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STORM WATER MANAGEMENT IN OTTERI NULLAH SUB BASIN, CHENNAI CORPORATION, CHENNAI, TAMILNADU

PURPOSE DRIVEN STUDY

(UNDER HYDROLOGY PROJECT-II)

आपो हिष्ठा मयो भुवः

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PREFACE

Chennai city often faces the problem of floods in many areas during rainy season. Heavy rain associated with cyclonic activity resulted to catastrophic flooding in Chennai during 1943, 1978, 1985, 2002 and 2005. In 2005, a 100 years return period rainfall of 40 cm in a day caused heavy inundation in and around the Chennai city and its suburban areas and more than 50,000 persons had to be evacuated from the existing low lying areas. Flooding of less catastrophic nature also occurs regularly in low-lying areas of the city and its suburbs. In order to have a scientific understanding the problem and a feasible solution, Govt. of Tamilnadu requested NIH to carry out 'Urban Hydrology of Chennai City'. As a part of Hydrology Project-II, a Purpose Driven Study (PDS) was initiated by NIH on 'Storm Water Management in Otteri Nullah sub basin, Chennai Corporation, Chennai with the consent of Govt., of Tamilnadu for micro level storm water modeling.

For the purpose of Storm Water Modelling of Otteri Nullah sub-basin, a 2-D dynamic rainfall-runoff storm water management model XP-SWMM was used. The hourly rainfall data at Nungambakkam for a period of 30 years from1980-2009 was used for rainfall frequency analysis and to derive IDF curve. The DEM and land use/cover maps of the study area was prepared from DGPS survey, satellite data and SOI maps. During the study, 5 tipping bucket rain gauges and 2 automatic water levels recorders were installed in the study area. Based on measured rainfall and water level data in the study area, few events were selected for the model performance in terms of runoff computation and to calibrate and validate the model. After successful calibration, it was found that even a peak discharge of 27.57 m^3/s generated from the hyetograph of 24 hour design storm of 2 years return period having maximum hourly rainfall of 48.89 mm is causing flood at many locations. The drain sections were then modified as proposed by PWD. The model simulation predicted adequacy of drainage upto design storm of 5 years. The impact of flood water diversion link from Otteri Nullah, west of Annanagar to Cooum river found that there is 38% of reduction in the peak flow against 2 -years return period storm. The hydrographs at outfall of the sub basin developed for various return period design storms computed by the model would be very useful for best management practices (BMP). The study would also provide a guideline that may be followed for macro level drainage of basin or other sub basins having similar hydrological conditions.

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ABSTRACT

Otteri Nullah sub basin spread over an area of 30.63 sq km is one of the 12 sub-basins of Chennai Municipal Corporation (174 sq km). The study entitled "Storm Water Management in Otteri Nullah Sub basin" envisages development of a 2-D dynamic rainfall-runoff storm water management model (XP-SWMM) to study the adequacy of existing drainage network of the sub-basin draining to Buckingham Canal for different return periods. The model predicts runoff hydrographs based on the input hyetograph and the physical characteristics of the sub basin. Since the only hourly rainfall data was available at Nungambakkam IMD station near to the Otteri Nullah sub basin, five more tipping bucket rain gauges and two automatic water levels recorders were installed in the study area.

The DEM and land use/cover maps of the sub-basin were prepared from DGPS survey and satellite data. The storm water drainage network details and Otteri Nullah longitudinal profiles/cross section details at every 30 m chainage were collected and GIS database was prepared. Using thematic layers of DEM, drainage network and road network, total 88 micro watersheds were delineated in the Otteri Nullah sub basin. The drainage network was. schematized into 121 nodes and 120 links in the XP-SWMM model. Model parameters like Node/link characteristics, pervious/impervious area, soil type, average width/slope and SCS-CN were computed for each micro watershed using GIS data base. Based on measured rainfall and water level data in the study area, few events were selected for the model performance in terms of runoff computation and to calibrate and validate the model.

After successful calibration, it was found that even a peak discharge of 27.57 m^3/s generated from the hyetograph of 24 hour design storm of 2 years return period having maximum hourly rainfall of 48.89 mm is causing flood at many locations. The drain sections were then modified as proposed by PWD. The model simulation predicted adequacy of drainage upto design storm of 5 years. The impact of flood water diversion link from Otteri Nullah, west of Annanagar to Cooum river found that there is 38% of reduction in the peak flow against 2 - years return period storm. The hydrographs at outfall of the sub basin developed for various return period would be very useful for adopting best management practices.

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1.0 INTRODUCTION

According to United Nations projections, 60% of the World's population will live in cities by 2030. Skyscrapers, paved roads, storm water drains, sewer drains and illuminated light system etc. are the symbol of urban areas. A typical urban land cover consisting of impervious rooftops, streets, and parking lots allow far less surface retention and infiltration. Urbanization increases storm water runoff volumes and rates and possibly causes flooding.

Urban development also has adverse effects on the quality of storm water runoff. Pollutants from various sources accumulated over the impervious surfaces during the dry periods are washed off when rain occurs, and they are quickly discharged into the drainage system. Main sources of the urban storm runoff pollution are dust and litter and possible dumping of untreated water from industries, fecal dropping from pets and other animals, oil spills from motor vehicles, garbage from residential areas, pesticides and fertilizers from lawns and gardens, and eroded soil from construction sites etc. Urban storm runoff can contain various types of toxic materials, and it is considered as a major threat to the receiving waters. During rainy seasons, urban areas are subjected to flooding due to non-provision/insufficient storm water drains to convey storm water safely to a suitable water body. If the water stagnates, then there is a likelihood of spreading water borne diseases. In order to overcome waterlogging and drainage congestion, the storm water drainage system has to be designed based on short duration rainfall (15min) and high-resolution topographical information. The design principle starts from the analysis of historical rainfall data. Estimation of surface runoff due to rainfall events is a key factor in drainage system network design. The rational method although is simple and widely used for the design of storm water drainage system, it allows estimation of discharge peak only. The method is unsuitable to account for large catchment for pipe routing, variations in rainfall intensity, a number of contributing areas and different rate of contribution. The rational method lumps of all physical parameters into two parameters namely runoff coefficient and time of concentration. This makes parameter estimation subjective, and prediction of discharge from observed rainfall is inaccurate.

As a part of Hydrology Project-II, a Purpose Driven Study (PDS) was initiated on 'Storm Water Management in Otteri Nullah sub basin, Chennai Corporation, Chennai' with the consent of Govt., of Tamilnadu. The study includes, installation of rain gauges to record short duration rainfall (15min) and installation of water level recorders for model calibration and validation. The study also incorporates collection of detailed topographical information employing DGPS field survey. Data were collected from Chennai Corporation, Indian Meteorological Department (IMD) and Public Works Department (PWD), Chennai. Based on field data, a suitable storm water management model XP-SWMM was setup to evaluate storm water flooding phenomena at Otteri Nullah sub basin.

1.1 Project Objectives

The main objectives of the project are given below:

1. Evaluation of existing storm water drainage network (adequacy of the existing drainage network) in the study area using mathematical model XP-SWMM.
2. To find out the inflow-outflow hydrograph at various outlets and the water surface profile along the storm water drains.
3. Feasibility to integrate the existing drainage network with other possible watercourses to mitigate urban storm water flooding in the study area.
4. Dissemination of results of the project through workshops/brain storming sessions/awareness programs with the help of NGO's/Govt., departments /Academic Institutions in the study area and elsewhere during the study period.

2.0 REVIEW OF LITERATURE

Mott MacDonald International (1993), Cambridge, UK studied three cases of catastrophic flooding occurred in Madras during 1943, 1976 and 1985. It was concluded that heavy rains associated with cyclonic activity caused these floods and these events were all attributed to failure of the major river drainage systems. In addition to these, it was also mentioned that flooding of a less catastrophic nature occurs regularly in low-lying areas of the city where the local drainage infrastructure is inadequate or inoperative. This report also brought together previous work carried out to assess the problems of flooding in Madras city, modern hydrological-hydraulic modeling techniques and measures to alleviate flooding in the northern part of the city. Revised criteria and a methodology for the design of new drains have been suggested especially 1 in 2 years with 60 minute design storm along with use of the MIDUSS (Microcomputer Interactive Design of Urban Storm water System) or similar drainage system analysis package. Thereafter, efforts were initiated by CMDA on urban hydrology of Chennai city using 15 minute rainfall and 0.25 m topographical details (www.cmdachennai.org).

2.1 Storm Water Runoff Computations

The first method to calculate storm water runoff was evolved in 1857 with a rule of thumb approach. The 20^{th} June 1857, on the Savoy street sewer at London, 25.4 mm rainfall occurred in 75 minutes and it produced a peak flow of 2.728×10^{-4} cumec/acre. This was the basis of the thumb approach. As a thumb rule (on English rate) it was "about half of the rainfall would appear as runoff from urban areas". These ideas were not scientifically based, but they were the stepping-stones to the development of modern hydrologic models.

Following the concept of early rate of thumb "empirical formulae" became the principle tool for quantifying the runoff. Most of this second generation approach was macroscopic. They considered the entire drainage area as a single unit, assumed the rainfall as uniformly distributed over the area and calculated the runoff only at the downstream point. The foremost example of this approach is the rational method (Kutchling, 1989) introduced in the United States. It was based on four years of rainfall data using non-recording rain gauges and one year of runoff data from pairs of white washed sticks. Five open ditches were used for flow determination. Rational method was used for over a half a century with changes in its original form. Even today professionals working in urban hydrology are using this method due to its simplicity. A second example of the macroscopic approach is the unit hydrograph method developed by Sherman (1932). Unit hydrograph is the hydrograph of a centimeter of runoff from a drainage basin produced by a uniform rainfall of unit duration. Originally, the unit hydrograph concept was applied mainly to river basins, but now a days it is used for urban watershed also, after the introduction of concept of instantaneous unit hydrograph (Nash, 1957). The third generation approach is "microscopic approach" characterized by all pertinent physical phenomena as input (rainfall) to the output (runoff) involving the following steps: a) determination of the design storm, b) calculation of the rainfall excess rate, c) determination of the flow to gutter from overland flow, d) routing of the gutter flow to the main channel, e) system and f) determination of the outflow hydrograph. The accuracy of the results is affected by accuracy of calculating the hydraulic phenomenon and the validity of assumptions employed. Tholin's hydrograph method (Tholin and Keifer, 1959) is an example of the microscopic approach. In the past, most of the microscopic approaches dealt with individual storm events. But with the advent of microcomputers, continuous simulation of hydrologic process is possible now and this trend is on increase (Crawford and Linsley, 1966). The fourth generation approach is "Simulator Models" for the urban watershed analysis. Physical models, analogue models or digital models may simulate hydrologic systems. Storm Water Management Model (SWMM) and Stanford watershed model are widely useful for urban watershed and small watershed studies. GIS has a long history of use in the water resources field. This is due in large part to the early availability of remotely sensed spatial data suited for this purpose. Work in natural area tends to focus on grid, or raster based, hydrology, whereas work in urban areas is more complex and requires more complex models that are vector based. Raster based approaches use rectangular as their fundamental unit within which hydrology characteristics are uniform. Vector based models use coordinate geometry to define unique boundaries of hydrologic characteristics.

2.2 Urban Hydrology Models

The first runoff model was developed based on Tholin Hydrograph method (Tholin and Keifer, 1959) and thereafter the urban runoff models was improved through British Road Research Laboratory Model, RRLM (Watkins, 1962), University of Cincinnati Urban Runoff Model, UCURM (Papadakis and Pruel, 1972), HEC-1 (Hydrological Engineering Centre, 1985), HEC-2 (Hydrological Engineering Centre 1982) and SWMM (Metcalf and Eddy, 1971). The EPA-SWMM (Rossman, 2005) is the latest model in use for urban drainage.

National Institute of Hydrology (1988-89) report described the details of the various available urban runoff models, their methodology and application. Thirteen models were discussed and compared which includes, SHE (System Hydrologic European) model, USDA model, SWMM model, UCURM model, RRLM model, HEC-1 and HEC-2. These are physical models, which simulate catchment transformation process based on physical processes involved with some degree of reality. The steps considered in the modeling of overland flow are a) to decide the method of spatial representation of the catchment, b) to decode upon the various key parameters to be used and finally c) to select an appropriate numerical method for solving the equations.

Effective Urban storm water management is highly dependent on appropriate consideration of the spatial variability of urban watershed characteristics (Huber and Dickinson, 1992). This realization has prompted increasing use of physical based urban watershed models such as the Environmental Protection Agency Storm Water Management model (EPA-SWMM). The use of spatially distributed, physically based model enhances the ability to simulate the dynamic response of urbanizing. Since continuously measured runoff discharge data are generally lacking in urban area for model calibration purposes, physically based models provide a means of predicting runoff based on other field measured data and map information. In addition, physically based models provide a stronger basis for evaluating the impacts of system with structural and non structural urban storm water management strategies. With this enhanced technical capability, the spatial data base makes the physically based modeling more realistic. Catchment information was constructed in an ARC/INFO database and transformation developed using this information to generate the input information necessary for operation of a SWMM-based catchment modeling system to simulate surface runoff. The application of the GIS to storm water management of urban development can be accommodated in a low cost, PC-based computing environment and GIS addresses issues such as data precision, accuracy, resolution, and degree of aggregation to provides an improved assessment of the reliability of estimated parameters as compared to traditional methods (Meyer et al., 1993).

Refsgaard et al. (1995) described the evolution of the Danish Hydraulic Institute's (DHI) land process hydrologic model SHE (System Hydrology European) and its extensive use of GIS.

Bellal et al. (1996) studied partly urbanized basins using GIS and hydrological model. The hydrological model was based on a non-urban water budget, with modifications to account for urbanization. The model inputs was based on a digital elevation model (DEM) and raster based land use data.

Feinberg and Uhrick (1997) discussed on integrating an infrastructure database in Broward County, Florida, with a GIS, water distribution and wastewater models. The HydroWorks model is used to simulate the wastewater collection system with close integration with database of infrastructure characteristics and the GIS.

Shamsi (1996) distinguishes three forms of information exchanges between ArcView GIS and the EPA storm water management model (SWMM): interchange, interface, and integration, listed in order of complexity. Integration, as defined by Shamsi (1996), combines a SWMM graphical user interface (GUI) with a GIS to provide a complete data environment.

Shamsi (1997) points out the advantages of a GUI and provides a summary of software features and needs for SWMM interfaces. Hellweger and Maidment (1999) developed an integrated application for delineating drainage basins and determining surface runoff in natural watershed using the HEC-HMS (Hydrology Engineering Center – Hydrologic Modeling System). Application of GIS in urban storm water system has been limited because of the need for large, expensive, and detailed spatial and temporal databases.

Esteves et al (2000) developed the two-dimensional model based on the explicit finite difference scheme (Mac Cormack) coupling the overland flow and infiltration processes for natural hill slope represented by topographic elevation and soil hydraulics parameters. This model allows modeling of hortorian overland flow and infiltration during complex rainfall events. They used Green-Ampt equation for reproducing of overland flows and transfer between different levels of catchments in the region. The accuracy of the results was tested by comparison with experimental field data on the basis of calibrated soil and surface friction parameters.

Zoppou (2001) presented review of urban storm water models. These models have been categorized in terms of their functionality, accessibility, water quantity and quality components along with their temporal and spatial scale. The overview of modeling approaches to simulate storm water quantity and quality and their limitations and assumptions are also included. The functionality and accessibility of representative models are given in table 1 and components in the quantity analysis in representative models are given in Table 2.

Delleur (2003) presented the details of evolution of urban hydrology: past, present, and future of urban hydrology after homage to professor Ven Te Chow. The main conclusion of the study was that in all urban water problems, whether runoff quantity or quality, or water supply and waste water treatment, can no longer be evaluated system in isolation but will have to be looked at in an integrated way at basin level.

Davies et al (2008 a, b) presented case studies on impacts of climatic change and urbanization on drainage in the Helsingborg, Sweden (Suburban storm water and also in combined sewer system). These two studies revealed that, urbanization was successfully simulated to reflect current trends in demographic and water management. It was also found that city growth and projected increases in precipitation, both together and alone, may worsen the current drainage problems. Conversely, installation of sustainable urban drainage systems (SUDS) has a positive effect on the urban environment.

Barco et al (2008) have developed auto calibration for US EPA SWMM model by applying to a large urban catchment in Southern California. An optimization procedure using the complex method of BOX was incorporated to estimate runoff parameters and ten storms were used for calibration and validation. The calibrated model predicted the observed outputs with reasonable accuracy. A sensitivity analysis showed the impact of the model parameters, and results were most sensitive to imperviousness and impervious depression storage and least sensitive to Manning's roughness for surface flow.

Amaguchi et al (2012) have developed Tokyo Storm Runoff (TSR) model and tested for urban Runoff analysis using two historical events in small and large urban watersheds. The recent advances in GIS technology and new data availability open up new possibilities concerning urban storm runoff modeling.

Fletcher et al (2013) have brought out a state of the art on 'Understanding, Management and Modeling of Urban Hydrology' and its consequences for receiving waters. It was mentioned that the ability to predict urban rainfall, with technology such as radar and microwave networks showing promise. It is highlighted that urban flood once regarded only as a nuisance, storm water is now increasingly regarded as a resource.

Karla and Malik (2014) have used storm CAD software for evaluating existing stormwater drainage network in the Chandigarh, India and found that the computed average runoff coefficient from the model is in good agreement with the rational method runoff coefficient, which was adopted for the study region. Literature review indicated that the storm water management is one of the important activities in urban towns in terms of flood management. If the quality of storm water is up to the mark, there is a huge scope for utilizing it as groundwater recharge. In order to augment storm water, there should be a systematic rainfall measurement, water levels measurements, periodic up gradation cross sectional profiles of storm water drains and high-resolution topographical information are very essential in the present contest. The available advanced computation tools are very useful for better storm water management in any region. It was also learnt that there is a lack of flood markings and measured flood discharges for successful calibration and validation of Mathematical models.

3.0 STUDY AREA

The urban population in India has grown from 25.7 million in 1901 to 286.1 million in 2001. Chennai (earlier called as Madras) was established in 1639 as one of the East India company's earliest trading ports and latter became the center for the company's control over southern India. By the end of the 18^{th} century, the north of Chennai city had become profoundly different from the south. The north Chennai was densely populated than south. By the time of year 1871, the population of the city had reached over 4 lakh. The Chennai Metropolis is expected to become one of the Mega Cities in the world with more than 10 million population, in the next 10 years. The Chennai city Corporation with 176 sq. km area may have to accommodate about 59 lakh population while rest of the metropolitan area with the extent of 1013 sq. km will accommodate about 66 lakh population by 2026. The location of study area with sub basin boundaries in Chennai Corporation and Chennai basin with geology of Chennai Corporation are shown in Figures 1 and 2 respectively.

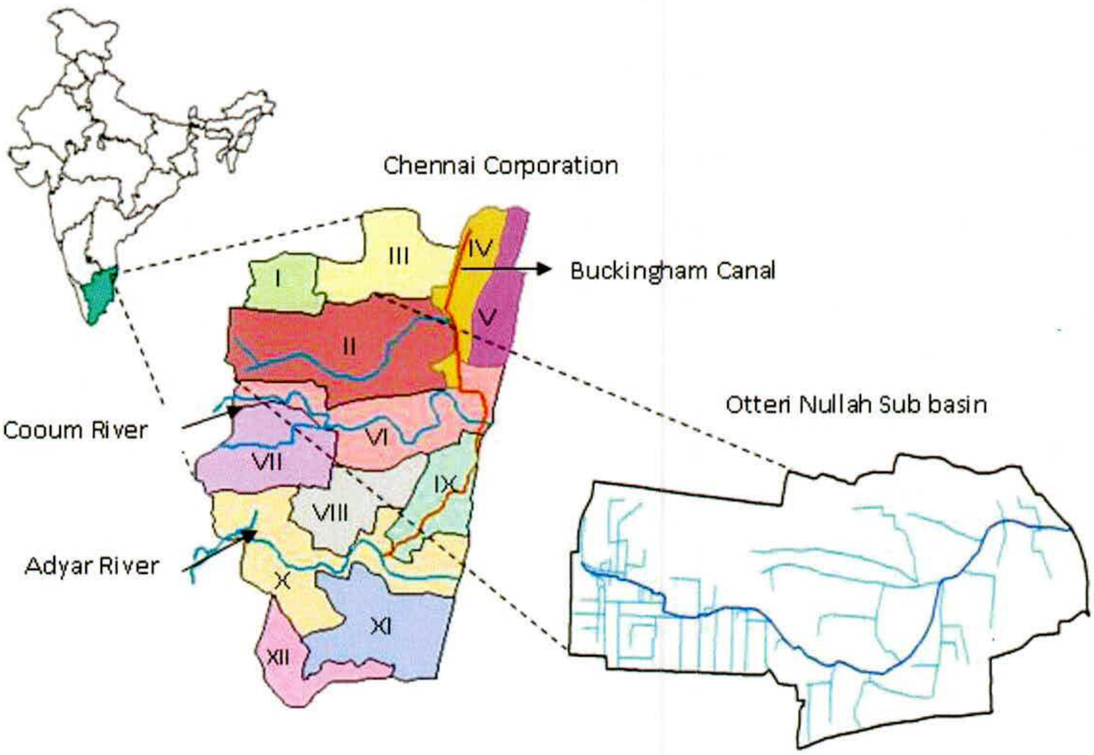

Figure 1. Location of study area and nearby sub basins in Chennai Corporation

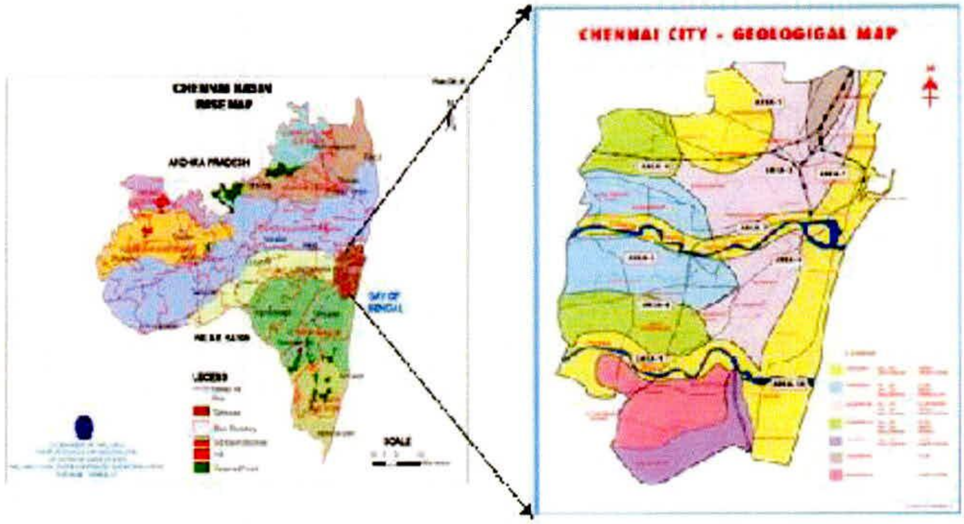

Figure 2. Chennai basin and geological map of Chennai Corporation

The boundaries of Chennai Municipal limits are spreading over 174 sq km. Mean annual rainfall in Chennai metropolitan is about 1200 mm and mean rainy days are about 52 days. The storm water drains and sewer lines are separate in the study area. Entire town drains the storm water into the Bay of Bengal mainly through two major rivers namely Cooum and Adyar Rivers. The entire city is divided into 12 watersheds based on the natural boundaries like rivers, channels, drains, roads, railway lines and contours (Figure 1). The following are the sub basins of Chennai Corporation with their geographical areas (Detailed Project Report, 2009).

Sub basin – I	Kolattur (6.92 sq.km)
Sub basin – II	Otteri Nullah (30.63 sq.km)
Sub basin – III	Captain Cotton Canal (12.81 sq.km)
Sub basin – IV	North Buckingham Canal (8.90 sq.km)
Sub basin – V	Royapuram East (9.66 sq.km)
Sub basin – VI	Cooum River (19.94 sq.km)
Sub basin – VII	Virugambakkam (13.71 sq.km)
Sub basin – VIII	Mambalam and Nandanam (10.93 sq.km)
Sub basin – IX	Central Buckingham Canal (9.74 sq.km)

Sub basin – X	Adyar River (25.5 sq.km)
Sub basin – X	South Buckingham Canal (16.97 sq.km)
Sub basin – XII	Velachery (8.3 sq.km)

These sub basins have different characteristics of their own having different types of land use pattern that affect the discharge. They have different soil characteristics, different permeability and flood absorption characteristics. Among these watersheds, the Otteri Nullah sub basin has been chosen for micro level urban storm water runoff modeling in consultation with Tamilnadu State Government. This sub basin is the largest sub basin among the sub basins of Chennai Corporation and the Otteri Nullah originates within Chennai Corporation and joins Buckingham canal.

3.1 Otteri Nullah sub basin

Otteri Nullah sub basin is located on the Northern part of Chennai city. It is surrounded by Kolattur sub basin (I) on western part, Cooum sub basin (VI) in southern part, North Buckingham canal sub basin (IV) on eastern side, Captain Cotton canal sub basin (III) on the Northern side and Otteri Nullah has a catchment area of 30.63 sq.kms. The total length of Otteri Nullah is 10.7 kms. This sub basin covers localities of Perembur, Konnur, Villivakkam, Ayanavaram, Purasavakkam, Kilpauk North, Mogapper, part of Kolattur, a part of Anna Nagar, Pulianthope and a part of Thattankulam. Micro closed drains like Anti Malarial Drain, Bricklin Road Drain, Sivagami street Drain, Konnur High Road Drain, 3rd Main Road Drain and Millers Road Drain join Otteri Nullah at different locations in addition to some road side drains that join directly Otteri Nullah. A microclosed Sivagami street drain joins micro open Ekangipuram channel and then finally drains into major Otteri Nullah. The Otteri Nullah sub basin contain 8.57 Sq.km of Commercial and Industrial, 19.45 Sq.km of Residential with high density, 0.18 Sq.km of Residential with low density and 2.43 Sq.km of Parks and open areas. The synoptic view of IRS-P6 satellite image of the study area is shown in Figure 3.

3.2 Climate

Chennai has a hot and humid climate for most of the year as it lies on the shores of the sea. The average elevation is not more than 7.0 meters. The hottest part of the year is in late May and early June, known locally as Agni Nakshatram (fire star) or as Kathiri Veyyil. Daytime

Figure 3. Synoptic view of IRS-P6 (L4Mx) satellite data (3-3-2008) of the study area

temperatures in summer ranges between 38°C and 42°C, though sometimes it goes beyond 42°C whereas the average temperature in the winters revolves around 24°C. The coolest part of the year is January, with minimum temperatures around 19-20°C. The lowest temperature recorded is 15.8°C and highest 45°C. It gets most of its annual rainfall from the north-east monsoon winds, from mid-October to mid December. The average annual rainfall is about 1,300 mm. Highest annual rainfall recorded is 2,570 mm in 2005. The Cooum (or Koovam) and Adyar rivers flow through the city. Chennai has several lakes like Red Hills, Sholavaram and Chembarambakkam Lake, which supply the city with potable water. The most prevailing winds in Chennai are the South-westerly between May and September and the North-easterly during the rest of the year.

3.3 Floods in Chennai

During the rainy season, Chennai faces the problem of floods in many areas. The information of the flood prone areas was collected by physical verification at onsite, interacting with the local people and from officials of Chennai Corporation. The last century records have shown that there were several catastrophic flooding in Chennai in 1943, 1978, 1985, 2002 and 2005 caused by heavy rain associated with cyclonic activity. These events of catastrophic flooding were found to be attributable to failure of the major rivers and other drainage systems. Flooding of less catastrophic nature occurs regularly in low-lying areas of the city and its suburbs because of inadequacy or inoperativeness of the local drainage infrastructure. The reasons for this state of affairs are three-fold. Most of the existing waterways are silted and their flow channels and banks are obstructed with encroachments and structures. Similar is the case with the reservoir and tanks. Secondly several of the areas under tanks and their anicuts have been developed as residential neighborhoods over the years. T. Nagar, Nungambakkam, Vyasarpadi are instances in this respect. The Taramani area has been developed as an institutional area. Thirdly the geological structure particularly in the south-west is not conductive to water infiltration.

3.3.1 Flood Experienced during last three decades

i) In 1946 the Chembarambakkam tank overflowed into Adayar River with a discharge of 20,000 Cusecs. In 1996 the Karanodai Bridge was collapsed.

ii) In 1976, Heavy flood and submergence was observed in Adayar-Kotturpuram TNHB quarters. Flood water could not enter the ultimate disposal point, the sea due to the prevalence of High Tide effects then.

iii) In 1985, Floods in Adayar was observed with a flood discharge of 63,000 Cusecs and the submergence of encroached flood plains.

iv) In 1996, Floods in Adayar, Cooum and Kosasthalaiyar rivers were observed. Poondi reservoir was overflowed with a flood discharge of 80,000 Cusecs.

v) In 1998, 3 persons were marooned and died in Thanikachalam nagar, a residential colony in the flood plains of Madhavaram tank's surplus course.

vi) In 2005, a 100 years recurring rainfall of 40 cm in a day caused heavy inundation in and around the Chennai city and its suburban areas and more than 50,000 persons were have to be evacuated from the existing low lying areas.

4.0 METHODOLOGY

The Storm Water and Waste Water Management Model (XP-SWMM) is a comprehensive mathematical model for simulation of urban storm water and combined sewer system. The SWMM is one of the most widely used models for analysis of urban runoff in quantity as well as quality. The SWMM transforms rainfall excess to runoff hydrograph using Manning's equation and a nonlinear runoff flow routing procedure. It is also capable of predicting and routing quantity and quality constituents of urban storm water runoff. Runoff hydrographs are predicted based on the input hyetograph and the physical characteristics of the sub catchment viz: area, average slope, degree of imperviousness, overland resistance factor, surface storage and overland flow distance. Rossman (2005) provided more details on SWMM.

In the present project, a dynamic rainfall-runoff simulation model XP-SWMM (Graphical Interface of SWMM 1-D and 2-D) used for single event and multiple days simulation of storm water runoff quantity in the study area. The runoff component operates on a collection of sub-catchment areas that receive precipitation and generate runoff. The routing portion includes runoff through system of pipes, channels, storage/treatment device, pumps and regulators. Model tracks the quantity of runoff generated within each sub-catchment and flow rate, flow depth in each pipe or channel during a simulation period consisting of multiple time steps. Catchment information is built up in GIS/image processing softwares like ARC-GIS / ERDAS and the same are transformed for developing the necessary inputs for mathematical model to simulate surface runoff processes. The following coverage (thematic maps) were developed and used in the study.

i) Sub basin and micro watershed boundaries

ii) Digital Elevation Model (DEM)

iii) Land use and soil map

iv) Storm water drainage network map

v) Drain exit points for all micro watersheds

The above coverages in turn define the model parameters like area of sub-catchment, length and slopes of channel/drains. The model routes the runoff collected from sub-catchments through the drainage network using St. Venant's equation (fully dynamic wave equation). The inputs required for developing runoff depth from each micro watershed using SWMM model is shown in Figure 4. Similarly the information required to generate runoff hydrograph is shown in Figure 5.

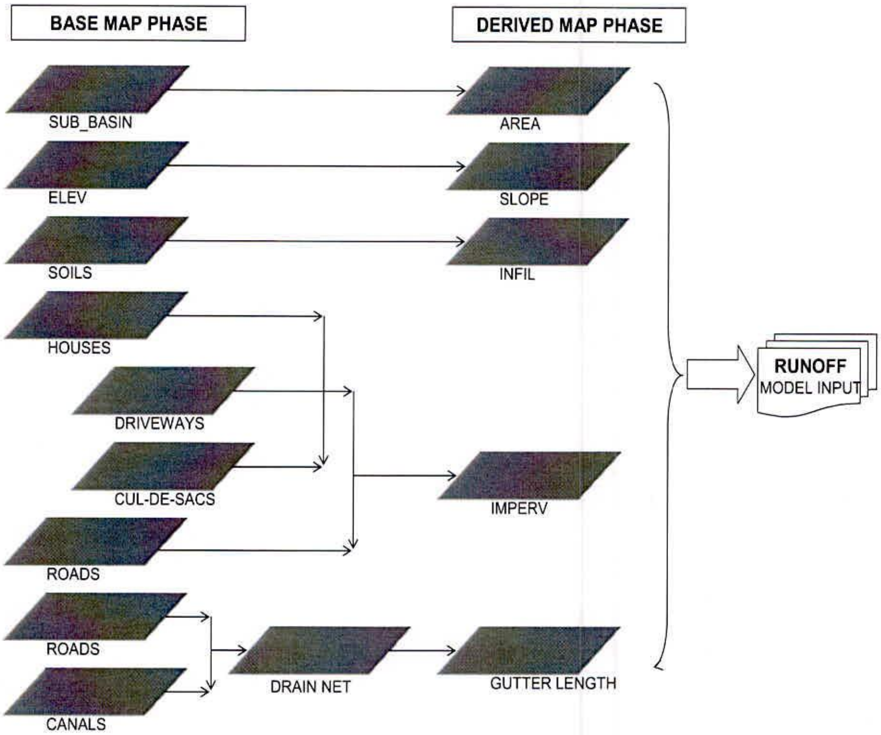

Figure 4. Flow chart showing model inputs for generating Runoff

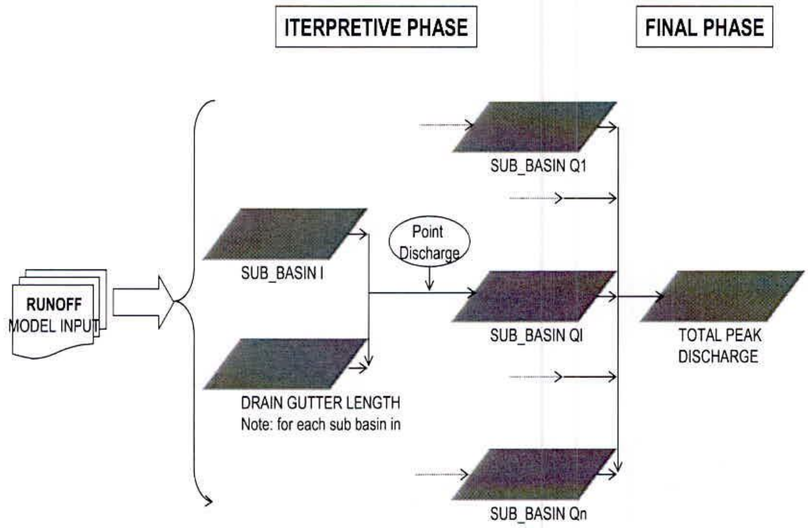

Figure 5. Flow chart showing model inputs for generating Runoff Hydrograph

4.1 Design Storm Analysis

A design storm represents the precipitation pattern used in the design of a hydrologic system. A design storm is the precipitation depth at a point, by a design hyetograph specifying the time distribution of precipitation during a storm, or by an isohyetal map specifying the spatial pattern of the precipitation. Usually the design storm serves as the system input, and the resulting rate of flow through the system are calculated using rainfall-runoff and flow routing. Design storm can be based upon historical precipitation data at site or can be constructed using the general characteristics of precipitation in the surrounding region. Their application ranges from the use of point precipitation value in the rational method for determining peak flow in the storm sewers and to use of hyetograph as input for rainfall-runoff analysis. Selection of the return period of the design storm depends on several factors such as the importance of the facilities being designed, the cost, the level of protection of the drainage facility provided, and the damages that would result from the failure of the facility. Based on the past experience and judgment, some generalized design criteria have been given for water control structures (Chow *et al.,* 1988) in Table 3.

4.2 Frequency analysis

Information on the frequency of heavy rainfall is often required by engineers and hydrologists involved in the water management and design of drainage systems. In the present study, Extreme Value Type 1 (EV1) distribution was used for frequency analysis of rainfall data (Chow *et al.,* 1988, Cunnane, 1989). The probability distribution function for EV1 is given by:

$$F(q) = P(Q \le q) = e^{-e^{-(q-u)\alpha}} \quad (1)$$

Where, u and α are location and scale parameters of the distribution and q is the threshold value. The parameters u and α are given by

$$u = P_m - 0.5772\alpha \quad (2)$$

$$\alpha = \frac{\sqrt{6}}{\pi} s \quad (3)$$

where, P_m and s are sample mean precipitation and sample standard deviation respectively. In the present study plotting position for the EV1 distribution as proposed by Gringorten (1963) was used.

$$F_i = \frac{i - 0.44}{N + 0.12} \quad (4)$$

where, i is the plotting position, N is the sample size and i is the rank with $i=1$, indicating the smallest sample member. The reduced variant of EV1 can be defined as

$$y_i = -\ln(-\ln(F_i)\ ) \tag{5}$$

$$y_{T_r} = -\ln(-\ln(1-\frac{1}{T_r})\ ) \tag{6}$$

where, T_r is the return period. Using the method of frequency factors, the expected value of P can be obtained from the relation (Eq. 7).

$$P_{T_r} = P_m + K_{T_r} s \tag{7}$$

where, K_{T_r} is the frequency factor given by (Eq. 8).

$$K_{T_r} = -\frac{\sqrt{6}}{\pi}\left(0.5772 + \ln\left\{-\ln\left(1-\frac{1}{T_r}\right)\right\}\right) \tag{8}$$

4.3 Overland Flow

As the rate of rainfall exceeds the infiltration capacity in an urban watershed, the excess rainfall will first satisfy the surface depression storage. Then, it will run off over the ground in the form of a thin sheet flow called overland flow. In many cases overland flow is the primary flow type in urban runoff. Even in rural watersheds, the volume of runoff is governed mainly by the rainfall-infiltration-overland flow processes.

Overland flow has a very small depth and a low Reynolds number. Therefore, it is often classified as laminar flow. We can use the Darcy-Weisbach formula to express the overland flow resistance as:

$$S_f = \frac{f_d q^2}{8gy^3} \tag{9}$$

Where S_f is friction slope, q is discharge per unit overland flow width, g is gravitational acceleration, y is overland flow depth and f_d is friction factor. For laminar flow $f_d = Cv/q$, C is laminar flow resistance factor and v is kinematic viscosity of water. Substituting f_d in Eq. 9, we get

$$q = \left(\frac{8gS_f}{Cv} \right) y^3 \tag{10}$$

However, overland flow resistance is affected by many more factors such as rainfall impact, partial canalization and abstractions due to rocks and litter. These factors will continuously introduce flow disturbances pulling the flow away from the laminar condition despite the low Reynold's number. The flow resistance can be approximated by an equation similar to Manning formula (Eq. 11).

$$q = \left(\frac{k}{n} \right) S_f^{1/2} y^{5/3} \tag{11}$$

Where $k = 1.0$ m$^{1/3}$/sec and n is the effective Manning roughness factor.

4.4 Sub Catchment Area

In the model, sub catchments are hydrologic unit of land whose topography and drainage system dispose the direct surface runoff to a single discharge point. The user is responsible for dividing a study area into appropriate number of sub catchments and identifying the outlet point of each sub catchment. Discharge outlet points can either be a node of the drainage system or other sub-catchment.

Sub-catchment area can be divided into pervious and impervious sub-areas. Surface runoff can infiltrate into the upper zone of the pervious sub-area, but not through the impervious sub-area. Impervious areas are themselves divided into two sub-areas, one that contains depression storage and another that does not. Runoff flow from one sub-area in a sub-catchment can be routed to the other sub-area, or both sub-areas can drain to the sub-catchment outlet.

4.5 Surface Runoff

The conceptual view of surface runoff used by SWMM is shown in the Figs. 4 and 5. Each sub catchment area, surface is treated as a non-linear reservoir. Inflow comes from the precipitation that fall on the designated upstream sub catchments. There are several outflows, including infiltration, evaporation and surface runoff. The capacity of this reservoir is the maximum interception, surface wetting and depression storage provided by ponding. Surface runoff per unit area, Q, occurs only when the depth of water in the reservoir exceeds the maximum depression storage, d_p, and the outflow is given by Manning's equation. Depth of water d over the sub catchment is continuous with time (t in sec.) can be solved the water balance equation numerically (Fig 6.).

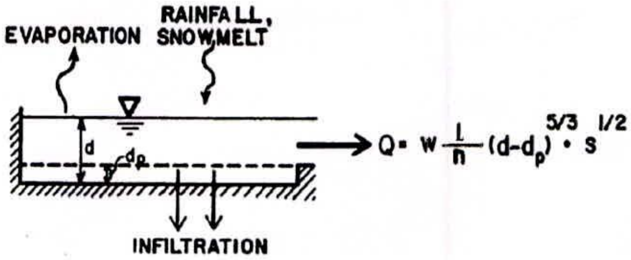

Figure 6. Conceptual view of surface runoff.

4.6 Infiltration

Infiltration is the process of entry of rainfall into the ground surface to the unsaturated soil zone of pervious sub catchments. SWMM offers three choices for modeling listed below:

4.6.1 Horton's Equation

This method is based on empirical observations showing that infiltration decreases exponentially from an initial maximum rate to some minimum rate over the course of rainfall event. Input parameters are maximum and minimum infiltration rates, a decay coefficient that describes how fast the rate decreases over time, and the time it takes a fully saturated soil to completely dry.

4.6.2 Green-Ampt Method

This method assumes that a sharp wetting front exists in the soil column, separating soil with some initial moisture content below saturated soil. The input parameters required are the initial moisture deficit of the soil, soil hydraulic conductivity, and the suction head at the wetting front.

4.6.3 Curve Number Method

This approach is adopted from the NRCS (SCS) Curve Number method for estimating runoff. It assumes that the total infiltration capacity of a soil can be found from the soil's tabulated Curve Number (Table 4).

During a rainfall event this capacity is depleted as a function of cumulative rainfall. The input parameters for this method are the curve number, the soil hydraulic conductivity and the time taken for a fully saturated soil to dry completely.

In the present study infiltration losses have been estimated by using SCS curve number method. Soil Conservation Service suggested an empirical model for rainfall abstractions which is based on the potential for the soil to absorb a certain amount of moisture. On the basis of field observations, this potential storage S (mm) was related to 'curve number' CN which is a characteristic of the soil type, land use and the initial degree of saturation known as the antecedent moisture condition. The value of S is defined by the following empirical expression.

$$S = \frac{25400}{CN} - 254 \qquad \text{(in mm)} \qquad (12$$

The effective rainfall is computed by the equation:

$$Q(t) = \frac{\left(P(t) - I_a\right)^2}{\left(P(t) + S - I_a\right)} \qquad (13)$$

Where, $Q(t)$ = accumulated depth of effective rainfall in time t, $P(t)$ = accumulated depth of rainfall in time t, I_a =initial abstraction, S =potential storage in the soil.

The original SCS method assumed the value of the initial abstraction I_a to be equal to 20% of the storage potential S.

$$I_a = 0.2S \qquad (14)$$

$$Q(t) = \frac{(P(t) - 0.2S)^2}{P(t) + 0.8S} \qquad (15)$$

4.7 Flow Routing

Flow routing within a conduit link in SWMM is governed by the Saint Venant equations based on conservation of mass and momentum for gradually varied and unsteady flow. The SWMM user has a choice on the level of sophistication used to solve any of the three options (i) Steady Flow Routing, (ii) Kinematic Wave Routing, and (iii) Dynamic Wave Routing.

4.7.1 Steady Flow Routing

Steady flow routing assumes that within each computational time step flow is uniform and steady. Thus it simply translates inflow hydrographs at the upstream end of the conduit to the downstream end, with no delay or change in shape. The Manning equation is used to relate flow rate to flow area (or depth). This type of routing cannot account for channel storage, backwater effects, entrance/exit losses, flow reversal or pressurized flow. It can only be used with dendritic conveyance network, where each node has only a single outflow link (unless he node is a divider in which two outflow links are required). This form of routing is insensitive to the time step employed and is really only appropriate only for preliminary analysis using long-term continuous simulations.

4.7.2 Kinematic Wave Routing

This routing method solves the continuity equation along with a simplified form of the momentum equation in each conduit. The latter requires that the slope of the water surface equal the slope of the conduit. The maximum flow that can be conveyed through a conduit is the full-flow Manning equation value. Any flow in excess of this entering the inlet node is either lost from the system or can pond atop the inlet node and be re-introduced into the conduit as capacity becomes available.

Kinematic wave routing allows flow and area to vary both spatially and temporally within a conduit. This can result in attenuated and delayed outflow hydrographs as inflow is routed through the channel. However this form of routing cannot account for backwater effects, entrance/exit losses, flow reversal, or pressurized flow, and is also restricted to dendritic network layouts. It can usually maintain numerical stability with moderately large time steps, on the order of 5 to 15 minutes. If the aforementioned effects are not expected to be significant then this alternative can be an accurate and efficient routing method, especially for long-term simulations.

4.7.3 Dynamic Wave Routing

Dynamic Wave routing solves the complete one-dimensional Saint Venant flow equations and gives more accurate result. These equations consist of the continuity and momentum equations for conduits and a volume continuity equation at nodes.

With this form of routing it is possible to represent pressurized flow when a closed conduit becomes full, such that flows can exceed the full-flow Manning equation value. Flooding occurs when the water depth at a node exceeds the maximum available depth, and the excess flow is either lost from the system or can pond atop the node and re-enter the drainage system. Dynamic wave routing can account for channel storage, backwater, entrance/exit losses, flow reversal, and pressurized flow. Because it couples together the solution for both water levels at nodes and flow in conduits it can be applied to any general network, even those containing multiple downstream diversions and loops. It is the method of choice for systems subjected to significant backwater effects due to downstream flow restrictions and with flow regulation via weirs and orifices.

4.8 Governing Equations

SWMM solve the conservation of mass and momentum equations that govern the unsteady flow of water through a drainage network of channels and pipes. These equations, known as the Saint Venant equations, can be expressed in the following form for flow along an individual conduit.

$$\frac{\partial Q}{\partial t} + \frac{\partial Q}{\partial x} = 0 \qquad \text{Continuity} \qquad (16)$$

$$\frac{\partial Q}{\partial t} + \frac{\partial (Q^2 / A)}{\partial x} + gA\frac{\partial H}{\partial x} + gAS_f + gAh_L = 0 \qquad \text{Momentum} \qquad (17)$$

Where S_f is the friction slope (head loss per unit length), h_L is the local energy loss per unit length of conduit, and g is the acceleration of gravity. Note that for a known cross sectional geometry, the area A is a known function of flow depth y, which in turn can be obtained from the head H. Thus the dependent variables in these equations are flow rate Q and head H, which are functions of distance x and time t. The friction slope S_f can be expressed in terms of the Manning equation as

$$S_f = \frac{n^2 V|V|}{k^2 R^{4/3}} \tag{18}$$

Where n is the Manning roughness coefficient, V is the flow velocity (equal to the flow rate Q divided by the cross-sectional area A), R is the hydraulic radius of the flow's cross section, and $k = 1.49$ for US units or 1.0 for metric units. The local loss term h_L can be expressed as

$$h_L = \frac{KV^2}{2gL} \tag{19}$$

where K is a local loss coefficient at location x and L is the conduit length.

To solve Eqn (16) and (17) over a single conduit, one needs a set of initial conditions for H and Q at time, t=0 as well as boundary conditions at $x = 0$ and $x = L$ for all times t. When analyzing a network of conduits, an additional continuity relationship is needed for the junction nodes that connect two or more conduits together (Fig. 7). In SWMM a continuous water surface is assumed to exist between the water elevation at the node and in the conduits that enter and leave the node (with the exception of free fall drop). The change in hydraulic head H at the node with respect to time can be expressed as:

$$\frac{\partial H}{\partial t} = \frac{\sum Q}{A_{store} + \sum As} \tag{20}$$

where A_{store} is the surface area of the node itself, $\sum As$ is the surface area contributed by the conduits connected to the node, and $\sum Q$ is the net flow into the node (inflow – outflow) contributed by all conduits connected to the node as well as any externally imposed inflows. Note that the flow depth at the end of a conduit connected to a node can be computed as the difference between the head at the node and the invert elevation of the conduit.

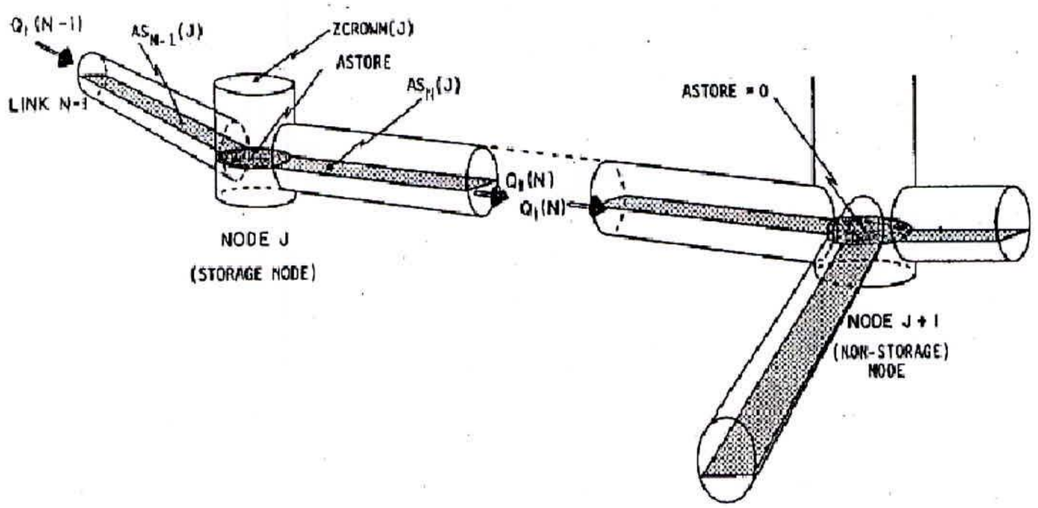

Figure 7: Node-Link Representation of a Drainage System in SWMM.

Eqn (16), (17) and (19) are solved in SWMM by converting them into an explicit set of finite difference formulas that compute the flow in each conduit and head at each node for time $t + \Delta t$ as functions of known values at time t. The equation solved for the flow in each conduit

$$Q_{t+\Delta t} = \frac{Q_t + \Delta Q_{gravity} + \Delta Q_{inertial}}{1 + \Delta Q_{friction} + \Delta Q_{losses}} \tag{21}$$

The individual ΔQ terms have been named for the type of force they represent and are given by the following expressions:

$$\Delta Q_{gravity} = g\overline{A}(H_1 - H_2)\Delta t / L \tag{22}$$

$$\Delta Q_{inertial} = 2\overline{V}(\overline{A} - A_t) + 2\overline{V}^2(A_2 - A_t)\Delta t / L \tag{23}$$

$$\Delta Q_{friction} = \frac{gn^2|\overline{V}|\Delta t}{k^2\overline{R}^{4/3}} \tag{24}$$

$$\Delta Q_{losses} = \frac{\sum_i^i K_i |V_i| \Delta t}{2L} \tag{25}$$

Where, $\overline{A}$ = average cross-sectional flow area in the conduit, $\overline{R}$ = average hydraulic radius in the conduit, $\overline{V}$ = average flow velocity in the conduit, V_i = local flow velocity at location 'i' along the conduit, K_i = local loss coefficient at location i along the conduit, H_1 = head at upstream node of conduit, H_2 = head at downstream node of conduit, A_1 = cross-sectional area at the upstream end of the conduit, and A_2 = cross-sectional area at the downstream end of the conduit.

The equation solved for the head at each node is:

$$H_{t+\Delta t} = H_1 + \frac{\Delta Vol}{\left(A_{store+\sum As}\right)_{t+\Delta t}} \tag{26}$$

Where, ΔVol is the net volume flowing through the node over the time step as given by

$$\Delta Vol = 0.5\left[\left(\sum Q\right)_t + \left(\sum Q\right)_{t+\Delta t}\right]\Delta t \tag{27}$$

4.9 Channel Capacity

Each reach of channel is assumed to be prismatic, *i.e* of constant cross-section and slope. As long as the channel flow has a free surface, the flow in each reach is assumed to be quasi-uniform, neglecting the variation of flow with time. For this condition the friction slope S_f and the water surface are assumed to be parallel to the bed slope S_0. The resistance is represented by the Manning equation to express the relationship between flow rate (Q), cross sectional area (A), hydraulics radius (R) and slope (S) in open channel and partially fully closed conduits

$$Q = \frac{1}{n} A R^{2/3} \sqrt{S} \tag{28}$$

Where n is the Manning roughness coefficient. For steady flow and kinematic wave routing, S is conduit slope. For dynamic wave flow routing it is friction slope S_f.

5.0 RESULTS AND DISCUSSION

5.1 Data collection tools and methods

The nearest available Indian Meteorological Department (IMD) rain gauge is at Nungambakkam and its hourly rainfall has been obtained from IMD, Chennai for a period of thirty years (1980 to 2009). Due to non-availability of raingauge in the study area, five tipping bucket rain gauges and two automatic water level recorders have been installed in the study area to collect short interval rainfall and water levels. The date of installation of equipments and its data availability is given in the Table 5 and their locations are shown in Figure 8.

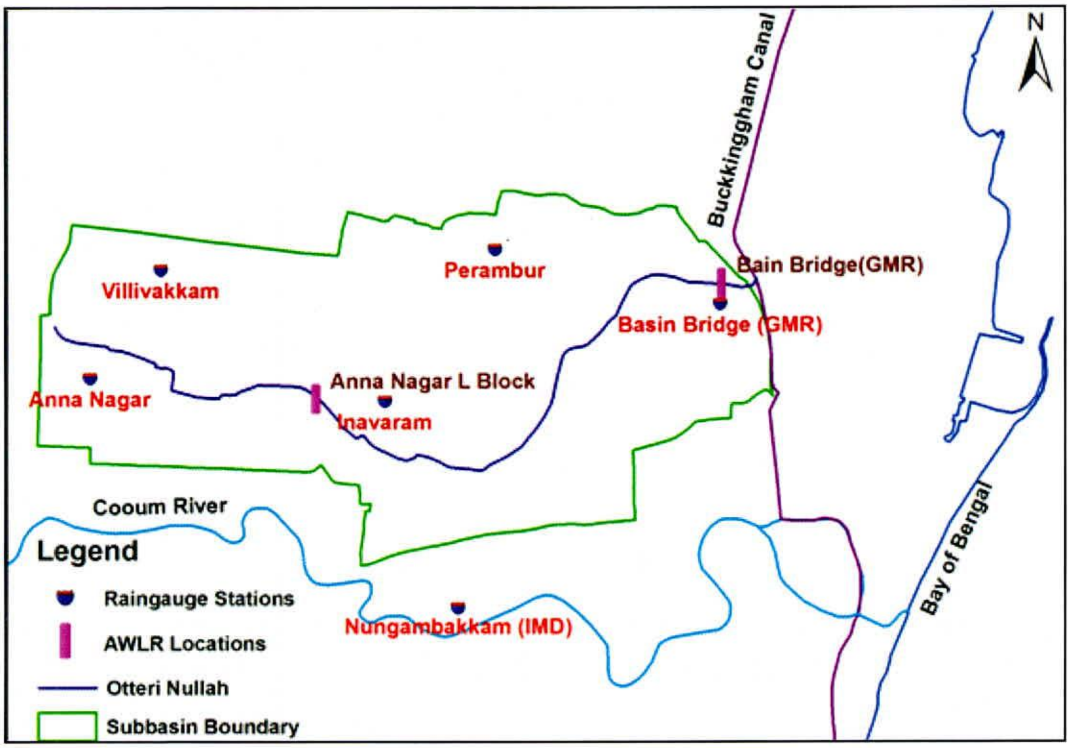

Figure 8. Location of Tipping bucket rain gauges and AWLR's in the study area

The collected data has been processed and analyzed. The average sub basin rainfall has been calculated using Thiessen polygon method and the influence area of each raingauge station is shown in Figure 9. Thiessen weights of Anna Nagar, Villivakkam, Inavaram, Perambur and Basin Bridge rain gauges are 0.134, 0.134, 0.348, 0.188 and 0.197 respectively.

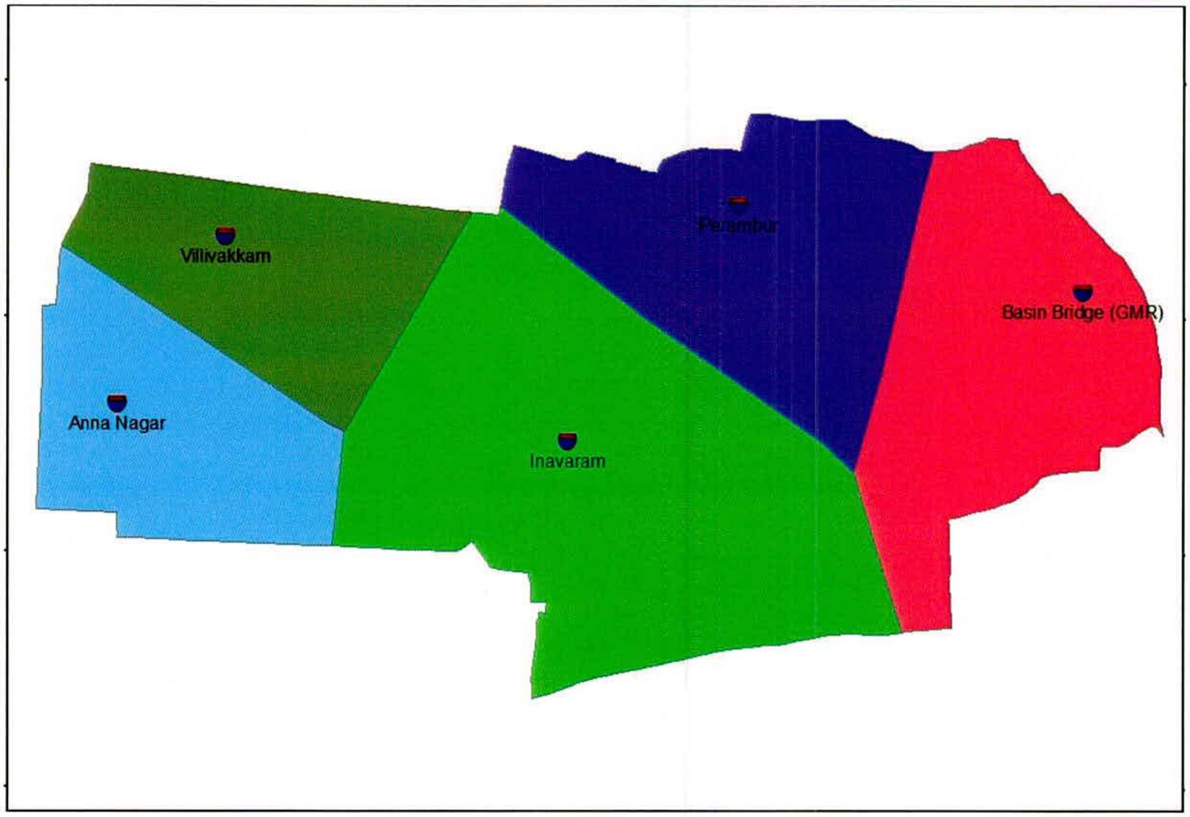

Figure 9. Map showing influence of each raingauge using Thiessen Polygon method

The responses of the average rainfall to observed water levels in Otteri Nullah at Anna Nagar and Basin Bridge within the study area are shown in Figures 10 and 11 respectively. DEM of the study area is prepared using spot heights obtained from Differential Global Positioning System (DGPS) and SOI topographical maps. The DEM of the study area and delineated drainage pattern from DEM are shown in Figure 12. This DEM drainage pattern and artificial storm water drainage network is different from each other. The maximum and minimum elevations found from the DEM in the study area are +10.97 and +2.27 m amsl respectively. The cross sectional details of Otteri Nullah drain at every 30 m interval, storm water drainage network details and its cross sections and bed levels were obtained from Chennai Corporation.

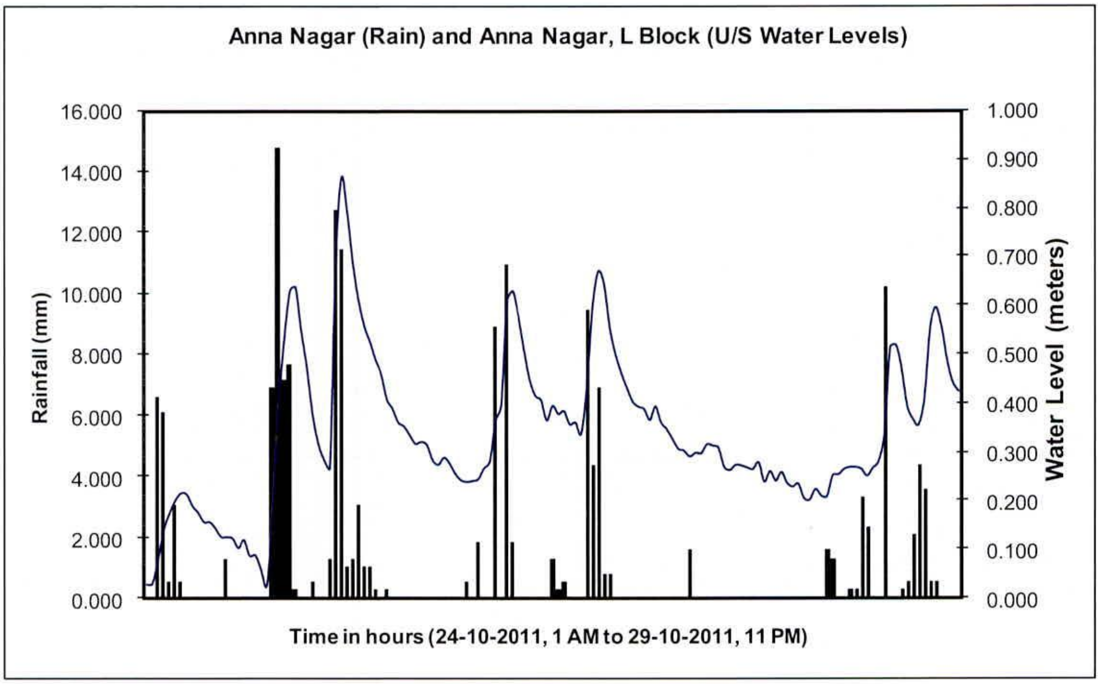

Figure 10. Hourly rainfall and its corresponding water level at Anna Nagar.

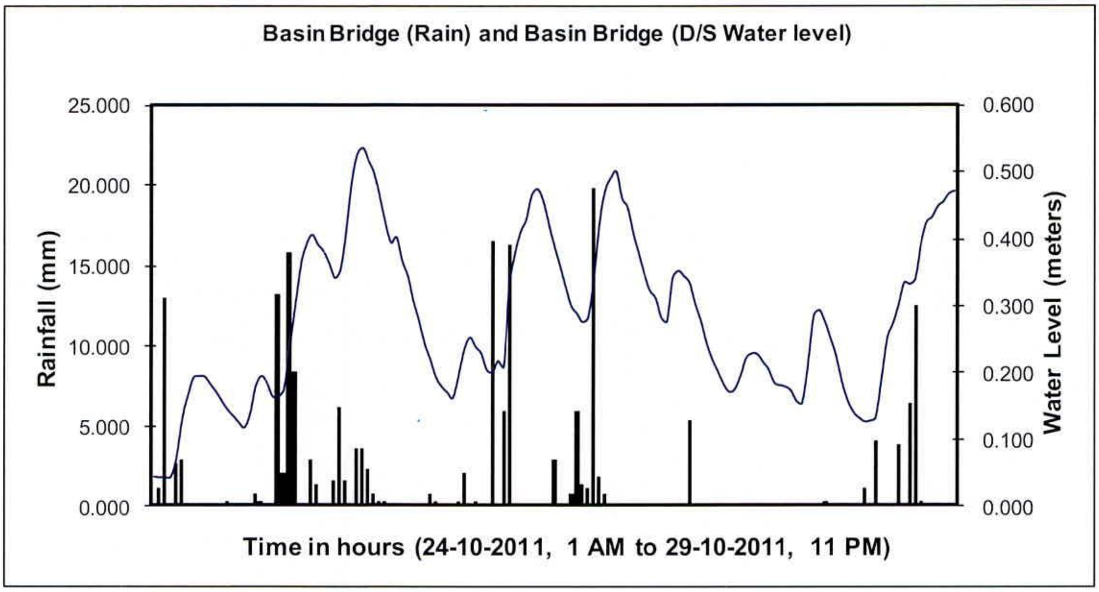

Figure 11. Hourly rainfall and its corresponding water level at Basin Bridge

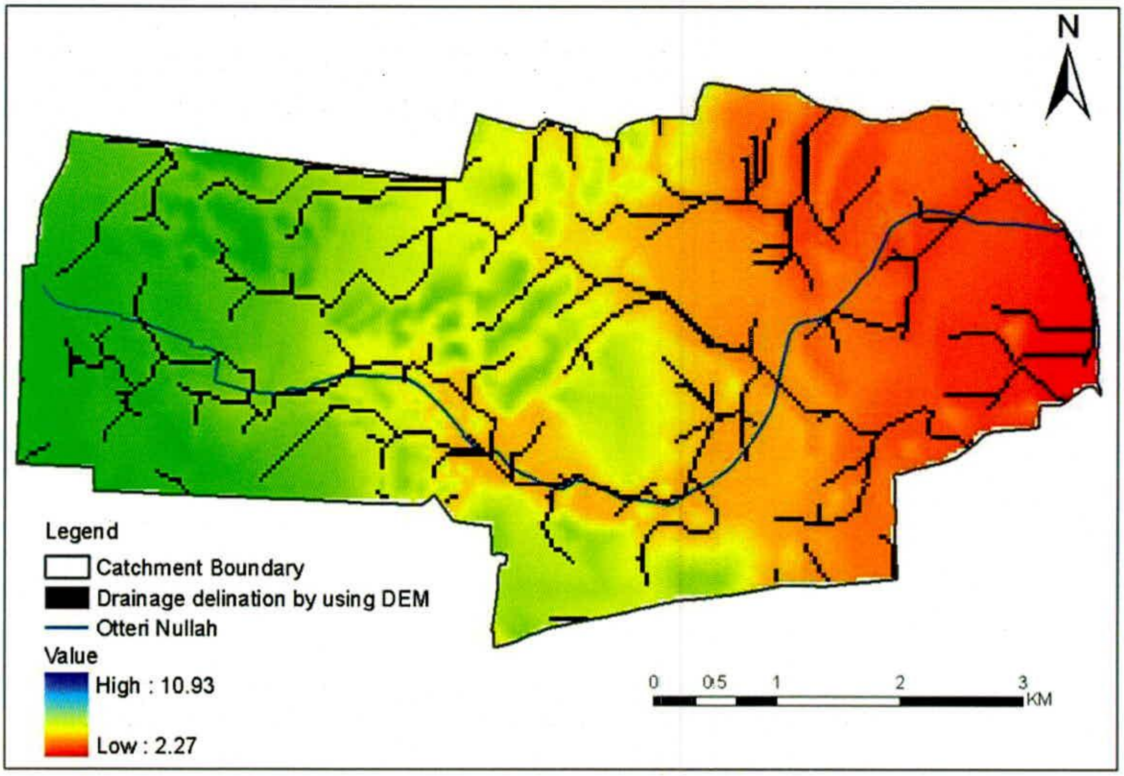

Figure 12. Digital Elevation Model (SOI Map+DGPS) and drainage pattern from DEM

5.2 Historical Rainfall Data Processing and Analysis

The analysis of 30 years (1980 to 2009) hourly rainfall data at Nungambakkam (of IMD) was carried out and Intensity Duration Frequency (IDF) curves were prepared using EV1 distribution. The highest annual rainfall observed during this period is 2489 mm in the year 2005. The maximum daily rainfall observed during this period is 394 mm on 27^{th} October 2005. The maximum number of rainy days observed is 83 in the year 1997. The Intensity Duration Frequency (IDF) curves prepared for Nugambakkam raingauge stations are given in Figure 13. The design storm of 24 hours with a return period of 2, 5, 10, and 25 years are given in Figure 14. The maximum hourly rainfall for 2, 5, 10 and 25 years return periods are 48.89, 64.10, 74.08 and 87.24 mm respectively. The observed maximum hourly rainfall at Nungambakkam raingauge station during the period 1980-2009 is shown Figure 15.

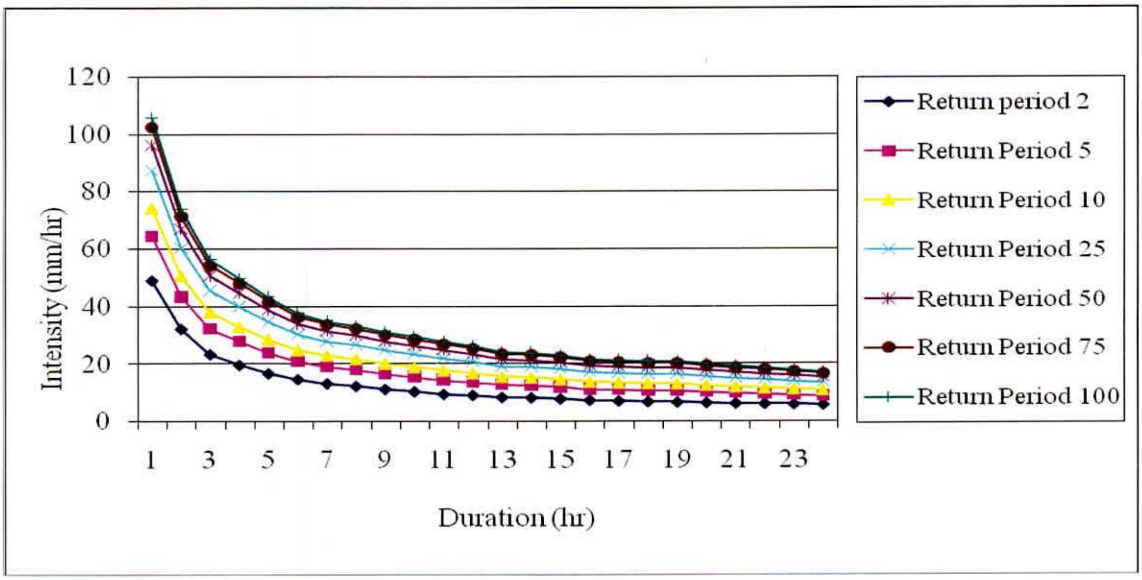

Figure 13. Intensity Duration Frequency (IDF) curves for Nungambakkam Raingauge station (IMD)

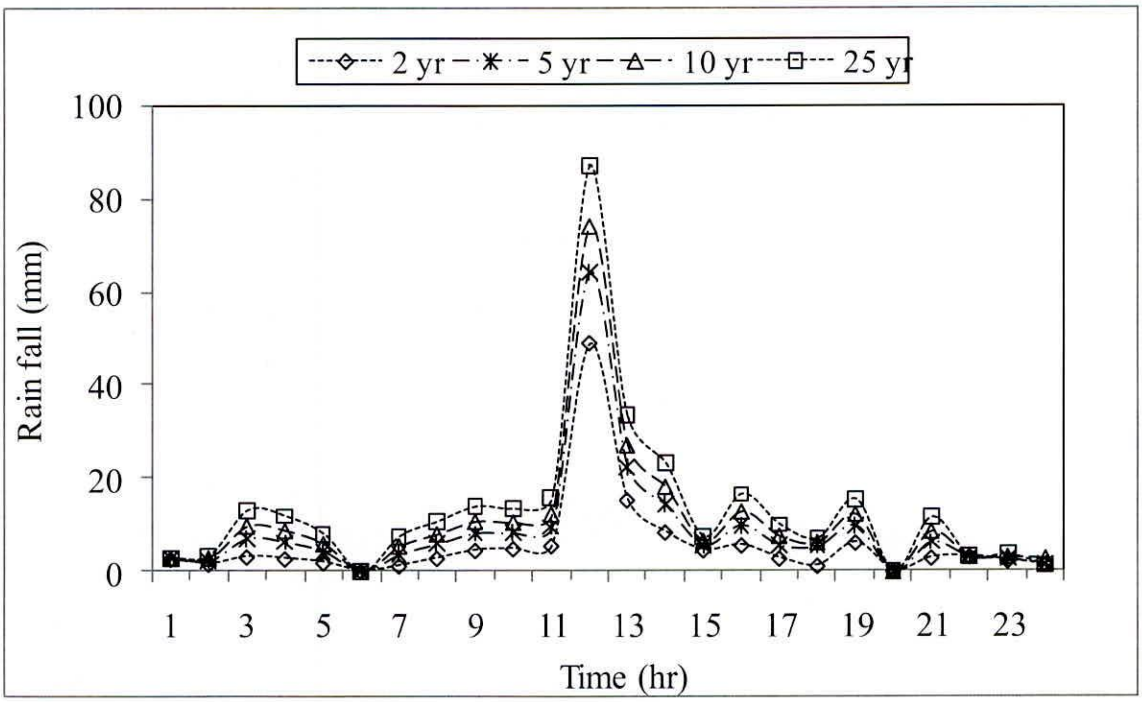

Figure 14. Design storms for various return periods in the study area

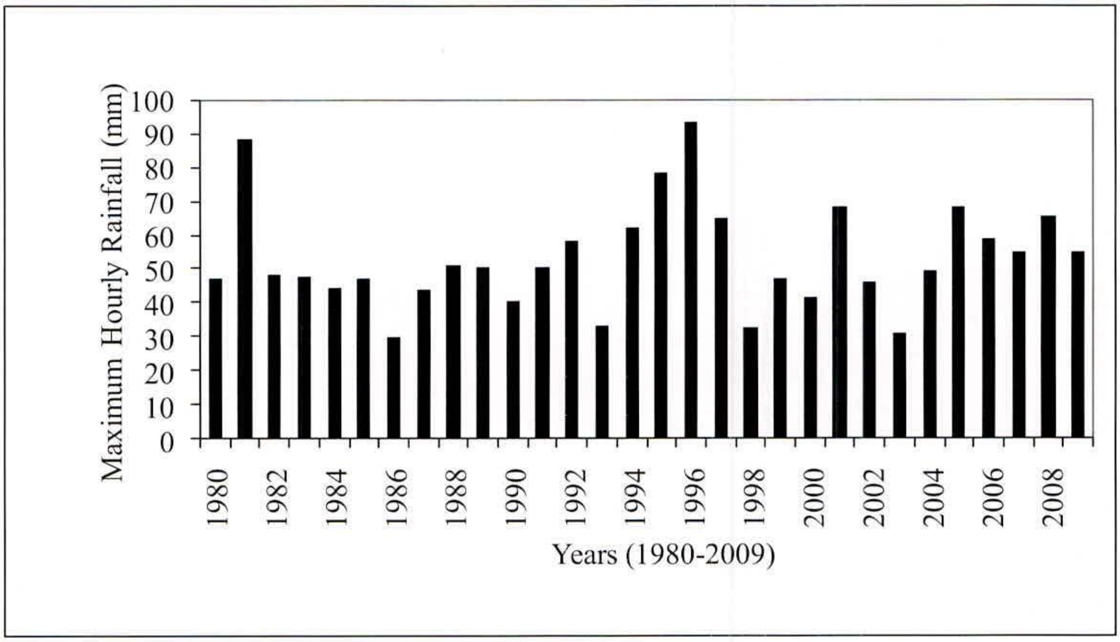

Figure 15. Maximum Hourly rainfall at Nungambakkam raingauge station (IMD)

5.3 Spatial analysis of observed rainfall within the study area

The spatial analysis of rainfall data observed in the study area through five tipping bucket rain gauges indicates that there is a significant spatial variation of rainfall found in the study area (Table 6). The present spatial variation of rainfall may be less during cyclonic storms and monsoon periods.

5.4 Model Setup

The study area has been descritised in the form of micro watersheds, nodes and conduits for setting up SWMM model and these details are given in Figure 16. Total 121 nodes, 120 conduits and 88 micro watersheds have been delineated in the study area using storm water drainage network and DEM. Among 121 nodes, 29 nodes are located on Otteri Nullah drain and rest of the nodes (92) is marked on storm water drains. Total 88 micro watersheds are connected to 85 nodes in the study area. Among 88 micro watersheds 52 micro watersheds are above Anna Nagar gauging station and rest of the micro watersheds are above Basin Bridge gauging station. The catchment area above Anna Nagar gauging station is 6.2 sq km where as the total catchment area of Otteri Nullah is around 30.63 sq km. The major land use and land cover map of the study area has been prepared from IRS P6 L4Mx (3-3-2008) satellite data and the same is shown in Figure 17. The total sub basin is having 67% of impervious and 33% of pervious area.

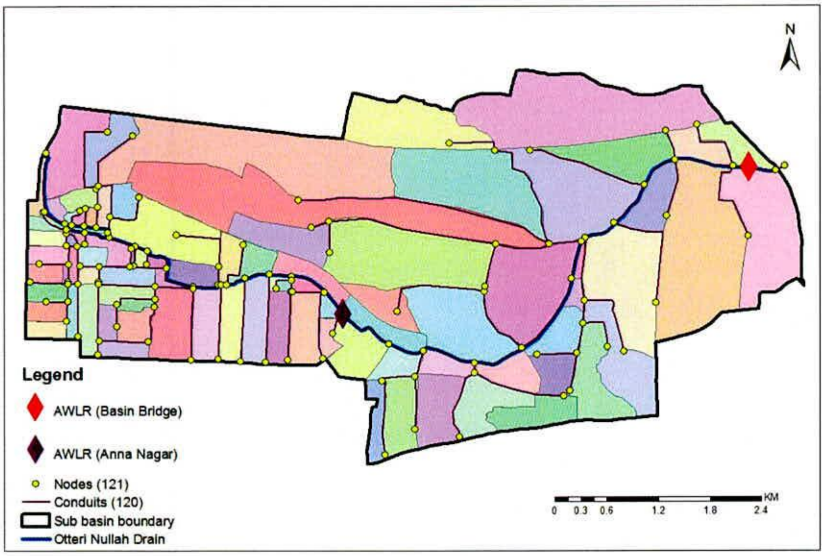

Figure 16. Micro watersheds delineation in the study area

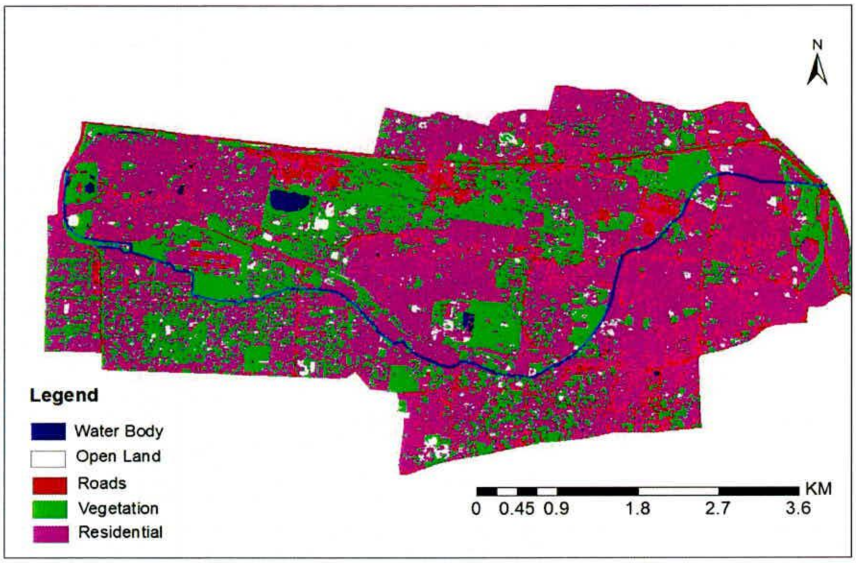

Figure 17. Study area land use classification

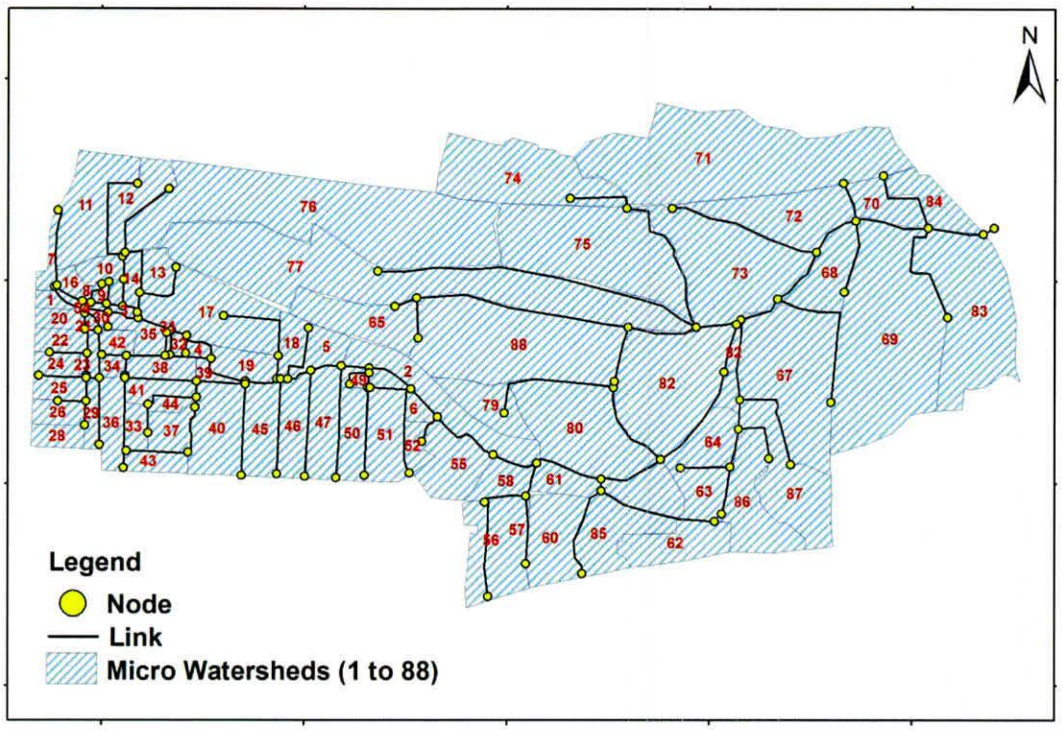

Figure 18. Delineated micro watersheds of the study area in XP-SWMM

Total length of storm water drains considered in the study area is 58,871 m. The total length of major Otteri Nullah is around 10.7 Kms. There are only three major soil types observed in the study area. They are sandy (Group A), clay (Group D) and sandy clay (Group D), and occupying 13.32%, 41.62% and 45.06% areas respectively in the study area. These soils are considered as Group A and D as per the SCS Hydrological soil groups. The major land uses observed in the study area are water body, open land, roads, vegetation and residential and their percentage of distribution is 0.6, 3.3, 10.9, 28.9 and 56.3 respectively. The range of computed curves number is 61 to 84 for 88 micro watersheds in the study area. The catchments characteristics considered in the XP-SWMM model and runoff coefficients obtained in the each micro watershed for two year return period of 24 hr design storm is given in Table 7. The range of micro watershed areas, % of impervious area, slope and runoff coefficients are 1.2-185 ha, 15-96, 0.001-0.008 and 0.39-0.96 respectively. The average runoff coefficient obtained for 2-year return period of 24 hrs design storm is found to be 0.74. The link details and storm water drainage connectivity considered in XP-SWMM model and computed flow velocities in each link for 2 yr return period of 24 hrs design storm are given in Table 8. The storm water drain length (Link) range in the model varies between 23-2,942 m. The range of discharge and velocity found to be 0.16-28 cumec and 0.64-4.8 m/s respectively for 2-year return period of 24 hrs design storm. The details of micro watershed identification and coding of Nodes and links in SWMM model has been given in Figures 18, 19 and 20 respectively.

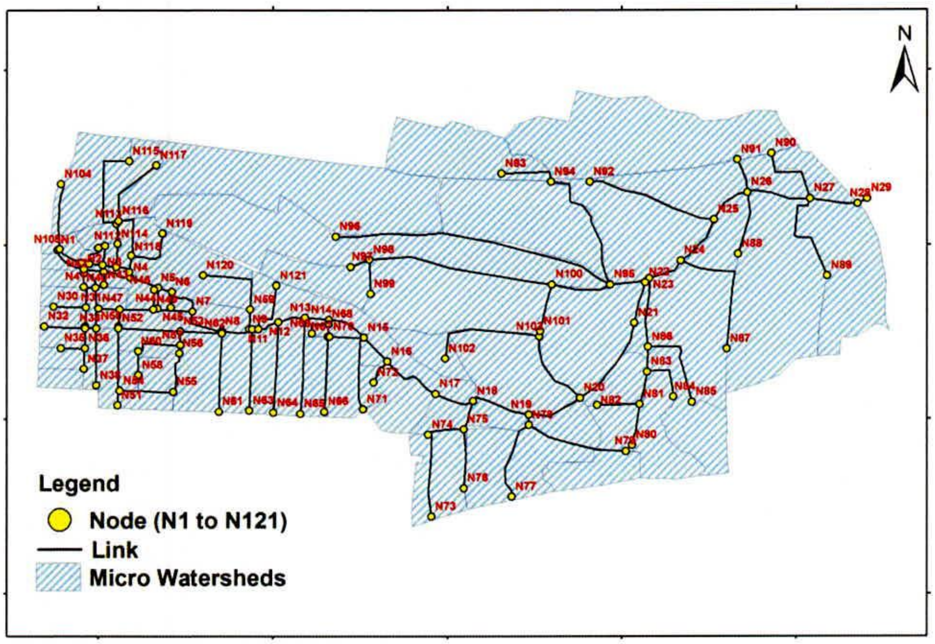

Figure 19. Connectivity of micro watersheds with nodes in XP-SWMM

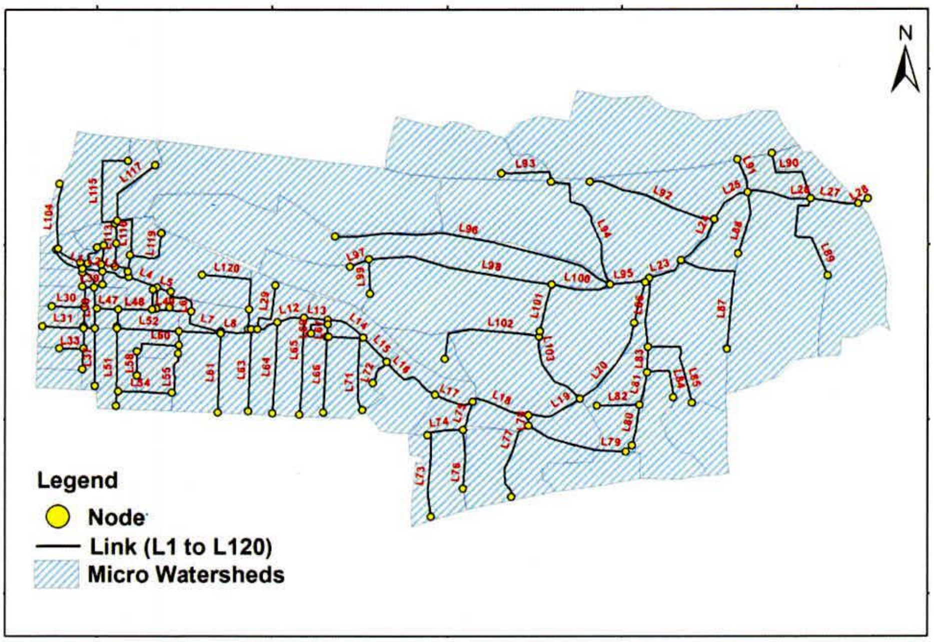

Figure 20. Details of links considered in the XP-SWMM

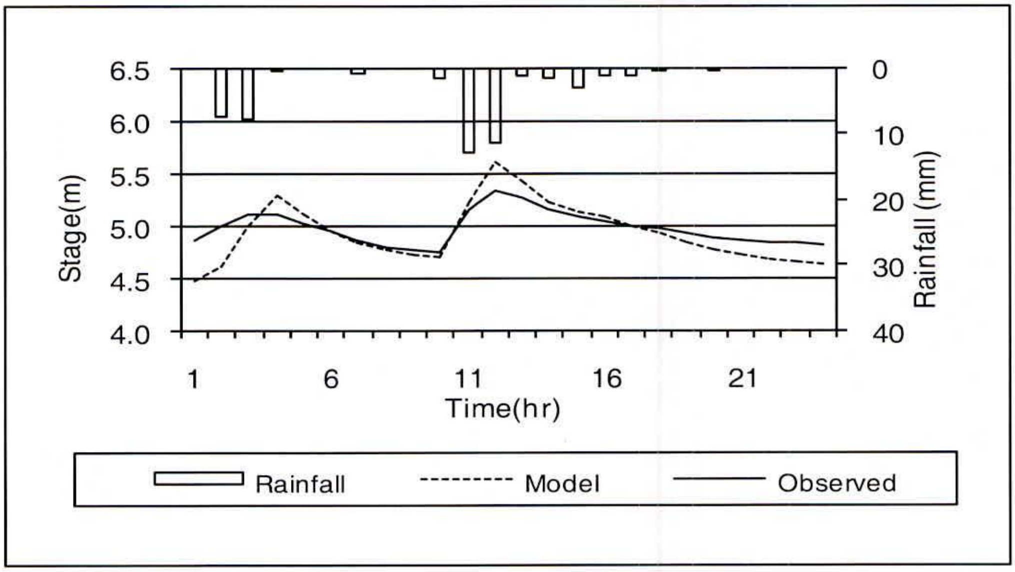

Figure 21. Comparison between observed and modeled stage at Anna Nagar (against rainfall event on 25 Oct. 2011)1

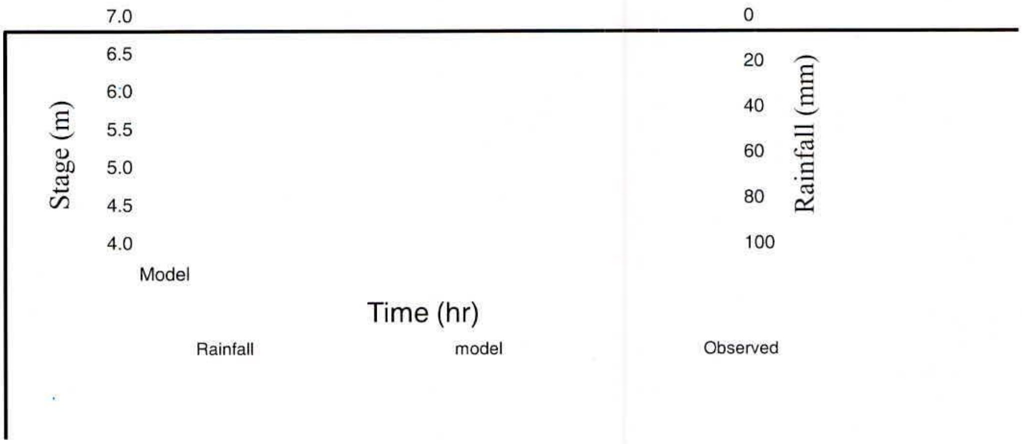

Figure 22. Comparison between observed and modeled stage at Anna Nagar (against Rainfall event on 4 Nov. 2011)

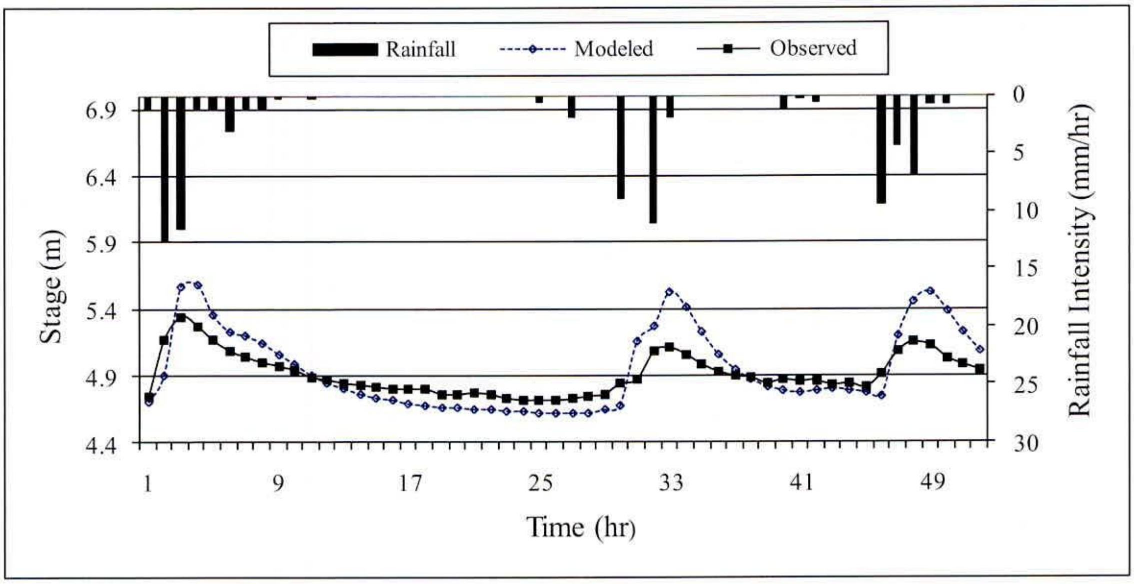

Figure 23. Comparison between observed and simulated stage at Anna Nagar

(25-10-2011, 9.00 to 27-10-2011, 12.00)

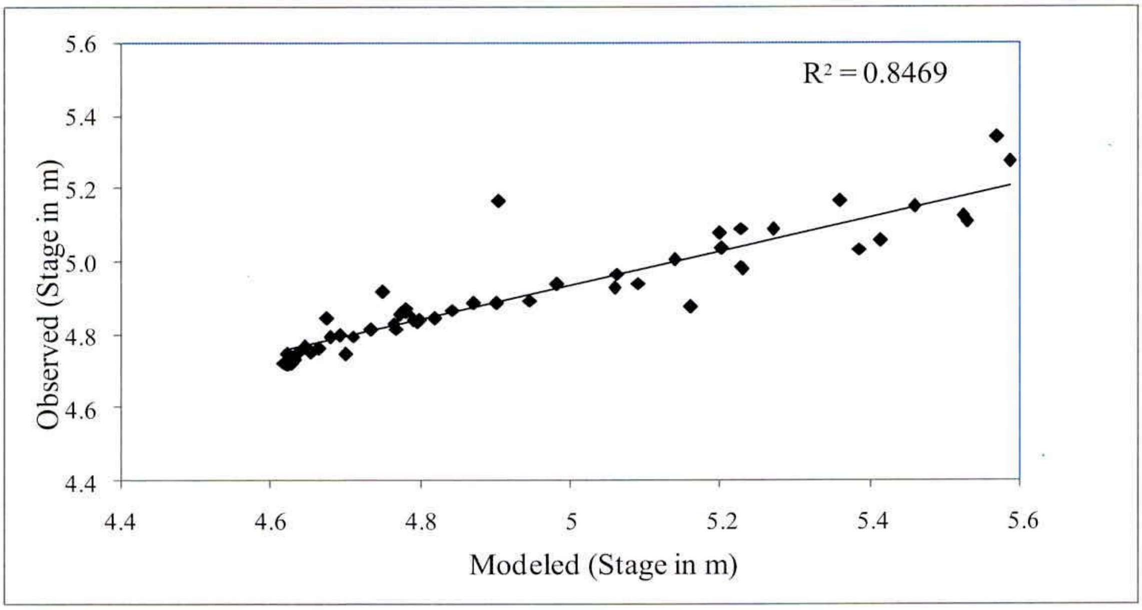

Figure 24. Correlation between observed and simulated stage at Anna Nagar

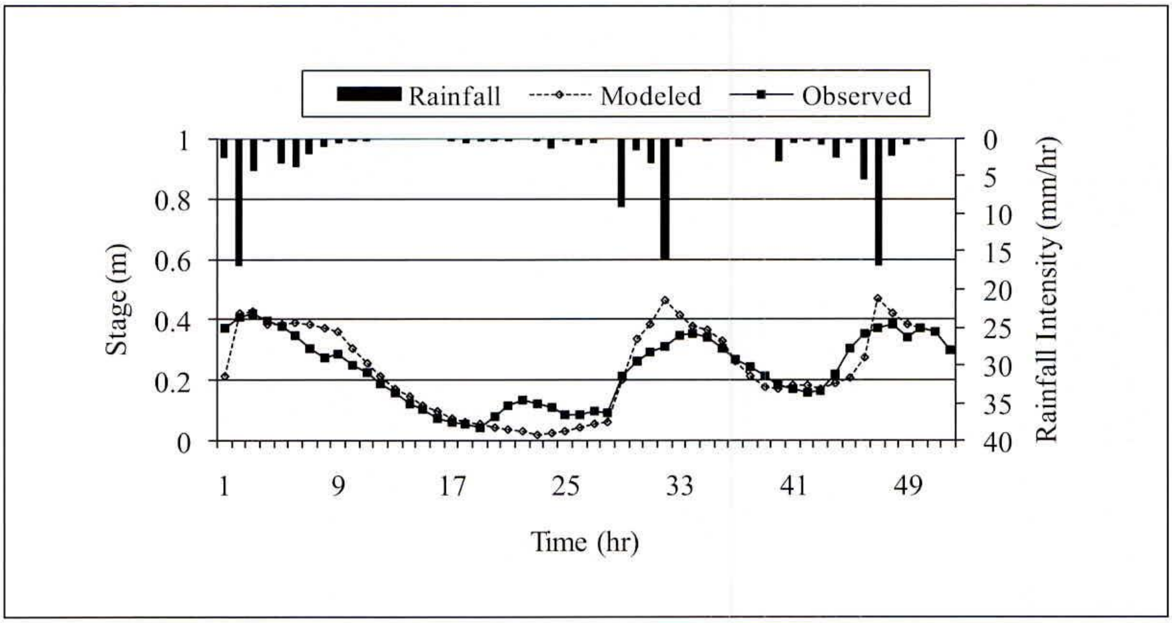

Figure 25. Comparison between observed and simulated stage at Basin Bridge (25-10-2011, 9.00 to 27-10-2011, 12.00 hrs)

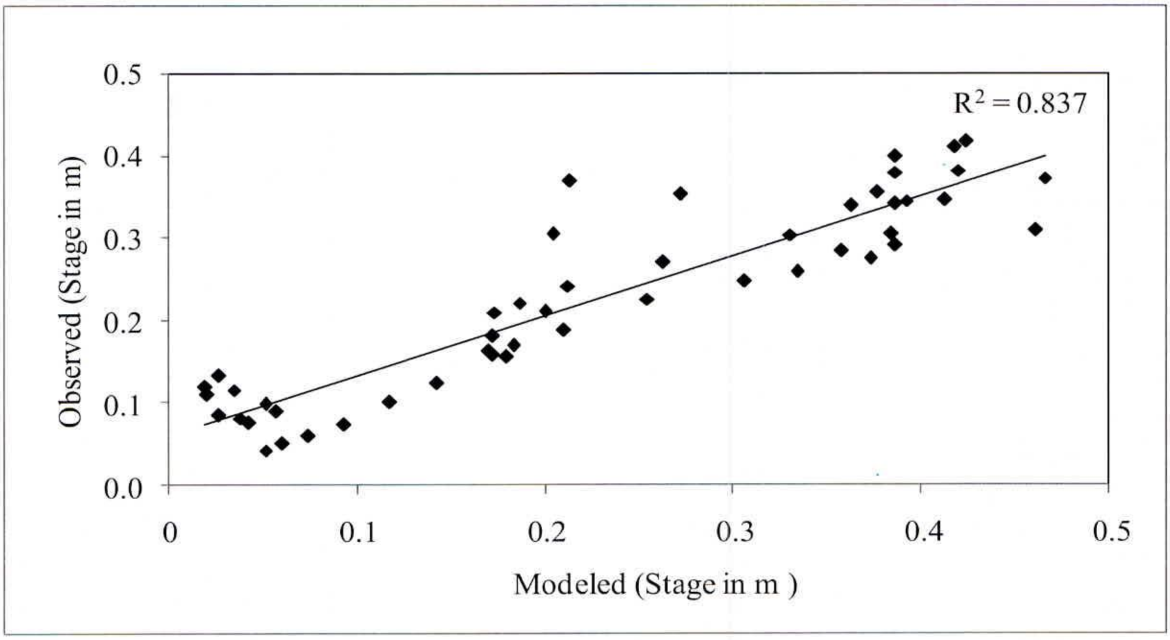

Figure 26. Correlation between observed and simulated stage at Basin Bridge

In order to test SWMM with field input parameters, two events (25th October 2011 and 4th November 2011) of observed hourly rainfall and water level data have been considered for model performance at Anna Nagar (Figures 21 and 22). Similarly continuous observed hourly rainfall during 25-27 October 2011 is also considered to test SWMM performance at Anna Nagar (Figure 23) and Basin Bridge (Figure 25). Analysis indicated that the model predicted water levels for continuous events at Anna Nagar and Basin Bridge are in good agreement with observed water levels (R^2 = 0.85 and 0.84 shown in Figures 24 and 26 respectively). It was observed that the peaks of observed stages are always less than the modeled stages. This is probably due to blocking of drains with garbage, floating material and improper interconnectivities between storm water drains. Improper maintenance of storm water drains causes the reduction of flow in the drains and the same process is reflected in the comparison of simulated and observed stages in the study area.

After successful testing of the XP-SWMM model, the storm water drainage network of Otteri Nullah sub basin was evaluated for different return period design storms computed from IDF curves for the basin (Figure 13). The design storms for 2, 5, 10 and 25 years return periods (Figure 14) were considered to evaluate present storm water drainage network and with existing longitudinal profile of Otteri Nullah. The design hyetographs and its corresponding hydrographs for 2, 5, 10 and 25 years return periods at basin outfall are shown in Figure 27. The flood peaks at basin outfall for 24 hrs design storm of 2, 5, 10 and 25 return periods are 27.57, 33.80, 37.58 and 42.37 cumec respectively. Further plan view of storm water drains and Otteri Nullah in the form of nodes and its corresponding flooding locations in the study area for 2-year return period design storm are shown in Figure 28. The existing longitudinal profile of Otteri Nullah with flooding locations is shown in Figure 29. The application of model with different return period storms indicated that the present networks of storm water drains with existing longitudinal profile of Otteri Nullah are not adequate even to drain off the runoff generated from the sub basin for 2-years return period storm.

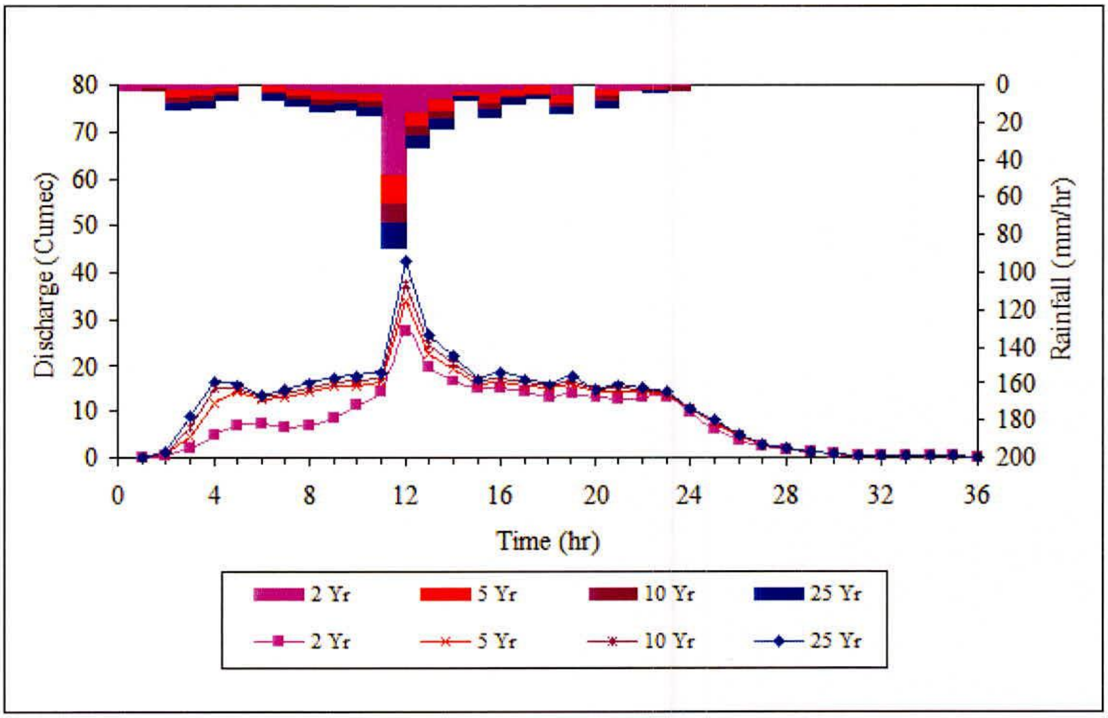

Figure 27. 24hr design hyetographs and its corresponding hydrographs at sub basin outfall

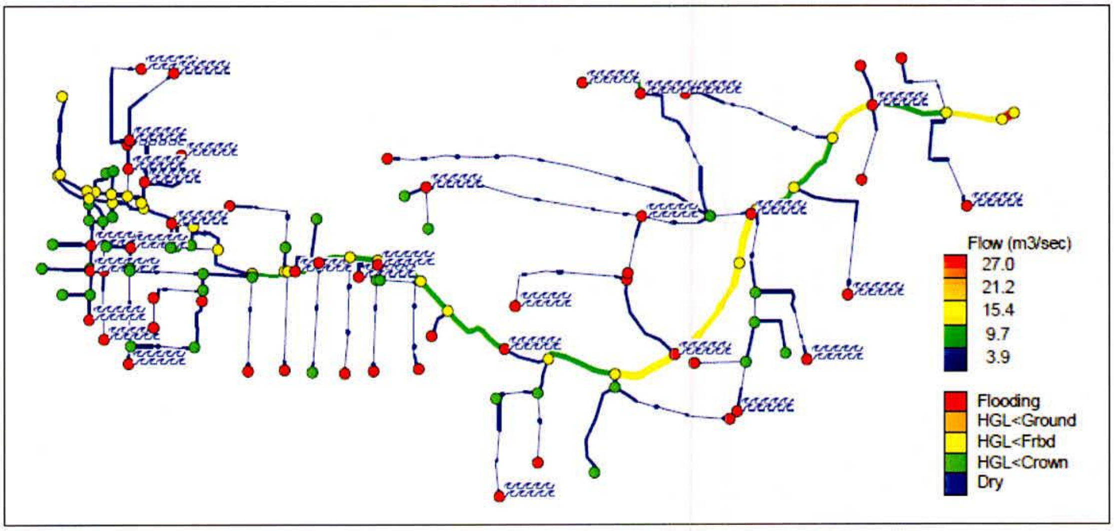

Figure 28. Plan view of flooding nodes in the study area for 2 yrs return period design storm

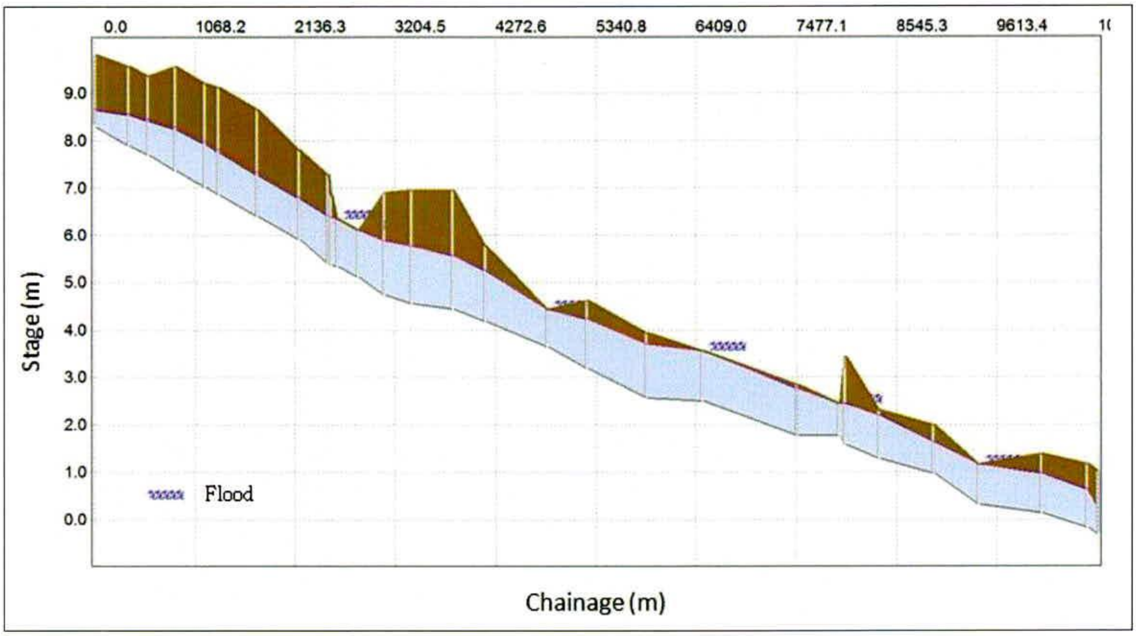

Figure 29. 2-years return period storm water surface profile with existing Longitudinal profile of Otteri Nullah.

5.5 Development of Scenarios as per PWD modifications

5.5.1 Impact of proposed longitudinal profile of Otteri Nullah

The Public Works Department (PWD), Chennai region has proposed to modify the longitudinal profile of Otteri Nullah drain as a part of flood mitigation measures in the sub basin. The proposed longitudinal profile and revised cross sections are incorporated in the model and tested the adequacy for storm water drainage network for different return period of storms. The design hyetographs and its corresponding hydrographs with proposed longitudinal profile are shown in Figure 30. The flood peaks at basin outfall for 24 hrs design storm of 2, 5, 10 and 25 return periods are 52, 61, 69 and 75 cumec respectively. Further, plan view of storm water drains and Otteri Nullah in the form of nodes and its corresponding flooding locations in the study area are shown in Figure 31. The maximum flow found in storm water drainage network is 51.2 cumec in the study area. The computed water surface profile with proposed longitudinal profile of Otteri Nullah with flooding locations is shown in Figure 32. It was observed that Otteri Nullah is capable of draining two years return period storm. Further, the Otteri Nullah with proposed longitudinal profile is also tested for 5-year return period storm and found that it is adequate to drain off storm water except at two nodes (Figure 33).

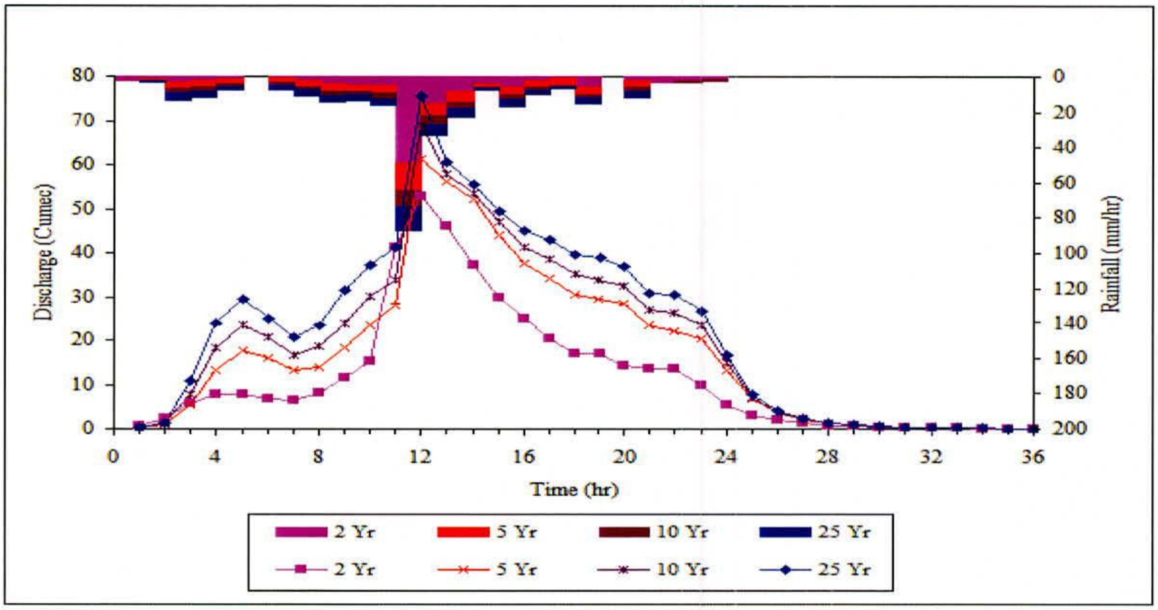

Figure 30. 24hr design hyetographs and its corresponding hydrographs at sub basin outfall with proposed longitudinal profile.

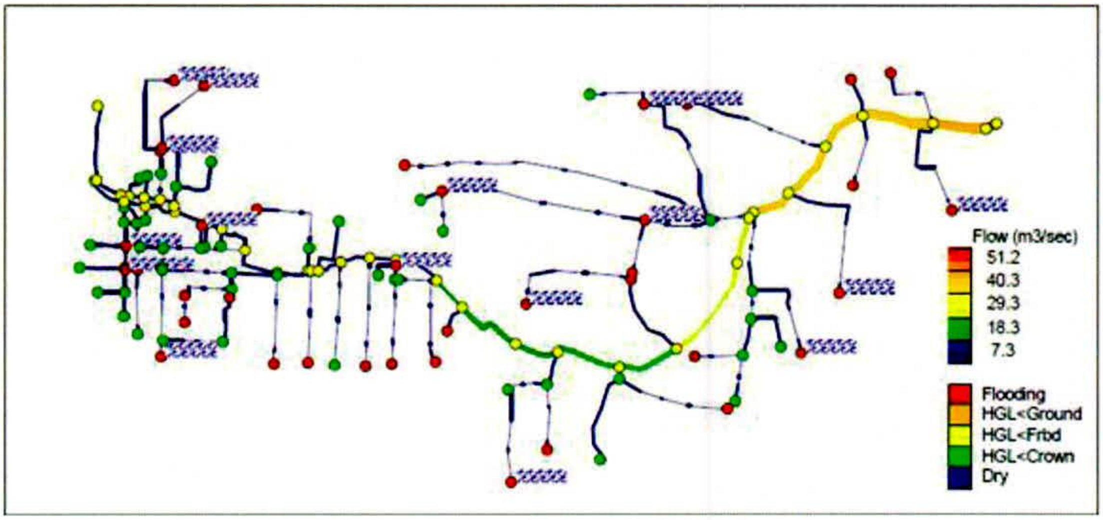

Figure 31. Plan view of flooding nodes in the study area for 2 yrs return period storm with proposed longitudinal profile of Otteri Nullah

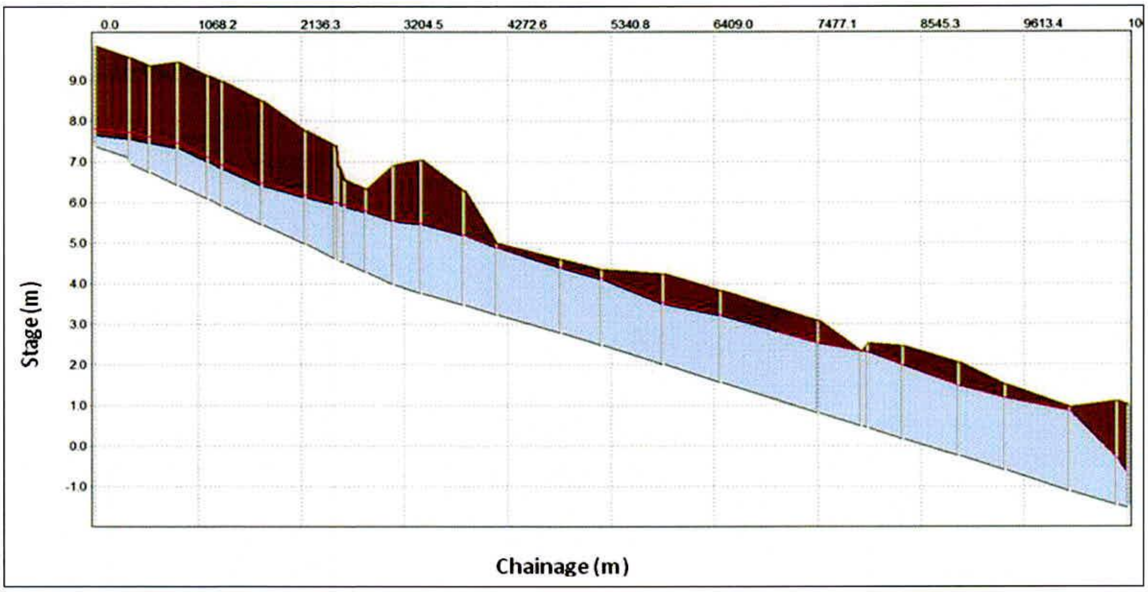

Figure 32. 2-years return period storm water surface profile with proposed longitudinal profile of Otteri Nullah

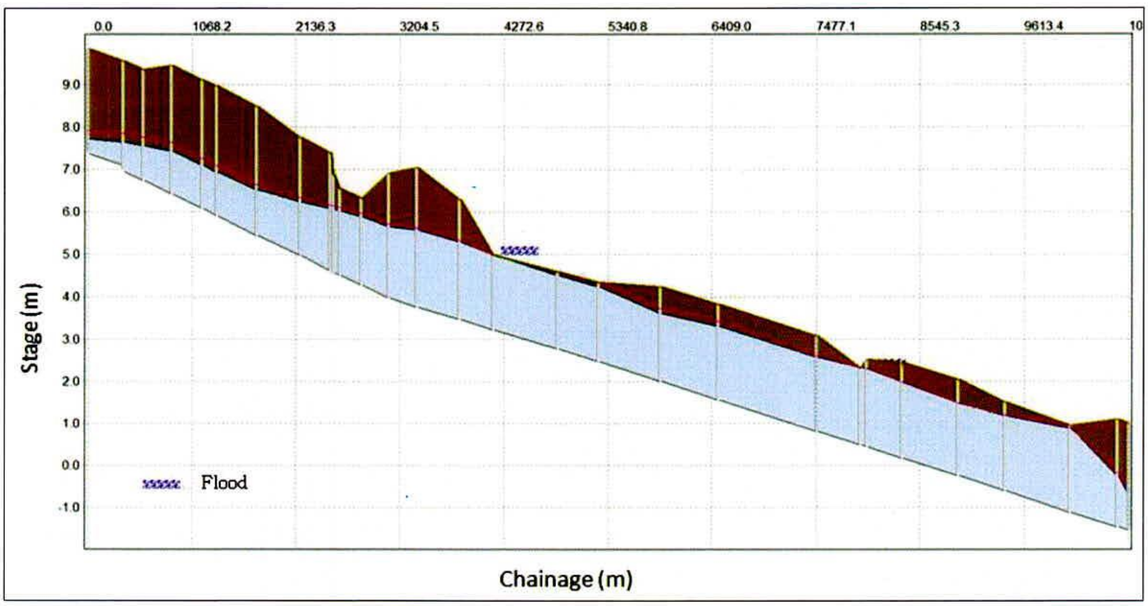

Figure 33. 5-years return period storm water surface profile with proposed longitudinal profile of Otteri Nullah

The flood peaks, system inflow, outflow volumes and percentage of error of the model at basin outfall with existing and proposed longitudinal profile of Otteri Nullah are given in Table 9. The minimum error indicates that the inflow volume and generated runoff volume are similar. The comparison between outfall hydrographs for various return period storms indicated that only 5-years return period flood peak is passing at basin outfall with proposed longitudinal profile and hydrographs more than five years return periods indicate flooding in the sub basin.

5.5.2 Impact of Flood Water Diversion

The PWD has proposed to divert floodwater from Otteri Nullah to Cooum river. The location flood diversion channel is shown in Figure 34. This proposed channel cross section is incorporated in the flow model and hydrographs have been computed at Diversion Channel with and without diversion. The computed hydrographs for 2-years return period of 24 hrs design storm are given in Figure 35. It was observed that reduction in peak flow is 38% after implementing diversion channel.

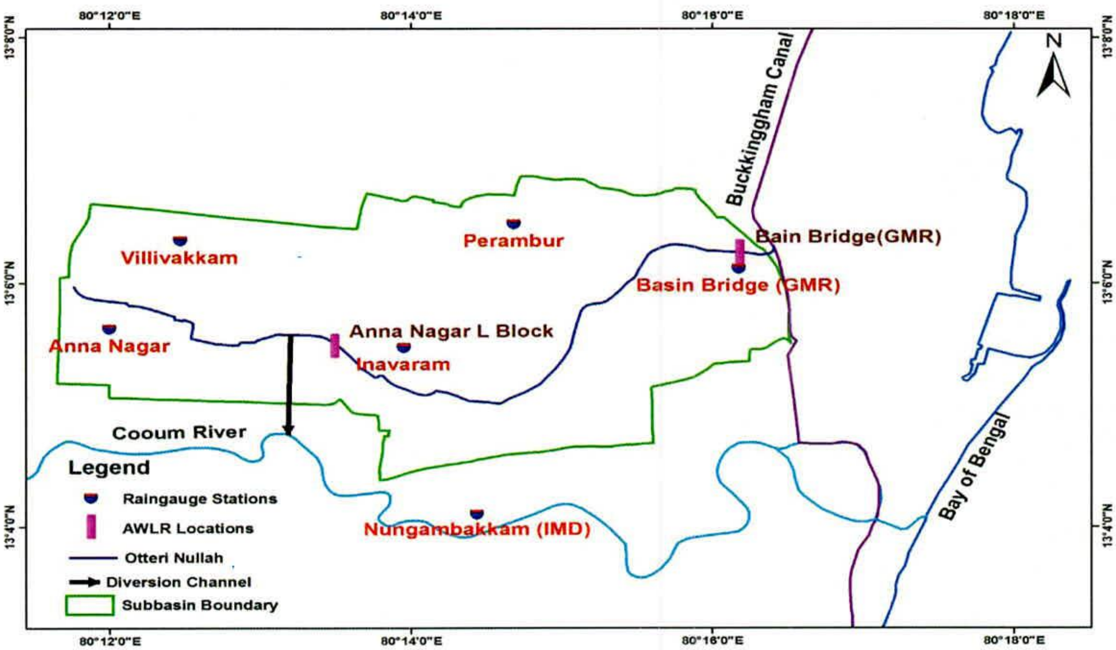

Figure 34. Location of proposed diversion channel by PWD in the study area

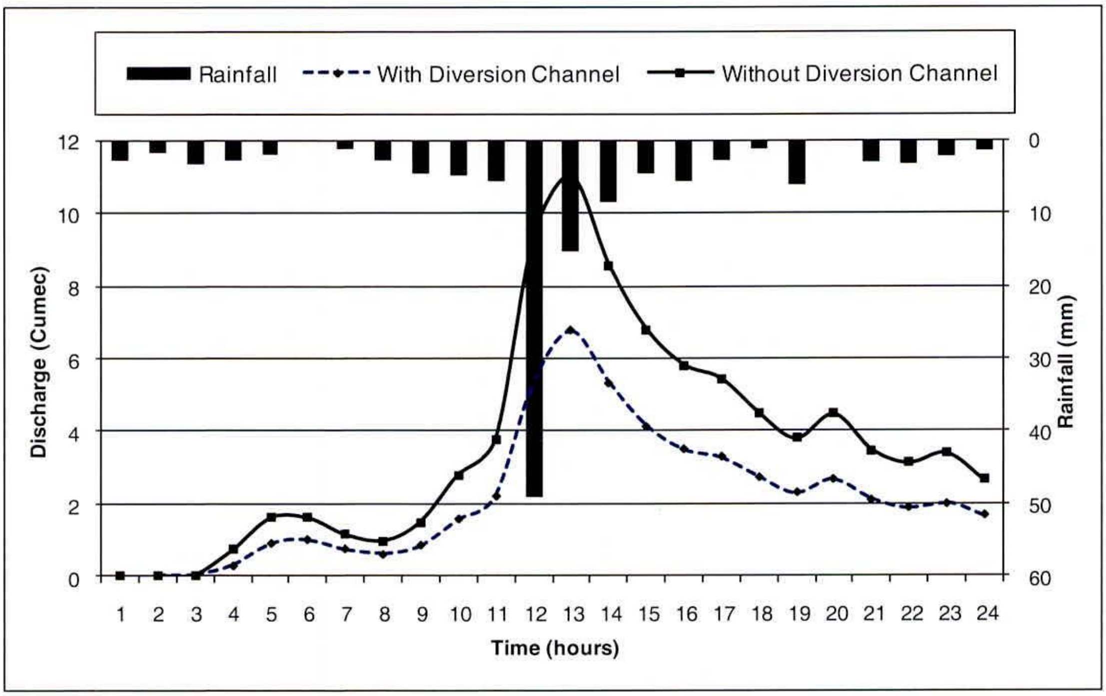

Figure 35. Two years return period of hyetograph and corresponding hydrograph at Anna Nagar with and without diversion channel

6.0 CONCLUSIONS

The storm water network in the Otteri Naullah sub basin has been studied. Historical hourl rainfall data (1980-2009) nearby locality at Nungambakkam (maintained by IMD) was collected and analyzed. The hourly rainfall computed for 2, 5, 10, and 25 years return periods using Extreme Value Type 1 distribution are 48.89, 64.10, 74.08, and 87.24 mm respectively. Due to non-availability of rainfall and water level data in the study area, five tipping bucket rain gauges and two automatic water level recorders were installed in the study area. Further, DGPS survey was conducted to substantiate spot heights from SOI maps to prepare Digital Elevation Model (DEM) for the study area. The land use/land cover map was prepared using IRS P6 satellite data. The storm water drainage network details and Otteri Nullah longitudinal profiles/cross section details at every 30 m were collected and GIS database was prepared. Using thematic layers of DEM, drainage network and road network, total 88 micro watersheds were delineated in the Otteri Nullah sub-basin. Using these micro watersheds, storm water drainage network and Otteri Naullah cross sections, the study area was schematized using 121 nodes and 120 links in the XP-SWMM environment. Model parameters like Node/link characteristics, pervious/impervious area, soil type, average width/slope and SCS-CN were computed for each micro watershed using GIS data base. Based on observed rainfall and water level data in the study area, few events were selected to analyze the performance of XP-SWMM model in terms of runoff computation in the study area. After successful testing of the model, the 24 hrs design storm for 2, 5 10 and 25 yrs return periods were considered to check the storm water drainage network efficiency in the study area. It was found that the present storm drainage network is not sufficient to drain storm water runoff even for two-year return period storm with existing longitudinal profile of Otteri Nullah. The hydrographs at outfall of the sub basin was developed for design storms of various return periods which would be useful to adopt best management practices (BMP).

Few scenarios were also developed for ongoing renovation activities proposed by PWD in the Otteri Nullah sub basin. The 2, 5, 10 and 25 return period storms were again tested for the modified sections and found that modified longitudinal profile is capable to drain five year return period storm. Flood water diversion link with proposed cross section were incorporated in the XP-SWMM model and found that the reduction of the flood peak at diversion link (above AnnaNagar) is 38% in Otteri Nullah basin. The outcome of the project was disseminated to user agencies through interaction workshops and training programs during the project period.

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Table 1. Functionality and accessibility of representative models

Programme name	Functionality			Accessibility	
	Planning	Operational	Design	Public domain	Commercial
Urban models					
DR_3M-QUAL	√		√	√	
HSPF	√		√	√	
MIKE-SWMM	√	√	√		√
QQS	√		√	?	
STORM	√			√	
SWMM	√		√	√	
SWMM Level 1	√			√	√
Wallingford Model	√	√	√		
Non-urban models					
BRASS		√	√		
HEC-5Q	√	√		√	
QUAL2E-UNCAS	√			√	
WQRRS	√		√	√	

Table 2. Components in the quantity analysis in representative models

Programme name	Model quantity component					
	Pipes	Open Channel	Retarding basins	Others	Natural Streams	Rainfall-runoff
Urban models						
DR_3M-QUAL	√	√	√		√	√
HSPF	√	√	1		√	√
MIKE-SWMM	√	√	√	2-7	√	√
QQS	3	√	√	2		√
STORM						
SWMM	√	√	√	4		√
SWMM Level 1						√
Wallingford Model	4	√	√	2-5		√
Non-urban models						
BRASS		√	1	7	√	√
HEC-5Q			1		√	
QUAL2E-UNCAS					√	
WQRRS		√	1		√	√

Table 3. Generalized criteria of design storm for various water control structures.

Return Period (yrs.)	Type of structures
02 – 25	Storm sewers-small cities
25 – 50	Storm sewers-large cities

Table 4. SCS Curve Numbers[1]

Land use description	Hydrologic Soil Group			
	A	**B**	**C**	**D**
Cultivated land without conservation treatment	72	81	88	91
Cultivated land with conservation treatment 71	62	71	78	81
Pasture or range land - Poor condition	68	79	86	89
Pasture or range land - Good condition	39	61	74	80
Meadow - Good condition	30	58	71	78
Wood or forest land - Thin stand, poor cover, no mulch	45	66	77	83
Wood or forest land - Good cover[2]	25	55	70	77
Open spaces, lawns, parks, golf courses, cemeteries, etc				
Good condition: grass cover on75% or more of the area	39	61	74	80
Fair condition: grass cover on50-75% of the area	49	69	79	84
Commercial and business areas (85% impervious)	89	92	94	95
Industrial districts (72% impervious)	81	88	91	93
Residential[3] Average lot size (% Impervious[4])				
1/8 ac or less (65)	77	85	90	92
1/4 ac (38)	61	75	83	87
1/3 ac (30)	57	72	81	86
1/2 ac (25)	54	70	80	85
1 ac (20)	51	68	79	84
Paved parking lots, roofs, driveways, etc. [5]	98	98	98	98
Streets and roads - Paved with curbs and storm sewers[5]	98	98	98	98
Streets and roads – Gravel	76	85	89	91
Streets and roads – Dirt	72	82	87	89

[1]Antecedent moisture condition II; Source: SCS Urban Hydrology for Small Watersheds, 2nd Ed.,(TR-55), June 1986. [2] Good cover is protected from grazing and litter and brush cover soil. [3] Curve numbers are computed assuming that the runoff from the house and driveway is directed toward the street with a minimum of roof water directed to lawns where additional infiltration could occur. [4] The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers. [5] In some warmer climates of the country a curve number of 95 may be used.

Table 5. Details of equipments installed in the study area

Sl.	Name of the Equipment	Location	Date of Installation	Data Processed and analyzed
1	Tipping Bucket Raingauges	Anna Nagar	19-09-2010	19-01-2014
2		Villivakkam	08-09-2011	19-01-2014
3		Inavaram	08-09-2011	19-01-2014
4		Perambur	09-07-2011	19-01-2014
5		Basin Bridge (GMR)	09-07-2011	19-01-2014
6	Automatic Water Level Recorders (Bubbler type)	Anna Nagar (L block)	29-09-2011	23-01-2014
7		Basin Bridge (GMR)	09-07-2011	22-01-2014

Table 6. Spatial variation analysis of observed rainfall data

	Highest Daily rainfall (mm) in the study area (2011)						
Raingauge Location	**30-Dec**	**25-Nov**	**25-Oct**	**16-Sep**	**25-Aug**	**21-Aug**	**27-Jul**
Anna Nagar	93	121	49	49	54	104	23
Basin Bridge (GMR)	60	112	48	57	73	68	17
Perambur	64	116	49	54	110	96	9
Villivakkam	74	147	59	56	DNA	DNA	DNA
Inavaram	74	96	72	74	DNA	DNA	DNA
DNA: Data Not Available							

Table 7. Micro watersheds characteristics of the study area in XP-SWMM

Subcatchment ID	Area (ha)	Width (m)	Impervious Area (%)	Slope (%)	Runoff Coefficient	Catchment outlet node
1	4.30	120	76.0	0.002	0.80	N1
2	20.60	252	69.4	0.004	0.78	N16
3	2.24	90	84.2	0.003	0.86	N3
4	3.09	216	15.4	0.003	0.39	N7
5	14.68	298	43.0	0.003	0.57	N12
6	5.75	140	72.7	0.006	0.78	N15
7	6.53	101	45.0	0.003	0.58	N104
8	1.79	100	53.6	0.001	0.65	N111

Subcatchment ID	Area (ha)	Width (m)	Impervious Area (%)	Slope (%)	Runoff Coefficient	Catchment outlet node
9	2.27	105	60.0	0.001	0.69	N112
10	9.16	229	83.7	0.003	0.85	N113
11	46.38	478	66.2	0.001	0.71	N115
12	24.93	250	87.6	0.002	0.85	N117
13	11.86	310	62.5	0.003	0.71	N119
14	7.68	163	66.2	0.004	0.73	N116
15	3.11	133	58.3	0.002	0.68	N114
16	5.42	195	45.9	0.001	0.59	N106
17	61.32	537	44.5	0.002	0.57	N120
18	12.36	175	39.5	0.003	0.54	N121
19	15.98	270	33.8	0.003	0.51	N59
20	8.06	183	57.5	0.003	0.67	N41
21	1.47	120	59.5	0.002	0.69	N40
22	10.12	211	65.2	0.003	0.72	N31
23	1.45	65	54.3	0.002	0.65	N33
24	8.98	229	67.6	0.003	0.74	N30
25	10.31	216	59.9	0.003	0.69	N32
26	10.55	225	62.5	0.002	0.70	N35
27	4.58	103	67.0	0.004	0.74	N39
28	9.71	200	86.6	0.002	0.86	N37
29	8.01	123	61.5	0.005	0.70	N38
30	2.65	100	64.0	0.001	0.72	N43
31	1.20	50	79.0	0.005	0.82	N44
32	3.11	142	66.4	0.003	0.74	N45
33	7.14	226	68.3	0.003	0.75	N47
34	4.81	234	64.0	0.004	0.72	N50
35	11.73	227	68.0	0.004	0.74	N46
36	18.67	250	65.9	0.002	0.72	N51
37	11.17	202	77.3	0.002	0.80	N54
38	12.91	200	63.5	0.005	0.71	N52
39	4.14	97	56.3	0.003	0.66	N53
40	38.44	480	49.1	0.004	0.61	N61
41	9.08	200	50.4	0.001	0.62	N58
42	11.52	194	41.5	0.004	0.56	N60
43	14.59	386	36.1	0.004	0.53	N55
44	5.19	124	30.8	0.005	0.49	N56

Subcatchment ID	Area (ha)	Width (m)	Impervious Area (%)	Slope (%)	Runoff Coefficient	Catchment outlet node
45	27.02	318	44.5	0.003	0.57	N63
46	24.13	261	57.8	0.002	0.66	N64
47	28.49	273	65.5	0.004	0.72	N65
48	2.43	121	70.6	0.007	0.77	N69
49	2.29	130	38.7	0.006	0.55	N67
50	20.70	238	55.7	0.005	0.65	N66
51	28.98	341	66.0	0.007	0.73	N71
52	8.48	220	63.0	0.008	0.71	N72
53	4.97	135	65.1	0.005	0.73	N70
54	3.56	70	78.5	0.004	0.81	N2
55	31.41	561	51.5	0.005	0.69	N17
56	16.75	956	71.5	0.004	0.81	N73
57	33.07	890	66.7	0.002	0.77	N76
58	15.54	316	71.6	0.005	0.83	N18
59	22.75	218	79.1	0.003	0.83	N18
60	38.30	820	57.3	0.005	0.79	N77
61	14.43	250	58.7	0.003	0.79	N19
62	29.28	319	69.4	0.004	0.86	N79
63	20.24	450	86.6	0.002	0.91	N80
64	23.60	410	85.8	0.004	0.91	N82
65	3.80	467	96.2	0.002	0.96	N23
66	22.46	350	61.4	0.004	0.84	N20
67	96.07	1097	88.6	0.002	0.91	N87
68	31.19	800	87.1	0.003	0.92	N88
69	158.21	1923	90.4	0.002	0.91	N89
70	20.03	297	90.4	0.002	0.92	N27
71	175.76	816	74.5	0.003	0.78	N90
72	48.58	300	45.3	0.003	0.63	N91
73	95.18	800	84.0	0.003	0.88	N92
74	76.27	520	71.1	0.004	0.83	N93
75	112.46	700	49.9	0.002	0.65	N94
76	185.13	500	57.1	0.004	0.75	N96
77	182.65	320	61.1	0.006	0.75	N97
78	37.59	300	61.7	0.008	0.74	N99
79	24.26	250	67.8	0.005	0.77	N102
80	74.62	750	45.3	0.004	0.64	N103

Subcatchment ID	Area (ha)	Width (m)	Impervious Area (%)	Slope (%)	Runoff Coefficient	Catchment outlet node
81	5.44	315	78.9	0.002	0.90	N21
82	92.88	1000	68.6	0.003	0.85	N22
83	99.85	1440	70.0	0.002	0.84	N28
84	22.31	347	65.9	0.001	0.84	N28
85	36.98	493	62.8	0.004	0.81	N78
86	43.78	1130	84.1	0.002	0.91	N84
87	52.63	1300	81.2	0.002	0.90	N85
88	122.86	708	77.8	0.008	0.86	N101

Table 8. Link properties in XP-SWMM for 2 yrs return period storm

Link Name	Upstream Node Name	Downstream Node Name	Length (m)	Max Flow cumec	Max Velocity m/s
L1	N1	N2	357	0.46	0.34
L16	N16	N17	663	12.45	1.07
L3	N3	N4	293	2.50	0.66
L4	N4	N5	311	4.91	0.87
L5	N5	N6	152	5.00	0.99
L6	N6	N7	411	5.20	1.02
L7	N7	N8	446	6.56	1.07
L8	N8	N9	307	7.71	1.12
L9	N9	N10	32	8.99	1.09
L10	N10	N11	65	10.03	0.95
L11	N11	N12	225	7.30	1.00
L12	N12	N13	282	8.50	1.08
L13	N13	N14	285	8.84	0.82
L14	N14	N15	442	8.66	0.72
L15	N15	N16	344	10.30	1.03
L104	N104	N105	720	0.47	0.59
L107	N107	N108	140	0.94	0.69
L108	N108	N109	150	1.14	0.79
L109	N109	N110	160	1.55	2.15
L110	N110	N4	60	2.48	1.85
L111	N111	N107	240	0.16	0.66
L112	N112	N108	210	0.22	0.78

Link Name	Upstream Node Name	Downstream Node Name	Length (m)	Max Flow cumec	Max Velocity m/s
L113	N113	N114	219	0.39	1.00
L115	N115	N116	1071	0.54	0.70
L117	N117	N116	784	0.48	0.59
L119	N119	N118	550	0.79	1.03
L118	N118	N110	183	0.94	1.26
L116	N116	N118	545	0.79	1.08
L114	N114	N109	244	0.41	1.09
L105	N105	N106	275	0.46	0.48
L106	N106	N107	75	0.79	0.62
L120	N120	N59	830	0.61	0.85
L29	N121	N11	603	0.84	1.45
L59	N59	N9	220	1.56	2.19
L41	N41	N42	148	0.70	0.87
L42	N42	N2	150	1.21	1.74
L40	N40	N42	228	0.67	0.94
L39	N31	N40	299	0.37	0.85
L32	N33	N31	202	0.14	0.19
L30	N30	N31	336	0.93	1.34
L31	N32	N34	435	0.84	1.08
L33	N35	N36	247	0.91	1.14
L38	N39	N40	437	0.40	0.52
L34	N37	N36	217	0.68	0.84
L35	N36	N34	333	0.54	0.67
L37	N38	N39	610	0.55	0.68
L36	N34	N39	113	0.75	0.92
L43	N43	N3	138	0.27	0.98
L44	N44	N5	231	0.14	0.51
L45	N45	N6	169	0.33	0.88
L48	N48	N49	354	0.88	1.19
L47	N47	N48	222	0.75	1.03
L50	N50	N48	185	0.40	0.58
L46	N46	N49	217	0.69	0.90
L49	N49	N7	408	1.33	1.83
L51	N51	N52	822	0.16	0.64
L54	N54	N55	567	0.67	0.86
L52	N52	N53	647	0.68	0.87

Link Name	Upstream Node Name	Downstream Node Name	Length (m)	Max Flow cumec	Max Velocity m/s
L53	N53	N62	437	0.70	0.99
L62	N62	N8	23	1.16	3.63
L61	N61	N62	1050	0.46	0.65
L58	N58	N60	252	0.32	0.66
L60	N60	N57	494	0.49	0.69
L57	N57	N53	150	0.77	0.95
L55	N55	N56	448	0.65	0.84
L56	N56	N57	87	0.72	0.88
L63	N63	N10	866	1.04	1.44
L64	N64	N12	967	1.11	1.51
L65	N65	N13	1025	0.80	1.15
L69	N69	N68	275	0.26	0.32
L67	N67	N68	135	0.46	0.58
L66	N66	N67	803	0.30	0.38
L71	N71	N15	800	0.78	1.11
L72	N72	N16	230	0.06	1.83
L70	N70	N15	370	0.51	1.28
L68	N68	N14	52	0.00	0.01
L2	N2	N3	207	2.03	0.60
L17	N17	N18	421	6.14	0.84
L73	N73	N74	890	0.27	0.68
L76	N76	N75	623	0.52	0.70
L75	N75	N18	314	0.78	2.09
L18	N18	N19	634	9.72	0.99
L77	N77	N78	854	0.80	0.80
L19	N19	N20	594	16.01	0.64
L79	N79	N78	1058	0.67	0.68
L80	N80	N81	445	2.00	0.93
L82	N82	N81	445	0.60	0.84
L81	N81	N83	350	2.30	1.16
L83	N83	N86	270	2.51	1.44
L86	N86	N23	730	2.67	1.84
L23	N23	N24	364	11.34	0.78
L20	N20	N21	1009	16.34	0.94
L87	N87	N24	1400	0.93	1.27
L24	N24	N25	577	11.98	0.84

Link Name	Upstream Node Name	Downstream Node Name	Length (m)	Max Flow cumec	Max Velocity m/s
L88	N88	N26	684	0.68	4.8
L26	N26	N27	671	10.37	0.63
L89	N89	N27	1123	2.28	1.79
L27	N27	N28	491	14.51	0.79
L90	N90	N27	757	0.67	1.62
L91	N91	N26	366	0.92	2.21
L92	N92	N25	1369	0.95	1.34
L25	N25	N26	479	12.93	0.83
L93	N93	N94	593	6.66	1.54
L94	N94	N95	1450	5.26	1.38
L95	N95	N23	413	6.18	2.89
L96	N96	N95	2942	0.44	0.64
L97	N97	N98	203	0.57	0.79
L98	N98	N100	1960	0.56	0.81
L99	N99	N98	370	0.16	0.64
L102	N102	N103	1233	0.26	0.66
L103	N103	N20	845	1.21	1.67
L100	N100	N95	594	0.53	0.81
L21	N21	N22	450	16.99	0.73
L22	N22	N23	63	8.06	0.50
L28	N28	N29	110	28.0	1.49
L78	N78	N19	112	4.73	4.86
L84	N84	N83	510	0.27	0.59
L85	N85	N86	937	0.17	0.46
L74	N74	N75	370	0.27	0.65
L101	N101	N100	523	0.86	1.13

Table 9. Details of water balance, flood peaks with existing and proposed longitudinal profile of Otteri Nullah against various return period of design storm.

24 hr design storm return period	Peak (m^3/s)		System Inflow (10^6 m^3)		System Outflow (10^6 m^3)		± % Error	
	Existing	Proposed	Existing	Proposed	Existing	Proposed	Existing	Proposed
2	27.57	53.00	2.7463	2.7463	2.7429	2.7538	0.065	-0.323
5	33.80	61.40	4.6879	4.6879	4.6817	4.6931	0.065	-0.110
10	37.58	69.31	5.9395	5.9395	5.9345	5.9411	0.031	-0.050
25	42.37	75.60	7.6090	7.6091	7.6056	7.6092	0.005	-0.001

Annexure I

Photographs of field situation and technology transfer activities

Annexure II

Consent Letter from Govt., of Tamilnadu

4063
18/8/08

Public Works Department,
Secretariat, Chennai - 9

Letter (D) No. 393/R.2/2008- 3, Dated. 12. 8.2008

From
Thiru S.AUDISESHIAH, I.A.S.,
Secretary to Government.

To
The Director,
National Institute of Hydrology,
Jalvigyan Bhawan,
Roorkee - 247 667.

Sir,

Sub: Hydrology Project Phase –II – Purpose Driven Study on "Urban Hydrology for the Chennai City" to be carried out by National Institute of Hydrology, Kakinada in collaboration with State Groundwater & Surface Water Resources Data Centre - Acceptance conveyed.

Ref: 1. Your Letter No. REMC/HP-II/NIH-08 dated 17.1.2008.
2. From the Chief Engineer (SG&SWRDC) Letter No. T2/144/2007, dated 12.2.2008 & 26.6.2008.
3. Your Letter No. NHI/SWH/PDS/ Tamil Nadu/2008, dated 9.7.2008.

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I am directed to invite your kind attention to the references cited. In view of the circumstances reported in your letter third cited, I am directed to convey the acceptance of the Government of Tamil Nadu for taking up the study on Urban Hydrology for Chennai City under "purpose Driven Studies" under Hydrology Project Phase-II, in collaboration with State Groundwater & Surface Water Resources Data Centre, without any Financial Commitment to Government of Tamil Nadu.

Yours faithfully,

S. [signature]

for Secretary to Government.

Copy to
The Chief Engineer (State Groundwater & Surface Water Resources Data Centre) Water Resources Department, Chennai -113.
The Senior Joint Commissioner, Ministry of Water Resources, Brahmaputra & Barek Wing, 5th Floor, Mohen Singh Place, Baba Kharak Singh Marg, Connaught Place, New Delhi – 110 001.
The Director, National Institute of Hydrology, Kakinada, Andhra Pradesh.
SF/SC