STORM WATER DRAINAGE PROBLEM AND SURFACE FLOODING OF PATNA TOWN - A CASE STUDY

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INTRODUCTION

Skyscrapers, paved roads, storm water drain, sewer drains illuminated light system etc. are the symbol of urban areas. The civil engineering structures change the urban area into paradise only if all structures function well through out the year. During rainy seasons, urban area is subjected to flooding due to non-provision of 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, which may affect the health of the people. In order to overcome the inundation, the storm water drainage system has to be properly designed. Manual design of storm water drain is tedious process and takes much time. Hence software accounting for the various aspects of flow processes is required.

The design principle starts from the analysis of historical rainfall data to simple hydraulic equations. Urbanization associated with increase due to in imperviousness and decreases in infiltration and evaporation causes increase in surface runoff volume in many floods. Hence, construction of drainage systems with sufficient capacity to carry the large volume of surface water is needed. Estimation of surface runoff due to rainfall events is a key factor in drainage system net work design. In the present study hydrologic and hydraulic models have been used in determining runoff hydrographs from rainfall excess.

The Rational method which is widely used for the design of storm drainage system is simple to use, but it allows only the determination of the discharge hydrograph peak. It is inadequate to account large catchment for pipe routing or variations in rainfall intensity, contributing area and rate of contribution. The further drawback in the method is the lumping of all physical factors into two parameters (runoff coefficient and time of concentration) which makes parameter estimation subjective and prediction from observed rainfall is inaccurate. At present, there is several efficient urban catchment simulation models are available and in this study application of Storm Water Management Model (SWMM) has been adopted.

SWMM uses information of GIS data (ERDAS & Arc View) together with a catchment modeling system. The main objects of the present study are (i) To develop the outfall hydrograph and water profiles along the storm channels and (ii) Evaluate the Runoff volume and surface flooding for the design storm of various return periods.

STUDY AREA AND DRAINAGE PROBLEM

Patna, capital of Bihar state is situated on the bank of river Ganga with Latitude 25^0 37' and longitude 85^0 10' and has a mean elevation of 49.68m above the MSL. Patna and its upland is sandwiched between the high Himalayan ranges in the far north and the high tracts of Chhotanagpur in the south. Due to its locàtion with relation to latitude and other features, Patna has a temperate climate, suitable for urban living. Patna is a linear city and is about 30Km long from east to west and 5-7Km from north to south. The city is situated between the river Ganga in the north, river Punpun in the south and river Sone in the west. The general slope of the city as indicated by the flow of these rivers is from north to south and from west to east. The gradient is so gentle that the whole land can be said to be featureless and flat expect for local undulations in the form of ditches, tanks and lakes.

Topographically, Patna might be called a flat city, though the ground slopes gently towards south. The main east-west road, the Ashok Raj Path running almost parallel to the river Ganga forms a ridge along north. Providing proper drainage for the city had been difficult, mainly for two reasons. Firstly, the city has been constantly threatened by floods from north for which the HFL of the river Ganga becomes higher than the general level of the city. Secondly vast area on the south of the eastern railway main line is subjected to annual inundations by flood water from the river Punpun during the monsoon period. The ridge line is the Ashok Raj Path. The area falling on north of this road up to the river bank drains into the river Ganga and the area south of it has slope towards south and ultimately drains into river Punpun as shown in the figure 1.

Details of the study area

For the purpose of storm water drainage and sewerage system, the city has been divided into four natural zones called the Western, the Central, the Eastern, and the Southern zone as shown in the figure 1. The boundaries of Western and Southern zones have been extended up to Khagaul-Danapur road on the west and Patna bypass road on the south as per proposed in the master plan of Patna.

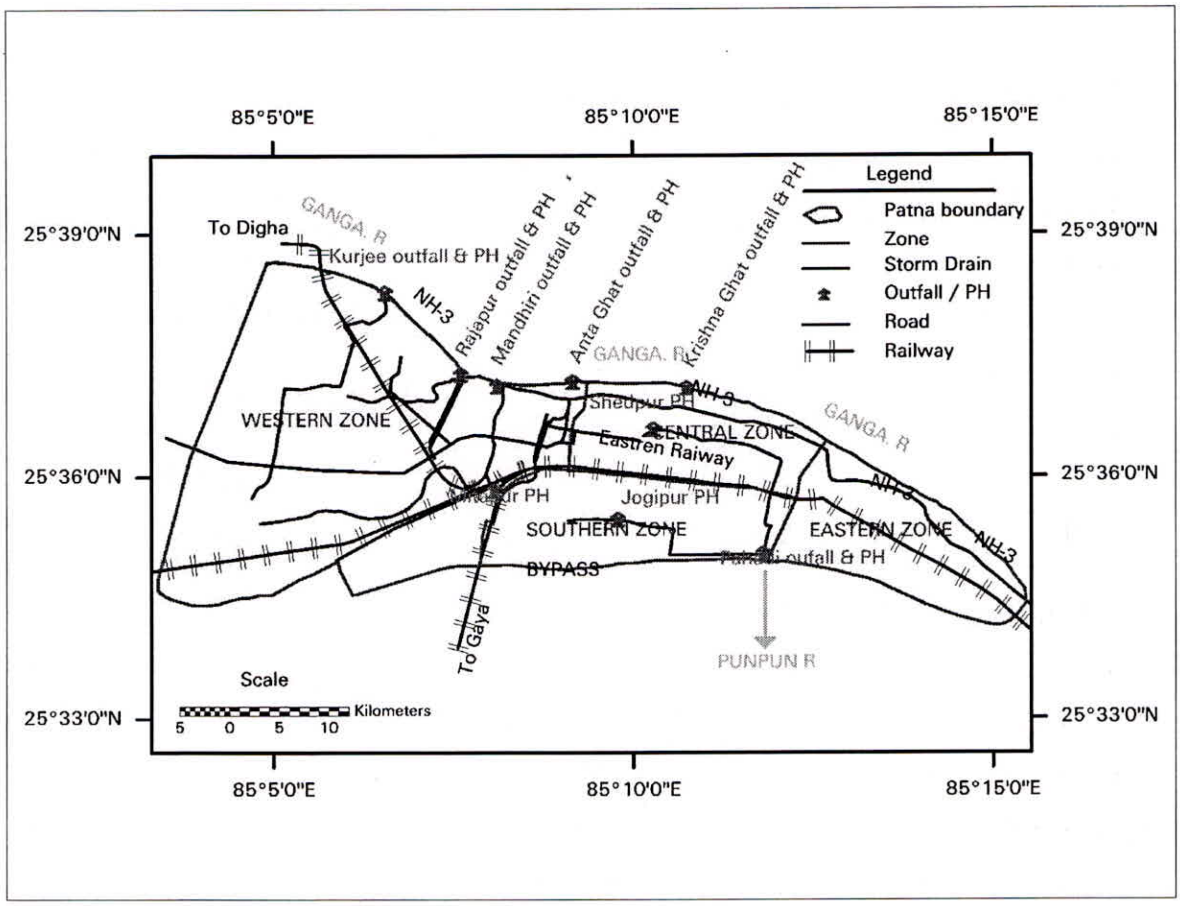


Figure 1. Details of the Patna Town

Western Zone

This zone comprises of the area bounded by Patna Gaya Road in the east, Danapur Khagaul road in the west and river Ganga in the north. Although on the south, main railway line is barrier, but storm water of areas such as Mithapur, Jakkanpur, Purandarpur,

Gardanibagh etc. which are to the south of main railway line and west of Patna Gaya railway line comes under gravity to a sump situated in Mithapur and from there to the Western Zone system through pumping station at Mithapur. Ground level of areas to the west of Patna Canal up to Danapur Khagaul road is high and generally free from flood.

Central Zone

Central Zone comprises of the area bounded by the Patna-Gaya road in the west, Ganga Bridge approach road in the east, river Ganga in the north and main Eastern Railway line in the south. Ganga is to the north of this zone and Punpun river is to the south though at a distance. Area north of Ashok Rajpath slopes towards Ganga . This area is prone to water logging specially when the level of river Ganga and Punpun rises and prevents the flow of storm water under gravity into the river Ganga.

Eastern Zone

The Eastern zone which is primarily old Patna town known as Patna City comprises of the area bounded by Pachhim Darwaja in the west, Deedarganj on the east,. And river Ganga on the north and Patna new by pass on the south. There is no proper drainage system for Patna city but there are two very old main outfalls.One at Pachhim Darwaja and the other for city area, which is called City Moat nalla. There is no proper system of surface drains to connect these nallas. Eastern Zone being unplanned and old there are narrow streets and it is difficult to lay either sewer or storm water conveying pipe. Thus combined system is suitable for this area. Presently part of the storm water goes to Ganga directly and part of it is logged in low-lying areas along the by-pass road.

Southern Zone

Southern zone comprises of the area bounded by Patna Gaya railway line in the west, Ganga Bridge in the east, main eastern railway line in the north and Patna new by-pass in the south. The existing system of storm drainage for this area has a pumping station at Jogipur which caters for only Lohiya Nagar Housing Board Colony with pumps of total capacity of 780 H.P. only, where as nearly 2600 H.P. is required for covering storm water from Chiraiyatanr, Karbigahiya and the adjoining areas. The areas west of Chiraiyatanr and Karbigahiya still have no proper drainage system . After construction of Patna new bye pass, natural drainage has been obstructed, causing water logging in the localities adjacent to by-pass road. In addition to the Housing Board Colony of Kankarbag, Bahadurpur and Hanuman Nagar, a number of private colonies have sprung up in a haphazard manner, without even surface drainage facilities. This poses a serious problem for management of drainage of this area even in dry season what to talk of rainy season.

Drainage Problem

Since the period of Buddha, Patna has been facing acute drainage problem due to its topography. Accordingly to historians and archaeologist that once ancient Patna was washed away because of floods of rivers Ganga, Sone and Punpun accompanied with the continuous rains for 17 days in the catchments.

In the recent past also, Patna was heavily flooded in the year 1975-76 and there was a severe water logging in the year 1990 & 1997 in the town. Due to topographical condition of Patna, storm water does not flow under gravity to river Ganga or Punpun during the period of flood. Pumping is the only means to dispose the rainwater of the town. Apart from this, new unplanned colonies and theirs commercial activities are fast developing in the western and southern parts of Patna without proper drainage system. This has aggravated the problem of water logging. More over construction of new By-Pass has blocked the natural drainage and thus aggravated the water logging problem in Kankarbagh and Anishabad area, as natural storm water flow towards south of by-pass road has been obstructed. Most of the private colonies have been developed in low-lying areas without adequate filling and have no regard to proper road and drainage network. There are the basic cause of water logging and drainage congestion of the Patna town.

METHODOLGY

Geographic information system (GIS) framework is a powerful tool to support the spatial data base requirement of urban storm water management. The physically based model SWMM offers input data formats that facilitate the sharing and exchange of data with GIS packages. The procedure for GIS to SWMM as shown in figure 2a &2b and for linking GIS and SWMM modeling for urban storm water management involves the following steps.

1. Acquisition and development of base map data layers and coverage required for SWMM.
2. Pre processing of model input data and parameters. Development of GIS techniques suitable for input of spatial information into the SWMM model.
3. Interfacing the GIS to the SWMM model through development of programs for converting GIS derived coverages to SWMM model input parameters.
4. SWMM model calculation of urban catchment discharges and depths.
5. Post processing of SWMM model output by returning portions of hydrographs to the GIS spatial display and analysis.
6. Analysis of hydrographs, including derivation of depths and extent of flooding in GIS environment.

The data on rainfall, topography, drainage network and remote sensing imagery are needed for the above analysis and to run SWMM model.

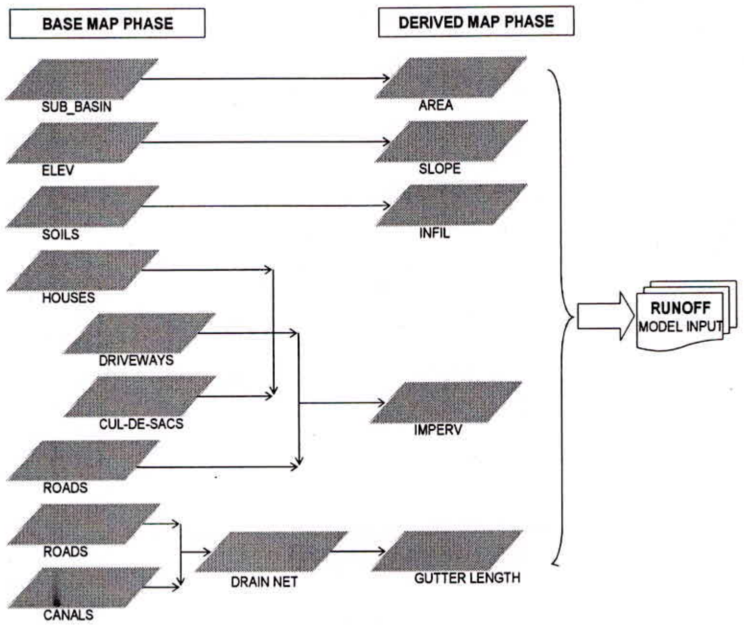


Fig. 2a: Procedure for GIS to Storm Water Management Model (SWMM).

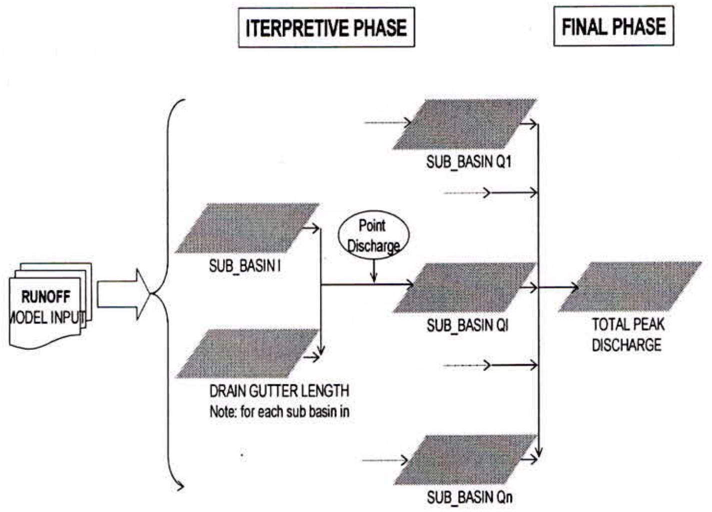


Fig. 2b: Procedure for GIS to Storm Water Management Model (SWMM).

Design Storm Analysis

A design storm is the precipitation pattern used in the design of a hydrologic system. Usually the design storm severs as the system input, and the resulting rate of flow through the system are calculated using rainfall runoff and flow routing. 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. 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 1.

Table 1: 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

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. Though many frequency distributions are reported in the literature, for the present study, it was decided to test the applicability of Extreme Value Type 1 (EV1) distribution to the available rainfall data. A brief description of the EV1 distribution follows (Chow *et al.,* 1988, Cunnane, 1989) the probability distribution function for EV1 is given by

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

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

$$y_i = -\ln(-\ln(F_i) \quad (5)$$

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

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).

$$P_{T_r} = P_m + K_{T_r} s \quad (7)$$

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

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

Temporal Distribution of Rainfall Intensity

There are a number of distributions that have been proposed in the literature to represent the intensity distribution of a design storm. For the present study, U.S. Department of Agricultural, Soil Conservation Service (SCS, 1986) model was used to develop synthetic storm hyetograph based on SCS standard 24 hour's distribution. This distribution is applicable in small and large catchments.

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 $f_d = \frac{Cv}{q}$, 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 form the laminar condition despite the low Reynold 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 = 1.49 $ft^{1/3}$/sec and n is the effective Manning roughness factor.

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 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 subarea, but not through the impervious subarea. Impervious areas are themselves divided into two subareas-one that contains depression storage and another that does not. Runoff flow from one subarea in a sub catchment can be routed to the other subarea, or both subareas can drain to the sub catchment outlet.

Surface Runoff

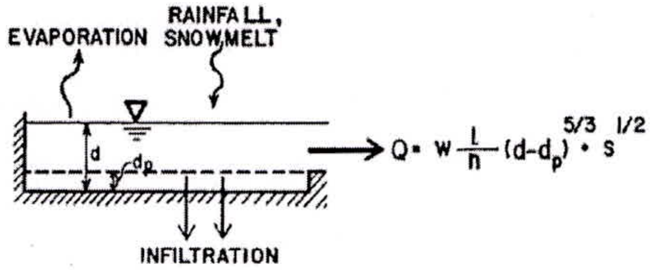


Fig 3: Conceptual view of surface runoff.

The conceptual view of surface runoff used by SWMM is shown in the Fig. 7. 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 depression storage provided by ponding, surface wetting and interception. 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 3.).

Infiltration

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

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 required by this method include the 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.

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.

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 2). 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.

SWMM model calculates infiltration losses 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:

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

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.

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

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

Table 2: SCS Curve Numbers[1]

	Hydrologic Soil Group			
Land use description	**A**	**B**	**C**	**D**
Cultivated land				
Without conservation treatment	72	81	88	91
With conservation treatment 71	62	71	78	81
Pasture or range land				
Poor condition	68	79	86	89
Good condition	39	61	74	80
Meadow				
Good condition	30	58	71	78
Wood or forest land				

Thin stand, poor cover, no mulch	45	66	77	83
Good cover[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
Gravel	76	85	89	91
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.*

Flow Routing

Flow routing within a conduit link in SWMM is governed by the Saint Venant equations base 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.

Steady Flow Routing

Steady Flow routing represents the simplest type of routing possible (actually no routing) by assuming 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 the 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.

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.

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.

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.

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

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

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

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

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

$$h_L = \frac{KV^2}{2gL} \qquad (19)$$

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

To solve equations (15) and (16) 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. 4). 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:

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

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.

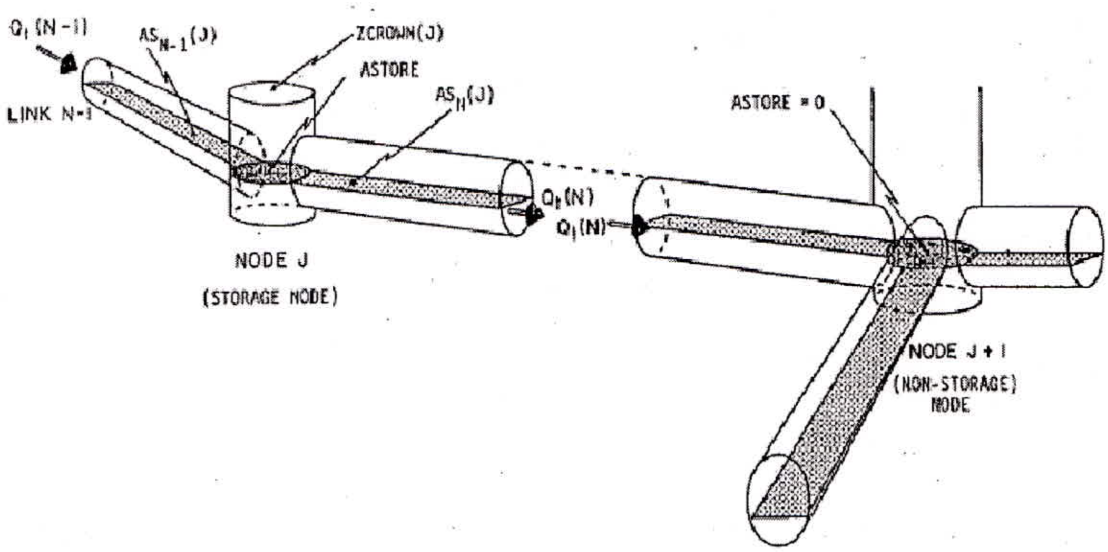


Fig 4: Node-Link Representation of a Drainage System in SWMM.

Equations (15), (16, 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:

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

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

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

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

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

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

Surface Flooding

Using DEM the elevation difference has an average area to compute the surface flooding maps. Associated water elevation h in the catchment is surface area A and volume V (Eq.28) to compute the surface flooding, which will appear occurs due to spill out the flow from the drainage network.

$$V_{j+1} = V_i + \left(\frac{A_j + A_{j+1}}{2}\right)\left(h_{j+1} - h_j\right) \tag{28}$$

Channel Capacity

Each reach of channel is assumed to be prismatic, *ie* 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 assumed to be 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

$$Q = \frac{1}{n} A R^{2/3} \sqrt{S} \tag{29}$$

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.

RESULTS AND DISCUSSIONS

As per the schematic diagram (Fig. 2a & 2b) the base map coverage was used to derive the coverage maps of elevation, land use, drainage and sub drainage area delineation, drainage network and outfall maps. The brief descriptions of the generated maps are as under:

Elevation map

The elevation map has been prepared using the 1 feet contour interval map supplied by the Bihar Rajya Jal Parshed (BRJP) and 1 feet spot height map supplied by the Patna Regional Development Authority (PRDA), Govt. of Bihar. These two maps were georeferenced in the GIS (ERDAS) environment and digitized the contours and spot heights. Digital elevation model (DEM) a raster file in which each gird cell represents an elevation value was prepared (Fig. 5). By using DEM data, the percentage slope map was prepared and color coded according to the steepness of the terrain. The study area was classified into seven classes (Fig. 6).

Land use map

Identification of various land cover of the study area was based on the digital image procured from NRSA. Classification is done selecting training sites generating signature and convoluting the training statistics over the entire map. The class so identified is also verified by ground truth data. This technique is known as supervised classification. For land use classification IRS P6-L4MX data has been used and performed supervised classifications (Fig. 7) and different class area shown (Table 3).

Table 3: Land use classification of the study area.

Sl	Name of the classification	Area (ha)	Sl	Name of the classification	Area (ha)
1	High density built up area	1619.34	5	Major opening area	446.24
2	Medium density built up area	929.89	6	Minor opening area	1320.48
3	Low density built up area	1471.59	7	Roads	479.58
4	Low lying area	89.19			

Sub area watershed delineations

In the first step the study area was divided into sub catchment. The delineations were based upon the topography of the study area, utilizing DEM (Fig. 6), following the existing drainage system and present land use pattern (Figs.1 & 7).The Town has been divided into four natural zones called the Western, the Central, the Eastern, and the Southern zone (Fig. 1). The Western zone is divided into four sub areas or sub catchments, central and southern zone is having one drainage area each, based on the existing drainage network. Finally each drainage area was divided into several sub drainage areas based on the slope and topography (Fig. 8).

Drainage network and outfall for all sub area

Western zone has four storm drains and four outfalls, Central zone has one storm drain which also drains some area of Southern zone, Southern zone have one storm drain and one outfall and Eastern zone has no storm drain (Fig. 1). Lengths of drains and name of outfalls are given in the Table 4. From the above base maps, the information was developed in GIS as derived maps. Then modeling parameters were prepared for input into the SWMM model by obtaining each data category in GIS coverage and then exporting these data into a spreadsheet for further analysis and weighted averaging

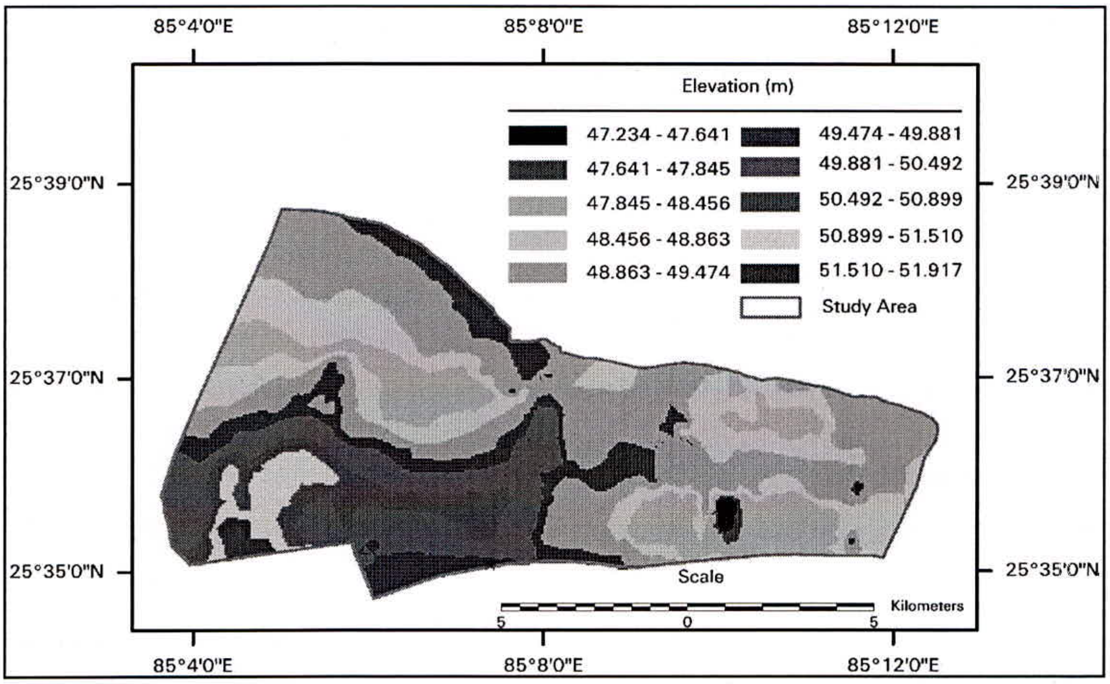


Fig. 5: DEM of the study area.

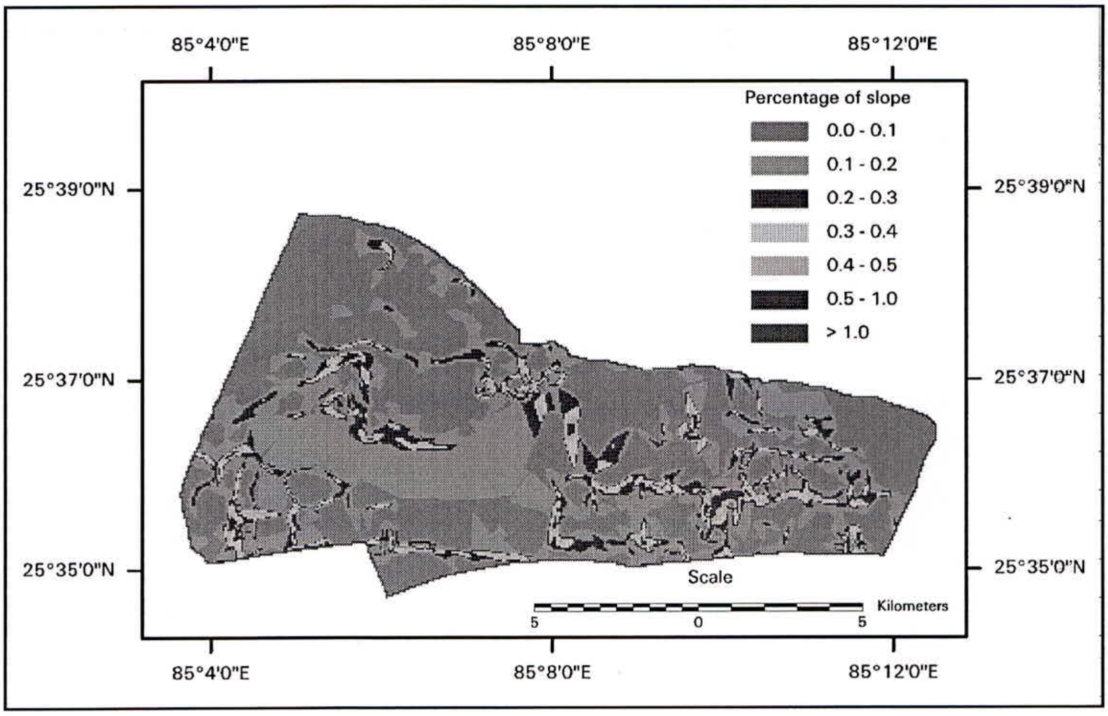


Fig.6: Percentage slope map of the study area

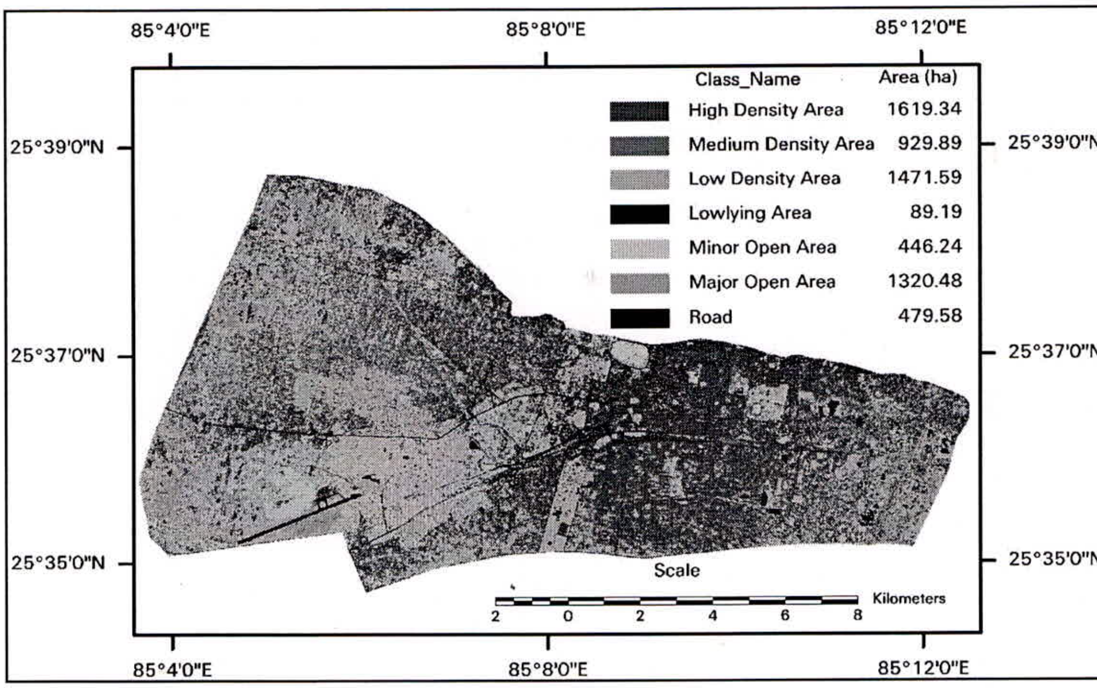


Fig. 7: Land use Classification using IRS-P6_L4MX data.

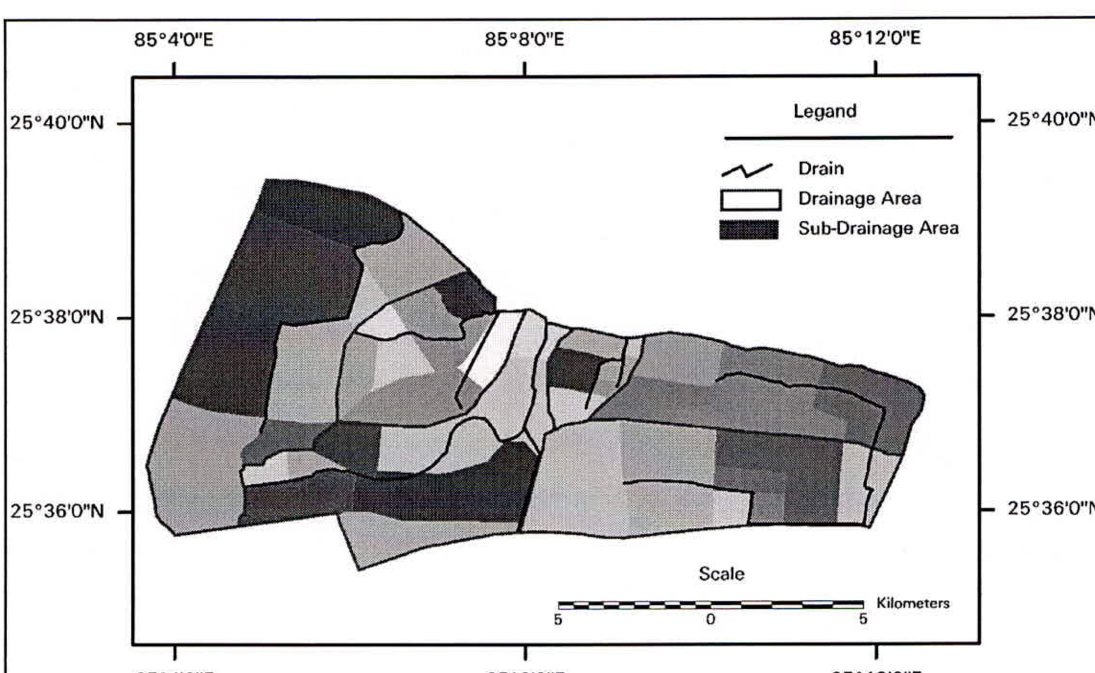


Fig. 8: Sub-catchments with drainage networks.

Table 4. Details of the storm drains and outfalls

Zone	Name of the Drain	Length of the drain (Km)	Name of the outfall
Western Zone	Kurjee	6.22	Kurjee
	S.K.Puri	3.30	Rajapur pooll
	Boring Road	2.06	
	Mandiri	8.36	Mandiri
	Mandiri Lateral (Mitapur)	2.12	
	Antaghat up NH Road	1.78	Antaghat
	Antaghat Lateral	0.72	
Central Zone	Agmkhan	5.78	Phadi
Southern Zone	Kankarbagh	5.28	

Model Simulation and Calibration

Calibration is the process of running a model using a set of input data and comparing the model results to actual measurements of the system. The calibration procedure takes into consideration the volume and peak rate of the event as well as the shape of the hydrograph. After the model is calibrated to a specific storm, it is validated by simulating one or more additional storms and comparing modeled and measured results. The model was calibrated using rainfall data obtained from the IMD Patna.

Initial calibration efforts showed that the model over-predicted due to conservative estimates of imperviousness or contributing area of the drainage basins. Thus, a detailed GIS analysis and field investigation was undertaken to determine the effective impervious area. The field investigation produced accurate delineation of drainage boundaries.

The objective of the application of a catchment model system is generally to determine peak flow rate and hydrograph shape. The evaluation criteria of relative error (RE) and root mean square error (RMSE) have been used to compare the simulated model output with the observed data.

Relative error (RE) for an arbitrary variable x

$$RE = \frac{x_o - x_s}{x_o} \tag{30}$$

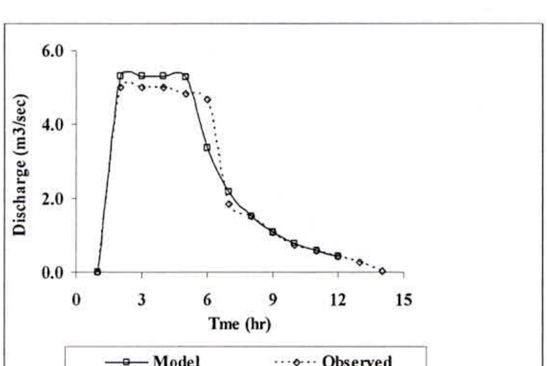
where x_o is observed value of a hydrograph characteristic and x_s is the simulated value of the same characteristic.

Root mean square error (RMSE) for discharge

$$RMSE = \sqrt{\frac{\sum_{i=1}^{n} [Q_o(i) - Q_s(i)]^2}{n}} \tag{31}$$

where $Q_s(i)$ and $Q_o(i)$ are the simulated and observed discharges, respectively, and, n is number of observations in the time series.

After calibration, the model was run and simulated the peak flow. The measured and simulated values of peak flow have been compared (Table 5). It is found that percentage errors at the four sites range from -0.90 to -6.20. The negative sign indicates that the simulated value overestimates the observed peak flow. RMSE criteria also imply that the prediction errors are well balanced. Comparison has been made between simulated and measured hydrographs (Figs. 9 to 12).



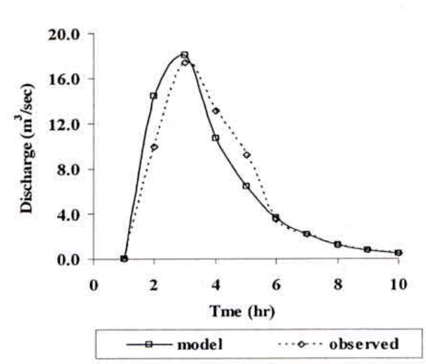


Fig. 9.Calibration for Kurjee outfall hydrograph

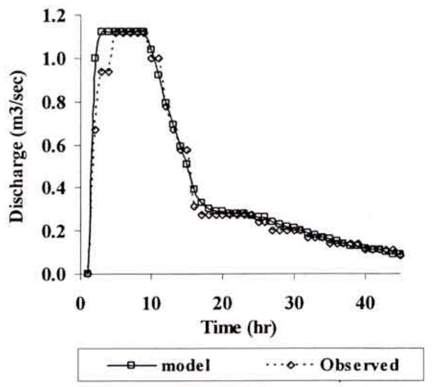


Fig.10. Calibration for Rajapur outfall hydrograph

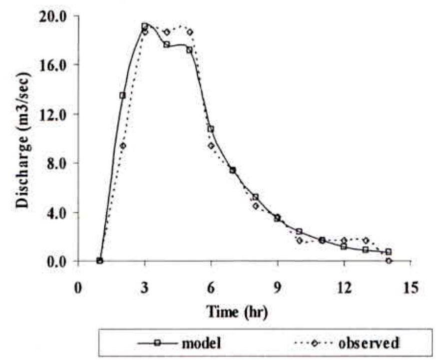


Fig.11. Calibration for Mandiri outfall hydrograph

Fig.12. Calibration for Pahadi outfall hydrograph

Table 5: Calibration of the model results with volume and peak flow

Sl	Name of the outfall	Observed peak flow (m^3/sec)	Simulated peak flow (m^3/sec)	RE (%)	RMSE
1	Krjee	4.99	5.3	-6.20	0.96
2	Rajpurl	17.36	18.04	-3.90	0.94
3	Mandiri	1.11	1.12	-0.90	0.97
4	Phadi	18.65	19.13	-2.40	0.96

Design Storm Analysis

The annual daily maximum rainfall data for the past 33 years (1975-2007) was obtained from IMD, Patna. SRRG charts of severe most storms were also recorded and analyzed. Gumble Extreme Value (EVI) distribution has been used to find out the maximum rainfall depth at different return periods (Table 6).

Table 6. Maximum daily rainfall depth for various return periods.

Return period	Maximum daily rainfall (mm)	Return period	Maximum daily rainfall (mm)
2	116.78	25	141.46
5	126.66	50	147.59
10	133.20	100	153.68
15	136.89		

Distribution of Rainfall

The intensity of rainfall usually varies on duration. Accurate representation of this temporal variation is important since runoff rates calculated from a rainfall runoff models are usually sensitivity to it. The U.S Department of Agriculture, Soil Conservation Service (SCS, 1986) developed synthetic storm hyetographs and the peak rainfall intensity was attained in the Type II curve. In general storm drainage system is designed for peak rainfall intensity, In the present study Type II curve and the 24 hours rainfall distribution for various return period values have been considered (Table 7). The hourly peak rainfall intensity more or equal to 60 mm/hr has been found in many years in 33 years of data.

Routing through the designed network with design storm

In the first hand analysis we applied the Kinematic Wave equation for flow routing to have an idea about the behavior of the existing drainage system. As this is a simplified approach that cannot deal with phenomena such as backwater effect, pressurized flow, flow reversal, and non-dendritic layouts, we adopted a Dynamic Wave routing procedure. The existing drainage system mostly consists of brick lining. The Roughness coefficient of the drainage system has been considered as 0.012 (SWMM user manual). Runoff generated from each sub drainage area is passed through a single out let which is connected to drainage node or junction, again it connects to the main drain and finally conveys the whole discharge to the outfall. Model simulation results of hydrographs of 2 year return period at various outfalls are shown in the Fig. 13.

Table 7: SCS Type II 24 hours distribution of Rainfall intensity with return period.

Time	SCS rainfall distribution Type-II for 24 duration

(hr)	2 year	5 year	10 year	15 year	25 year	50 year	100 year
1.00	1.28	1.39	1.47	1.51	1.56	1.62	1.69
2.00	1.40	1.52	1.60	1.64	1.70	1.77	1.84
3.00	1.40	1.52	1.60	1.64	1.70	1.77	1.84
4.00	1.52	1.65	1.73	1.78	1.84	1.92	2.00
5.00	1.87	2.03	2.13	2.19	2.26	2.36	2.46
6.00	1.87	2.03	2.13	2.19	2.26	2.36	2.46
7.00	2.34	2.53	2.66	2.74	2.83	2.95	3.07
8.00	2.34	2.53	2.66	2.74	2.83	2.95	3.07
9.00	3.15	3.42	3.60	3.70	3.82	3.98	4.15
10.00	3.97	4.31	4.53	4.65	4.81	5.02	5.23
11.00	6.42	6.97	7.33	7.53	7.78	8.12	8.45
12.00	***49.87***	***54.08***	***56.88***	***58.45***	***60.40***	***63.02***	***65.62***
13.00	13.20	14.31	15.05	15.47	15.98	16.68	17.37
14.00	5.72	6.21	6.53	6.71	6.93	7.23	7.53
15.00	3.62	3.93	4.13	4.24	4.39	4.58	4.76
16.00	2.92	3.17	3.33	3.42	3.54	3.69	3.84
17.00	2.57	2.79	2.93	3.01	3.11	3.25	3.38
18.00	2.22	2.41	2.53	2.60	2.69	2.80	2.92
19.00	1.87	2.03	2.13	2.19	2.26	2.36	2.46
20.00	1.75	1.90	2.00	2.05	2.12	2.21	2.31
21.00	1.40	1.52	1.60	1.64	1.70	1.77	1.84
22.00	1.40	1.52	1.60	1.64	1.70	1.77	1.84
23.00	1.40	1.52	1.60	1.64	1.70	1.77	1.84
24.00	1.28	1.39	1.47	1.51	1.56	1.62	1.69

Table 8: Surface flooding of the study area for various return periods.

Return period	**Rain fall (mm)**	**Surface flooding (ha-m)**	**Return period**	**Rain fall (mm)**	**Surface flooding (ha-m)**
2 Year	116.78	126.818	25 Year	141.46	194.980
5 Year	126.66	152.616	50 Year	147.88	213.750
10 Year	133.20	170.836	100 year	153.68	233.067
15 year	136.89	181.408			

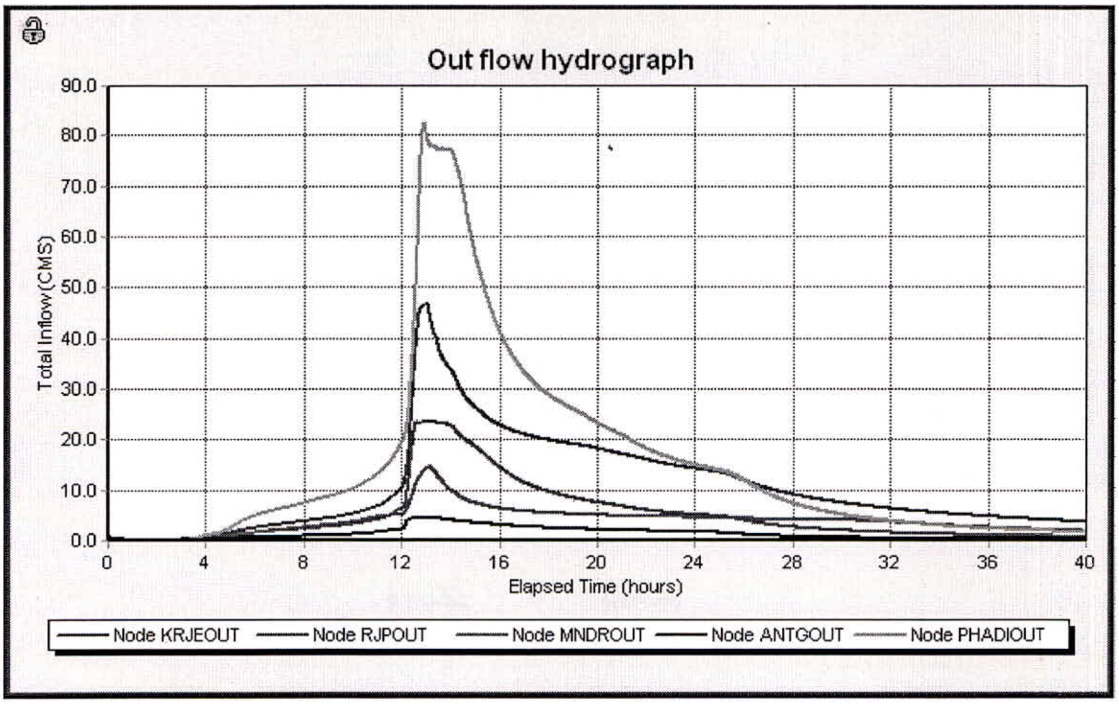


Fig. 13: Outfall hydrographs of 2 year return period hyetograph at various out lets

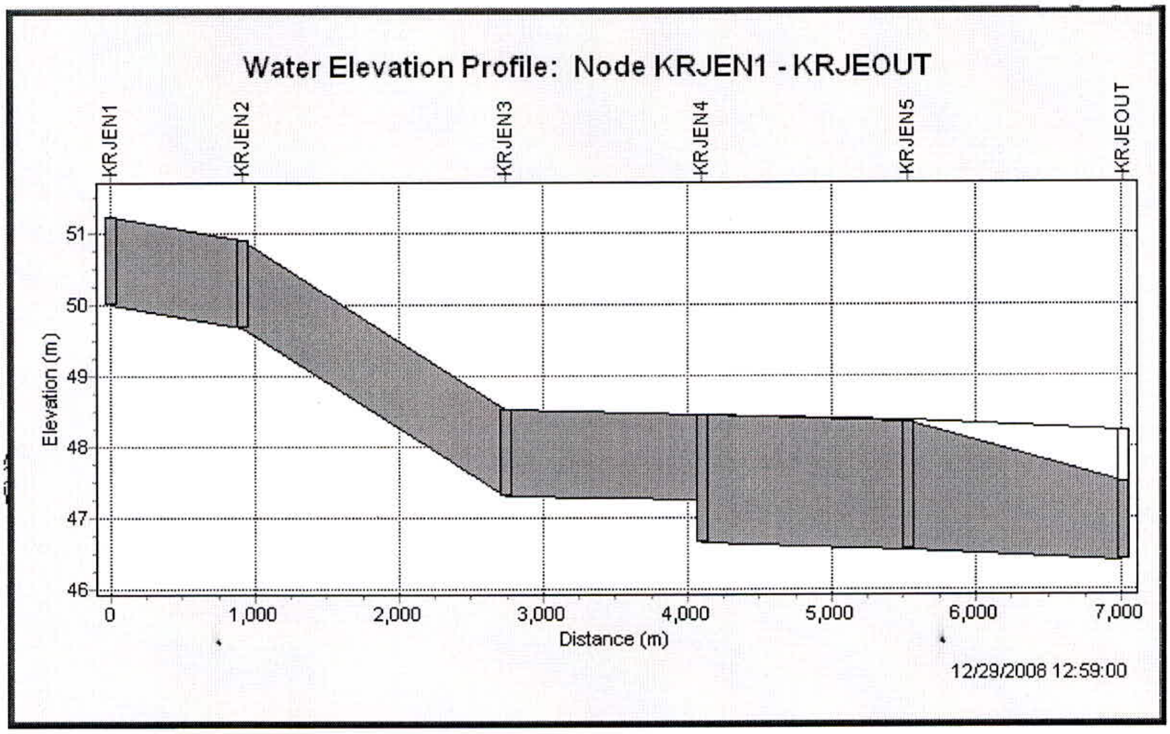


Fig. 24a: Water elevation profile of the Kurjee drain

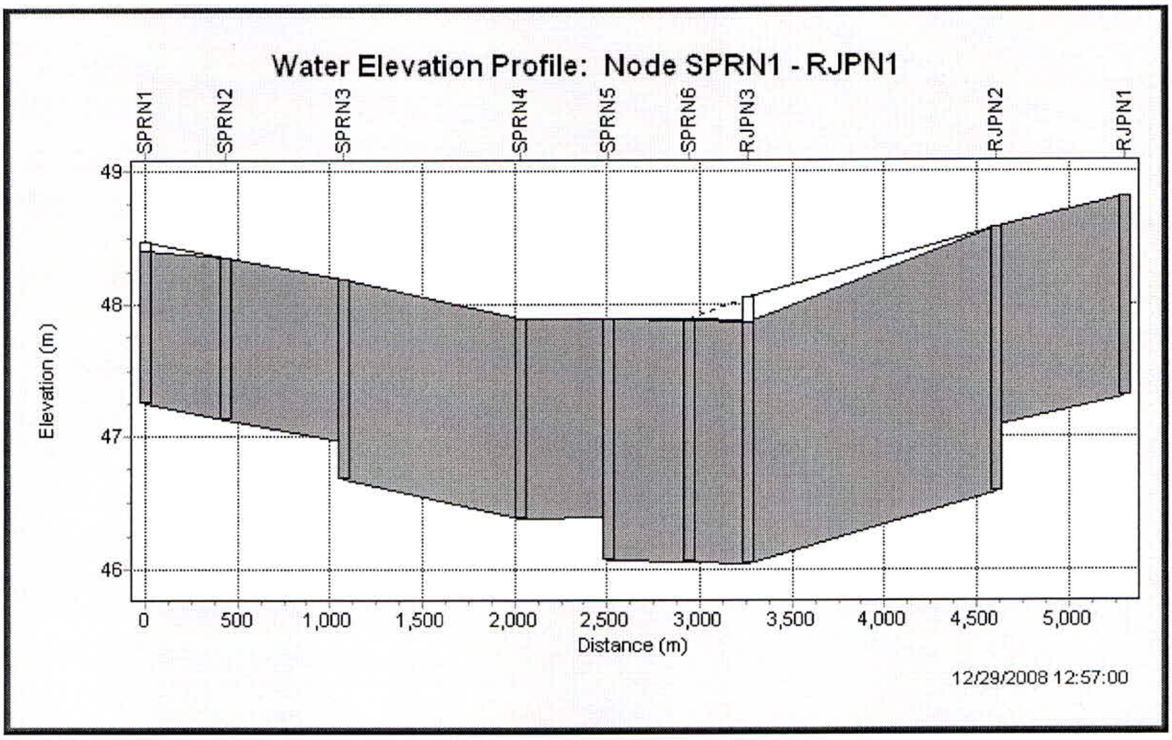


Fig. 14b: Water elevation profile of the SK puri and Boring canal drain.

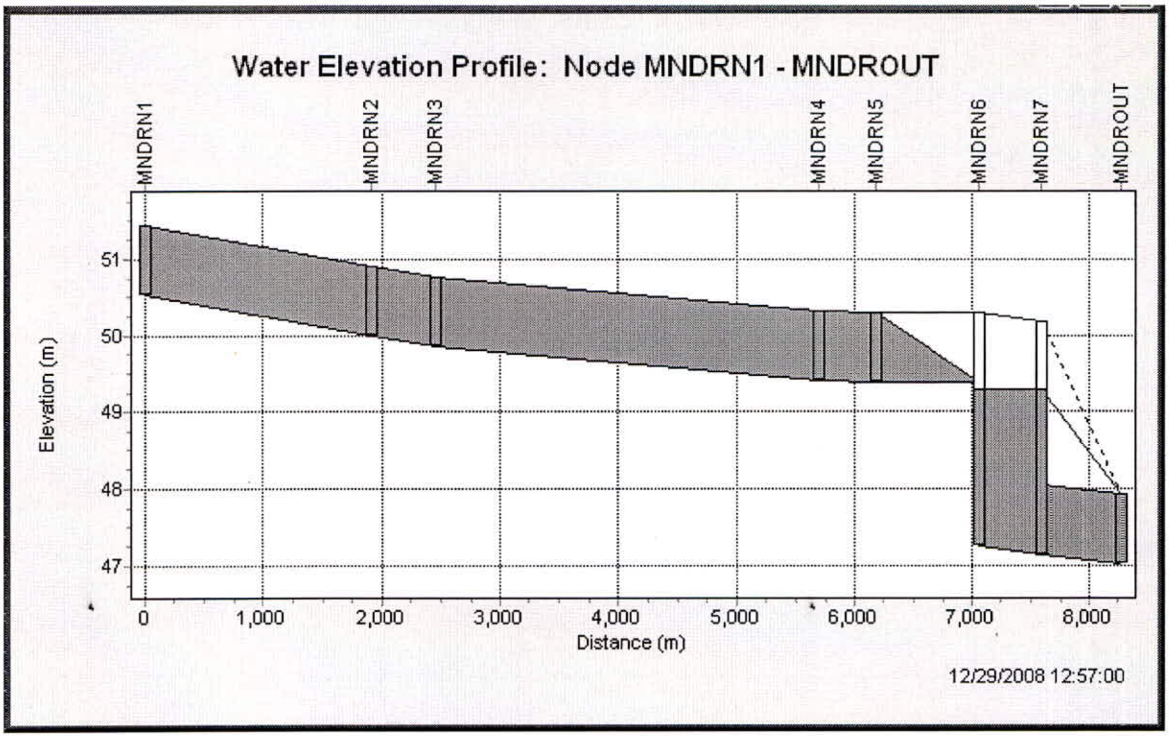


Fig. 14c: Water elevation profile of the Mandiri drain

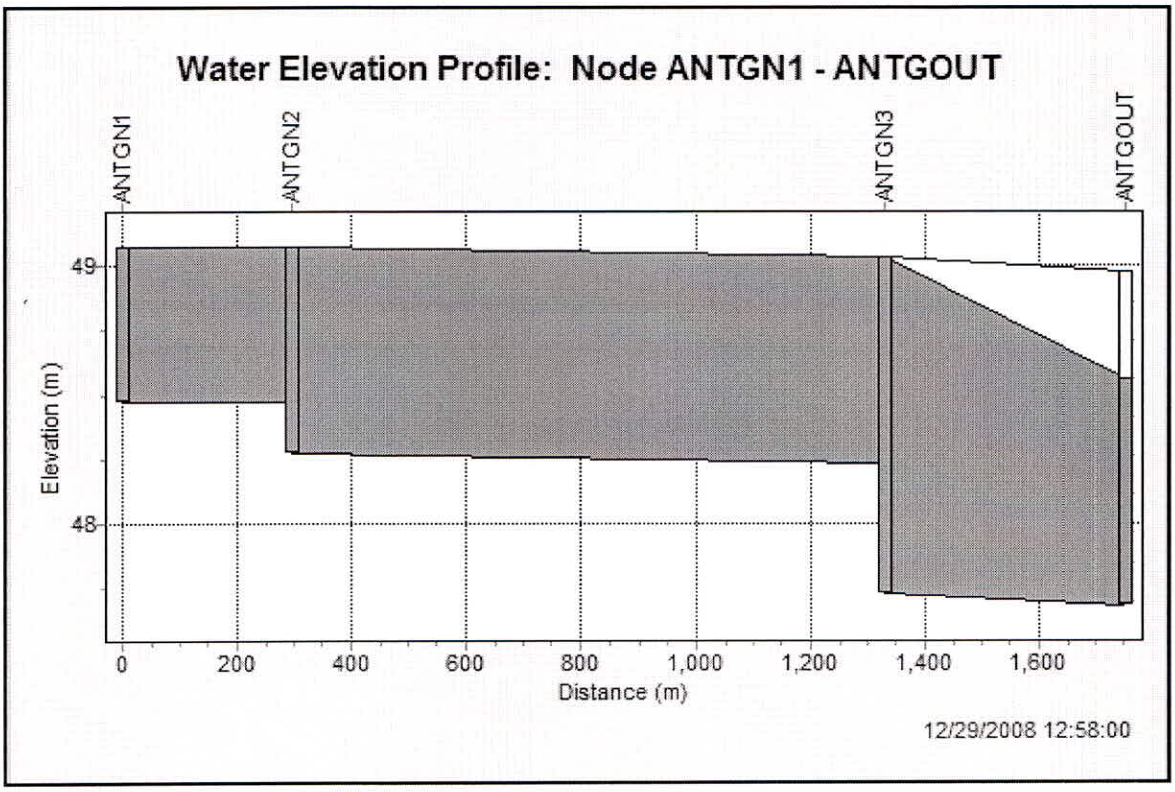


Fig. 14d: Water elevation profile Antaghat drain

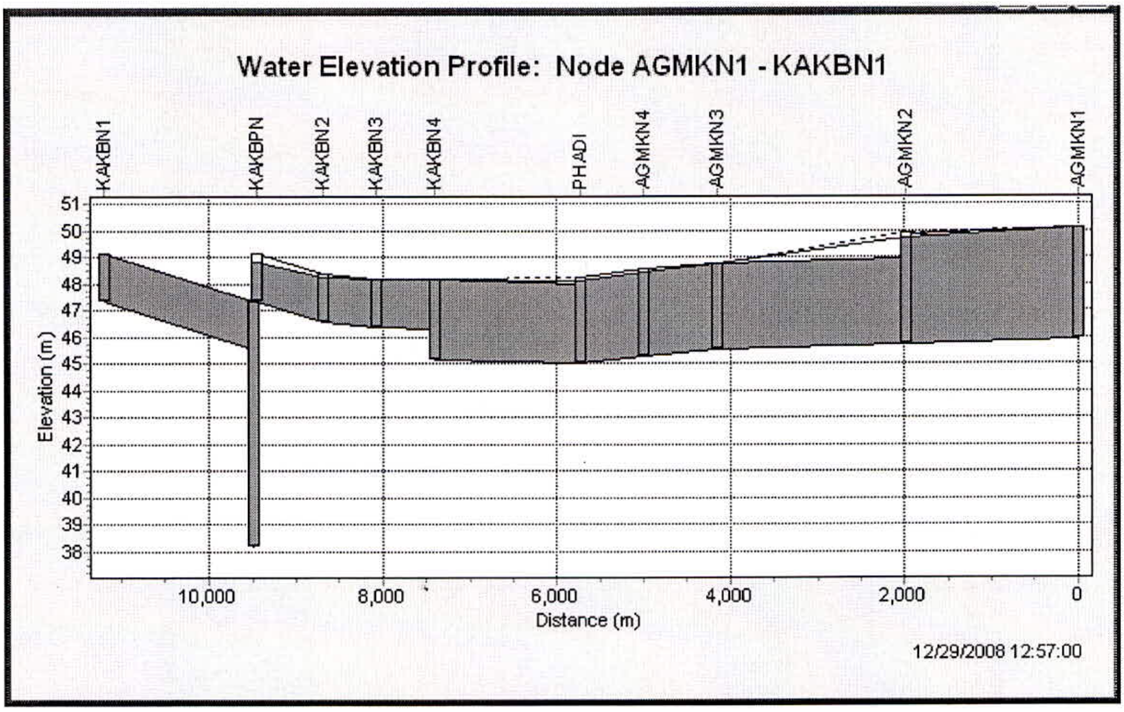


Fig. 14e: Water elevation profile of the Kankarbagh and Agumkhan drain

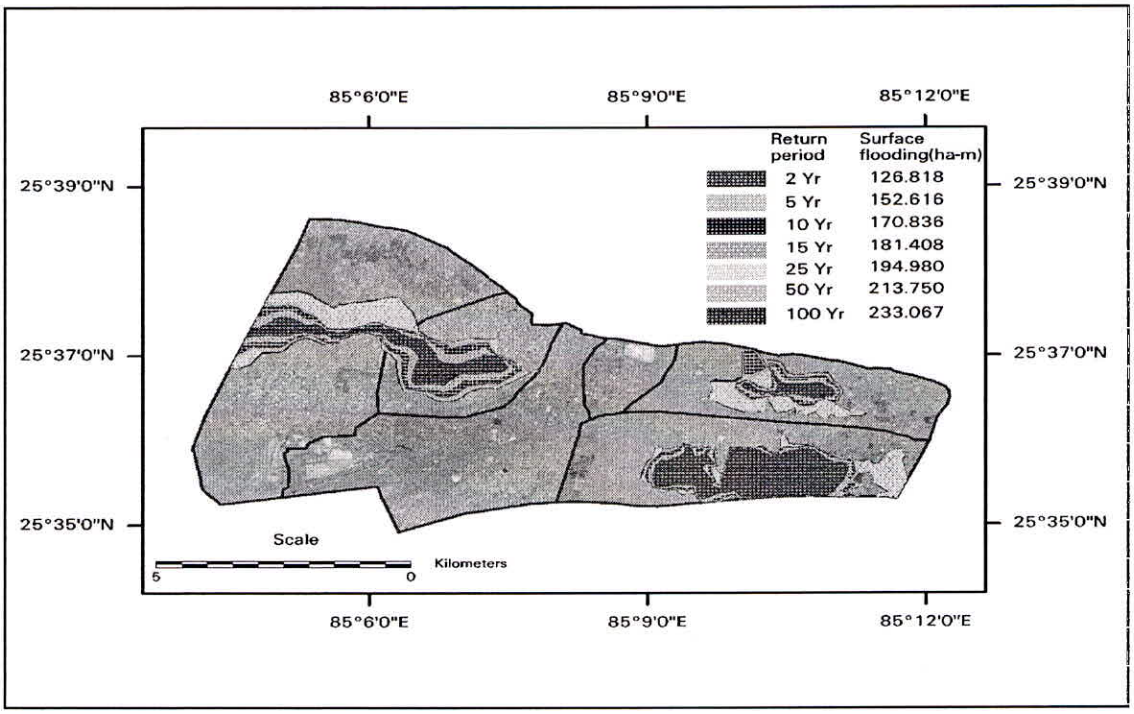


Fig. 15: Surface flooding for various design storm of the designed drainage network system.

In SWMM, flooding will occur whenever the water surface at a node exceeds the maximum defined depth of the drains. Normally such water will be lost from the system. The water elevation profile of drains (Figs. 14a to 14e), reveal that the drains are inadequate to carry flow to the outfalls. This flow spills out along the drains and accumulates in the low lying areas, causing flood inundation (Table 8). From the analysis we found that the existing drainage system is inadequate even to drain off runoff generated from 2 year return period rainfall. The inundation area with return period was identified. Using DEM and extent of the surface flooding, maps for various design storms were generated (Fig.15) based on (Eq. 28).

Design of Channel Capacity

The flow in drains are assumed free surface flow through prismatic channel having constant cross sections at different reaches. Manning's formula was used for design of channel assuming uniform flow (friction slope parallel to the bed slope) to accommodate peak flow. The existing drainage system was made several decades back. After subsequent developments in the study area there is no scope at present for widening the channel in most of the locations. The only way out to accommodate peak flow is by changing the geometrical properties of the drainage system without changing bed slope and roughness coefficient of the channel.

For design of the drainage system for 25 years return period hyetograph, spreaded uniformly over the entire area has been considered. Modified geometrical properties and proposed drain of the drainage system is given in the Table 9. From the analysis it reveals that modified design as proposed is adequate to convey the storm runoff to the outfalls without any spilling and flooding.

The shape, volume and peak rate of system inflow and outflow hydrographs (Fig. 16) are matching with the corresponding outfall hydrographs at various out lets (Fig. 17) Further, water elevation profile along the drains shows the zero flooding and without spilling at nodes of the drainage system (Figs. 18a to 18e). The modified designed system was verified with the observed rainfall (15 min interval) of 63 mm/hr *i.e* the maximum hourly peak on 13th August .2007, This rainfall intensity was even greater than the 25 years return period hyetograph peak value. The system result shows flooding has occurred only to an extent of 5.754ha-m (Fig. 19).

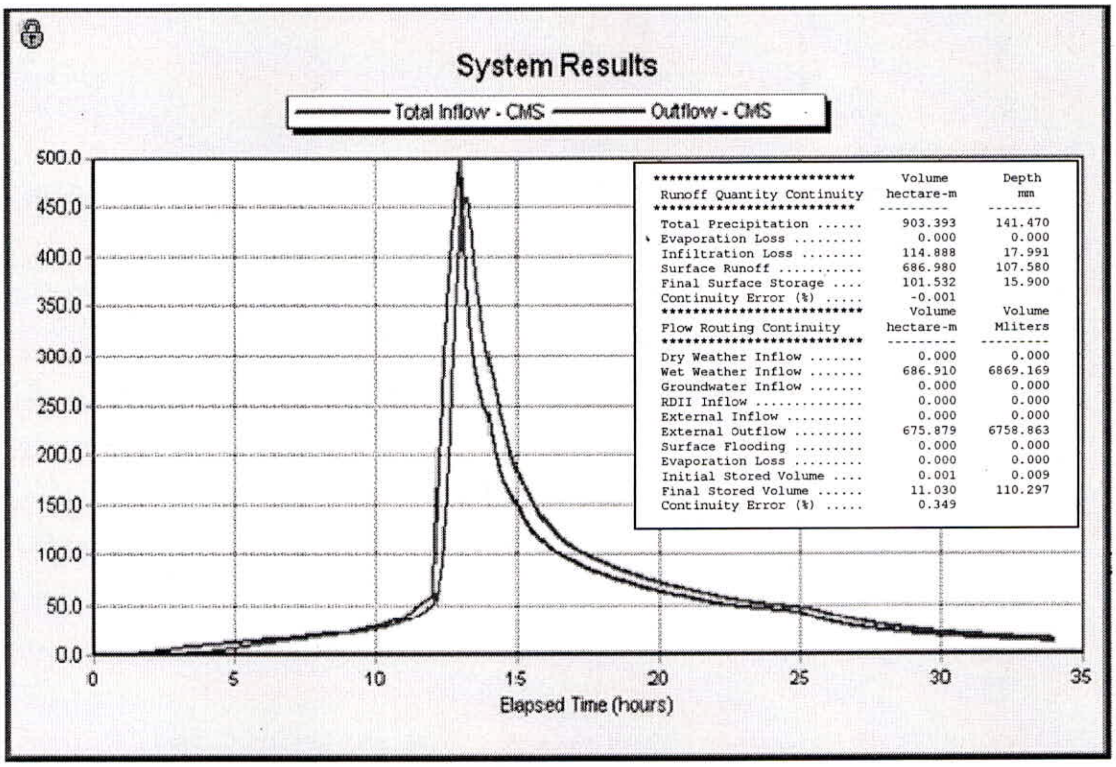


Fig. 16: System inflow and outflow hydrograph of the design storm 25 year return period hyetograph.

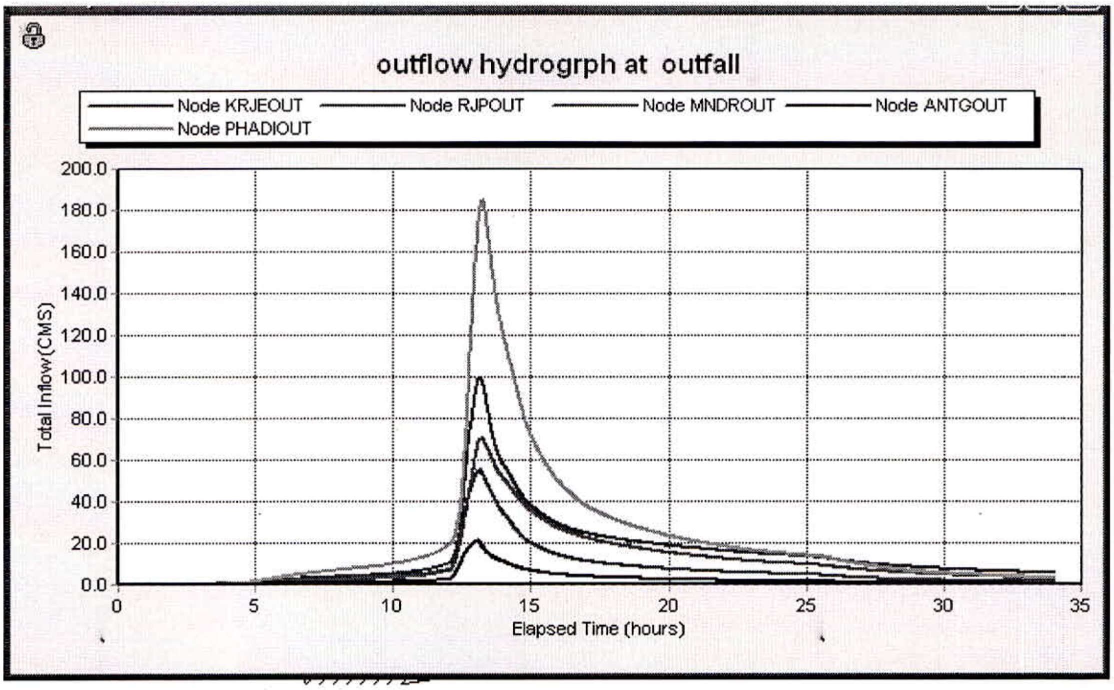


Fig. 17: Outfall hydrograph at various outfall of the drainage system.

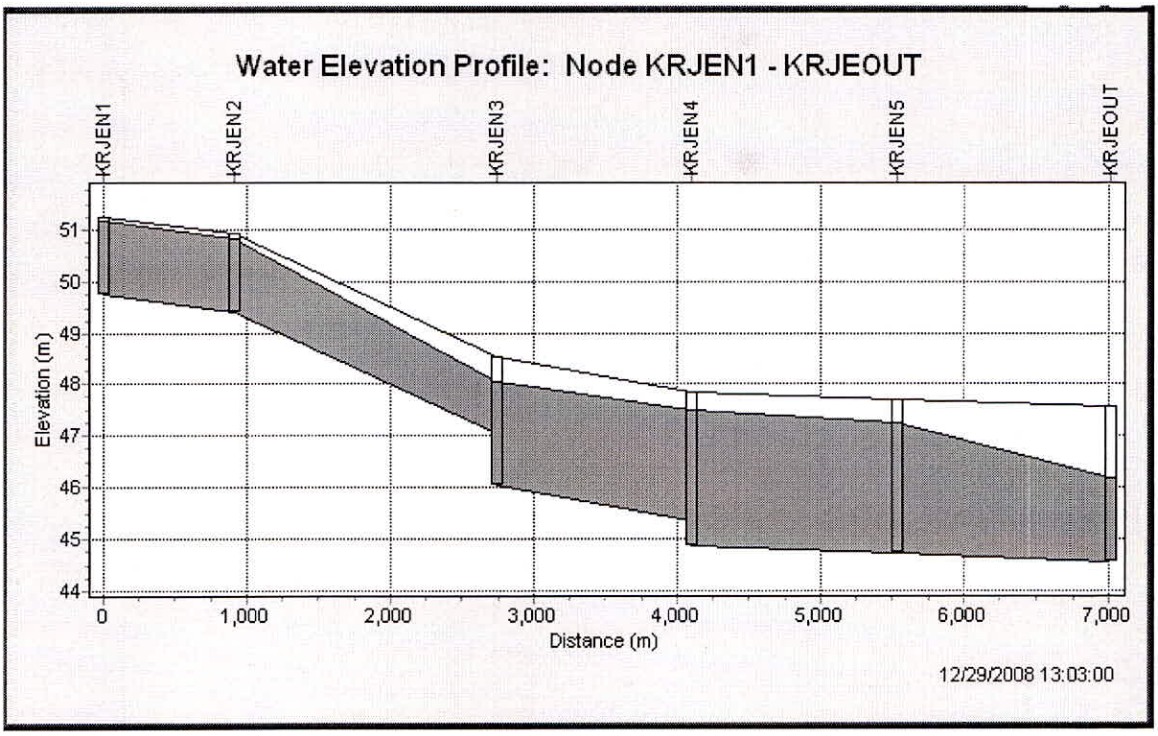


Fig. 18a: Water elevation profile of the modified Kurjee drain.

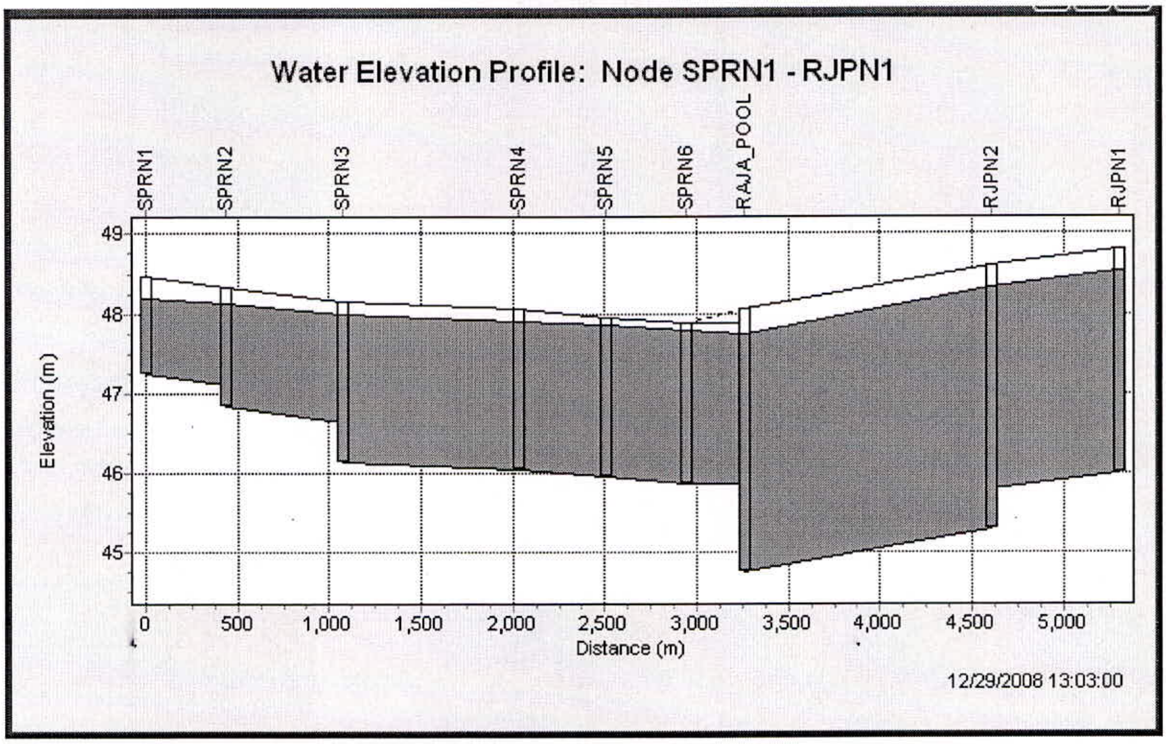


Fig. 18b: Water elevation profile of the modified SK Puri and Boring canal drain.

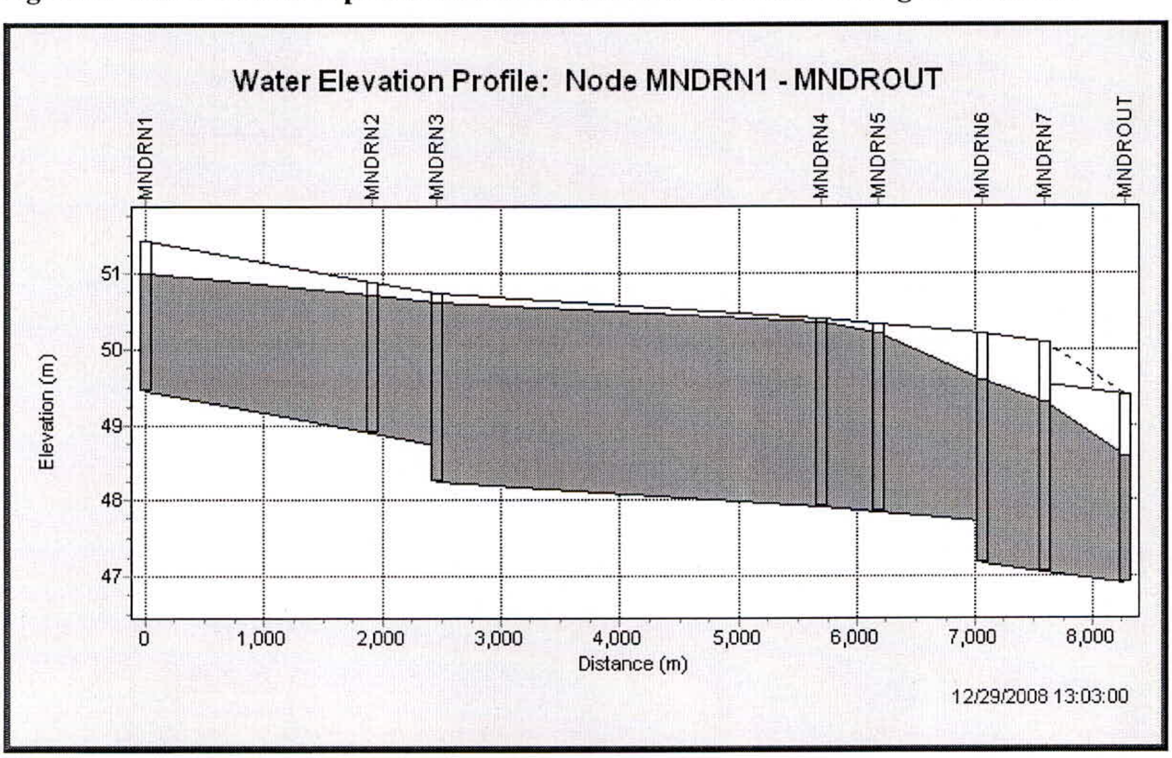


Fig. 18c: Water elevation profile of the modified Mandiri drain.

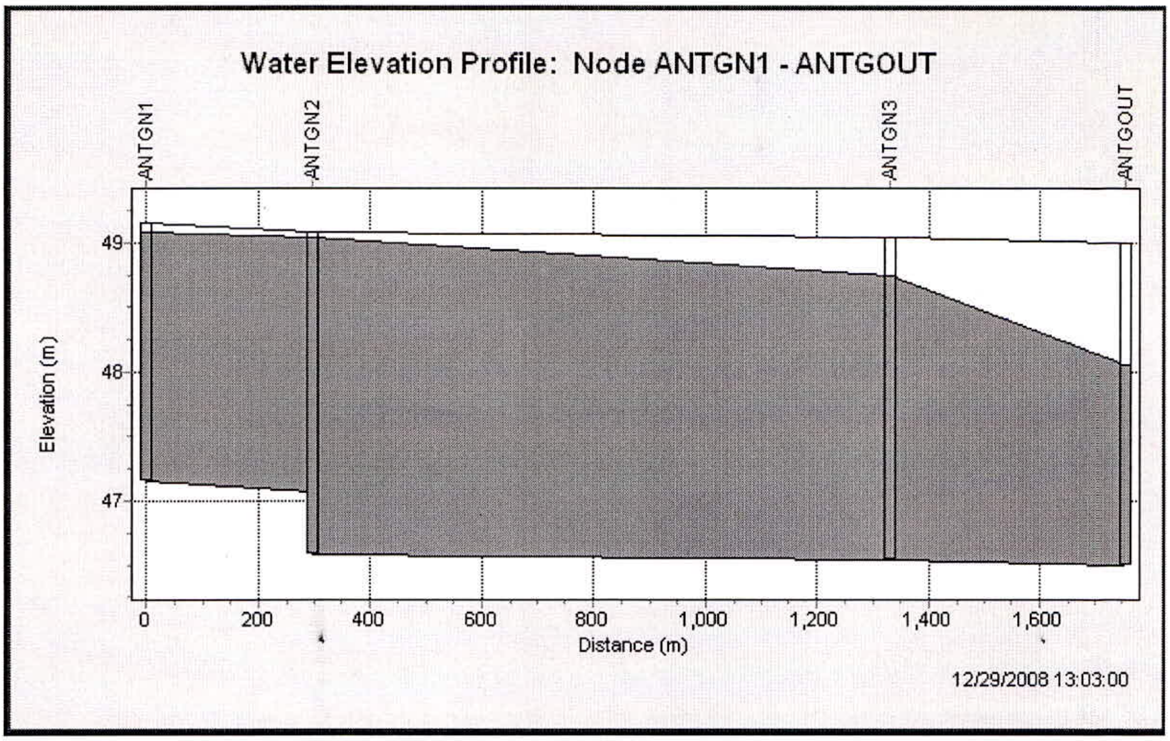


Fig. 18d: Water elevation profile of the modified Antaghat drain.

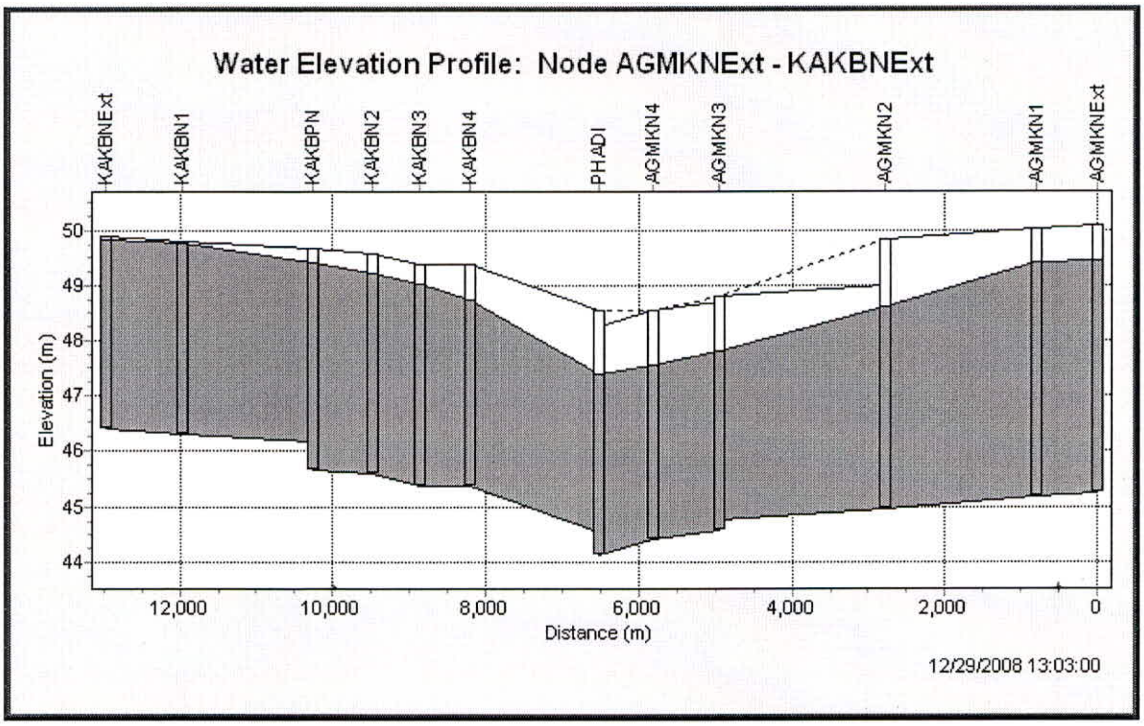


Fig. 18e: Water elevation profile of the modified Kankarbagh and Agamkhan drain.

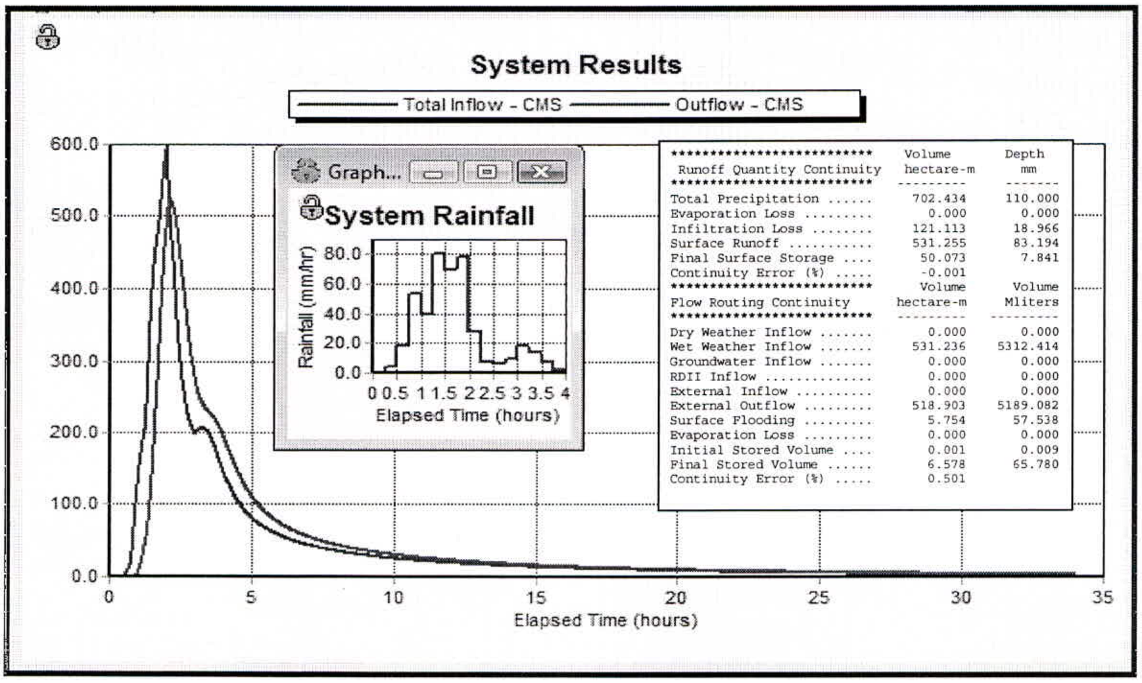


Fig. 19: System inflow and outflow hydrograph of the modified drainage system with observed hyetograph on 13th August 2007.

CONCLUSION

The storm drainage system of Patna Town has been analyzed in the present study. EPA SWMM has been used in GIS environment to simulate the storm runoff and the same has been routed through the storm drains. Thirty tree years of daily maximum peak rainfall data have been used to estimate design storms and analyzed the simulated runoff results.

The SWMM model was run to dispose of the present runoff generated considering the originally designed dimensions in British era of the drainage networks without any blockage and we found it is inadequate to dispose of the runoff. The condition of the existing drainage networks of Patna town is not even able to dispose off the rainfall depth of 2 year return period. As a result Patna town is facing flooding and water logging every year. The maintenance of the conveyance system is very poor and people are less concern about the health of the system. The drains are chocked due to throwing of garbage, polythene bags, wastages of vegetable and fish markets etc. directly into it. Thus people need to be awakened and attention should be given for renovation of the existing drains. Micro level drainage system is very poor and many areas are not connected to the main drainage system. Thus development of micro level drainage should be strengthened and connected to the main drainage system.

Simulation studies were performed on the originally designed drains to carry the existing runoff with various design storms. It is found that the discharge spills out of the original drains and the system is not adequate to carry the storm runoff to the outfalls. The flow is thus accumulated in lower elevation area and appears as surface flooding. Surface flooding maps have been prepared by using DEM for various design storms of the originally designed system.

In this study the geometrical properties of the drain (design) has been modified and extended the length of Shedpur and Kankarbagh drains. The simulated runoff with design storm of 25 year return period has been routed through the modified drainage system. Water surface profile along the drains has been developed and we found that this storm runoff is well accommodated in the drains. Again the model was run to simulate the flow with design storm greater than 25 years (considering intensity of rainfall events with 15 min duration) and it is found surface flooding of 5.25 ha-m only. Thus surface flooding has been reduced considerably for the modified drains. It is recommended that the drains need periodic cleaning and proper maintenance following the modified design criteria, the surface flooding may be reduced to a great extent.

The hydrographs at different outfalls have been developed for various design storms (2, 5, 10, 15, 25, 50 and 100 year) of Patna town. This is very useful information and recommended for Best Management Practice (BMP) for design of sumps or ponds and also to decide pump capacity.

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Table 9: St.Trapezoidal = Stepped Trapezoidal, Rect.open = Rectangle open, Rect.closed = Rectangle closed, Horiz. Ellipse = Horizontal Ellipse, Agmkhan Ext = Agamkhan Extension, Kankarbagh Ext = Kankarbagh Extension and L = lenhth

Conduit	Designed Drainage System					Modified designed Drainage System				
	Shape	Depth (m)	Area (sq.m)	Hydraulic Radaius (m)	Top Width (m)	Shape	Depth (m)	Area (sq.m)	Hydraulic Radaius (m)	Top Width (m)
Agmkhan Ext.	Proposed drain with 1Km length.					Trapezoidal	4.85	29.43	1.96	7.70
Agmkhan L1	St.Trapezoidal	4.19	21.75	1.70	7.70	Trapezoidal	4.85	32.74	2.05	7.70
Agmkhan L2	St.Trapezoidal	3.28	22.15	1.49	12.19	Trapezoidal	4.04	38.02	2.30	12.19
Agmkhan L3	St.Trapezoidal	3.28	22.15	1.49	12.19	Trapezoidal	4.14	42.54	2.42	12.19
Agmkhan L4	St.Trapezoidal	3.28	22.15	1.49	12.19	Trapezoidal	4.14	42.54	2.42	12.19
Antaghat L1	Trapezoidal	0.60	0.94	0.34	2.30	Rect. open	2.00	5.00	0.77	2.50
Antaghat L2	Trapezoidal	0.80	1.75	0.47	3.17	Rect. open	2.50	8.25	0.99	3.30
Antaghat L3	Trapezoidal	1.30	2.79	0.63	2.99	Rect. open	2.50	8.75	1.03	3.50
Antaghat L4	Trapezoidal	0.60	0.94	0.34	2.30	Rect. open	1.70	3.40	0.63	2.00
Antaghat L5	Trapezoidal	0.60	0.94	0.34	2.30	Rect. open	1.70	3.40	0.63	2.00
Kankarbagh Ext.	Proposed drain with 1Km length.					Rect.closed	3.50	28.00	1.22	8.00
Kankarbagh L1	Trapezoidal	1.80	6.53	0.94	5.43	Rect. open	3.50	33.25	2.02	9.50
Kankarbagh L2	Trapezoidal	1.80	6.53	0.94	5.43	Rect. open	4.00	34.00	2.06	8.50
Kankarbagh L3	Trapezoidal	1.83	14.66	1.28	10.03	Rect. open	4.00	32.00	2.00	8.00
Kankarbagh L4	Trapezoidal	1.83	14.66	1.28	10.03	Rect. open	4.00	32.00	2.00	8.00
Kankarbagh L5	Trapezoidal	3.00	18.68	1.58	9.45	Rect. open	4.00	32.00	1.33	8.00
KurjeeL1	Trapezoidal	1.22	3.29	0.67	3.90	Rect. open	1.50	10.50	1.05	7.00
KurjeeL2	Trapezoidal	1.22	5.17	0.80	5.48	Rect. open	1.50	11.25	1.07	7.50
KurjeeL3	Trapezoidal	1.22	8.87	0.93	8.54	Rect. open	2.50	25.00	1.67	10.00
KurjeeL4	Trapezoidal	1.82	17.18	1.35	11.26	Rect. open	3.00	45.00	2.14	15.00
KurjeeL5	Trapezoidal	1.82	25.52	1.47	15.84	Rect. open	3.00	45.00	2.14	15.00
Mandiri L1	Trapezoidal	0.91	4.16	0.67	5.48	Rect. open	2.00	10.00	1.11	5.00
Mandiri L2	Trapezoidal	0.91	4.16	0.67	5.48	Rect. open	2.00	10.00	1.11	5.00
Mandiri L3	Trapezoidal	0.91	4.16	0.67	5.48	Rect. open	2.50	20.00	0.95	8.00
Mandiri L4	Trapezoidal	0.91	4.16	0.67	5.48	Rect. open	2.50	20.00	1.54	8.00
Mandiri L5	Trapezoidal	0.91	4.16	0.67	5.48	Rect. open	2.50	23.75	1.64	9.50
Mandiri L6	Rect. open	3.05	29.74	1.88	9.75	Rect. open	3.05	29.74	1.88	9.75
Mandiri L7	Trapezoidal	0.91	4.16	0.67	5.48	Rect. open	2.50	24.37	1.65	9.75
Mitapur L1	Rect. closed	0.90	2.25	0.33	2.50	Rect. open	2.00	10.00	0.71	5.00
Boring canal L1	Horiz. ellipse	1.50	2.86	0.46	2.00	Rect.closed	2.80	8.40	0.98	3.00
Boring canal L2	Horiz. ellipse	2.00	5.08	0.61	3.00	Rect.closed	3.30	11.55	1.14	3.50
Boring canal L3	Horiz. ellipse	2.00	5.08	0.61	3.00	Rect.closed	3.30	11.55	0.85	3.50
SK Puri L1	Trapezoidal	1.22	5.95	0.84	6.10	Trapezoidal	1.22	5.95	0.84	6.10
SK Puri L2	Trapezoidal	1.22	7.44	0.89	7.32	Rect.closed	1.50	10.50	0.62	7.00
SK Puri L3	Trapezoidal	1.50	4.08	0.74	3.00	Rect.closed	2.00	17.00	0.81	8.50
SK Puri L4	Trapezoidal	1.50	12.75	1.13	10.00	Trapezoidal	2.00	21.00	1.48	12.50
SK Puri L5	Trapezoidal	1.83	17.86	1.36	11.59	Trapezoidal	2.00	21.00	1.48	12.50
SK Puri L6	Trapezoidal	1.83	18.79	1.38	12.00	Trapezoidal	2.00	21.00	1.48	12.50