4, Fig. 4.19: Topographical maps for sub catchments 1, 2 and 3



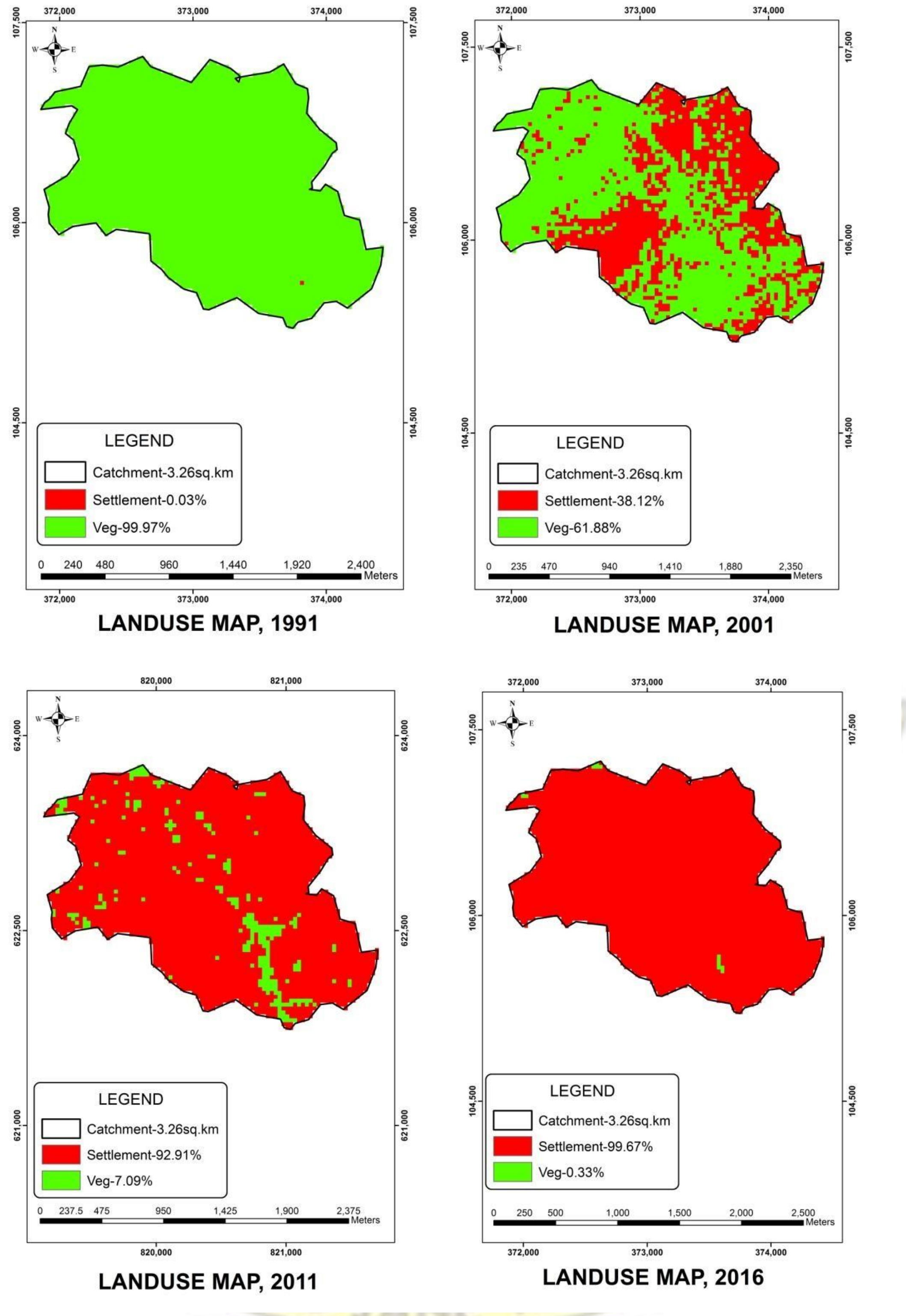
Appendix 4, Fig 4.20: Culvert at construction year 1994



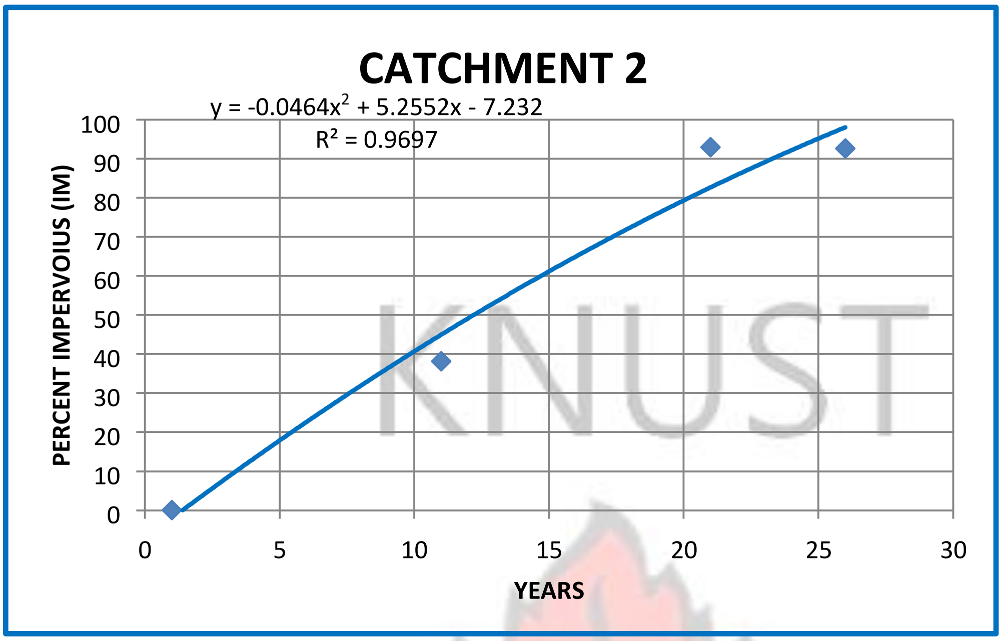
Appendix 4, Fig 4.21: Performance of culvert at 2016



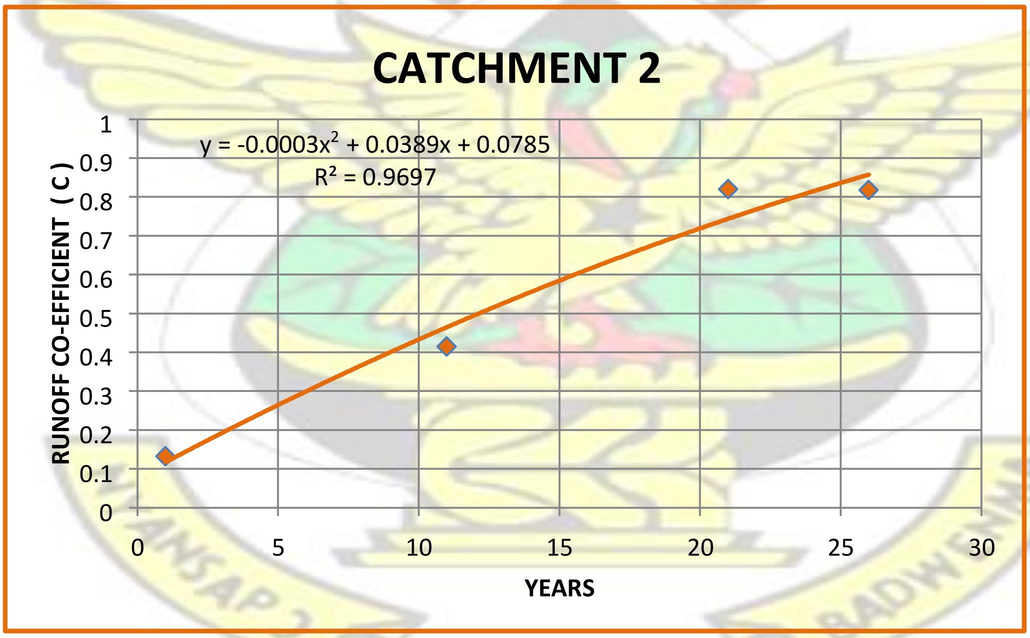
Appendix 4, Fig 4.22: Proposed culvert



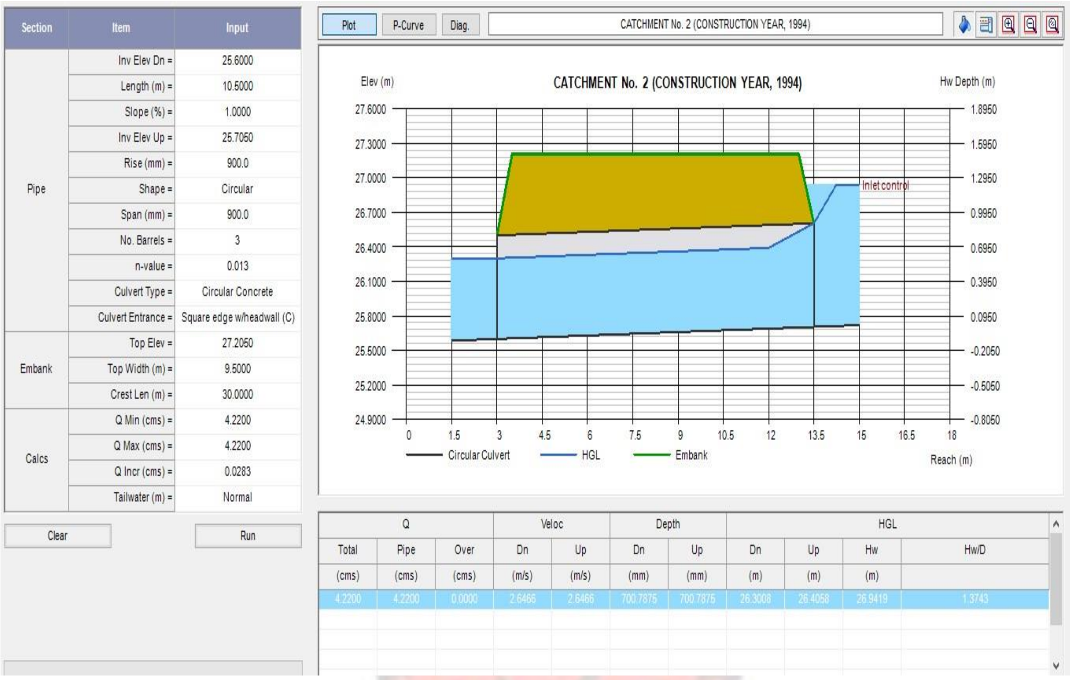
Appendix 4, Fig. 4.23; Land use, land cover maps of sub catchment no. 2



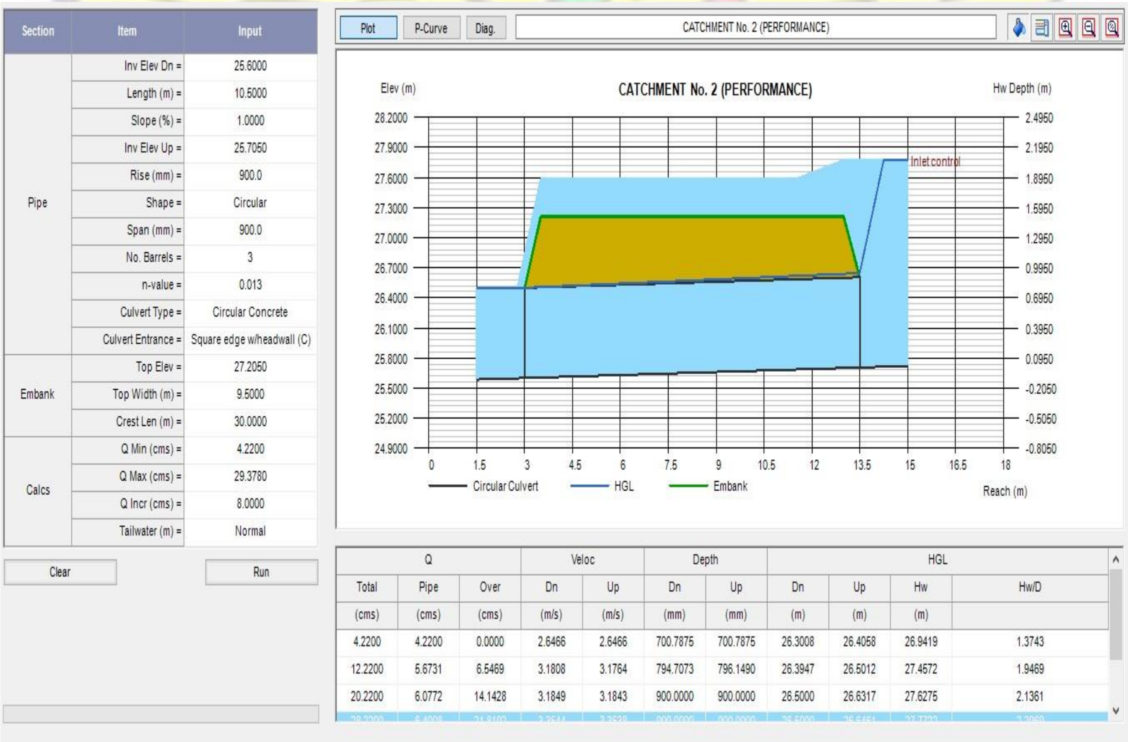
Appendix 4, Fig 4.24: Graph of percent impervious against years for catchment 2



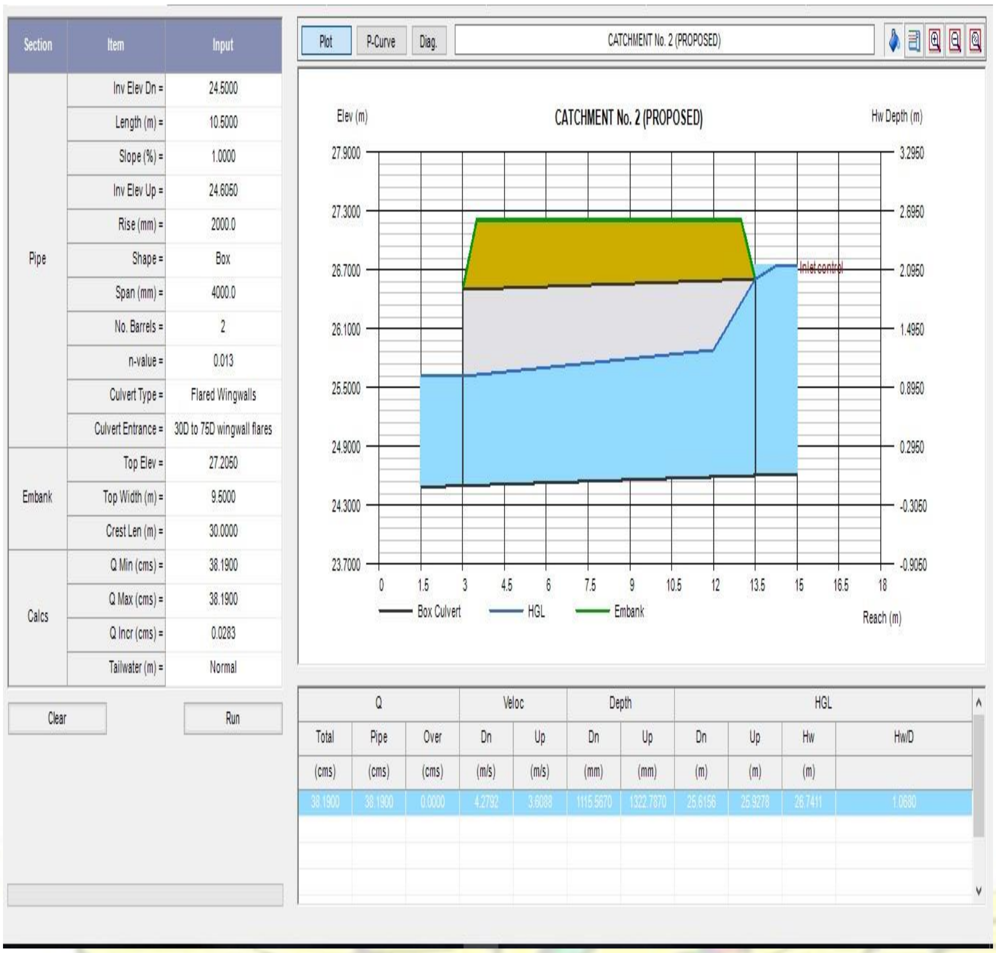
Appendix 4, Fig 4.25: Graph of runoff co-efficient against years for catchment 2



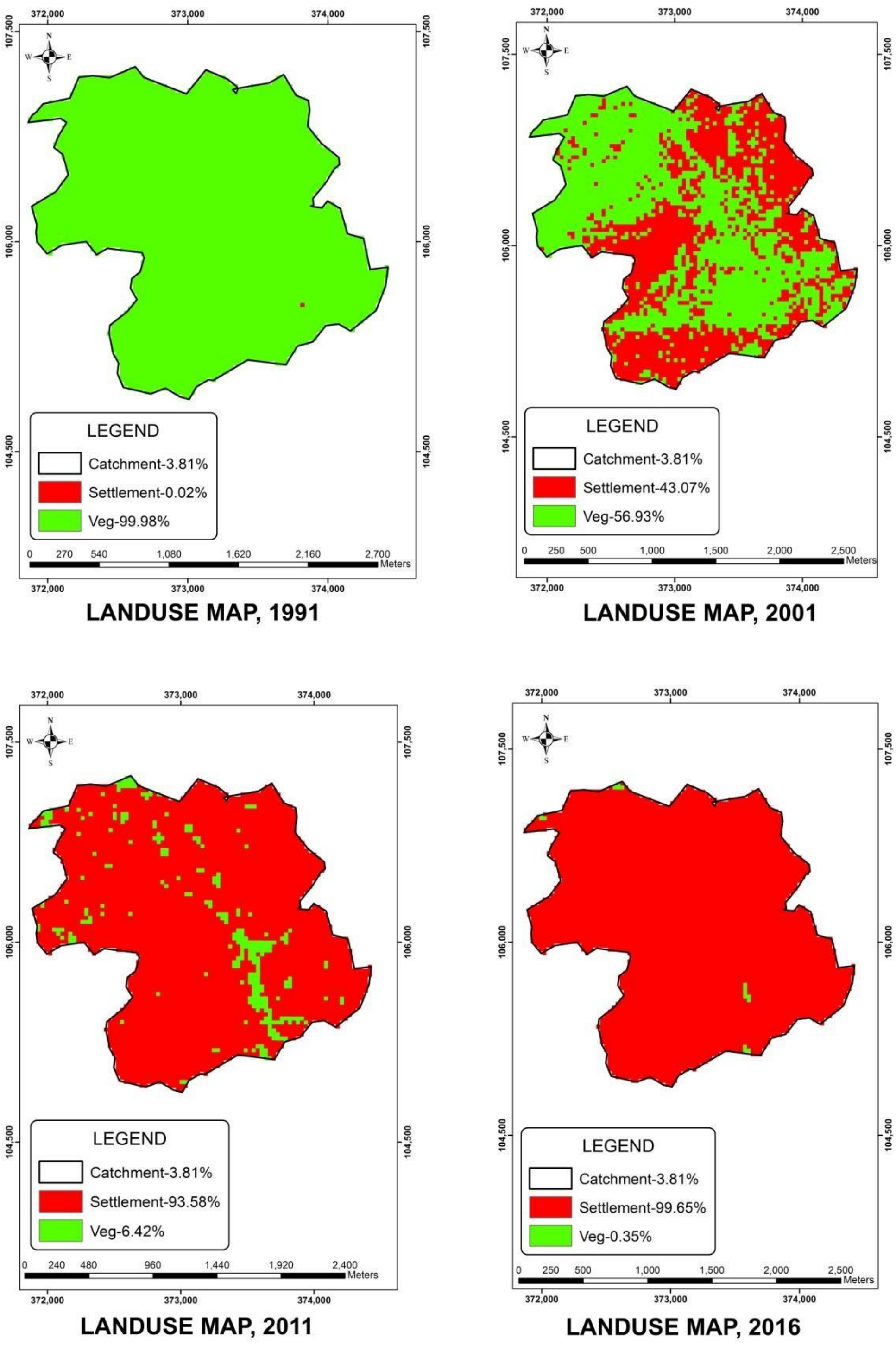
Appendix 4, Fig 4.26: Culvert at construction year 1994



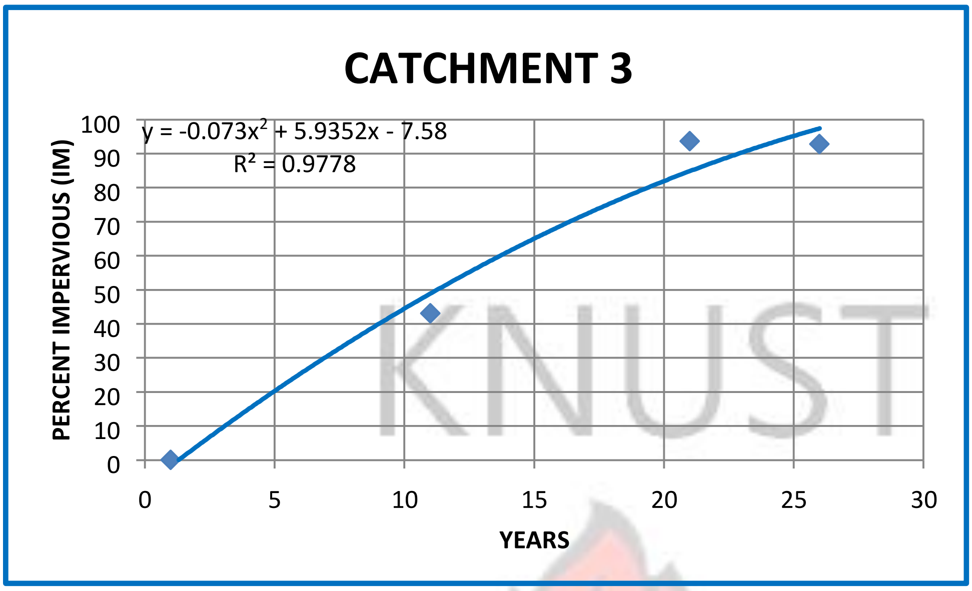
Appendix 4, Fig 4.27: Performance of culvert at 2016



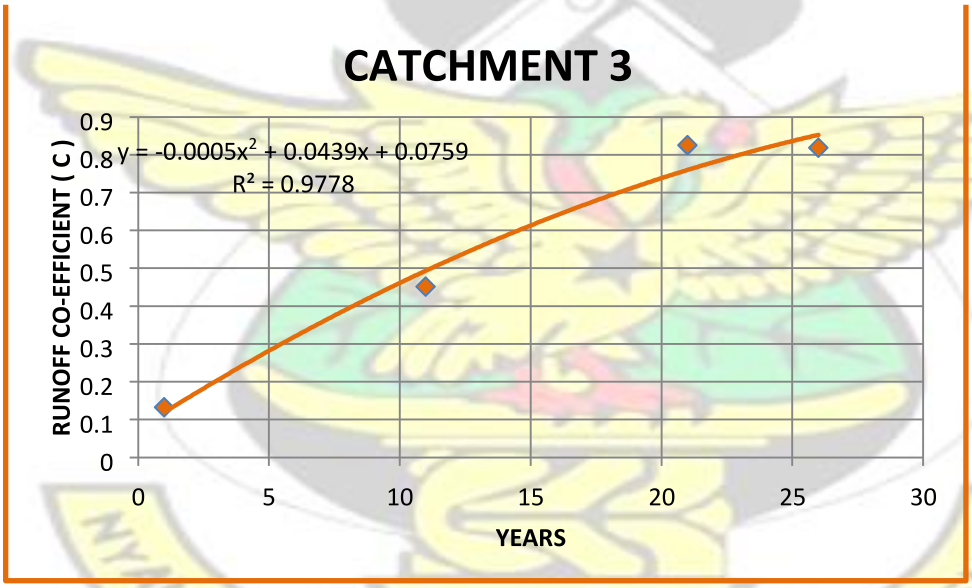
Appendix 4, Fig 4.28: Proposed culvert



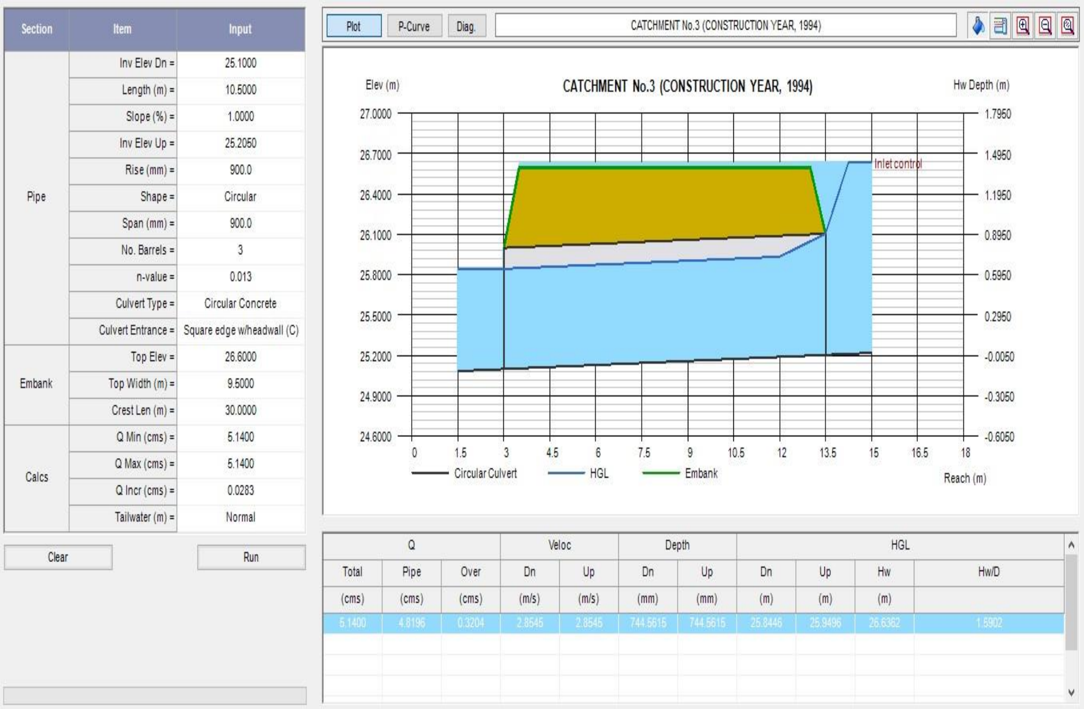
Appendix 4, Fig. 4.29; Land use, land cover maps of sub catchment no. 3



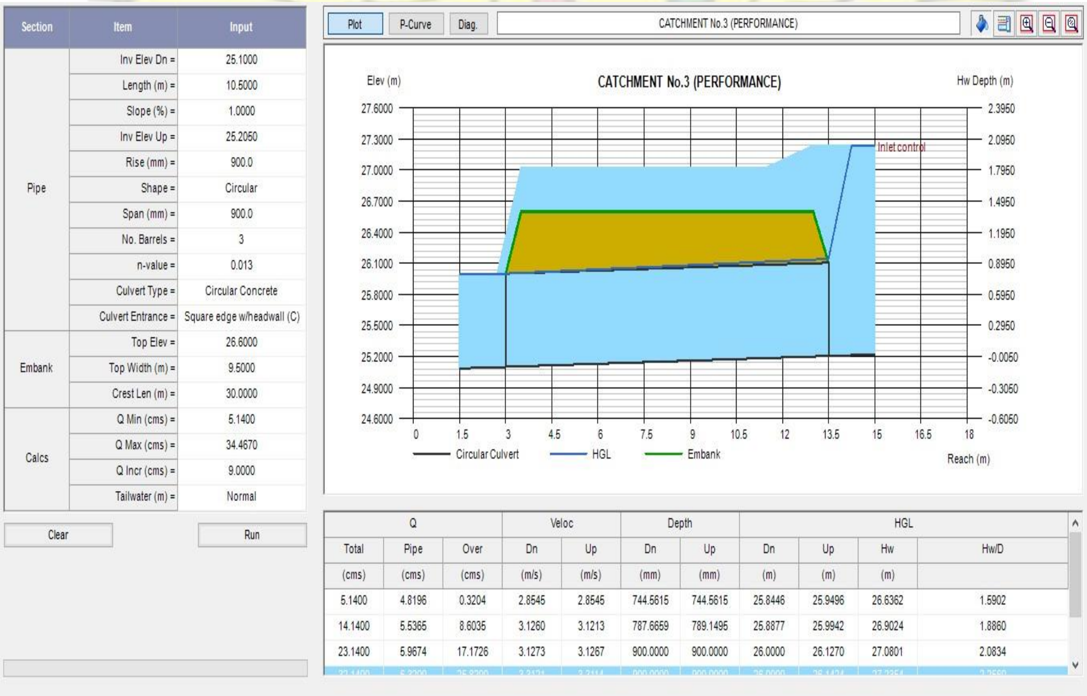
Appendix 4, Fig 4.30: Graph of percent impervious against years for catchment 3



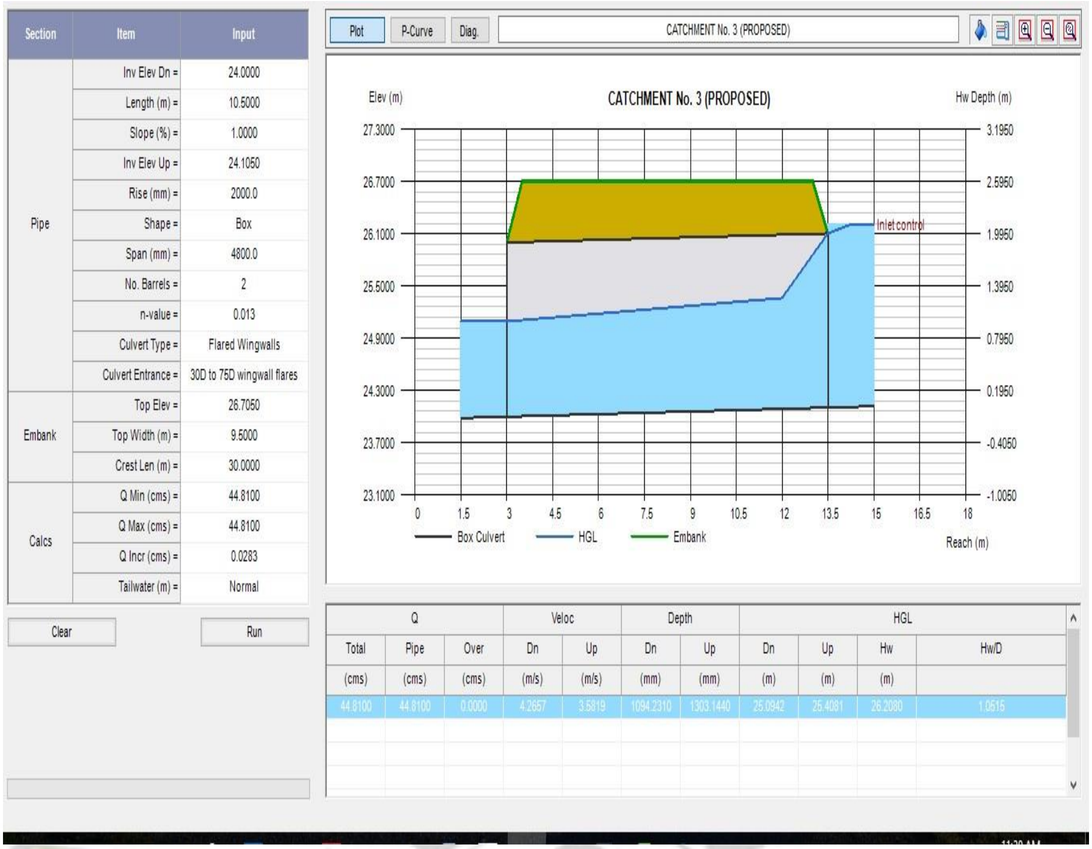
Appendix 4, Fig 4.31: Graph of runoff co-efficient against years for catchment 3



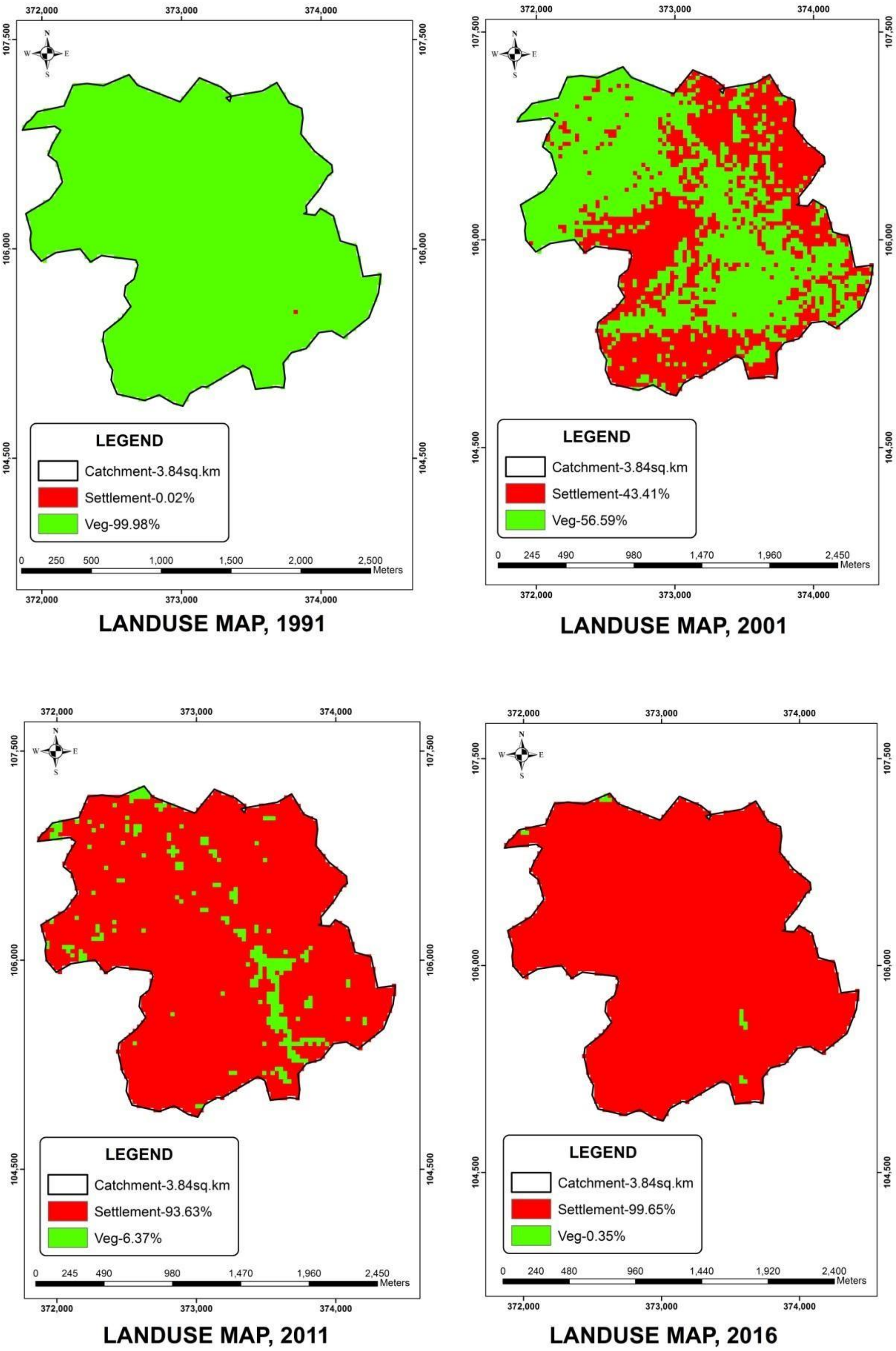
Appendix 4, Fig 4.32: Culvert at construction year 1994



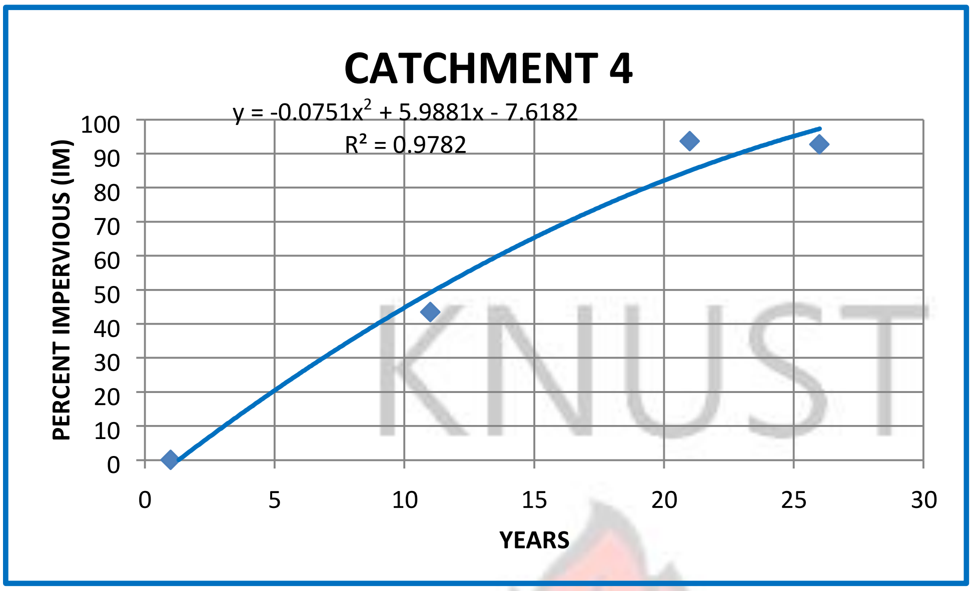
Appendix 4, Fig 4.33: Performance of culvert at 2016



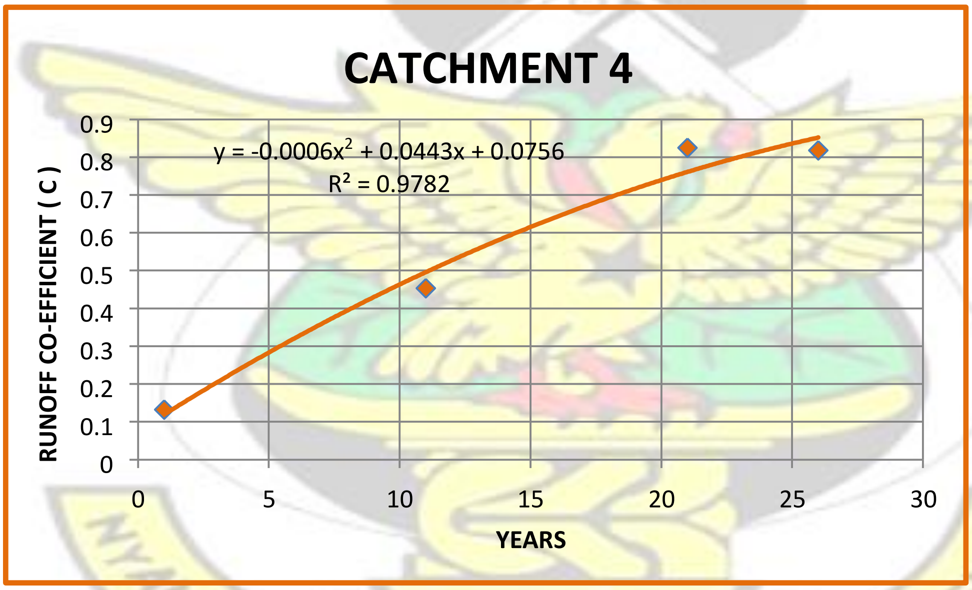
Appendix 4, Fig 4.34: Proposed culvert



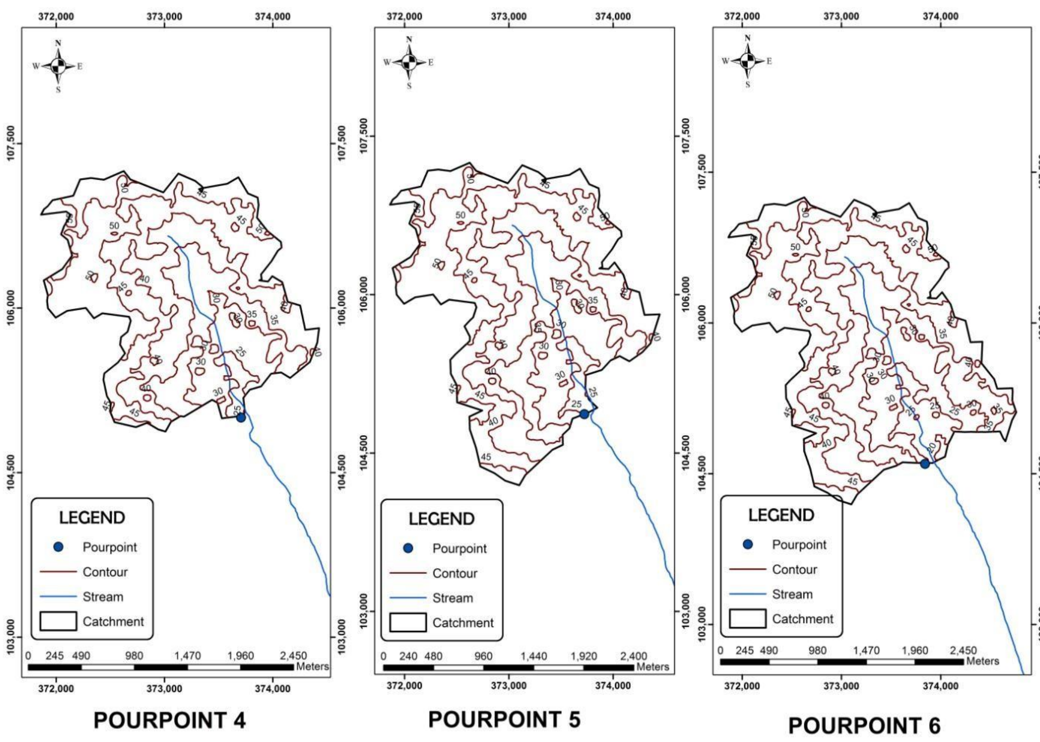
Appendix 4, Fig. 4.35; Land use, land cover maps of sub catchment no. 4



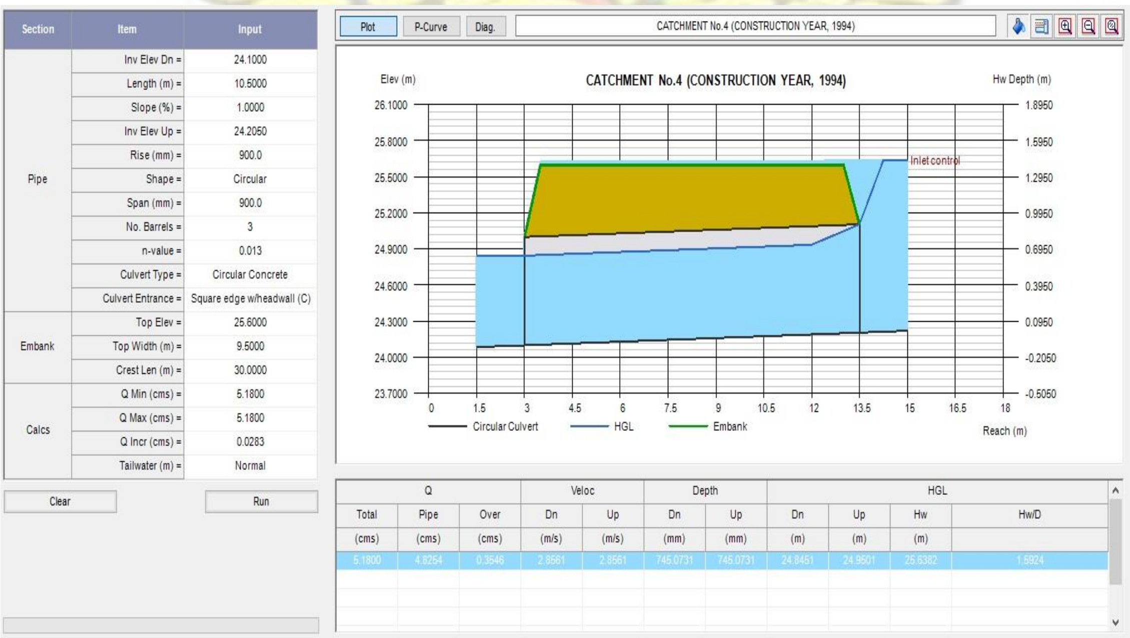
Appendix 4, Fig 4.36: Graph of percent impervious against years for catchment 4



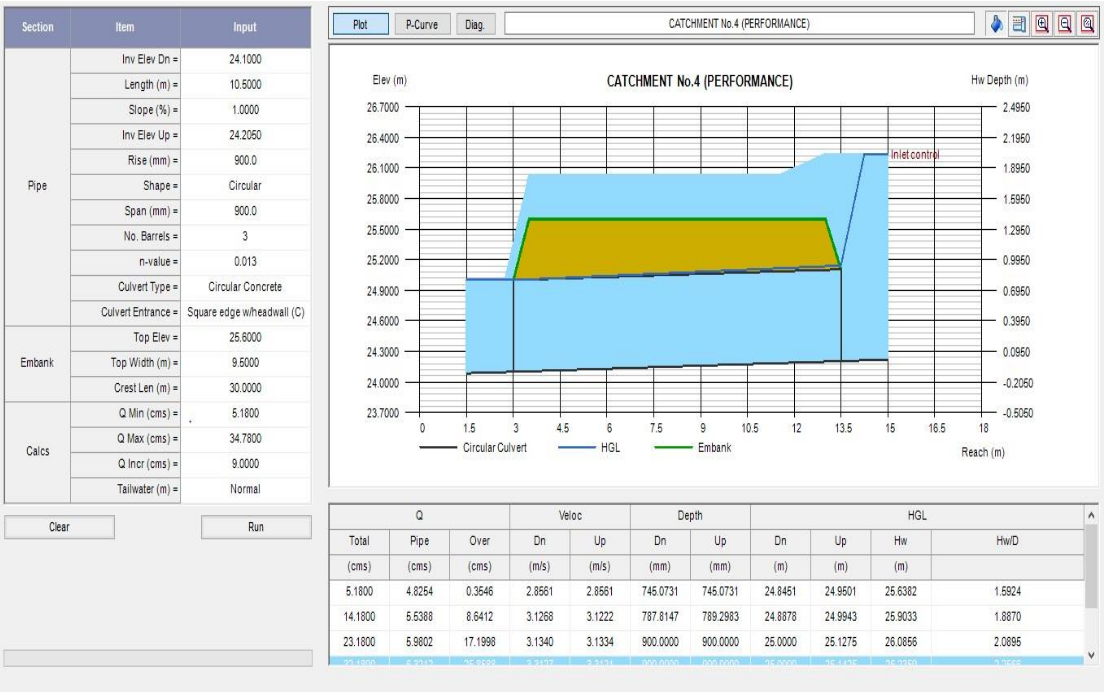
Appendix 4, Fig 4.37: Graph of runoff co-efficient against years for catchment 4



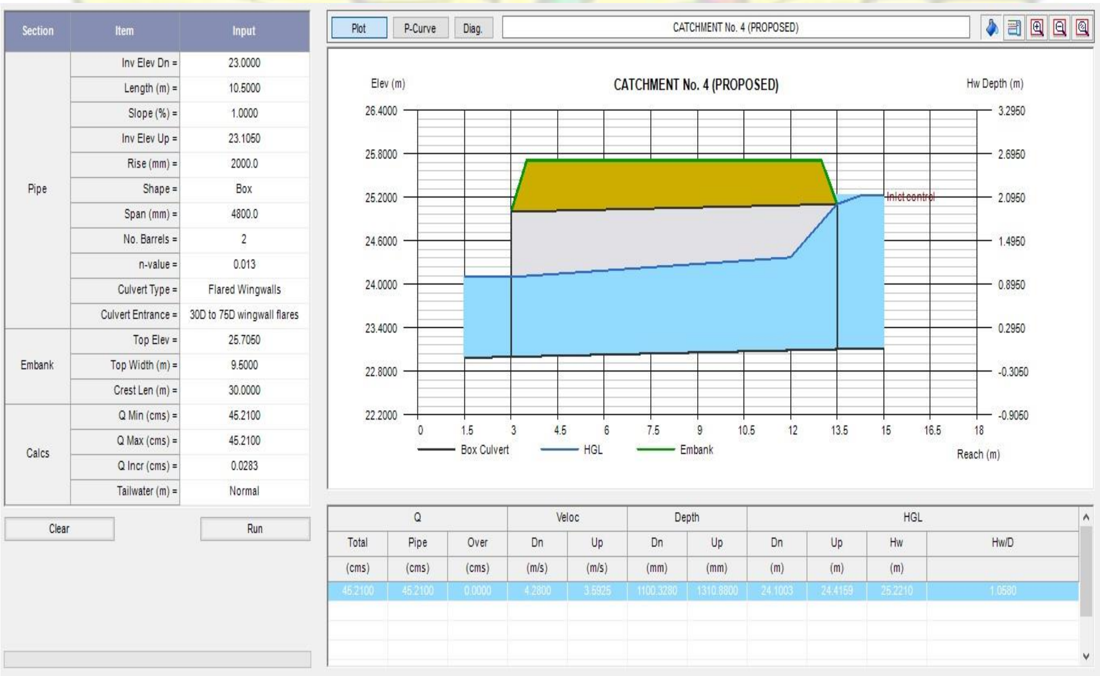
Appendix 4, Fig. 4.38: Topographical maps of sub catchments 4, 5 and 6



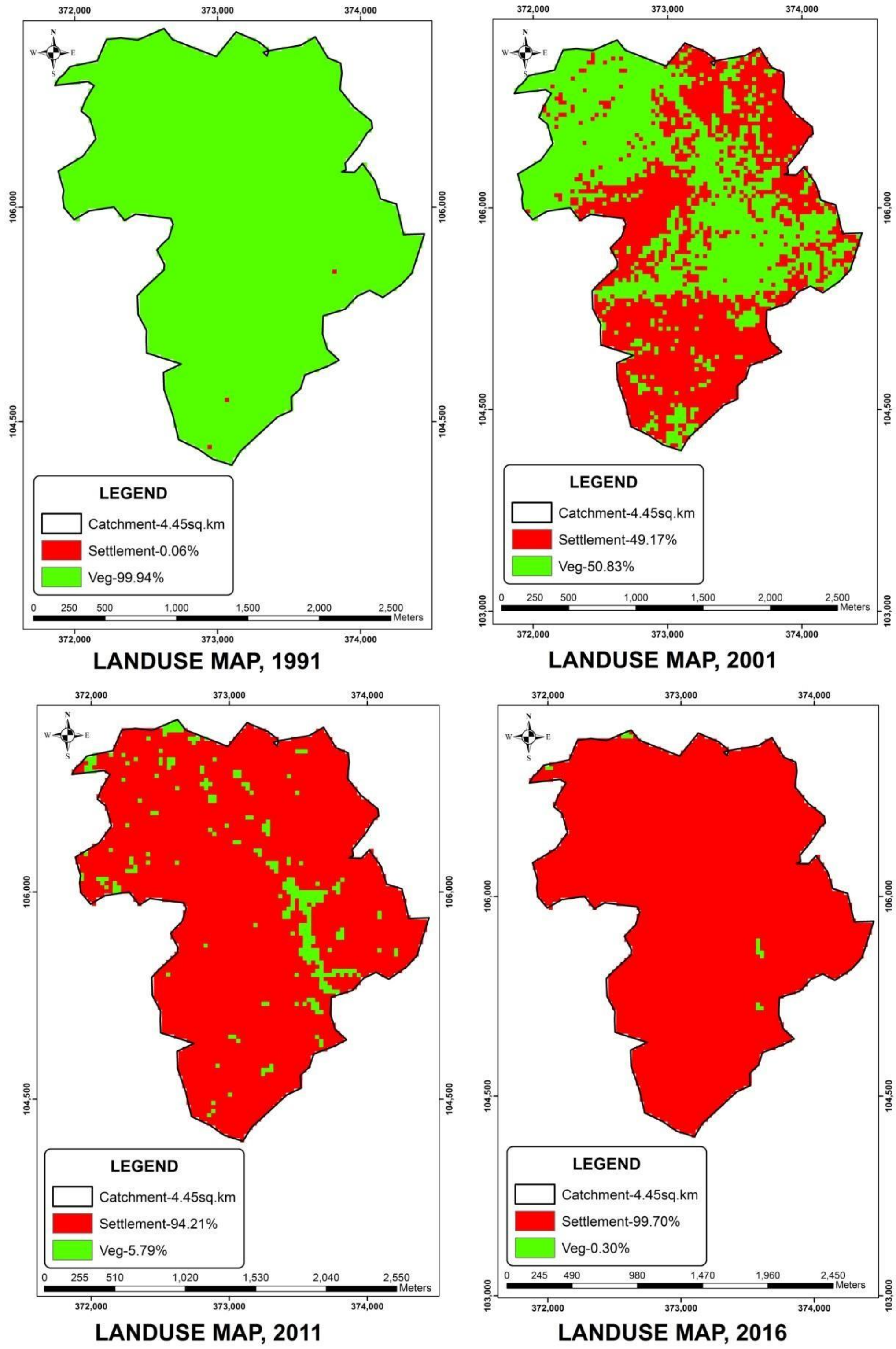
Appendix 4, Fig 4.39: Culvert at construction year 1994



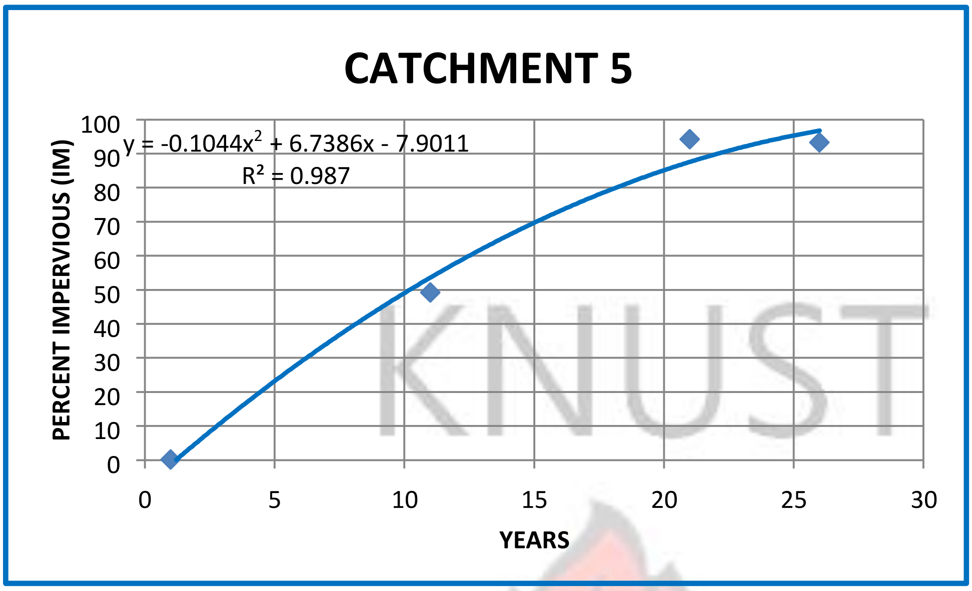
Appendix 4, Fig 4.40: Performance of culvert at 2016



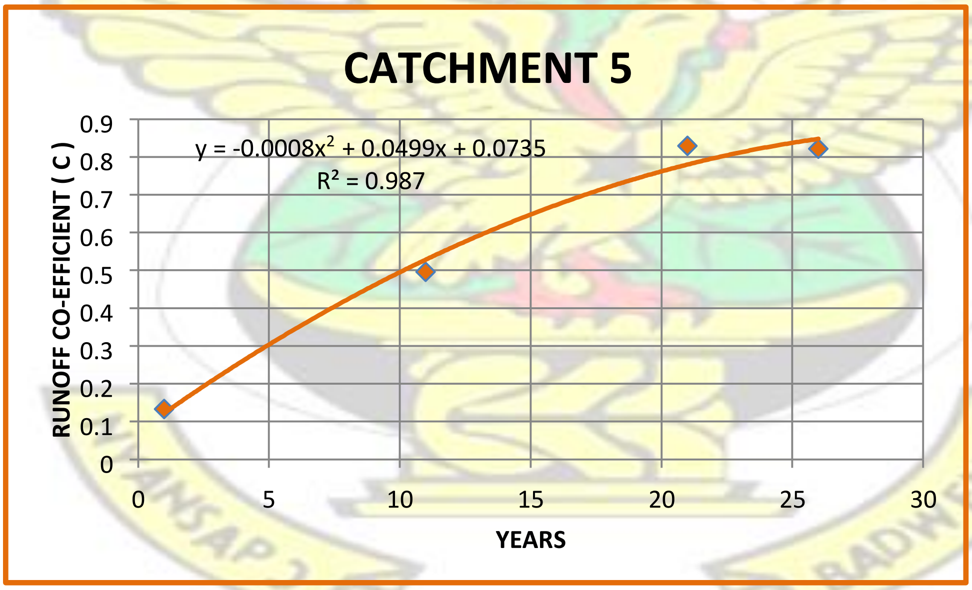
Appendix 4, Fig 4.41: Propose culvert



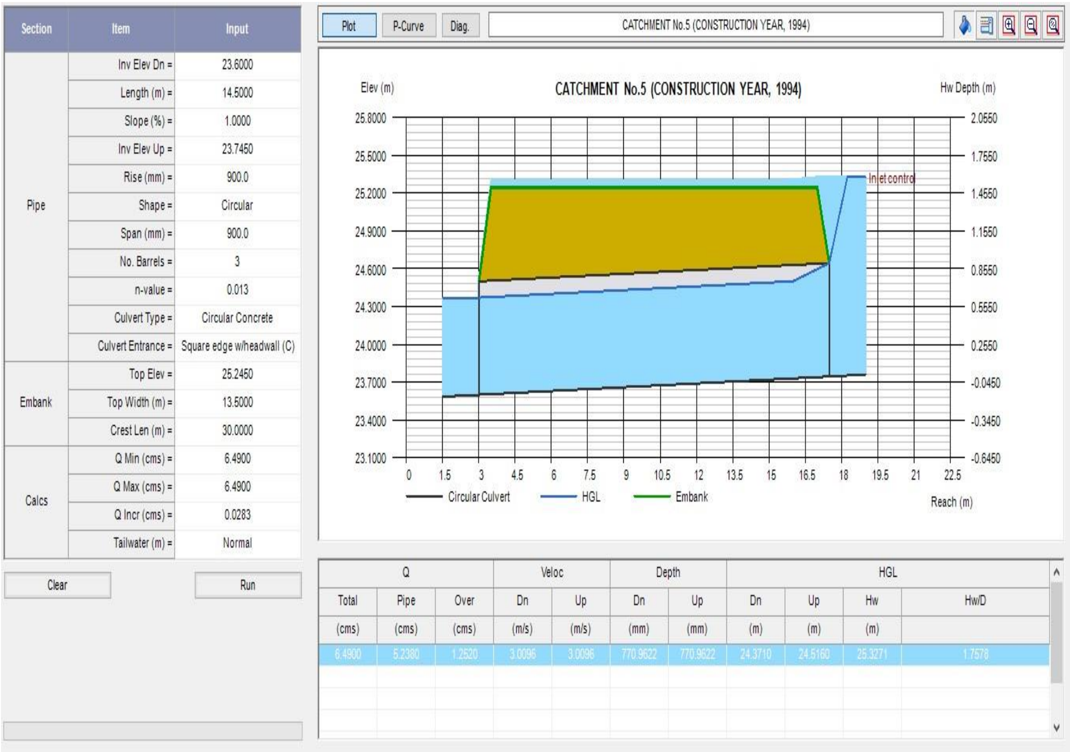
Appendix 4, Fig. 4.42; Land use, land cover maps of sub catchment no. 5



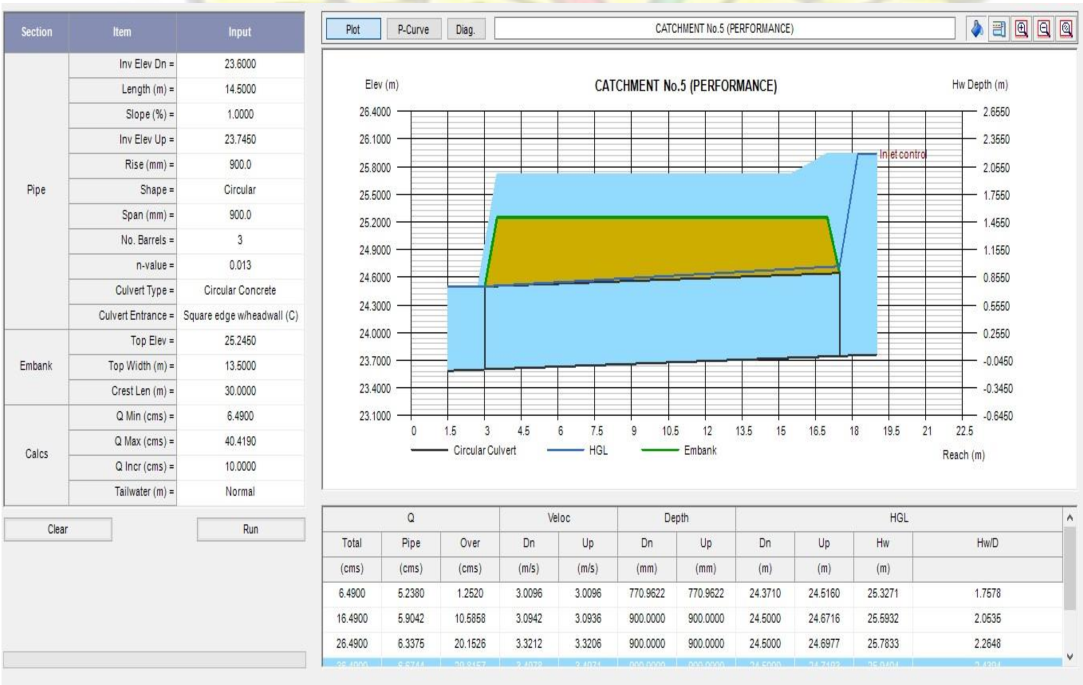
Appendix 4, Fig 4.43: Graph of percent impervious against years for catchment 5



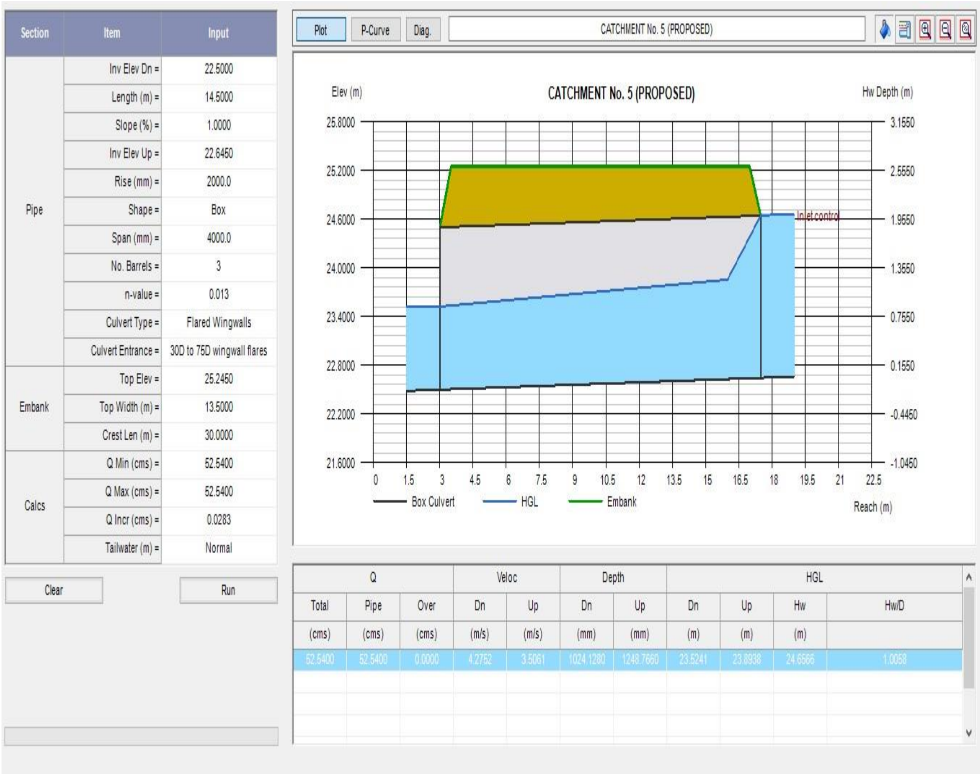
Appendix 4, Fig 4.44: Graph of runoff co-efficient against years for catchment 5



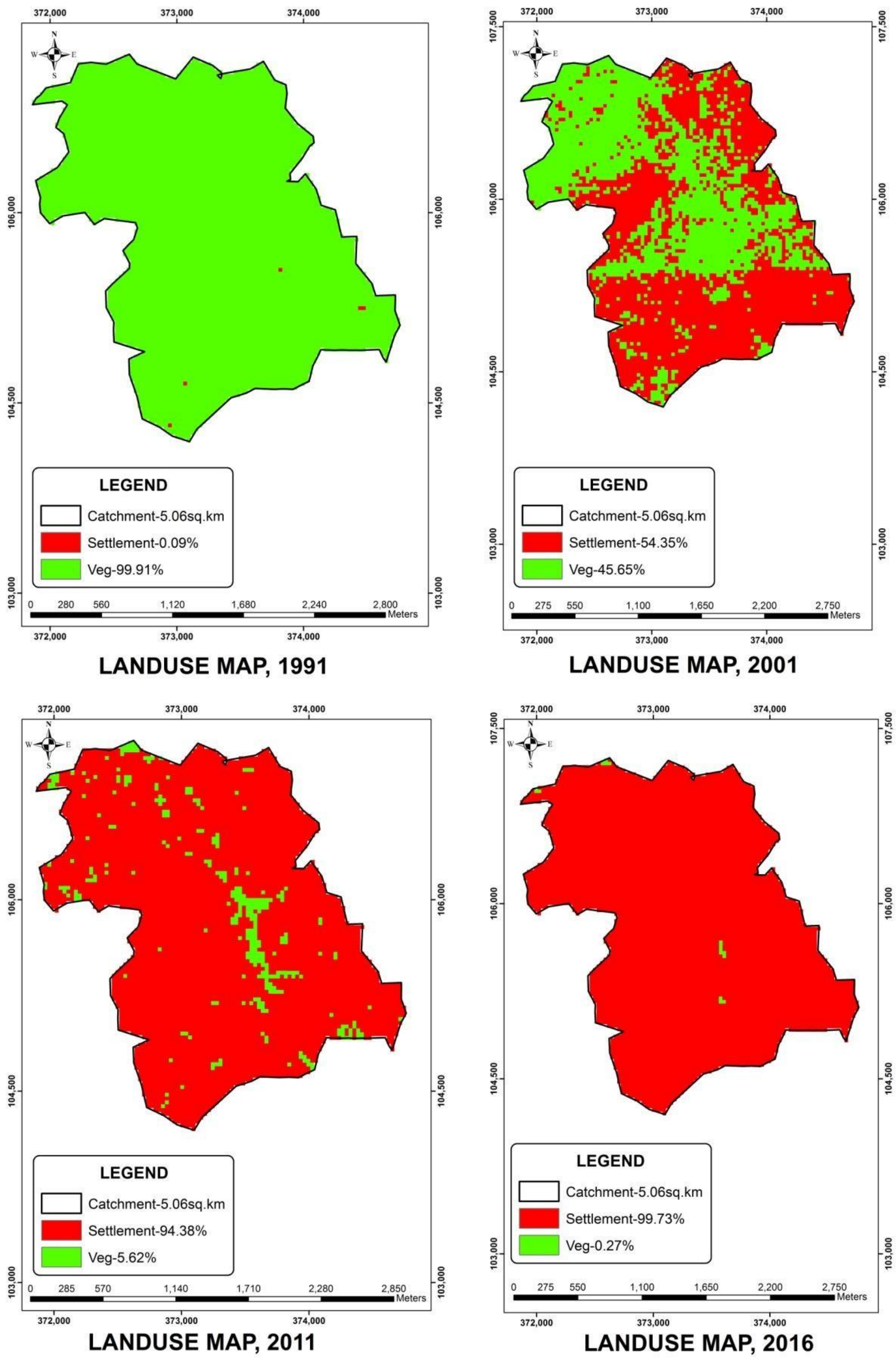
Appendix 4, Fig 4.45: Culvert at construction year 1994



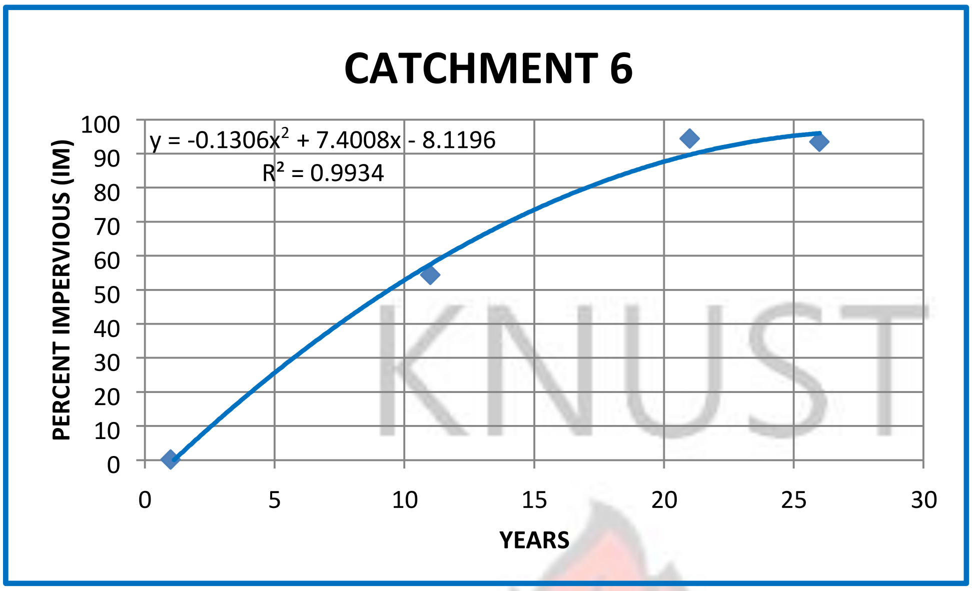
Appendix 4, Fig 4.46: Performance of culvert at 2016



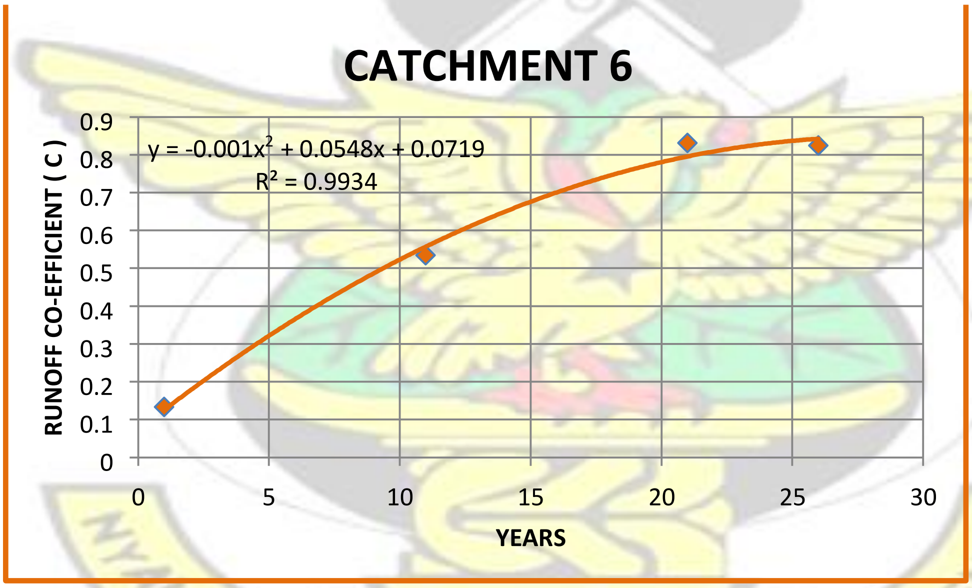
Appendix 4, Fig 4.47: Proposed Culvert



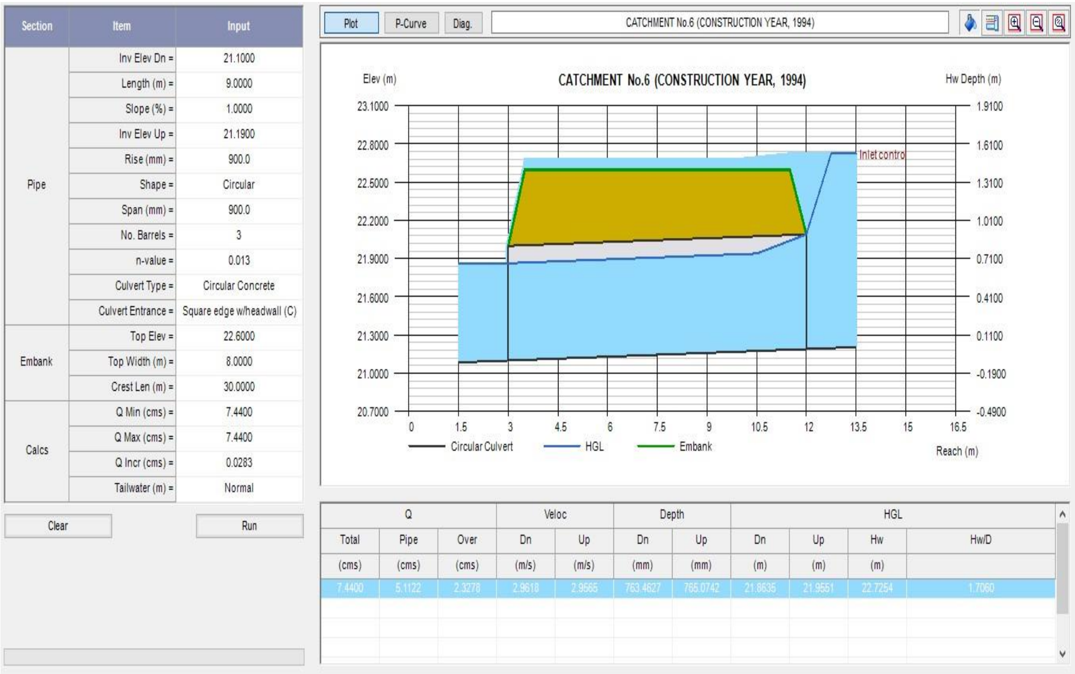
Appendix 4, Fig. 4.48; Land use, land cover maps of sub catchment no. 6



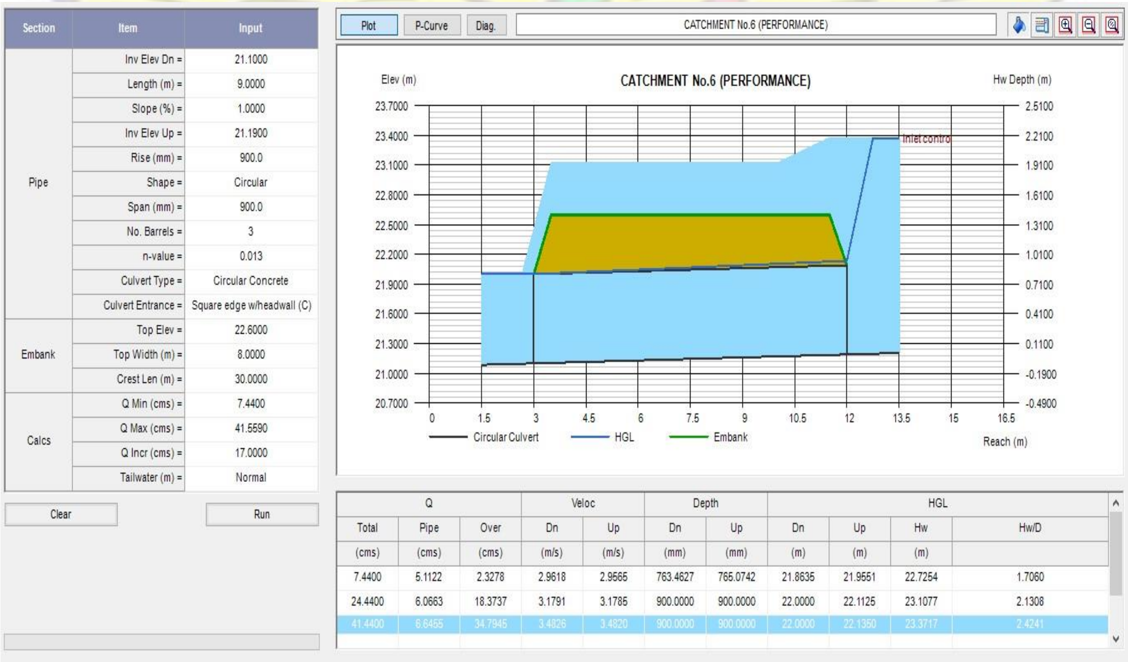
Appendix 4, Fig 4.49: Graph of percent impervious against years for catchment 6



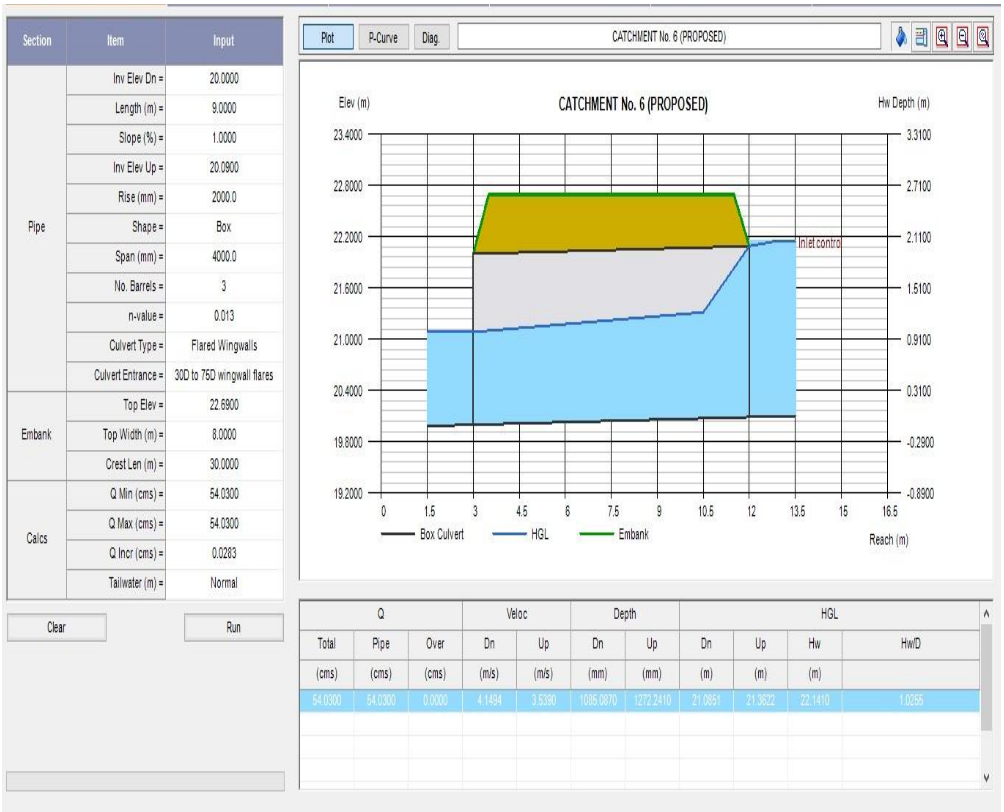
Appendix 4, Fig 4.50: Graph of runoff co-efficient against years for catchment 6



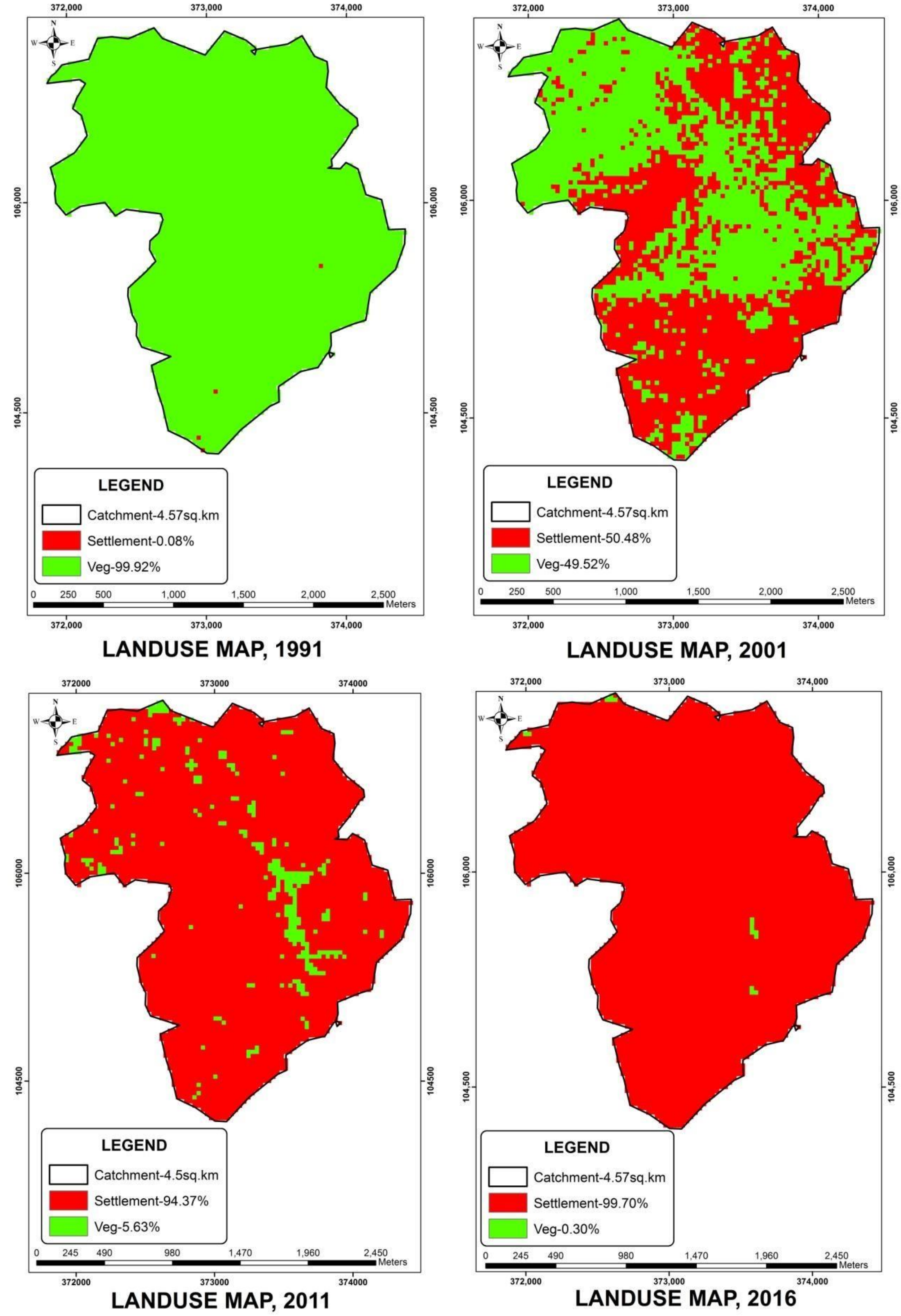
Appendix 4, Fig 4.51: Culvert at construction year 1994



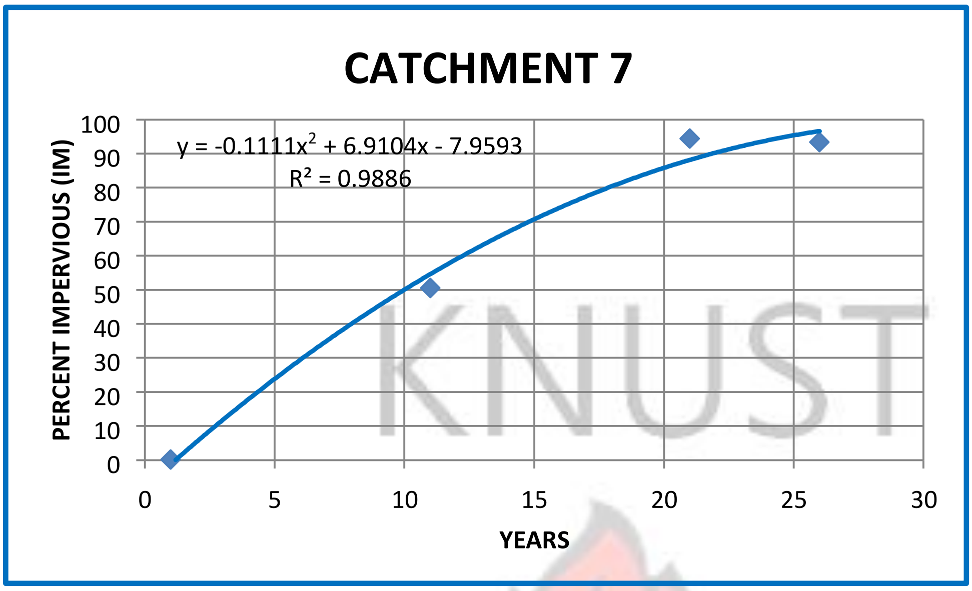
Appendix 4, Fig 4.52: Performance of culvert at 2016



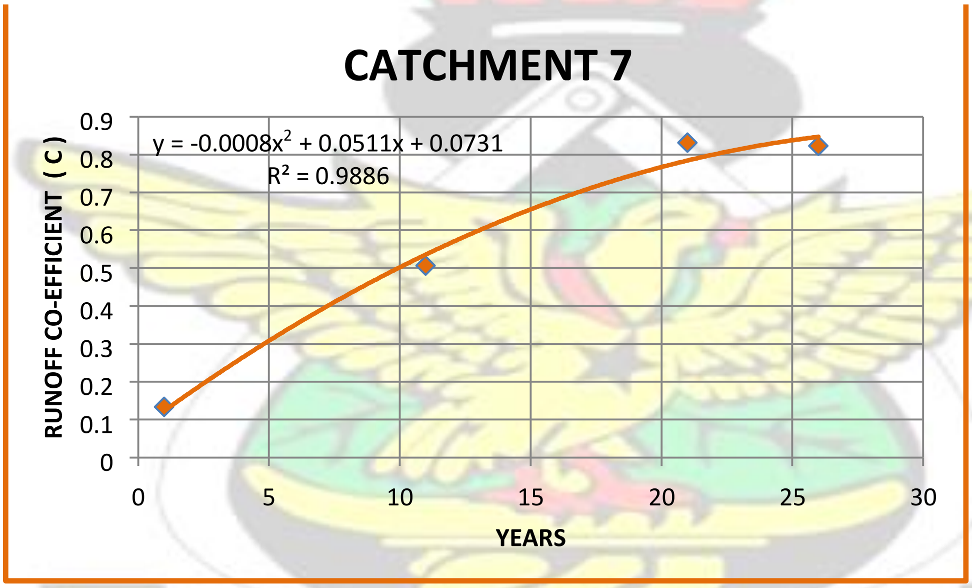
Appendix 4, Fig 4.53: Proposed culvert



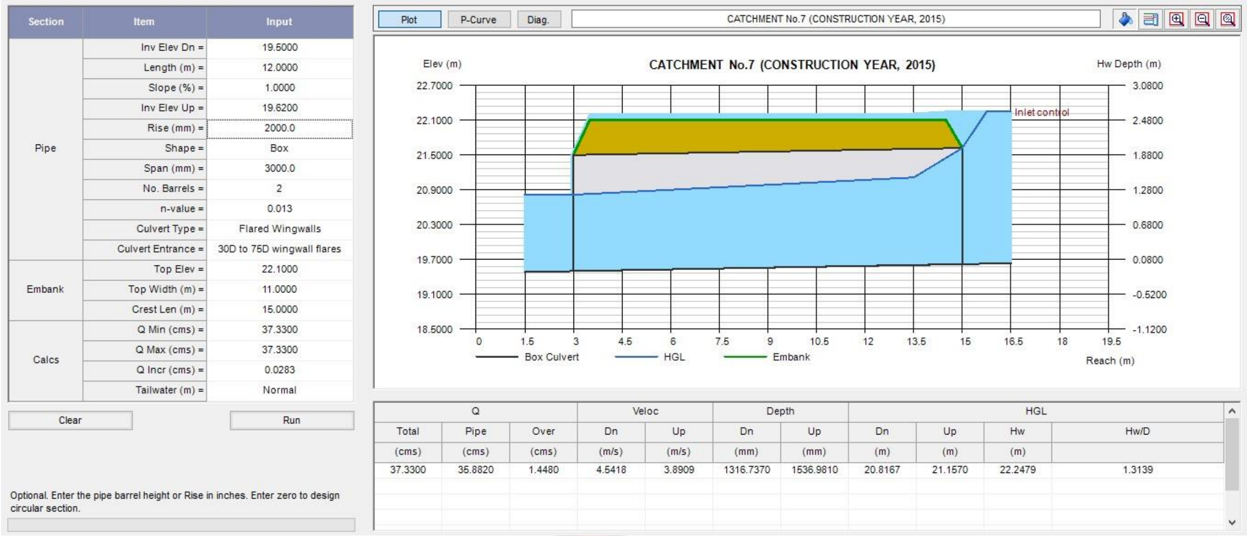
Appendix 4, Fig. 4.54; Land use, land cover maps of sub catchment no. 7



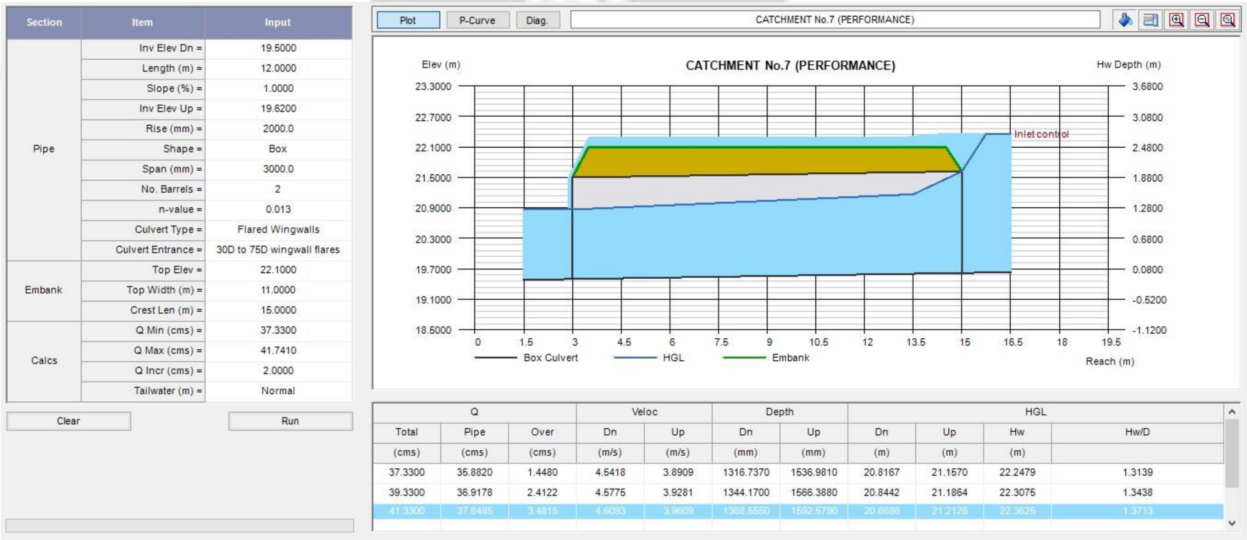
Appendix 4, Fig 4.55: Graph of percent impervious against years for catchment 7



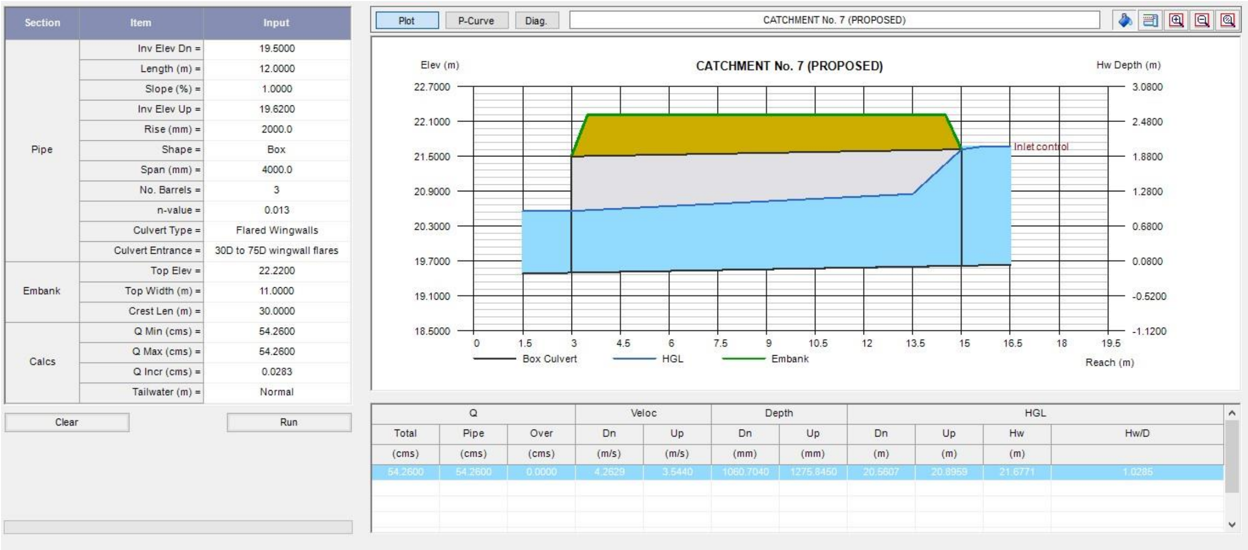
Appendix 4, Fig 4.56: Graph of runoff co-efficient against years for catchment 7



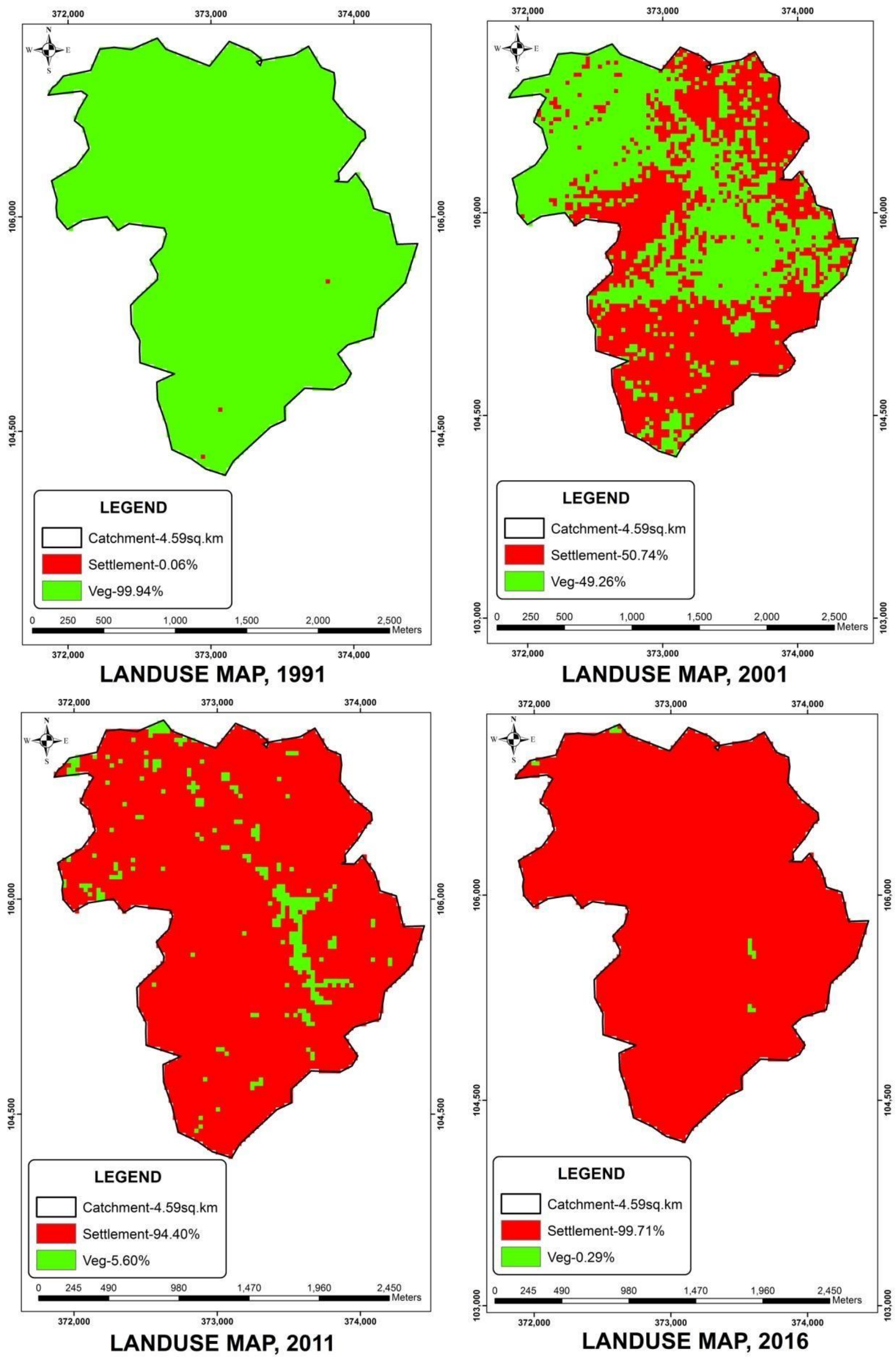
Appendix 4, Fig 4.57: Culvert at construction year 2015



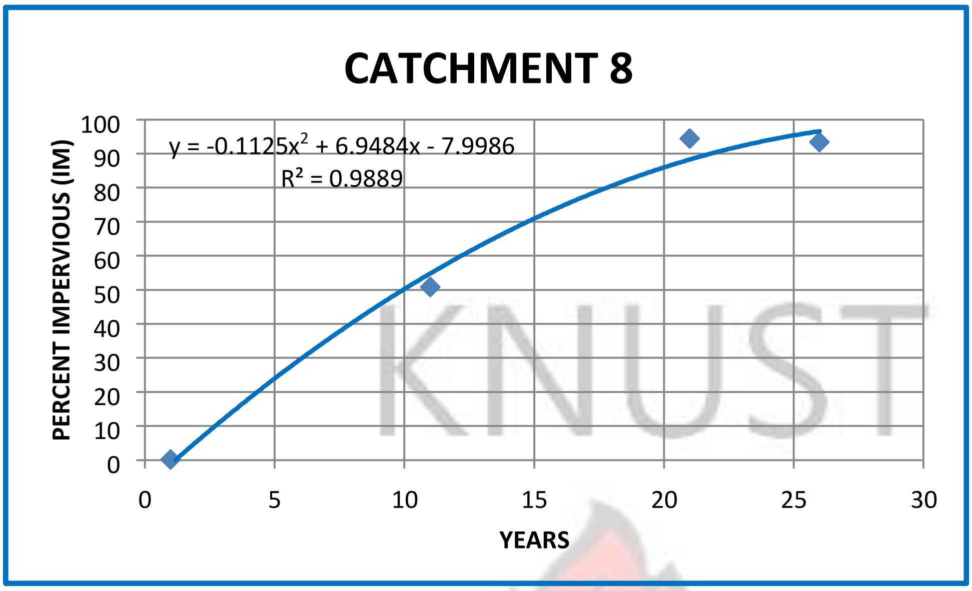
Appendix 4, Fig 4.58: Performance of culvert at 2016



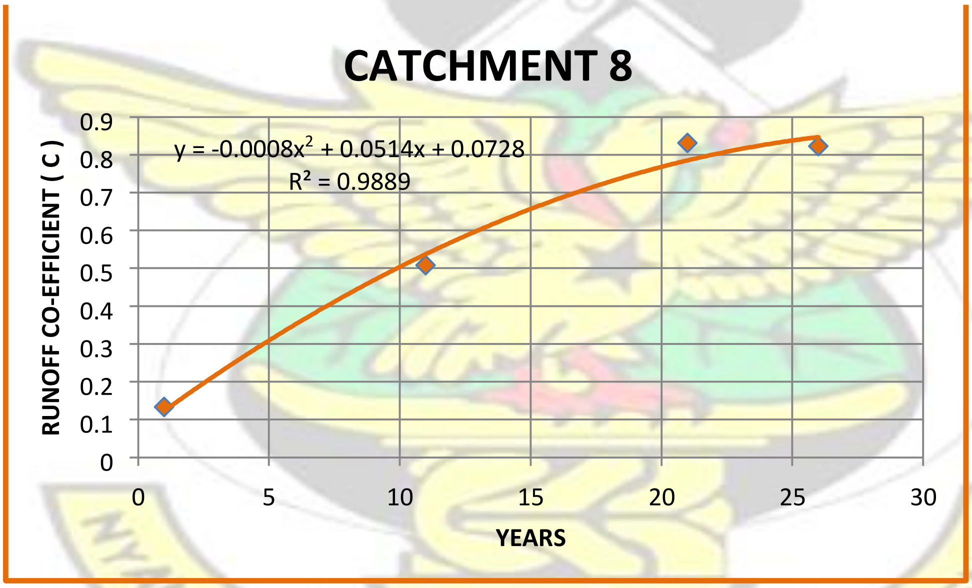
Appendix 4, Fig 4.59: Proposed culvert



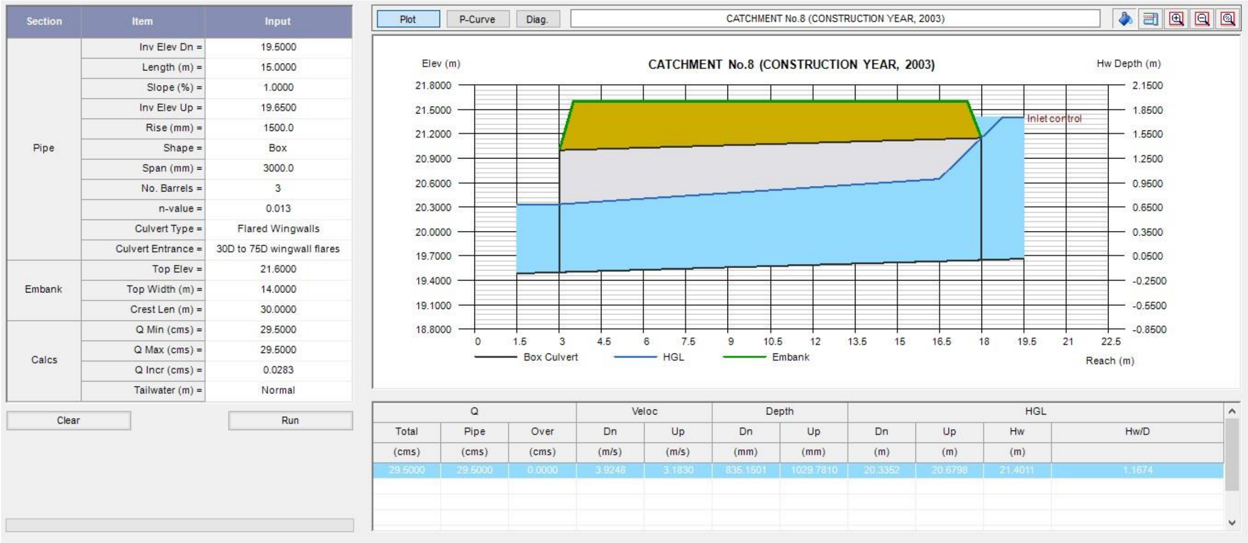
Appendix 4, Fig. 4.60; Land use, land cover maps of sub catchment no. 8



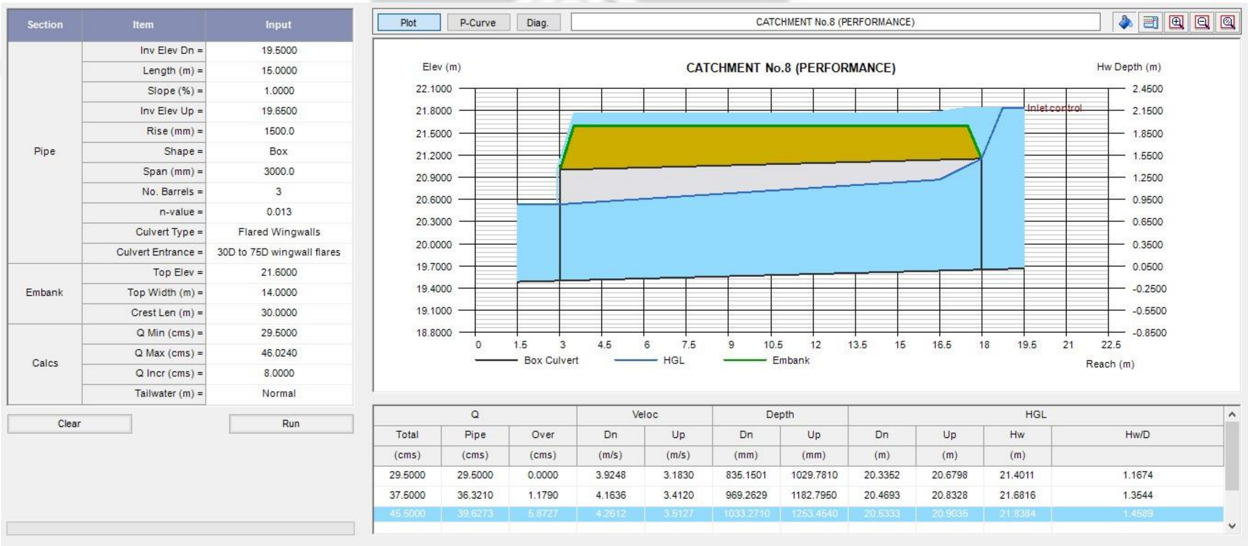
Appendix 4, Fig 4.61: Graph of percent impervious against years for catchment 8



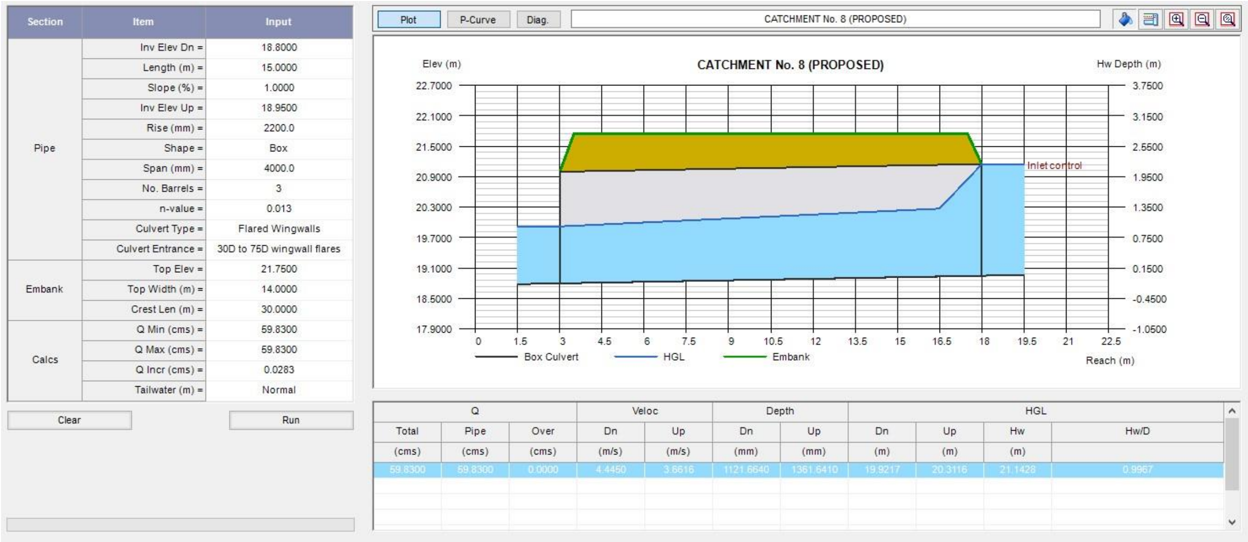
Appendix 4, Fig 4.62: Graph of runoff co-efficient against years for catchment 8



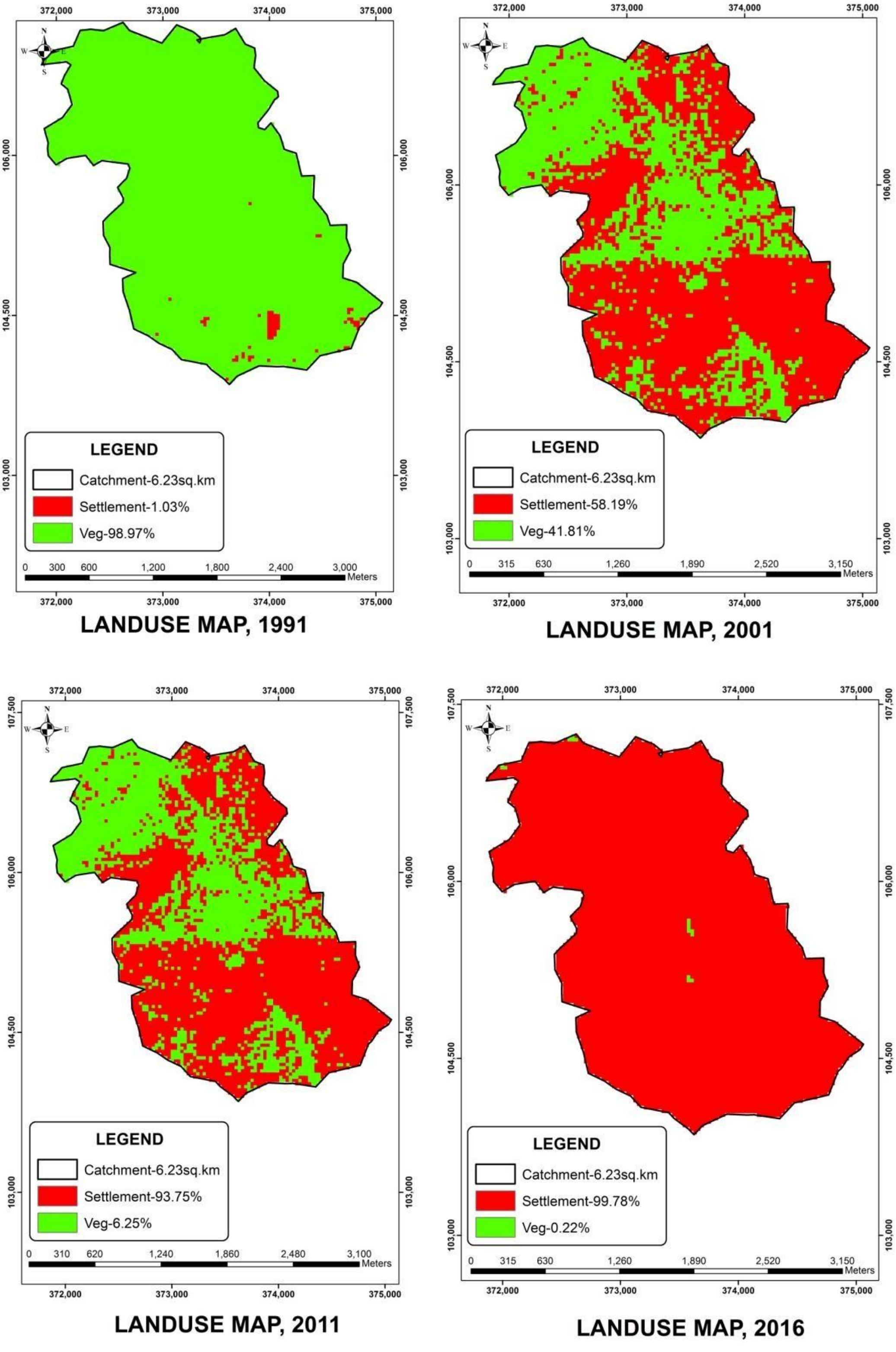
Appendix 4, Fig 4.63: Culvert at construction year 2003



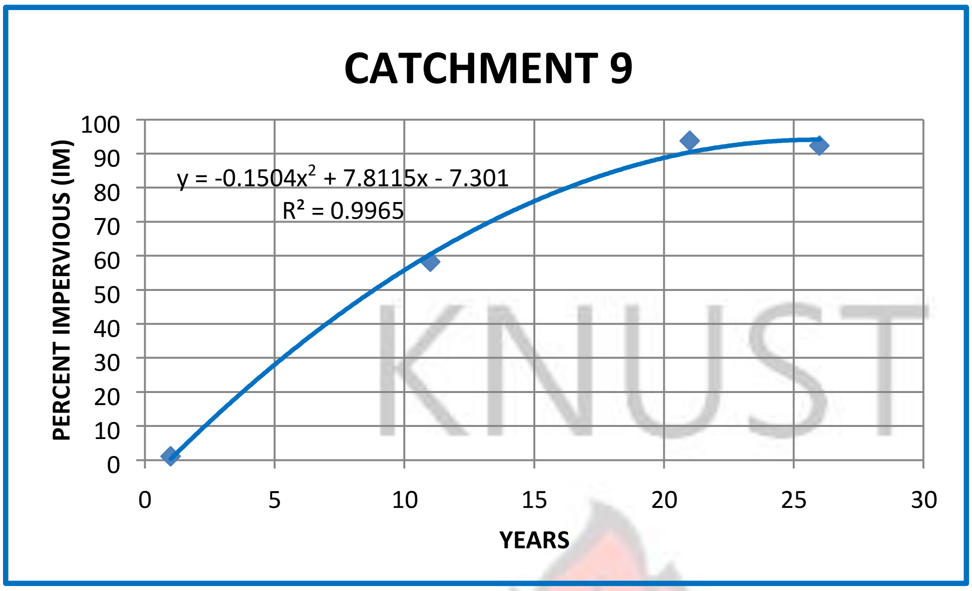
Appendix 4, Fig 4.64: Performance of culvert at 2016



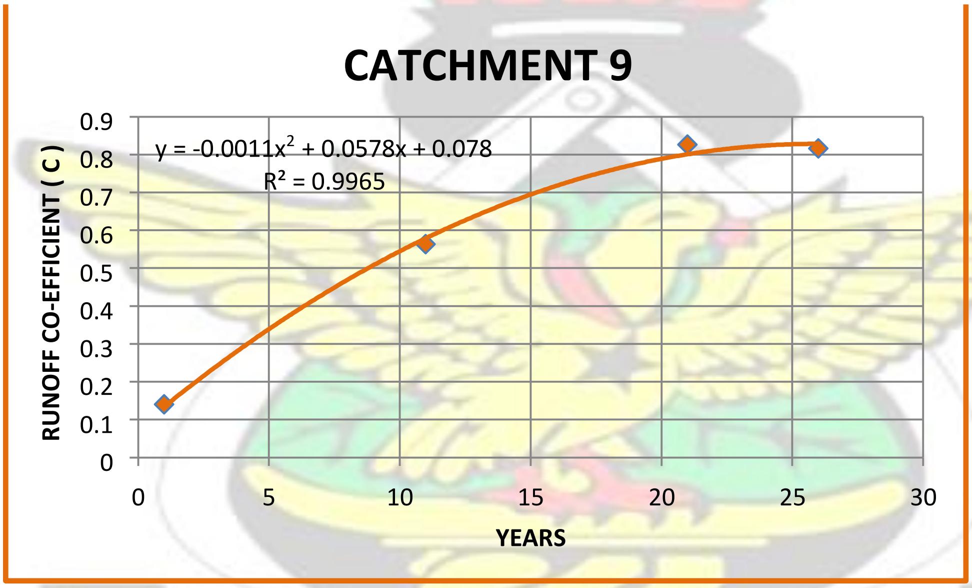
Appendix 4, Fig 4.65: Proposed Culvert.



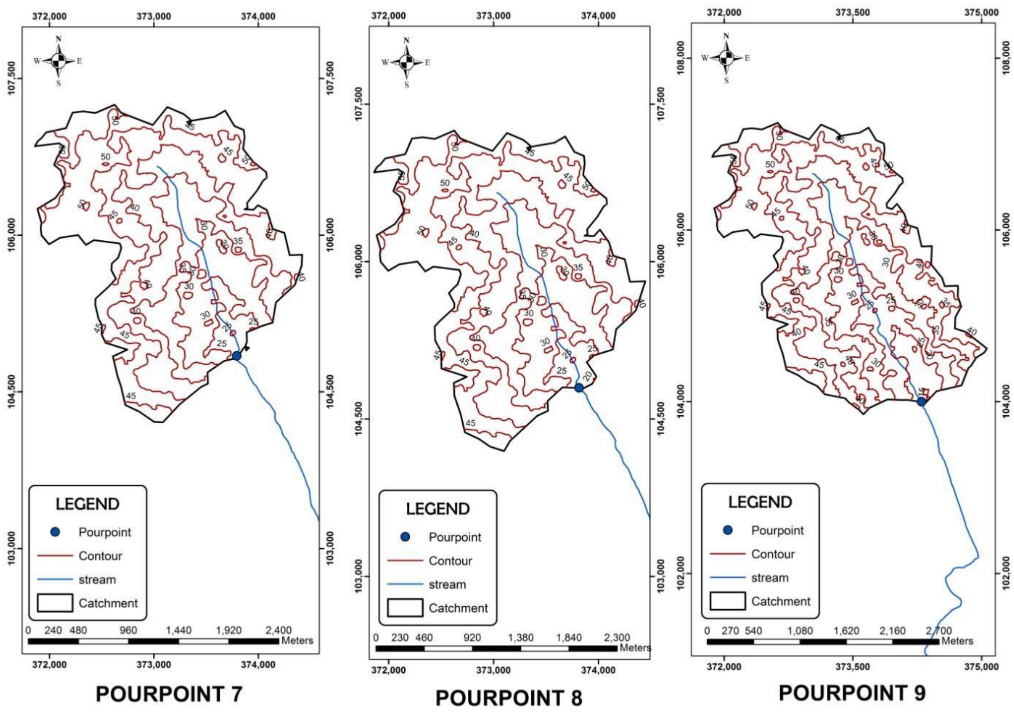
Appendix 4, Fig. 4.66; Land use, land cover maps of sub catchment no. 9



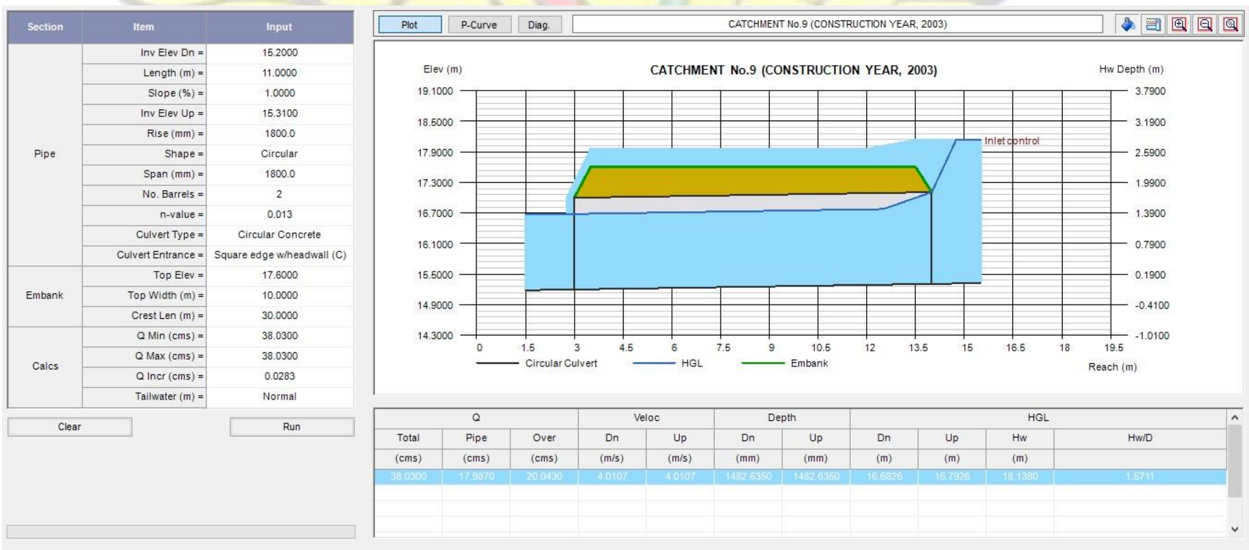
Appendix 4, Fig 4.67: Graph of percent impervious against years for catchment 9



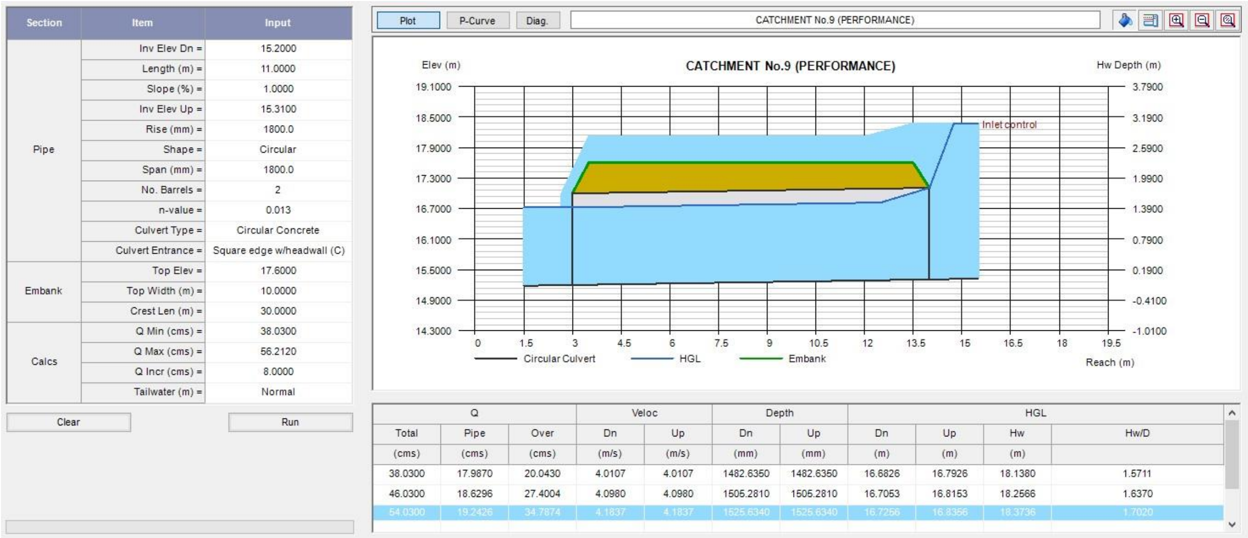
Appendix 4, Fig 4.68: Graph of runoff co-efficient against years for catchment 9



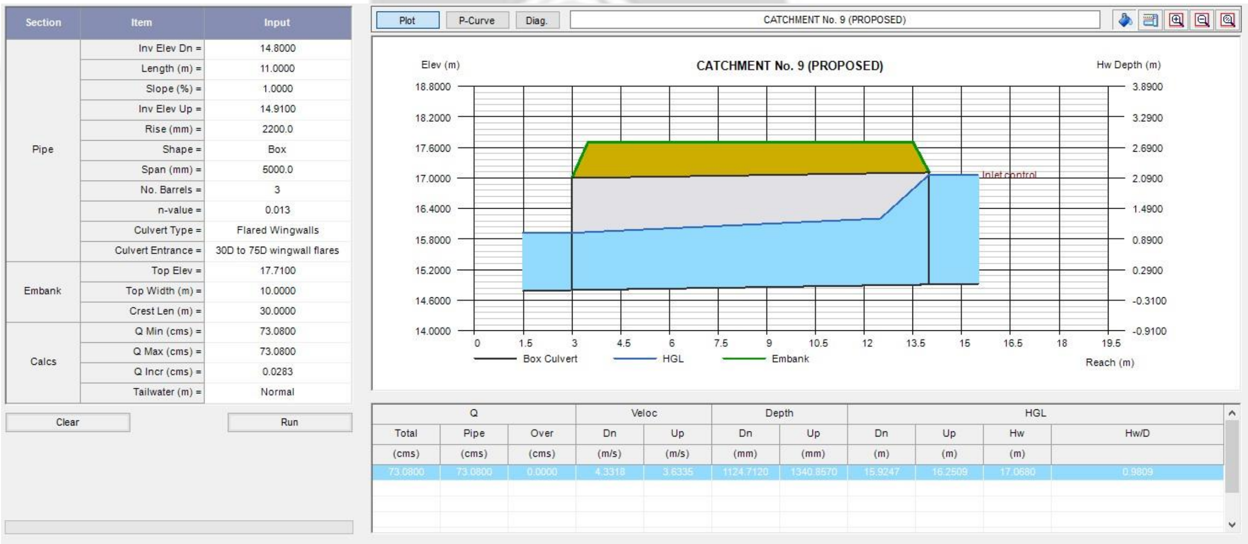
Appendix 4, Fig. 4.69: Topographical maps of sub catchments 7, 8 and 9



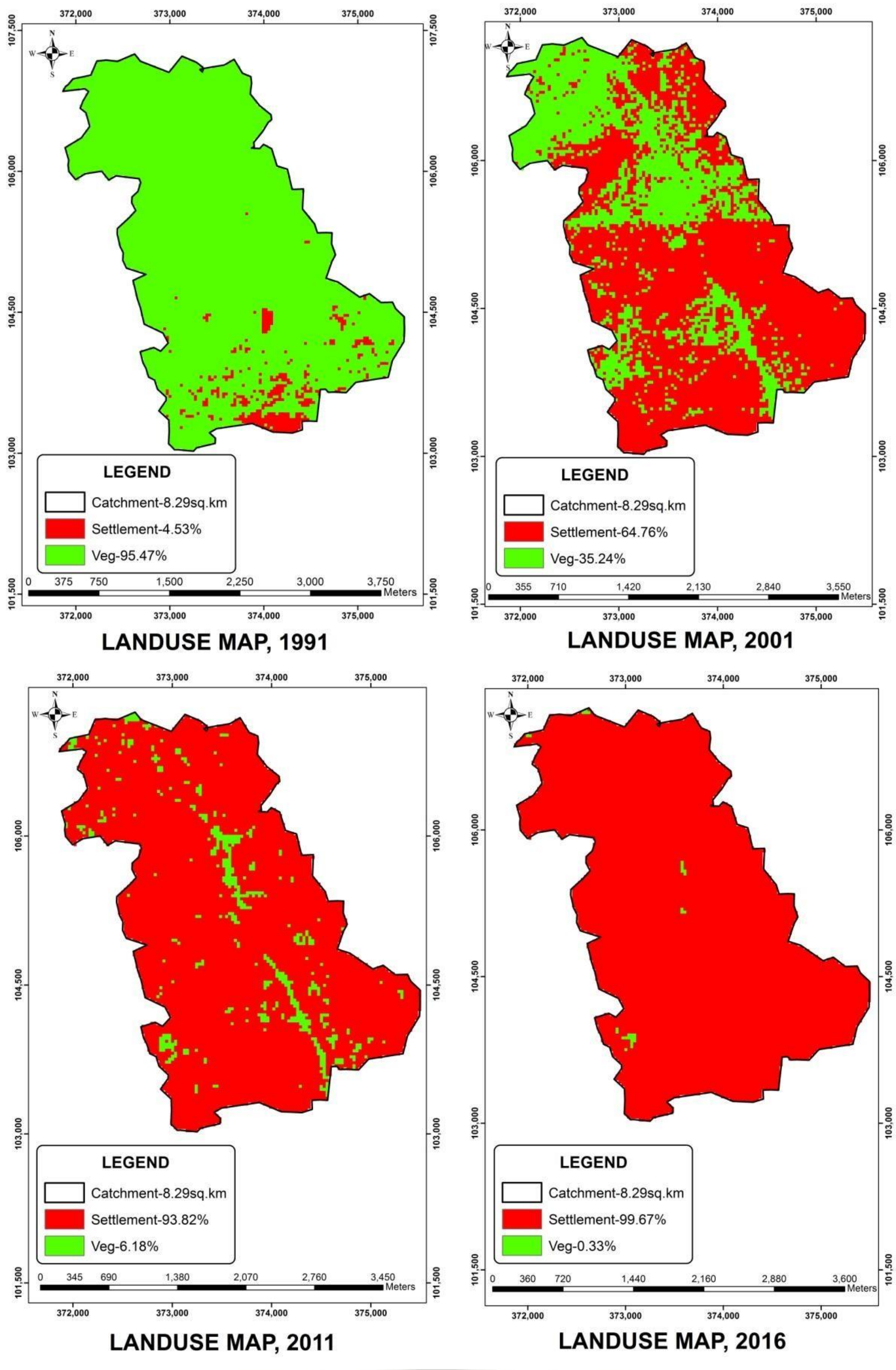
Appendix 4, Fig 4.70: Culvert at construction year 2003



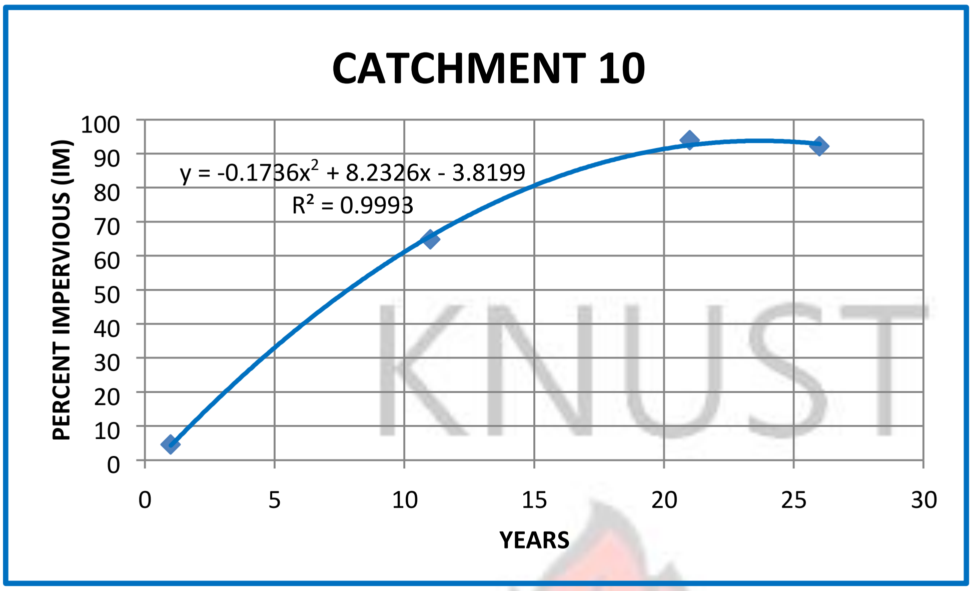
Appendix 4, Fig 4.71: Performance of culvert at 2016



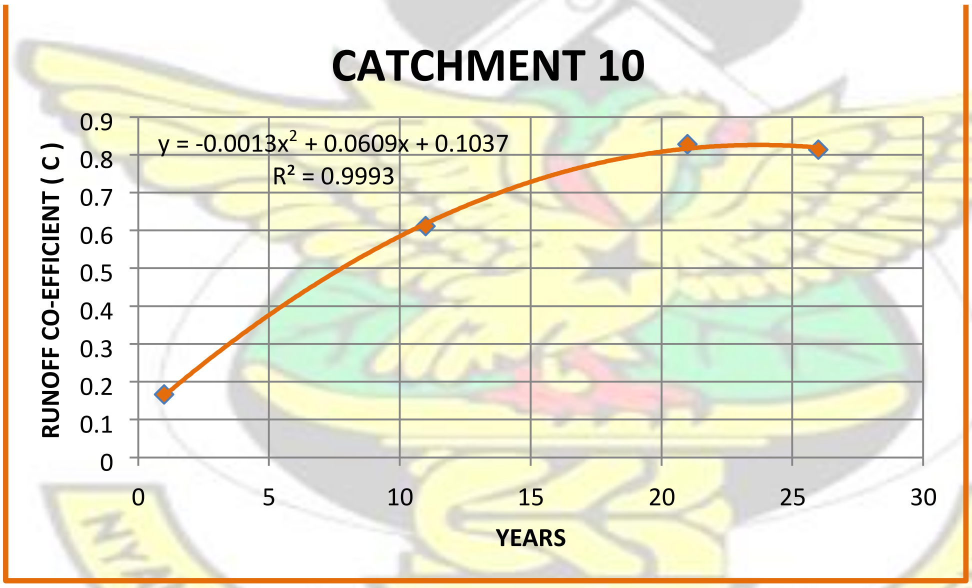
Appendix 4, Fig 4.72: Proposed culvert.



. Appendix 4, 4.73; Land use, land cover maps of sub catchment no. 10

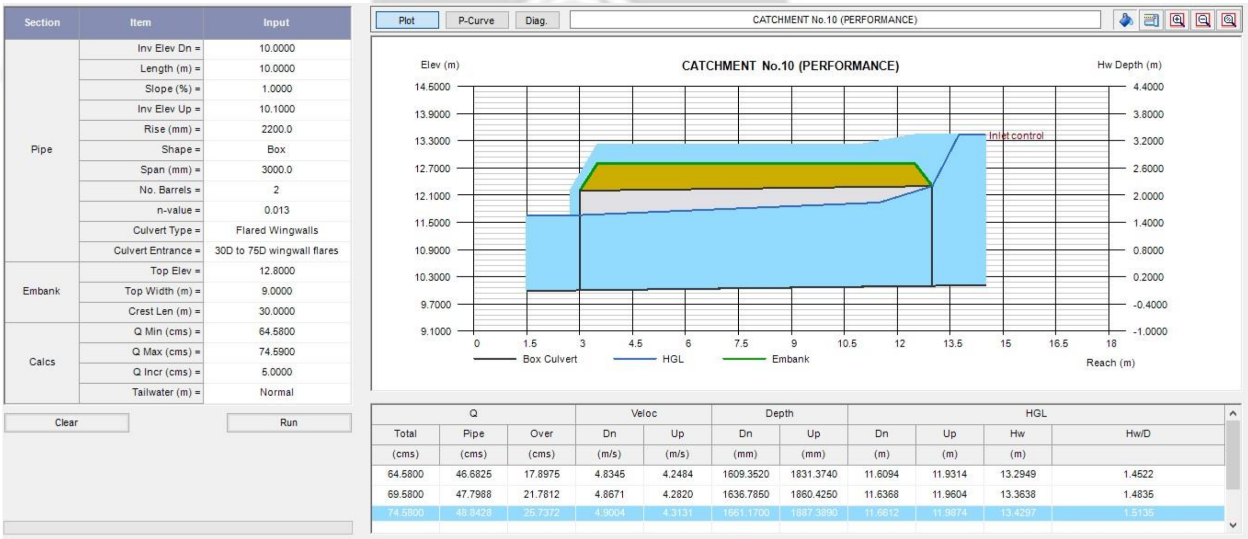


Appendix 4, Fig 4.74: Graph of percent impervious against years for catchment 10

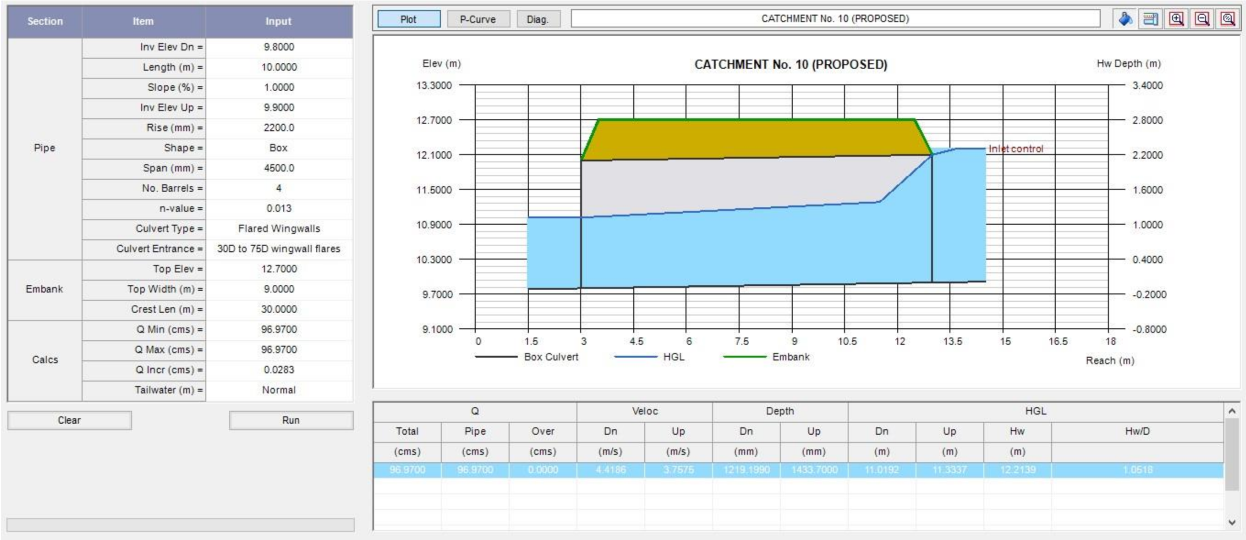


Appendix 4, Fig 4.75: Graph of runoff co-efficient against years for catchment 10

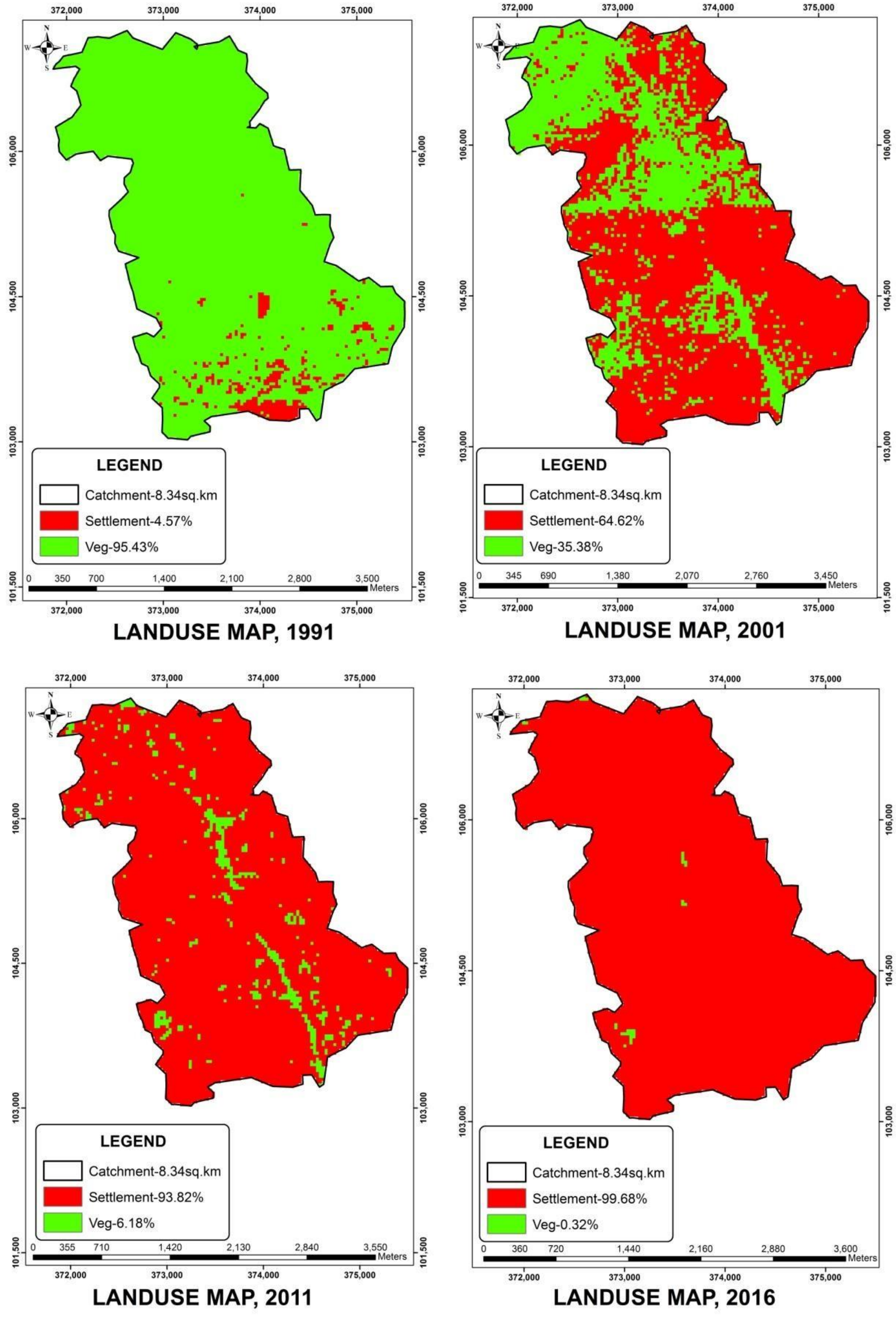
Appendix 4, Fig 4.76: Culvert at construction year 2014



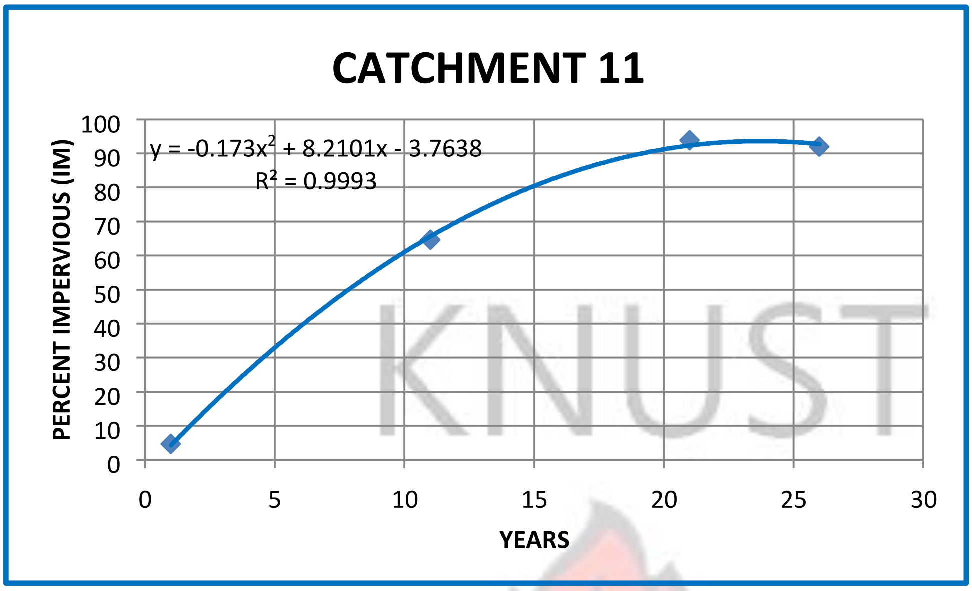
Appendix 4, Fig 4.77: Performance of culvert at 2016



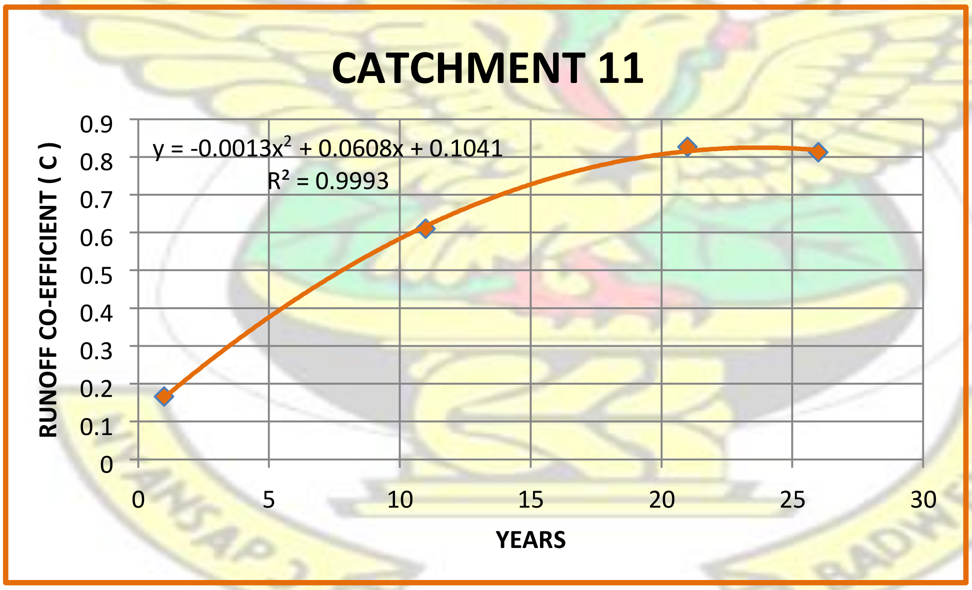
Appendix 4, Fig 4.78: Proposed culvert



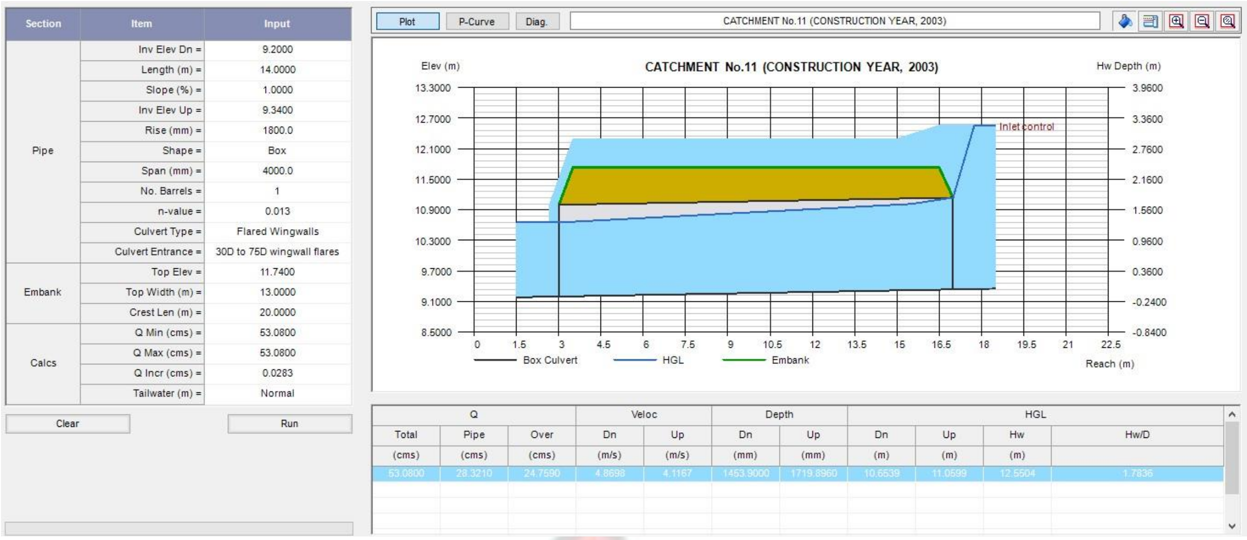
Appendix 4, Fig. 4.79; Land use, land cover maps of sub catchment no. 11



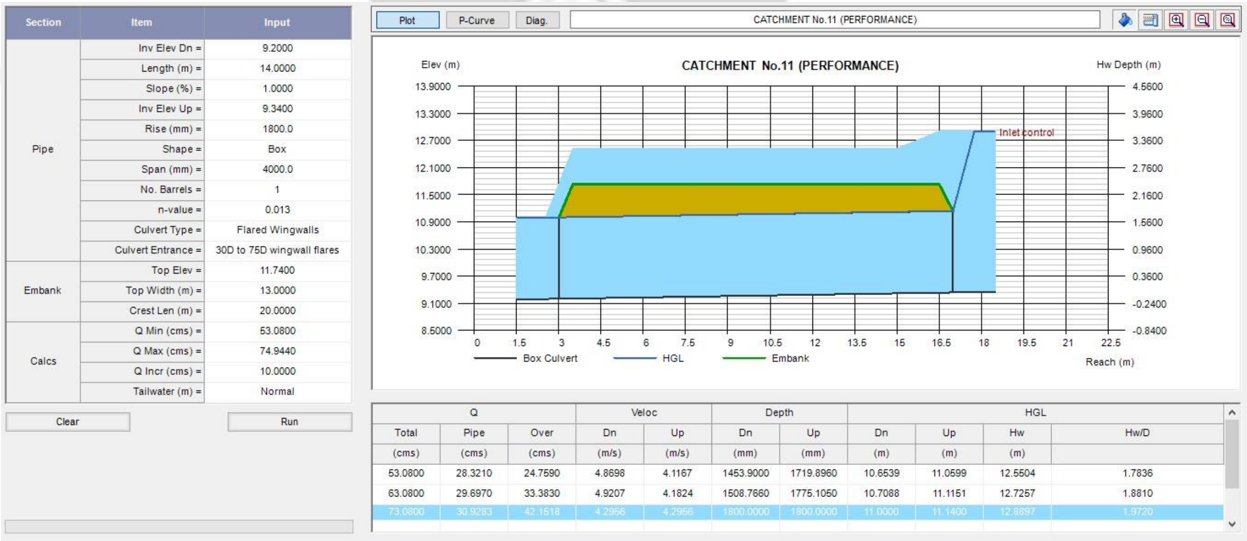
Appendix 4, Fig 4.80: Graph of percent impervious against years for catchment 11



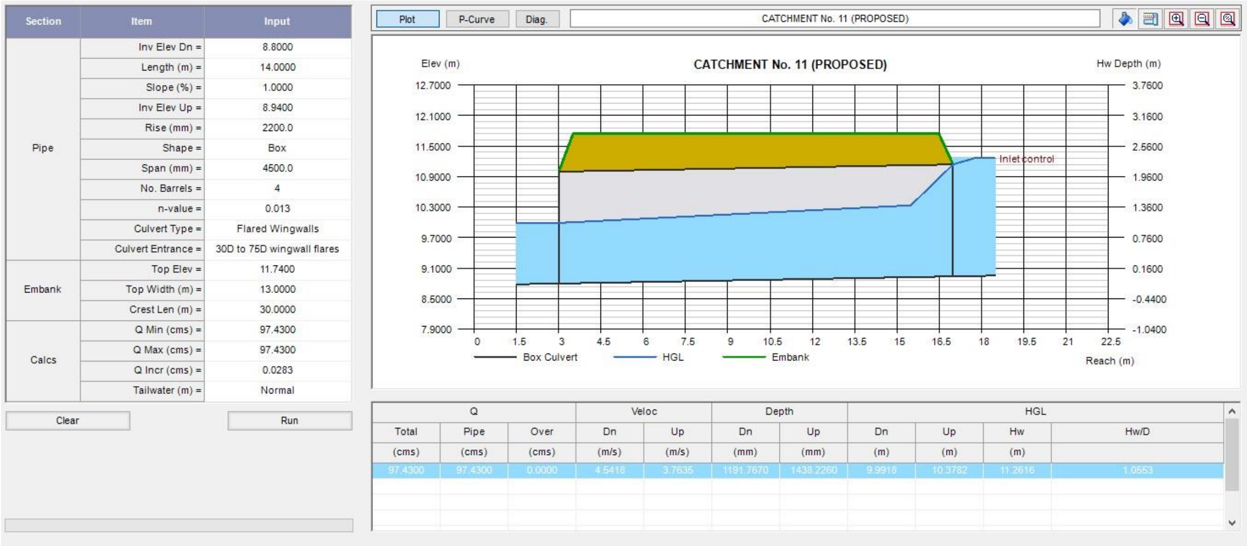
Appendix 4, Fig 4.81: Graph of runoff co-efficient against years for catchment 11



Appendix 4, Fig 4.82: Culvert at construction year 2003



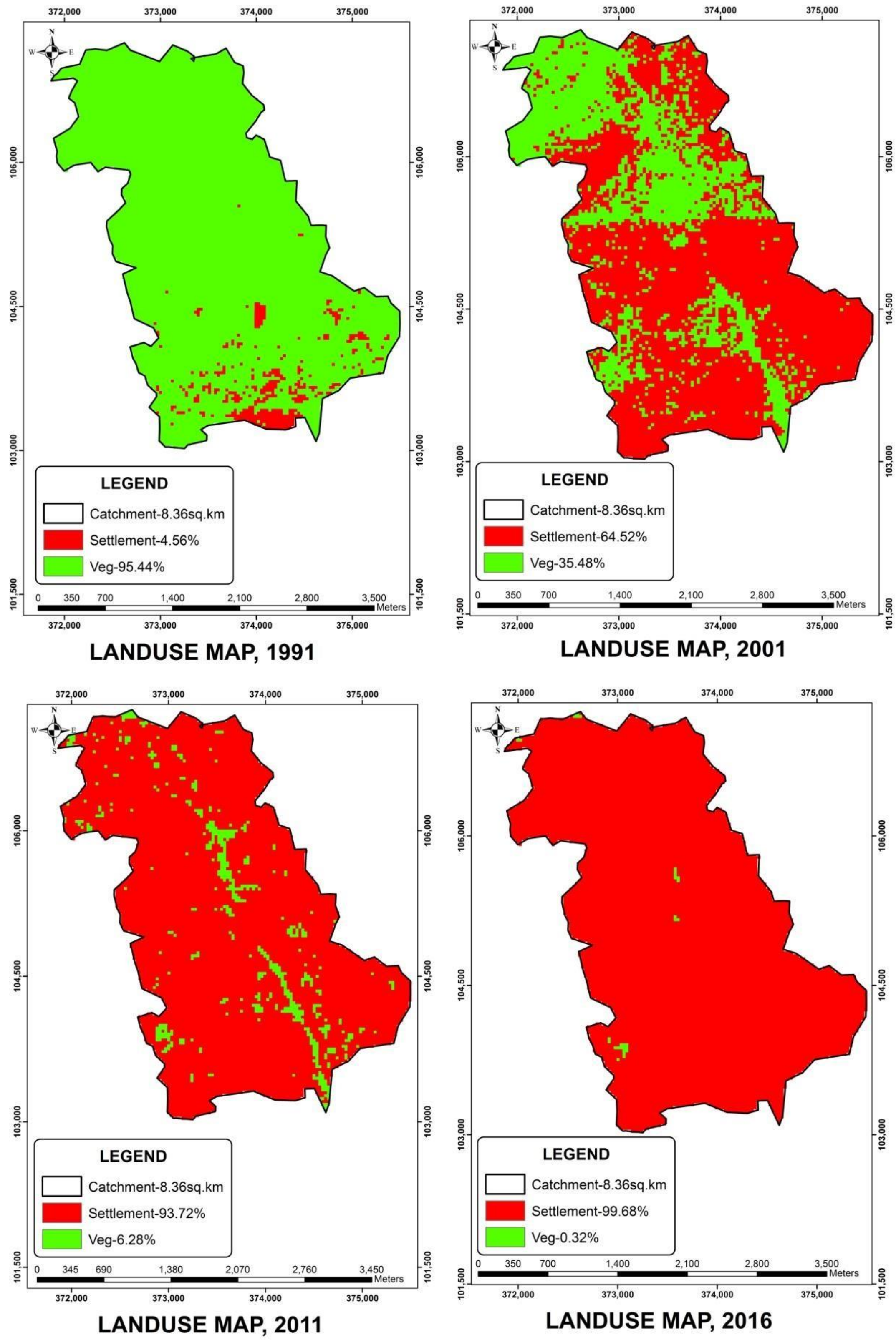
Appendix 4, Fig 4.83: Performance of culvert at 2016



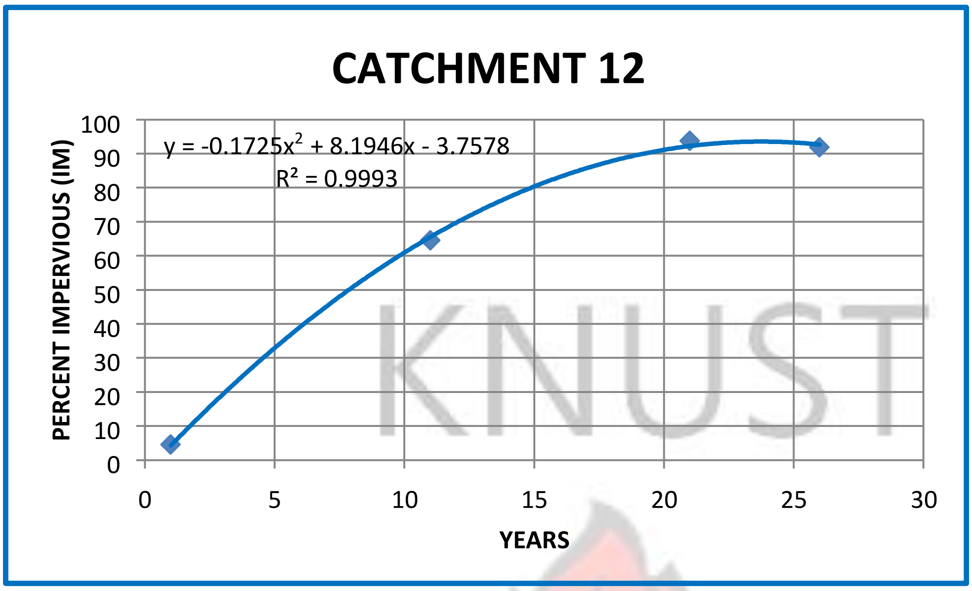
Section	Item	Input
Pipe	Inv Elev Dn =	8.8000
	Length (m) =	14.0000
	Slope (%) =	1.0000
	Inv Elev Up =	8.9400
	Rise (mm) =	2200.0
	Shape =	Box
	Span (mm) =	4500.0
	No. Barrels =	4
	n-value =	0.013
	Culvert Type =	Flared Wingwalls
	Culvert Entrance =	30D to 75D wingwall flares
Embank	Top Elev =	11.7400
	Top Width (m) =	13.0000
	Crest Len (m) =	30.0000
Calcs	Q Min (cms) =	97.4300
	Q Max (cms) =	97.4300
	Q Incr (cms) =	0.0283
	Tailwater (m) =	Normal

Q			Veloc		Depth		HGL			
Total	Pipe	Over	Dn	Up	Dn	Up	Dn	Up	Hw	Hw/D
(cms)	(cms)	(cms)	(m/s)	(m/s)	(mm)	(mm)	(m)	(m)	(m)	
97.4300	97.4300	0.0000	4.5418	3.7635	1191.7670	1438.2260	9.9918	10.3782	11.2616	1.0553

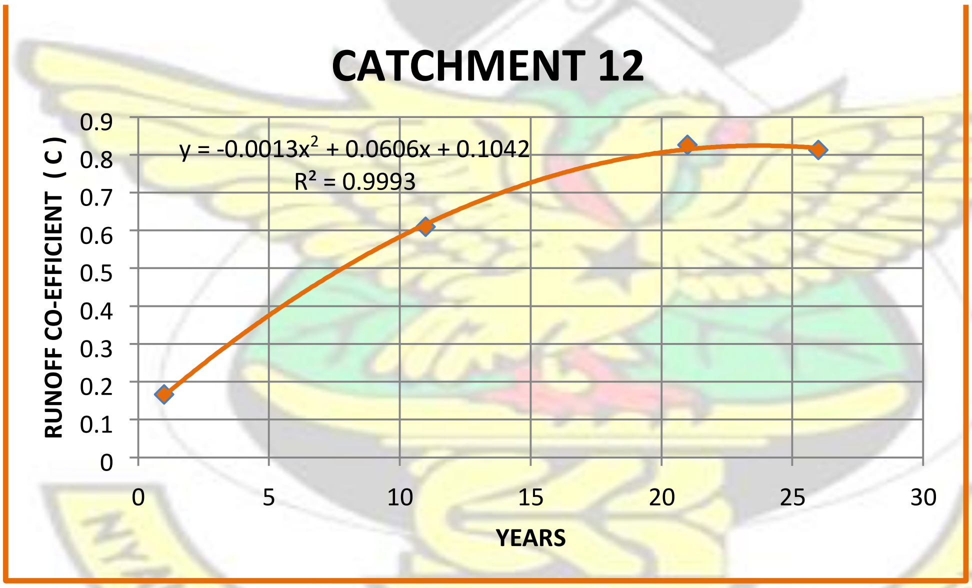
Appendix 4, Fig 4.84: Proposed culvert



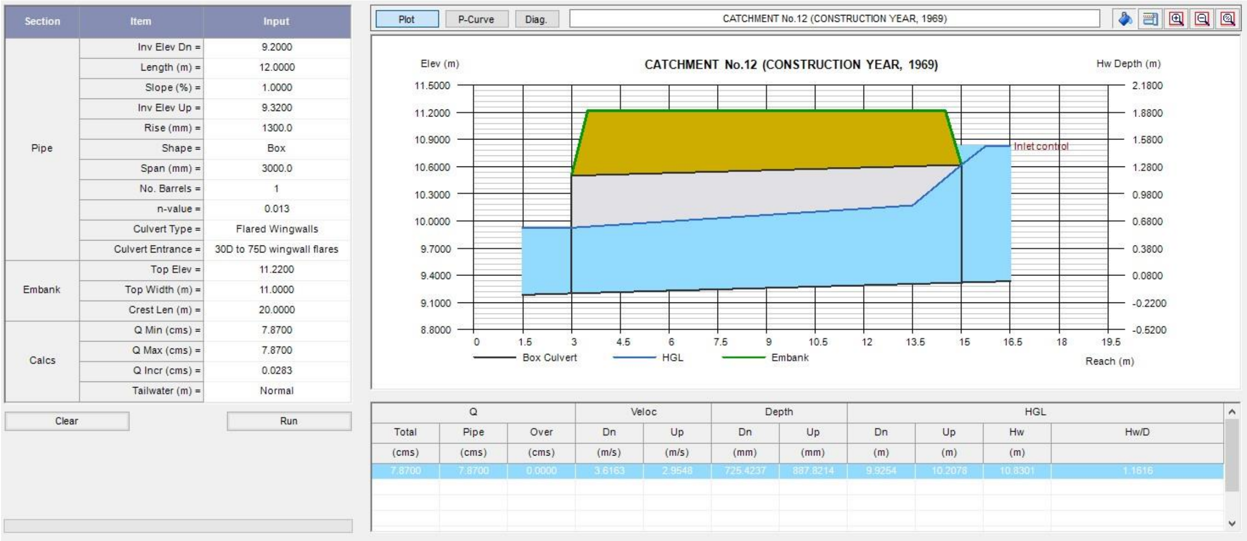
Appendix 4, Fig. 4.85; Land use, land cover maps of sub catchment no. 12



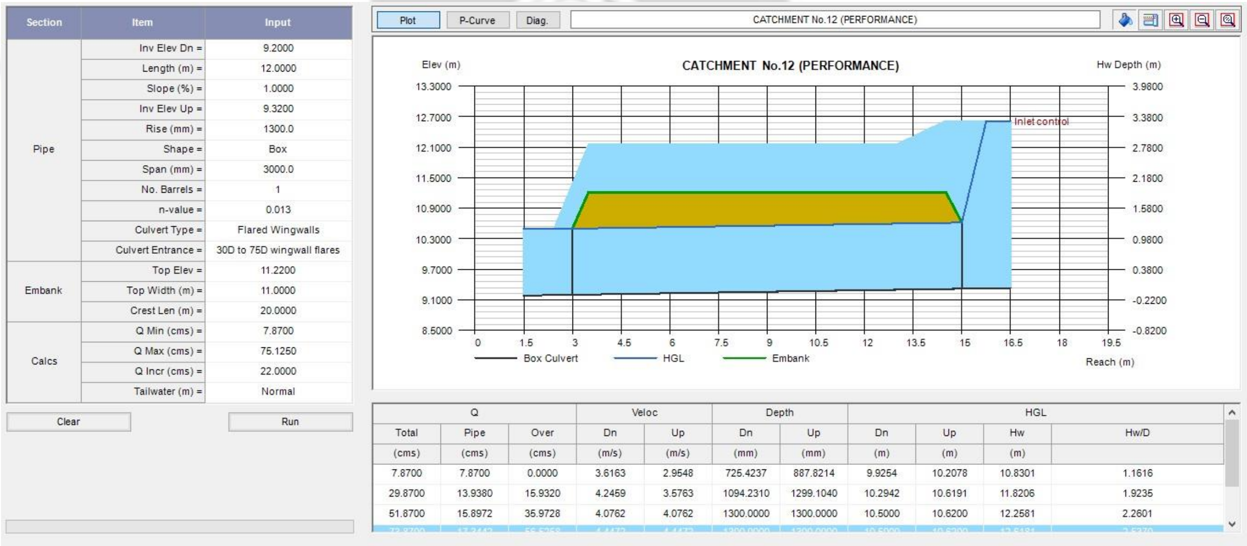
Appendix 4, Fig 4.86: Graph of percent impervious against years for catchment 12



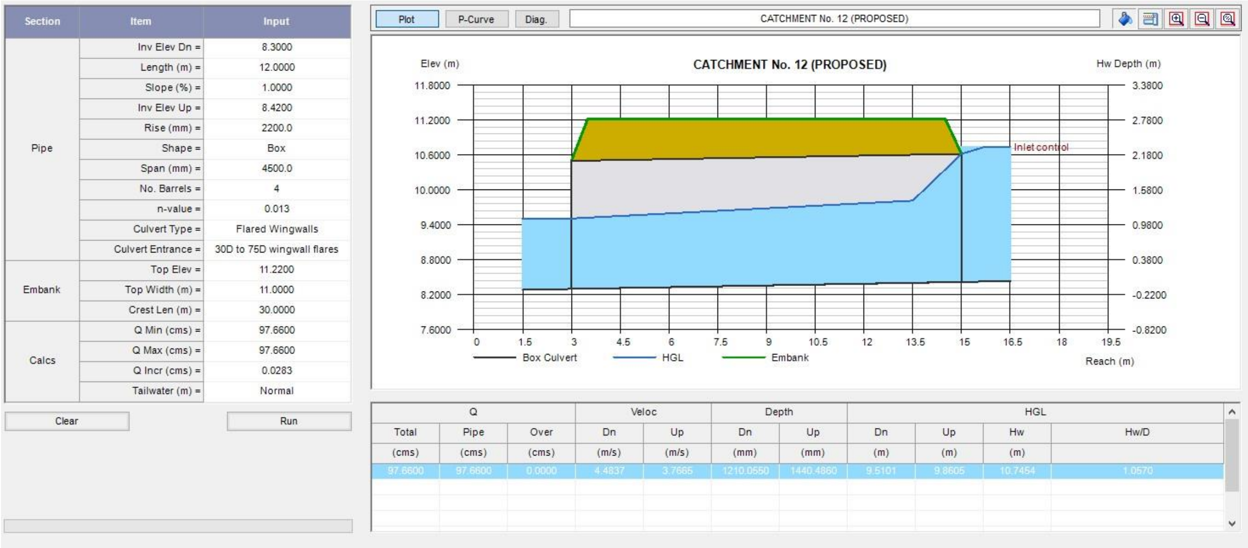
Appendix 4, Fig 4.87: Graph of runoff co-efficient against years for catchment 12



Appendix 4, Fig 4.88: Culvert at construction year 1969



Appendix 4, Fig 4.89: Performance of culvert at 2016



Section	Item	Input
Pipe	Inv Elev Dn =	8.3000
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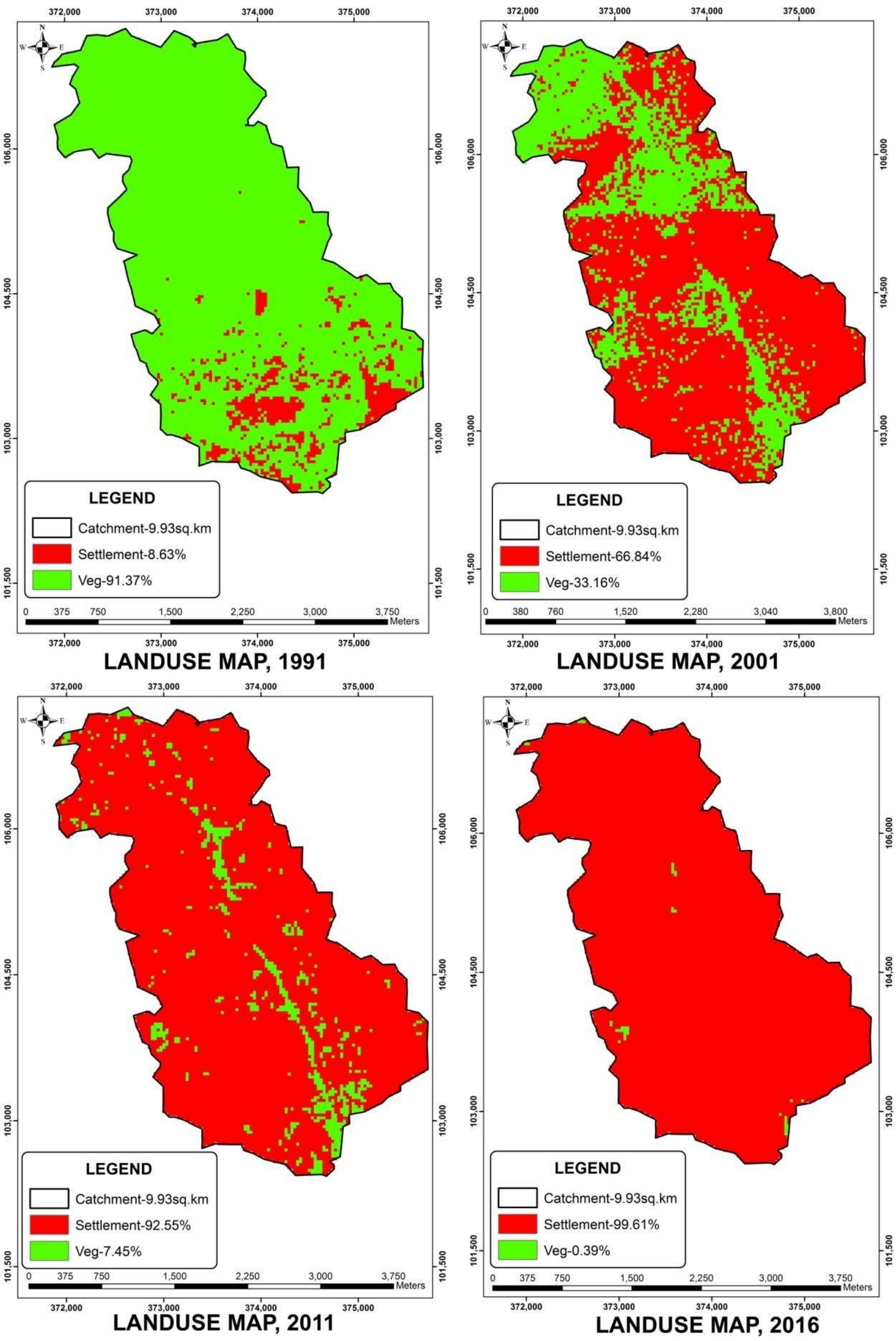
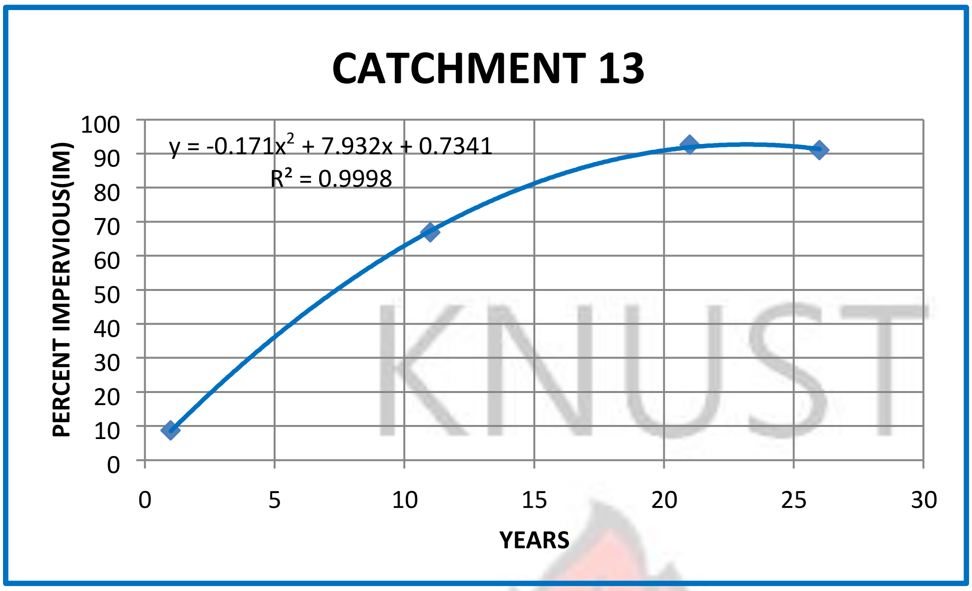
	Slope (%) =	1.0000
	Inv Elev Up =	8.4200
	Rise (mm) =	2200.0
	Shape =	Box
	Span (mm) =	4500.0
	No. Barrels =	4
	n-value =	0.013
	Culvert Type =	Flared Wingwalls
	Culvert Entrance =	30D to 75D wingwall flares
Embank	Top Elev =	11.2200
	Top Width (m) =	11.0000
	Crest Len (m) =	30.0000
Calcs	Q Min (cms) =	97.6600
	Q Max (cms) =	97.6600
	Q Incr (cms) =	0.0283
	Tailwater (m) =	Normal

Q			Veloc		Depth		HGL			
Total	Pipe	Over	Dn	Up	Dn	Up	Dn	Up	Hw	Hw/D
(cms)	(cms)	(cms)	(m/s)	(m/s)	(mm)	(mm)	(m)	(m)	(m)	
97.6600	97.6600	0.0000	4.4837	3.7665	1210.0550	1440.4860	9.5101	9.8605	10.7454	1.0570

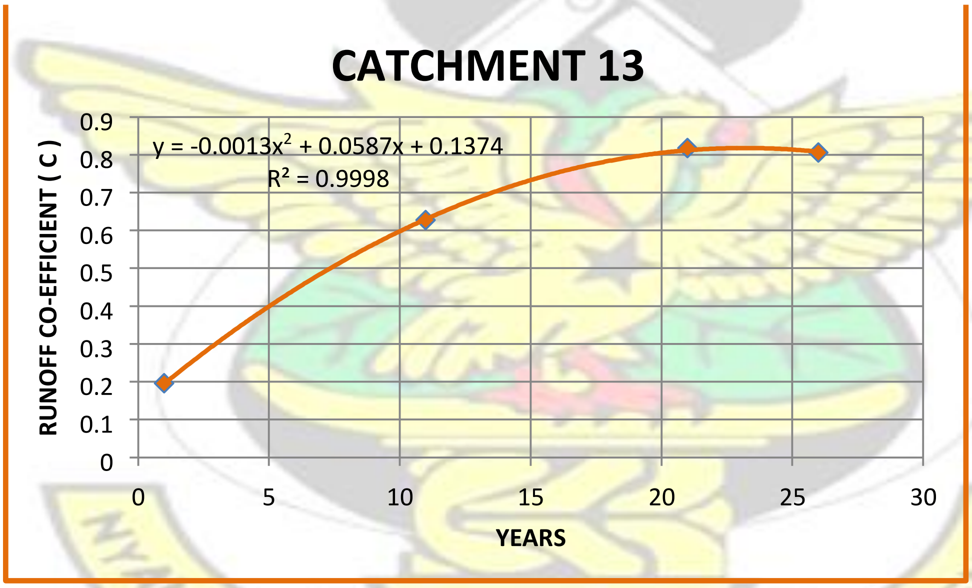
Appendix 4, Fig 4.90: Proposed Culvert



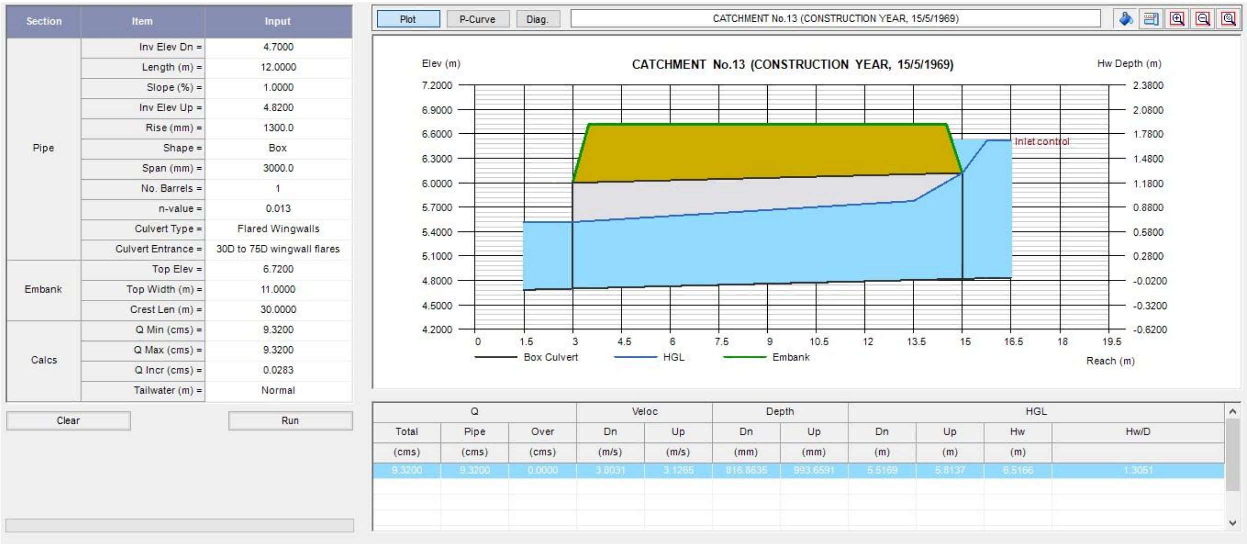
Appendix 4, Fig. 4.91; Land use, land cover maps of sub catchment no. 13



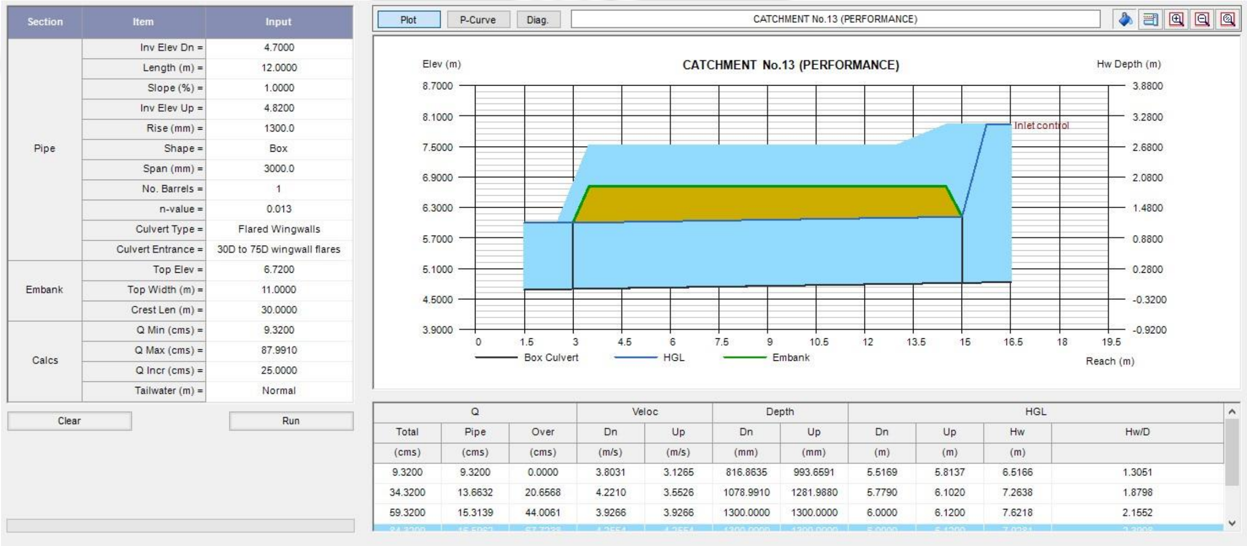
Appendix 4, Fig 4.92: Graph of percent impervious against years for catchment 13



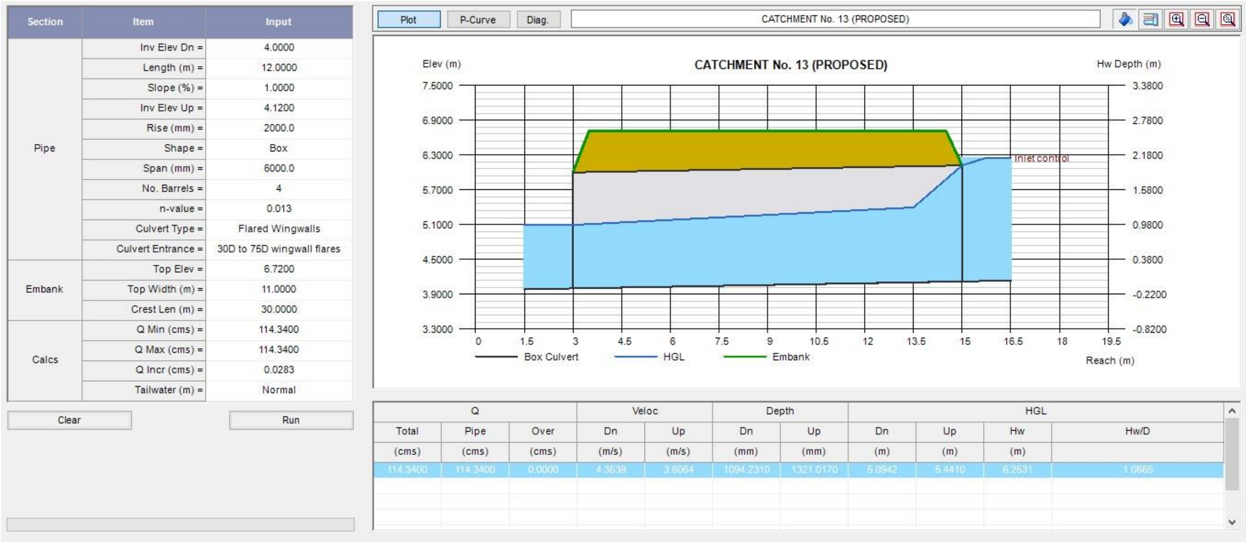
Appendix 4, Fig 4.93: Graph of runoff co-efficient against years for catchment 13



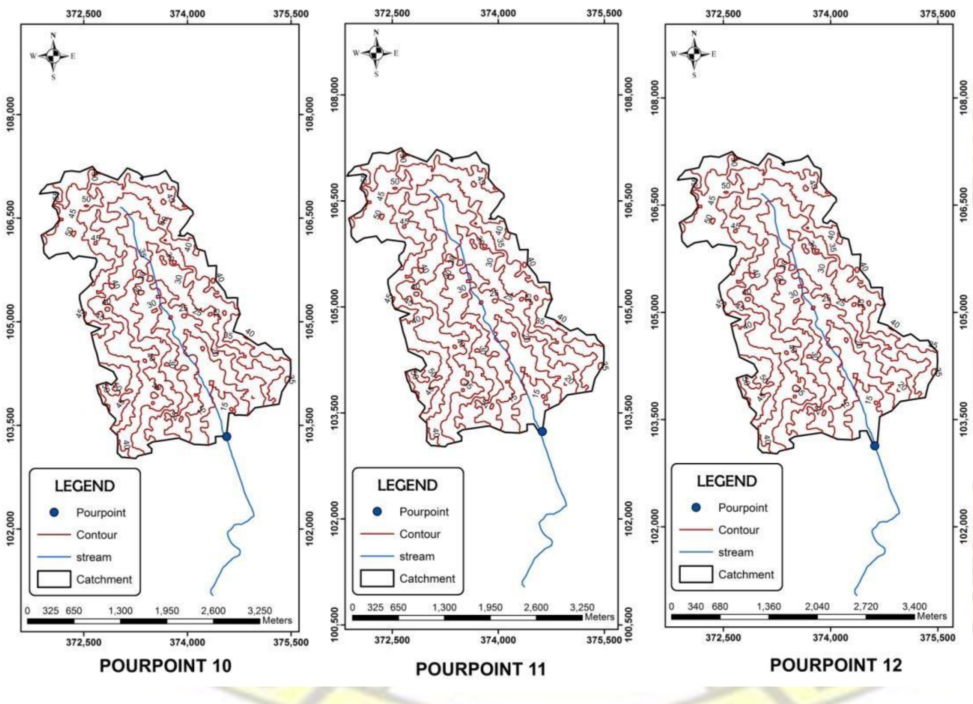
Appendix 4, Fig 4.94: Culvert at construction year 1969



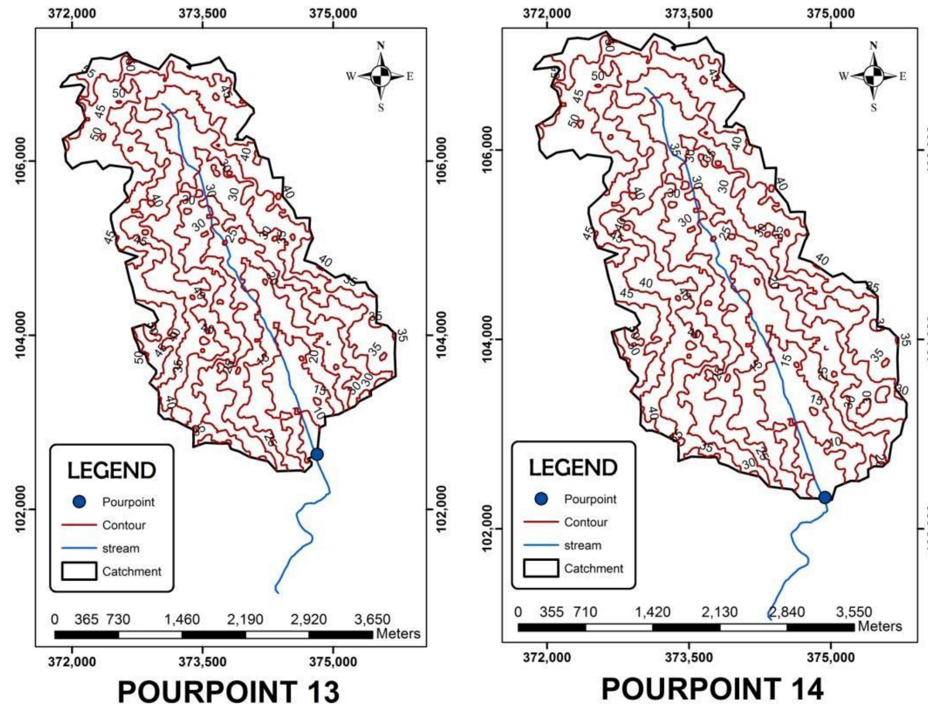
Appendix 4, Fig 4.95: Performance of culvert at 2016



Appendix 4, Fig 4.96: Proposed Culvert



Appendix 4, Fig. 4.97 : Topographical maps of sub catchments 10, 11 and 12



Appendix 4, Fig. 4.98: Topographical maps of sub catchments 13 and 14