respectively. Flow measurement and sampling frequency were approximately 15 minutes during all storm events.

Monitoring at the storm sewer locations were straightforward since measurements were done within a pipe in the downstream terminus of each subcatchment. Measurements conducted with the CSO chambers, however, were more complicated due to surcharging conditions at most CSO chambers and influences of tide gates. This problem was overcome by collecting data at two locations, the influent and overflow pipes of each chamber.

The increased effort required for collection of this additional data was well justified, since it provided a more reliable database for model calibration and verification.

The water quality variables monitored at both the CSO and storm sewer locations were similar to the data collected for the adoption of the river model. These variables include:

(a) Flow	(f) C-$BOD_{5\&20}$	(k) DO
(b) Temperature	(g) NH_3-N	(l) Organic Phosphate
(c) pH	(h) NO_2-N	(m) Orthophosphate
(d) Turbidity	(i) NO_3-N	(n) Total Phosphorus
(e) Suspended Solids	(j) TKN	(o) Fecal Coliform

The collected database was used in calibration and verification of the SWMM-4 model, which is described in detail in Chapter 4.

3.3 Tidal Marsh Studies

In addition to the field measurements collected at instream and sewer outfall locations, experiments were also performed at selected tributary and tidal marsh stations. The purpose of tidal marsh experiments was to quantify the potentially important role of tidal marsh areas as nutrient sources or sinks to the Lower Hackensack River.

Nutrient exchange processes in adjacent tidal marsh/mud flat regions may influence the dissolved oxygen level and overall water quality of an estuary such as the Hackensack. For example, marsh vegetation may assimilate or release significant quantities of nutrients, while marsh sediments may exert a significant oxygen demand upon overlying waters. Also, landfill sites within some of the marsh areas may discharge significant leachate loadings to the system. These nutrient loadings and oxygen demands from the lower, tidal marsh areas of the estuary may be transported by tidal action to the upper reaches of the estuary and may mask the effect of BCUA's outfall. For this reason, one must account for the relative contributions from these sources and sinks when modeling the individual impact of BCUA's discharges.

A monitoring plan was developed to incorporate these source/sink effects into the adapted estuary model. Towards this objective, nutrient fluxes were measured at the sediment/water interface and at the outlets of three representative tributary marsh systems (Sawmill Creek, Mill Creek and Berrys Creek Canal) of the estuary. Loadings generated through this methodology were extrapolated to the remaining landfills and marshlands within the study area to assess the loadings from the entire network of marshes/mudflats and landfills.

A river boat survey of the study area was conducted by the staff of Najarian Associates, L.P., on September 7, 1988. The survey team included Dr. Jay Taft of Harvard University, one staff member from General Testing Corp. and four staff members from Najarian Associates, L.P. The intent of the survey was to determine possible sampling locations within the relevant tributaries draining into the Hackensack River. The tributaries considered were Moonachie Creek, Mill Creek, Sawmill Creek, Berrys Creek and Berrys Creek Canal, respectively. Three of these, Mill Creek, Sawmill Creek and Berrys Creek Canal were targeted for further study. These three tributaries were selected because of their characteristics described below.

a. **Sawmill Creek**

Sawmill Creek is identified as being the tributary which has the greatest tidal exchange with the main Hackensack River. This tributary basin contains the largest active landfills (HMDC 1C and Balefill Sanitary Landfills) in the Hackensack River watershed. The basin also includes extensive tidal marshes and mudflats located between the landfill and the mouth of the Creek. To quantify nutrient contributions (or uptake) from these potential sources over specified tidal cycles, six sampling stations (S1, S2 S2A, S3, S4, S5) were installed within Sawmill Creek and four stations in Sawmill Creek (S2A, M1-M3 and N1) in the adjoining marshes and mudflats. The locations of all sampling stations are shown in Figure 3.4 and listed in Table 3.4.

b. **Berrys Creek Canal**

Berrys Creek Canal drains a large industrial/commercial watershed in its upstream end, a small tidal marsh in its central eastern section, and dredge spoil fill in its lower end. Sampling of sediments in the heavily polluted canal provided the means to quantify an additional source of oxygen demand in this industry dominated tributary. Three sampling stations (S6 to S8) were installed for Benthic sampling along the canal reach as shown in Figure 3.4.

c. **Mill Creek**

Mill Creek is the only tributary selected on the eastern bank of the Hackensack River. The Creek has a sewage treatment plant with capacities of 5.12 mgd of Level III effluent at its upper end. Tidal marshes flank the

Table 3.4 - Tributary Hydraulic and Water Quality Sampling Stations

Station	Location	Approximate Miles Upstream of Mouth	Qualifier	Parameters(1) Analyzed	Duration of Sampling (Days)	Frequency(2) Sampling (Events/Day)
S1	Sawmill Creek Channel at Mouth	0.0	Hackensack River Boundary #1	All,B	2	12
S2	Sawmill Creek North Channel	0.0	Hackensack River Boundary #2	All,B	2	12
S2A	Sawmill Creek Channel	0.7	Benthic Station	B	1	1
S3	Sawmill Creek Channel at NJ Turnpike Bridge	2.3	Instream Station	All,B,Q	2	12
S4	Ditch to Sawmill Creek near PSE&G Powerline culvert at HMDC 1C	2.8	Tributary Station	All,B	2	12
S6	Berrys Creek Canal	0.6	Benthic Station	B	1	1
S7	Berrys Creek above Route 3 bridge	1.3	Benthic Station	B	1	1
S8	Berrys Creek	2.3	Benthic Station	B	1	1
S9	Mill Creek above Mitigation site	0.4	Hackensack River Boundary	All,B	2	12
S9A	Mill Creek below STP	0.9	Instream Station	All,B	2	12
S10	Secaucus STP	1.0	STP	All,Q	2	6
S11	Mill Creek at Huber Street	1.2	Upstream Boundary Station	All,B	2	12
S14	Berrys Creek at NJ Turnpike Bridge	0.3	Hackensack River Boundary	All,B,Q	2	12
S15	Berrys Creek	1.0	Instream Station	All,B	2	12

(1) All Parameters as described in text; B - Benthic Station
(2) At equal intervals of time

entire length of the eastern side of the Creek. The lower section of the Creek is the site of an extensive marsh renovation project, which involves downgrading the marsh surface, removal of Phragmites and subsequent planting of Spartina. Four sampling stations were installed within Mill Creek, three in the Creek (S9-S11) and one station (S10) to monitor the effluent of the sewage treatment plant.

Review of the April data indicated the possibility of de-nitrification occuring in selected sections of the Hackensack River. Based on this review, and consultation with Dr. Jay Taft, de-nitrification experiments were conducted at two locations: one in Sawmill Creek mudflat (N1) and the other in the main stem of the Hackensack River (N2) (Part II, Appendix I).

Table 3.4 shows the sampling locations, frequencies and parameters analyzed. Tributary sampling was conducted over a period of two days (four tidal cycles) with a sampling frequency of about two hours. The parameters analyzed were:

(a) Salinity	(f) DO	(k) TKN
(b) Temperature	(g) Suspended Solids	(l) Total Phosphorus
(c) pH	(h) NH_3-N	(m) Organic Phosphorus
(d) C-$BOD_{5,20}$	(i) NO_2-N	(n) Ortho Phosphorus
(e) Chlorophyll-a	(j) NO_3-N	

Data was collected at hourly intervals during the period when the marsh was inundated with tidal waters (8 hours/cycle). Concurrent with water quality data, flows were measured entering and leaving the marsh/mudflat stations. From the measured flows and concentrations, instantaneous fluxes were computed. Integration of these fluxes over the corresponding tidal cycles yielded the mass transport of each constituent into or out of the sampled marsh/channel system. In areas where small channels could not be isolated for placement of suitable flow meters, flow was estimated using the adopted hydrodynamic model.

De-Nitrification experiments were started during the sampling period and continued in the GTC laboratories for 45 days. The methodology for this experiment was similar to the benthic procedure proposed by Dr. Jay Taft, and is presented in Part II, Appendix I.

All data collection and analysis was conducted by General Testing Corporation (GTC). The data in the tributaries and marsh/mudflats was collected during the first two weeks of November, a potential period of nutrient release from the marshes. This experiment was repeated during the months of July/August 1989 to determine the seasonal variation of the nutrient exchange process. Results of the Tidal Marsh Study are presented in detail in Part II of the report.

4. RUNOFF MODEL DESCRIPTION AND ADAPTATION

4.1 Land Surface Runoff Model Description

In this study, the pollutant loads generated from the extensively urbanized sections of the lower Hackensack River Basin via land surface runoff were computed using the SWMM-4 model. In adopting the SWMM model, the first step is the generation of an accurate description of both the short-term and long-term rainfall distribution within the watershed. This analysis was conducted using USEPA's rainfall model SYNOP (USEPA, 1976).

4.1.1 Analysis of Short-term and Long-term Rainfall Characteristics: SYNOP

To generate realistic stormwater and CSO loads, several rainfall gages were located within the watershed during the study period. Rain gages were located in the vicinity of all major CSO's and storm sewers monitored during the different storm events. This technique provided a greater spatial resolution of the rainfall data resulting in a more accurate calibration and verification of the model. In addition to these short-term stations, two long-term rainfall stations are located at the Northern and Southern boundaries of the basin at Oradell Dam, and Newark Airport, respectively.

The purpose of conducting a long-term rainfall analysis is to statistically characterize the different variables of interest (volume, duration, intensity and time between storms) of specific storms using historical records of local rainfall data. This differs from a rainfall analysis performed to generate peak flood hydrographs, where only extreme and rare events are of concern.

The two rainfall records were analyzed using the EPA rainfall analysis program SYNOP. This model is designed to generate relevant statistical data on the parameters of concern, using the long-term rainfall record. Thus, it is possible to interpret the characteristics of a particular storm of interest in relation to the long-term characteristics generated by the program.

To provide sufficient confidence in the rainfall characterization, a minimum of five years of continuous record is required by the model. In this study, 40 years of continuous, hourly rainfall data (1949-1989) were available at both gaging stations. These data were obtained on computer tape from the National Weather Service (NWS), in a format compatible with the requirements of the model. The model was then exercised over the 40 year rainfall record for each station.

The analysis and interpretation of the rainfall characteristics of the two major storm events that occurred during the river model calibration/verification effort in July and August of 1988 are presented in Table 4.1.

4.1.2 SWMM-4 Model Description

The SWMM-4 Model (Huber et al., 1988) is used to assess pollutant loadings to receiving waters from the entire watershed. These areas include storm sewered areas, CSO areas and portions of the watershed contributing direct surface runoff via tributaries and overbank flow. The quantity and quality of surface runoff are first computed and routed using the SWMM model. Generated outputs of the transport block of SWMM are then used as lateral loading inputs to the river model, MIT-DNM. **The SWMM-4 model was selected for this study since it is designed as a deterministic model, whereby parameters can be adjusted through the process of model calibration and verification with locally observed data prior to extensive application.** Furthermore, the routing of storm water can be simulated through the use of the transport block of SWMM, a feature not available in some other models like the STORM model developed by the U.S. Army Corps of Engineers. In the SWMM application, the size of the drainage basin under study is a somewhat critical parameter as it governs the routing characteristics affecting the transient behavior of receiving water. Although SWMM has other options such as storage and treatment,

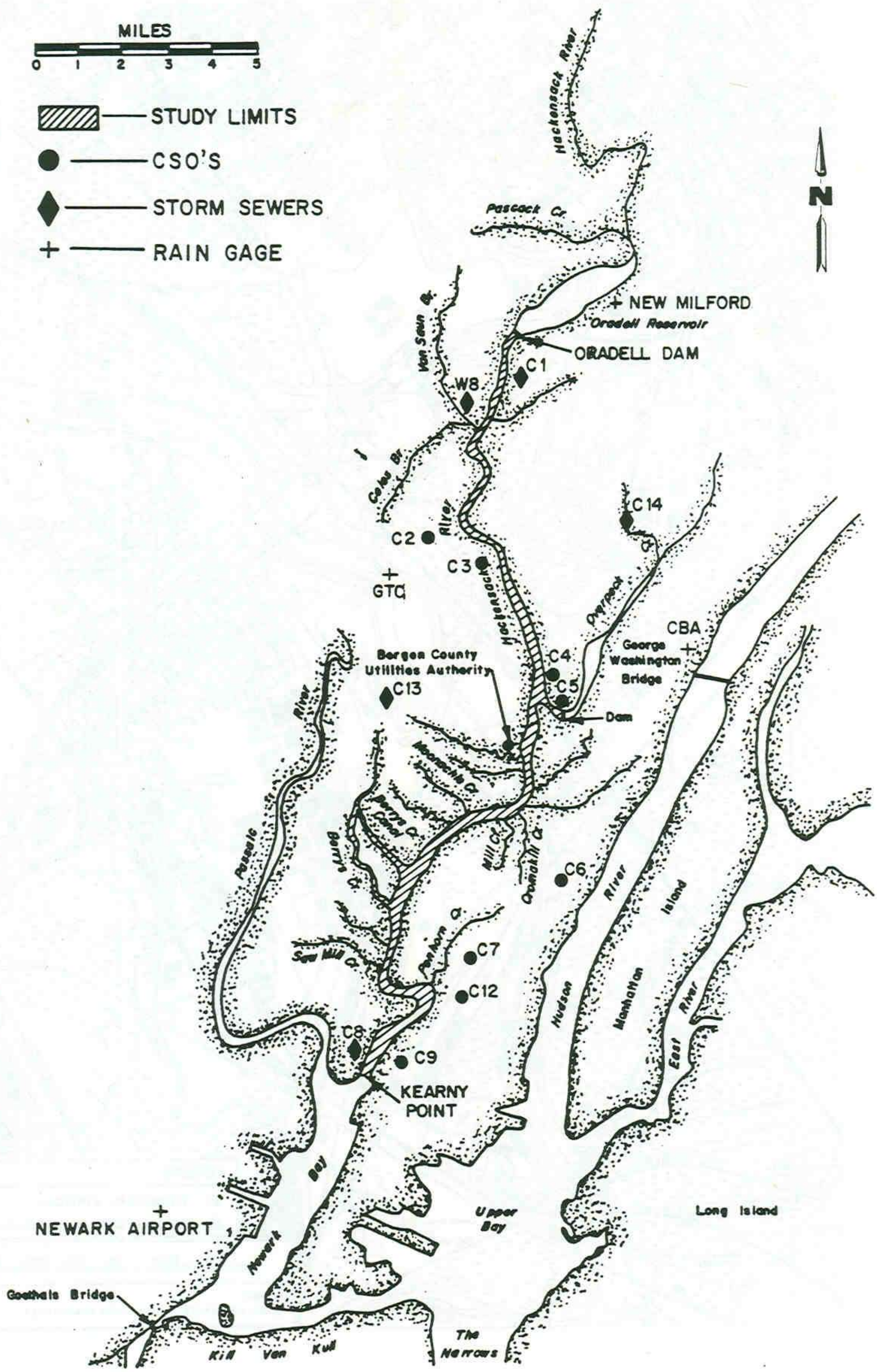

Figure 3.3 Storm Sewer & C&D Monitoring Locations, Lower Hackensack River Study

Figure 3.4: Marsh Study, Location of Monitoring Stations

Table 4.1 - Statistical Data on Two Observed Storm Events

STORM PARAMETERS	Storm Event #1 (July 19-22, 1988)		Storm Event #2 (August 24-25, 1988)		Median Storm*	
	New Milford	Newark	New Milford	Newark	New Milford	Newark
Total Volume (inch)	1.28	1.57	1.29	1.02	0.17	0.12
% Prob. < Volume	93	97	94	91.3	50	50
Return Period (mo.)	1.7	2.8	1.7	1.1	0.22	0.18
Duration (hours)	8	7	5	3	4	3
% Prob. < Duration	72	72	55	48	50	50
Return Period (mo.)	0.40	0.34	0.25	0.18	0.23	0.17
Mean Intensity (in/hr)	0.16	0.22	0.26	0.34	0.038	0.03
% Prob. < Intensity	92	97	96	98.8	50	50
Return Period (mo.)	1.32	3.0	2.9	8.3	0.22	0.19

* Based on 30-year rainfall record analysis: 1949-1979 at New Milford and 40-year rainfall record analysis: 1949-1989 at Newark Airport

NOTE: Interevent Time between storms = 3 hours receiving water effects and statistics, only the Runoff and Transport Blocks are used in the present study.

Description of the Runoff Block

The Runoff Block models the drainage basin by an aggregate of idealized subcatchments and gutters, which takes in a hydrograph of rainfall or snowfall and makes a step-by-step accounting of snow melt, infiltration losses in pervious areas, surface detention, overland flow, gutter flow, and the water quality constituents washed into inlets. This ultimately leads to the computation of a number of inlet hydrographs and pollutographs. The relevant sections of the Runoff Block are discussed below.

Runoff Quantity Computations

Two infiltration models are available in SWMM, namely Horton's equation and Green-Ampt's equation. The latter is selected for this application, since it is a technique based on accepted engineering theory.

$$F_s = \frac{S \cdot IMD}{i/K_s - 1} \quad \text{for } i > K_s$$

$$F_s = \text{infinity} \quad \text{for } i \leq K_s \tag{4.1}$$

$$f = i \quad \text{for } F < F_s$$

$$f = K_s \left(1 + \frac{S \cdot IMD}{F}\right) \quad \text{for } F \geq F_s \tag{4.2}$$

Where:

- f = Infiltration rate, ft/sec
- i = Rainfall intensity ft/sec
- F = Cummulative infiltration, this event, ft
- F_s = Cummulative infiltration volume required to cause surface saturation, ft

S = Average capillary suction at the wetting front, ft. water

IMD = Initial moisture deficit for this event, ft/ft

K_s = Saturated hydraulic conductivity of soil, ft/sec

At each time step the initial moisture deficit "IMD" is updated, and the saturation condition tested based on the calculated value of "F_s". Next, the infiltration rate, "f", is computed by the Green-Ampt equation in an integrated form, or is made equal to the rainfall intensity when the surface is not saturated.

In the Runoff Block, rainfall excess (i.e., rainfall/snowmelt intensity minus evaporation and infiltration rates) is routed to the downstream inlet of a subcatchment by overland flow or by rough gutters and pipes using the non-linear reservoir model. The outflow to the gutter/pipes and inlets is computed as:

$$Q = W \cdot \frac{1.49}{n} (d - dp)^{5/3} S^{1/2} \tag{4.3}$$

Where:

Q = Subcatchment outflow, cfs

W = subcatchment width, ft

n = Manning's roughness coefficient

d = water depth, ft

dp = depth of depression (retention) storage, ft

s = slope (ft/ft)

where the water depth d, is related to rainfall excess i* by integrating the equations:

$$A \frac{dd}{dt} = A \cdot i^* - Q \tag{4.4}$$

Where:

A = surface area of subcatchment, ft^2

i^* = rainfall excess, ft/sec

The subcatchment width, W, is a parameter sensitive to the timing of the routed storm, i.e., an increase in subcatchment width reduces the peak time of the runoff hydrograph. In other words, the shape of the runoff hydrograph is sensitive to the sub-catchment width. If the subcatchment is an idealized rectangular surface with a uniform slope, then the width of the subcatchment becomes the physical width of overland flow. If the gutter or pipe is located in the center of the subcatchment, dividing the subcatchment into two equal parts, the subcatchment width is twice the length of the gutter or pipe. In cases where the channel is off center, a skew factor is suggested as given in equation 4.5.

$$\gamma = \frac{A_2 - A_1}{A} \tag{4.5}$$

Where:

γ = Skew factor, O $\leq$ $\leq$ 1.0

A_1 = area to one side of channel, ft^2

A_2 = area to other side of channel, ft^2

A = total area, ft^2

Then the subcatchment width is adjusted as W = (2-) , where is the length of the main drainage channel in feet.

When drainage subareas are lumped into a smaller number of subcatchments in order to reduce the cost of computation, the total subcatchment width has to be reduced since the accuracy of the drainage network storage computations are lost during this aggregation. This limitation can be compensated by reducing the subcatchment width.

Runoff Quality Computation

Water quality constituents are generated in the Runoff Block basically by two mechanisms, build-up and wash-off. Water quality constituents are assumed to build-up on the land surface during periods of dry weather, preceding a storm event. Factors affecting build-up are rather complicated and vary according to land use and area. Variations depend upon type of land use, population density, traffic flow, street sweeping, etc. Build up generally accumulates with time until a certain limit is reached, and then is washed off during a storm event into the drainage system.

In the Runoff block there are two techniques of computing build-up. A straight-forward method is to specify independently, the forms and parameters of build up functions for each constituent simulated. Another method is to assume all pollutants to be related to the amount of dust and dirt accumulations. Once the proportions of pollutants related to dust and dirt are defined, the computations for build up are reduced to relationships associated with dust and dirt. The build up of dust and dirt is calculated by different functions and parameters for different land uses.

Three types of build ups are available in the model:

1. Power-linear $M = at^b$ (4.6)

2. Exponential $M = M_o(l-e^{-ct})$ (4.7)

3. Michaelis-Menton $M = \frac{M_o t}{(a+t)}$ (4.8)

Where:

M = accumulated mass built up, lb/acre or lb/100 ft-curb

M_o = upper limit of mass built up, lb/acre or lb/100 ft-curb

t = time, day

a,b,c = parameters to be calibrated

The upper limit of build up, or the asymtotic build up value is also supplied by the user for the power-linear type build up. In this study, the Michaelis-Menton formulation was selected to compute the pollutant build up.

Wash-off is simulated in SWMM using an exponential decay relationship as a function of runoff rate and time, whereby the pollutant mass that remained on the surface at the end of a time step is determined as

$$m(t+\Delta t) = m(t) \cdot \exp\left[-C_w \cdot \frac{r(t)^k + r(t+\Delta t)^k}{2} \cdot \Delta t\right] \tag{4.9}$$

Where:

$m(t)$ = mass remaining on the surface at time t

C_w = wash off coefficient, constant

r = runoff rate

k = constant

δt = time step

Consequently, the concentration of pollutants at the end of the time step is derived from Equation 4.9.

$$C(t+\Delta t) = \frac{1}{Q}\frac{dm}{dt} = \frac{m(t+\Delta t) \cdot r^{k-1}(t+\Delta t)}{A} \tag{4.10}$$

Where:

$c(t + \Delta t)$ = concentration at the end of time step

A = the area of subcatchment

Varying the values of the two parameters C_w and k can change the shape and value of wash-off and concentration profiles. Hence, careful adjustment of these two values is essential to obtain a satisfactory calibration. Detailed discussion concerning the effects of these two parameters on wash-off computation is elaborated upon in the SWMM User's Manual, Version 4 (Huber et al., 1988). The model also provides the option of supplying a user specified rating curve for particular applications where the exponential decay function does not represent the system to be modeled.

Description of the Transport Block

Although the Runoff Block provides moderate routing ability using the non-linear reservoir method, applications are limited to small subcatchments or to small drainage channels. Another facility in SWMM, namely the Transport Block, is employed in the stormwater routing simulation. The Transport Block, based on kinematic wave theory, receives the resulting hydrograph and pollutograph from the Runoff Block at inlets of each subcatchment. It then routes them downstream through the sewer system, represented by elements of pipes, conduits and manholes, and produces routed hydrographs and pollutographs at outlet manholes. The Transport Block also accounts for internal storage, sewer infiltration and dry weather flow. Surcharge situations are also simulated by keeping excess runoff water at an upstream element until the conduit reclaims its stormwater carrying capacity.

4.1.3 Runoff Model Schematization

The basic components of the lower Hackensack River Basin are incorporated into the SWMM-4 formulation through model schematization techniques. Schematization or "discretization" refers to the development of a simplified representation of the watershed system in which actual sub-basins and streams are replaced by a series of idealized components called "model subcatchments", "channels", and "pipes". A model subcatchment is defined as an idealized runoff area having spatially uniform hydrologic and hydraulic properties (e.g., infiltration rates, detention depth, ground slope, roughness, etc.) and a uniform hydrologic and water quality response. Likewise, a channel is defined as a non-tidal stream segment having spatially uniform hydraulic properties and pollutant constituent concentrations. These averaged hydraulic properties are incorporated into the SWMM model through the input of representative model parameters. A network of subcatchments, channels, and pipes is developed to approximate the hydrologic and hydraulic components of a watershed system.

In general, drainage basins with detailed layouts of storm and combined sewer systems were lumped into simplified, single-channel subcatchments. This procedure is recommended by Zaghloul (1981) for cases where bulk loading effects are of primary concern. A detailed schematization of a drainage basin into a network of sewer pipes along with corresponding subcatchments would result in more accurate peak flows and timing with the expense of labor and computing cost. This approach is necessary when the main modeling concern is flooding and surcharging of the drainage system. Also, the Transport Block and in many cases, the Extran Block of SWMM using fixed time steps in the order of seconds are needed to maintain the stability and accuracy of the model. Thus, in most applications, only short term simulations (single event or design storms) are practical in such a detail schematization. On the contrary, the Runoff Block of SWMM Model, with certain routing capability, generally uses larger computational time steps (5 minutes to an hour). It can also automatically switch to a much larger time step during periods of dry weather to save computing costs. These features are superior for the purpose of continuous, long-term simulations. In this study, the major objective of watershed modeling is to predict long-term pollutant loadings to the recieving water from storm water runoff. Therefore, it is resonable to employ the lumped scheme and utilize the full capability of the Runoff Block to minimize schematization and computing efforts.

In the case of the Overpeck Creek, a major tributary of Lower Hackensack River with three major branches in the upstream reaches and a large detention storage reach at downstream, Transport Block of SWMM was rigorously applied. The Overpeck drainage basin was schematized into four subcatchments (see figure 4.2). Runoff block was first applied to each subcatchment to generate a hydrograph and pollutograph individually. Next, the Transport Block was used to route upstream hydrographs and pollutographs to the downstream reaches. Storage and weir elements in the Transport Block were used to simulate the backwater effect due to a tidal dam 4,500 feet upstream of its confluence with the Hackensack River. Furthermore, the Combine Block was used to combine hydrographs and pollutographs at the confluences of different reaches.

Figure 4.2 is a schematic representation of the entire lower Hackensack River Basin. It consists of 23 subcatchments, 6 CSO areas, 3 channels, and numerous model segments (reaches) of the Hackensack River watershed. The subcatchments of the watershed were delineated (Figure 4.1) on U.S. Geological Survey topographic maps based on the natural drainage divides of the Lower Hackensack River Basin.

4.1.4 SWMM-4 Model Input

Upon schematizing the entire watershed into discrete sub-basins, information regarding land use, soil type, meteorology, channel and catchment geometry, etc., were developed as required by the model input. Precipitation data was assembled at four stations indicated in Figure 4.1; namely, at Newark Airport, Hackensack Water Company (HWC), New Milford, General Testing Corporation (GTC) office in Hackensack, and Clinton Bogert Associates, Inc. (CBA) at Fort Lee, respectively. At the Newark Airport and HWC, data was obtained from the National Weather Service (NWS). At the remaining two stations, fifteen-minute data were collected by GTC and CBA. In this study, monthly evaporation rate estimates for the watershed were based upon published NWS data at Newark Airport. Portable rain gages were provided in the latter part of collection in the vicinity of all CSO and storm sewer stations to provide greater spatial resolution of the rainfall data.

A set of representative model input parameters assembled on the basis of local site conditions and literature values are provided in Table 4.2. As indicated in this table, most parameter values selected for this study lie within a range reported in the literature. The relevant parameter selection techniques for the present study are discussed below.

Geometric parameters - such as subcatchment areas, lengths, widths, and slopes - were estimated from detailed topographic maps obtained from the USCE and HMDC. Parameters for channel geometry also were based on these maps, and additional field surveys conducted by Najarian Associates, L.P. during the spring of 1989.

Parameter values for "percent impervious" were based on land use distribution data for the basin. To this end, an inventory of the land use data described in section 2.1.2 was assembled for each individual subcatchment, based on recent land use survey and zoning maps compiled by the Bergen County and Hudson County Planning Boards. Land use data was digitized and entered into a geographical information system, "SAGIS" (U.S. Fish and Wildlife Service), to determine the percent coverage within each subcatchment area of the following composite land use groups: low density residential, high density residential, industrial, commercial, and open space.

Table 4.2 - Input Parameter Ranges for the Adapted SWMM-4 Model

Parameter Symbol - Name	Present Study Values	Literature Range
WW(3) - Percent imperviousness of subcatchment	32.6-87	0-100
WW(5) - Impervious area Manning's roughness	0.023	0.01-0.036
WW(6) - Pervious area Manning's roughness	0.3	0.01-0.48
WW(7) - Impervious area depression storage (in)	0.015	0.005-0.11
WW(8) - Pervious area depression storage (in)	0.06	0.06-0.25
SUCT - Average capillary suction (in)	8.96-11.97	4-12
HYDCON - Saturated soil hydraulic conductivity (in/hr)	0.032-0.165	0-0.45
SMDMAX - Initial soil moisture deficit fraction	0.211-0.286	0.21-0.34
CBVOL - Average individual catchbasin storage	16 (ft^3)	2.8-78
DRYBSN - Catchbasin recharge time (days)	5	--
WASHPO - Washoff exponent	1.8	0.8-2.6
RCOEF - Washoff coefficient	40	0.052-100
CBFACT - Initial catchbasin BOD conc. (mg/l)	1.0	5-1500
CBFACT - Initial catchbasin NH_3-N conc. (mg/l)	0.05	0.75-9.0
CONCRN - BOD Concentration in precipitation (mg/l)	1.0	4-22
CONCRN - NH_3 Concentration in precipitation (mg/l)	0.05	0.01-0.04

Individual percentages of imperviousness associated with each of these land use groups were then prescribed as follows: Low density residential (38%), high density residential (65%), industrial (72%), commercial (85%), and open space (0%). These latter percentages were obtained from published Soil Conservation Service data (U.S. Department of Agriculture, Technical Release No. 55). A weighted-average of the parameter called "percent imperviousness" was computed for each subcatchment area by summing the products of these imperviousness percentages with corresponding land use composition data. Table 4.3 shows an example spreadsheet used to calculate the composite percentage of imperviousness, soil infiltration parameters and build-up rates for a specific subcatchment.

In an analogous fashion, percent coverage of each hydrologic soil group within each catchment was computed by application of the SAGIS geographical information system. These percent coverages, along with other pertinent data, were used to determine representative Green-Ampt soil infiltration parameters such as capillary suction, hydraulic conductivity, and soil moisture deficit. Thus, the weighted-average capillary suction parameters for each catchment were computed from these areal percentages and from typical suction values for each soil group given by Carlisle et al. (1981) (Table 4-11 of the SWMM-4 users manual). Likewise, representative values for the saturated hydraulic conductivity parameter were computed from the areal percentages and typical permeability values for each soil group given by Musgrave (1955) (Table 4-7 of the SWMM-4 users manual). Finally, weighted-average soil moisture deficit parameters were computed for each catchment on the basis of values given for each soil group by Clapp and Hornberger (1973) (Table 4-10 of the SWMM-4 users manual).

For the watershed water quality simulations, the build-up/washoff method was chosen over the rating-curve method at all but one subcatchment (Coles Brook, W8). The rating curve method is relatively simple in its application and requires minimal

Table 4.3: Hirsh Brook Sub-Basin Composition

LOWER HACKENSACK RIVER STUDY

HIRSH BROOK SUB-BASIN SOIL TYPE AND LAND USE COMPOSITION

Soil Type	Area (AC)	% AC	Suct	ΣSuct*	Ks	ΣKs	IMD	ΣIMD
A	0.0	0.0	0.0	0.000	0.00	0.000	0.00	0.000
B	1608.2	55.16	8.0	4.413	0.22	0.121	0.31	0.171
C	917.7	31.48	10.0	3.148	0.10	0.031	0.26	0.082
D	389.5	13.36	12.0	1.603	0.03	0.004	0.21	0.028
Composite	2915.4	100.00		9.164		0.156		0.281

					Buildup Rate (lbs/acre/day)			
Land Use	Area (AC)	% AC	%IMP	Σ%IMP	BOD	ΣBOD	NH_3-N	ΣNH_3-N
L.D. Resid.	1924.6	66.01	38.0	25.1	0.211	0.13929	0.015	0.00990
H.D. Resid	28.4	0.97	65.0	0.6	0.240	0.00234	0.018	0.00018
Industrial	230.8	7.92	72.0	5.7	0.325	0.02572	0.044	0.00348
Commercial	293.1	10.05	85.0	8.5	0.895	0.08998	0.045	0.00452
Open	436.8	15.05	0.0	0.0	0.058	0.00873	0.003	0.00044
Composite	2915.4	100.00		40.0		0.266		0.0185

* Suct - Capillary Suction (in)
Ks - Hydraulic Conductivity (in/hr)
IMD - Initial Moisture Deficit
Σ - Proration by Area

parameterization. However, this method requires extensive field data. The build-up/washoff method, on the other hand, requires specification of a greater number of parameters and equations than the rating-curve method. However, the more "flexible" build-up/washoff method often reproduces the observed water quality trends much better than the other methods.

The rating curve method was employed only at the Coles Brook (W8) monitoring site. This instream tributary station is located near river mile 18 of the lower Hackensack River. The extensive flow and water quality data collected at this site facilitated the rating curve method. Appropriate runoff coefficients and exponents for this site were computed from log-log plots (Figure 4.3) of flow versus loading for each modeled constituent during two storm events.

The SWMM-4 model, has two options for simulating the build-up/washoff process. In the first option, build-up/washoff rates are determined on the basis of dust and dirt accumulation rates. These accumulation rates, in turn, vary with land use. Build-up rates for each constituent are assumed to be a prescribed fraction of the dust and dirt accumulation rate. For a given subcatchment, however, only one land use type can be considered. As a result, this option does not allow for the variation of constituent build-up/washoff rates due to land use variations within a subcatchment.

In the second option, dust and dirt accumulation are not considered. Instead, separate build-up/washoff rates are assigned for each constituent. For the present study, separate build-up/washoff coefficients were selected not only for each constituent, but also for different land use types. Build-up/washoff rates within a subcatchment were then computed via an arithmetic mean of the various rates for a particular constituent weighted by the relative percentage of the various land use types within that subcatchment.

4.2 Runoff Model Calibration and Verification

4.2.1 Storm Sewers

Data collected at five storm sewer areas during 1988 and 1989 were used for the calibration and verification of the SWMM-4 model. These include Overpeck (C14), East Riser (C13), Coles (W8), Kearny (C8) and New Milford (C1), respectively (Figure 4.1). These storm sewer areas were selected to represent a mixture of land use types typical for the watershed, Saun and Overpeck catchments being predominantly residential, East Riser being a combination of commercial and residential, and Kearny being largely industrial.

The SWMM-4 model was calibrated for the relevant hydraulic and water quality parameters at the five storm sewer monitoring sites described above. The selection of the relevant hydraulic and water quality variables for model application were determined by requirements of the receiving water model. Since the receiving water analysis was primarily concerned with the DO regime of the river, the pollutants that impact the DO regime were targeted for the SWMM-4 analysis. Thus, the calibration effort was limited to the variables flow, BOD and NH_3-N, respectively. The two parameters for computing pollutant build-up rates obtained during this calibration process applied to the entire watershed.

The following general strategy was employed in the water quality calibration effort of the SWMM-4 model. Rather than developing a specific set of calibration coefficients for each subcatchment, an attempt was made to develop a set of area-wide build-up/washoff coefficients for the entire Lower Hackensack River Basin. A Michaelis-Menton formulation was chosen to estimate the surface build-up rates during the antecedent dry days.

The SWMM-4 model has four adjustable parameters for pollutant build-up and washoff. Pollutant Washoff is modeled via two adjustable parameters, a washoff coefficient and exponent. Michaelis-Menton build-up is adjusted by two time constants - a half-time constant and a maximum build-up concentration. These four "degrees of freedom" afford the user many opportunities to tune model predictions to match observed data. For this reason, a systematic approach was undertaken to limit artificial tuning of these parameters within individual subcatchments.

Since there is no obvious physical reason for the washoff coefficient and exponent to differ from one subcatchment to another, these parameters were held fixed for each subcatchment in the latter stages of the calibration process. Representative washoff coefficient and exponent values of 40 and 1.8, respectively, were selected for all subcatchments. Likewise, the half-time concentration (i.e. the time required for the concentration to build-up to half its limiting value) was also held constant at 0.9 days, as recommended in the SWMM User's Manual, for each subcatchment. The limiting build-up concentration, however, was assumed to be land use-specific, and thus to vary among subcatchments.

The adopted approach, therefore, was to first calibrate the limiting build-up rate parameters for BOD and NH_3-N for five land use categories in a manner that would assess average constituent loading contributions associated with each land use within the watershed. Next, the loading contributions from an individual catchment were determined based on its relative land use composition. For example, the Kearney subcatchment, monitored at station C8, is comprised predominately of industrial land use. Calibration runs at this station were used to fine-tune literature values for limiting industrial build-up concentrations. These loading rates were then used as the industrial land use components of the weighted-average estimates for the remaining subcatchments.

The implicit assumption in this approach is that loading rates associated with a given land use are nearly uniform throughout the watershed. While this assumption may not be valid for all subcatchments within the watershed, it will be valid for the majority of subcatchments. In any case, it is clear that a generic approach such as the method described above is required for this study, since the results need extrapolation to the 18 remaining subcatchments.

A set of sub-basin-averaged limiting build-up rates (or "loading potentials") were determined for each composite land use. These rates were determined through a series of iterative model simulations in which average build-up coefficients (weighted by the relative percentage of each land use) were chosen that best fit the data at the five storm sewer monitoring sites. Literature values were used as a "first guess" for these coefficients, and subsequent model computations were used to refine these preliminary estimates. Successive trial runs of the model indicated that reasonable estimates for the limiting build-up concentration were equivalent to the simulated loading rate (units - lbs/acre/day) integrated over a 4 day period.

Build-up/washoff coefficients were then derived for each sub-basin of the entire watershed, based on computations for the mean coefficients for a given subcatchment, weighted by the relative percentages of the five land use types.

Results of the SWMM-4 calibration effort for the storm sewer areas are displayed in Figures 4.4-4.13. Since the times of concentration corresponding to each model subcatchment range from several minutes (e.g., Saun subcatchment) to a few hours (e.g., Coles Brook subcatchment), it is unlikely that extensive biochemical reactions (e.g., decay) have a discernible impact on the quality of transported stormwater. Consequently, the observed water quality time-histories are largely flow-dependent - where peaks and troughs of constituent concentration closely mimic the time histories of the runoff hydrographs. The model serves to elucidate this inherent feature of the observed data. The rainfall characteristics of the different storms used in model calibration and verification are contained in Table 4.4.

Table 4.4 - Rainfall Volume (ins) at Different Locations in 1988 and 1989

EVENT	LOCATION			
	NEWARK AIRPORT	GTC*	CBA**	NEW MILFORD
July River Model Calibration (storm) July 11-22, 1988	6.770	5.830	4.520	4.190
August River Model Verification (storm) August 23-26, 1988	1.450	1.510	1.630	1.750
August River Model Verification (storm) August 29-30, 1988	0.302	0.260	0.220	0.500
October SWMM Model Calibration October 21-23, 1988	1.36	1.300	-----	-----
May SWMM Model Calibration May 1 -3, 1989	1.43	1.520	-----	-----
May SWMM Model Verification May 23-25, 1989	1.60	1.140	-----	-----

* GTC - General Testing Corporation Gage at Hackensack
** CBA - Clinton Bogert Associates Gage at Fort Lee

NOTE: Values indicate total volume of all storms during specified periods. For daily values, see Appendices.

Calibration/Verification of the New Milford Storm Sewer

Figure 4.4 shows a plan of the New Milford watershed draining to the sampling station C1. The monitoring station of New Milford Storm Sewer is located at a storm sewer manhole on Heanley Avenue, upstream of the outlet into Hackensack River. The land use within this 138 acre drainage basin consists of predominantly medium density residential (76.5%), 11.4% Low density residential, 10% commercial, and 2.1% open space, respectively. The overall percentage of imperviousness is 40%. The schematization of the subcatchment is shown in Figure 4.4. The July and August 1988 storm events were used for calibration and verification of the SWMM model. The results of the calibration and verification are shown in Figures 4.5 and 4.6, respectively. The rainfall records are shown at the top of the figure where each vertical line represents the fifteen minute rainfall volume in inches. The hydrograph and pollutographs simulated by the model are shown as the solid lines while the symbols represent the observed data.

The hydrographs for the July events match well with the observed data with the exception of the July 20 and July 22 events. The storm events that occurred in July 1988 had large temporal and spatial variations. The rain gage used in this simulation was located at Hackensack Water Company Water Treatment Plant in New Milford, approximately three miles North of the New Milford storm sewer site. However, a careful review of Figure 4.5 reveals that during the July 20 storm event, the observed runoff at the site actually preceeded the recorded rainfall, a discrepancy clearly implying a moving storm. Also during the July 23 storm event, runoff was observed with no recorded rainfall at the HWC gage. Another rain gage at General Testing Co., four miles south of the site, recorded some precipitation during that storm event. Rainfall data for other storm events recorded at the GTC rain gage performed poorly and were not selected for the calibration/verification purposes. Model simulations of water quality concentrations generally conform well with the observed data, except for a few extreme values. These high values are considered very unusual for a storm sewer and thus neglected in the analysis. In general however, model results compare well in both magnitudes and temporal variation for both BOD and NH_3-N, respectively.

The model was then verified using the data collected during the August 1988 storm event. All parameters generated during the calibration effort were unchanged for model verification. The results of model verification are shown in Figure 4.6. The model predicted remarkably well both the flow and water quality during both storm events. Both the peaks and timing of the hydrographs were reproduced by the model with great accuracy. Unfortunately, the first flush of the August 24 event was not captured during the data collection effort. The model predicted a high concentration at the first flush due to a prolonged dry period (three weeks) before this storm event.

Calibration/Verification of the Coles Brook Sub-Area

The monitoring station of the Saun storm sewer area (station number W8) was located at the confluence of Coles Brook and van Saun Mill Brook above the lower Hackensack River. These two tributaries drain a total area of 4,472 acres (7.0 square miles). The land use of this basin consists of predominantly low and medium density residential (65.9%), 2.2% high density residential, 7.6% industrial, 5% commercial and 19.3% open space, respectively. The overall percentage of imperviousness is 36.2%. This station was designated as an instream monitoring stations, thus the flows were sampled continuously during July and August of 1988. The SWMM-4 model was applied for the storm events from July 8 to July 28 in a continuous mode for the calibration of the model. The GTC rain gage data were used in this simulation. The results of the SWMM model calibration are show in Figure 4.7. The period of July 17-26, which had significant rainfall, is shown here for purposes of presentation. No significant rainfall occured during the remaining days and therefore are not illustrated here. The solid line shown in the hydrographs represents the computed flow, while the dotted line represents the observed flow at the station. The flows predicted by the model consistently match the observed data, with the exception of July 23, which again can be attributed to inconsistencies in the rainfall data. Water quality predictions of the model are also good in both amplitude and timing. The model seems to underpredict the concentrations during the period of July 17-29, during two minor storm events. However, although the observed flows were negligible, BOD and NH_3-N concentrations remained high, contrary to what would be expected in comparison with observations during the major storm events.

The storm events in August 1988 were used as a verification of the SWMM model of the Saun basin. All parameters generated during the calibration effort were kept unchanged for model verification. The results of model verification are shown in Figure 4.8. The model again predicted very well both the hydrographs and pollutographs of the August 24 storm event.

Calibration/Verification of the East Riser Storm Sewer

The East Riser Ditch storm sewer monitoring station (station C14) is located at the intersection of Huyler street and Route 46 in the town of South Hackensack (see Figure 4.9). This station has a drainage area of 417 acres consisting of 58% commercial and 42% medium and low density residential land use, respectively. The overall percentage of imperviousness is 57.7%. As the previous stations served to assess the contribution of the runoff and buildup/washoff from the residential area, this basin was used to identify the contribution from commercial land use. Since this station was added later in the study, only two storm events were monitored, during May 1-3, 1989 and May 23-25, 1989, respectively. Thus, the first storm event was used for calibration, while the second event was used for verification of the SWMM model on this sub-basin.

The results of the model calibration and verification are shown in Figure 4.10 and Figure 4.11, respectively. The hydrographs matched well for the first storm event but overpredicted slightly the second storm event. The pollutant concentrations were reasonably within the range of the observed data. Note the concentrations of BOD and NH_3-N were significantly higher compared with those in the New Milford and

Saun storm sewer areas due to the contribution from the highly commercial watershed.

Calibration/Verification of the Kearny Storm Sewer

The Kearny storm sewer monitoring station (station C8) is located at the manhole on Hackensack Avenue, between Second and Third Avenues. This station controls a drainage area of purely industrial land use. The overall percentage of imperviousness is 72%. The SWMM model was applied for the storm events between July 8-28, 1988 in a continuous mode for the calibration, while the two storm events in August, 1988 were used to verify the model. The Newark Airport rain gage data were used in the SWMM simulations.

The results of the model calibration and verification are shown in Figure 4.12 and Figure 4.13, respectively. The hydrographs performed reasonably well for the July storm events, except in two instances. This maybe attributed to the inconsistencies of the rainfall data. The pollutant concentrations were within the range of the observed data, except for a few isolated high concentrations sampled. The nature of these values indicate data noise rather than reality. Again the concentrations of BOD and NH_3-N were significantly higher compared to those in the New Milford and Saun storm sewer areas but compatible with data in the commercial areas. During model verification, it is unfortunate that no data was available to test the effects of the first flush, computed by the model. Despite this limitation, the verification run was satisfactory especially for the prediction of the peak concentrations.

4.2.2 CSO Areas

Combined sewers overflowing into Lower Hackensack River involves four municipalities, namely Hackensack, Ridgefield Park, North Bergen and Jersey City with a total service area of over 4,000 acres (6.3 square miles) In previous studies, The BCUA I/I analysis (Clinton Bogert Associates, 1981) and Hudson County 201 CSO study (Havens and Emerson and Hazen and Sawyer, 1980) documented these

CSO areas in great detail. In this study, a more detailed assessment of pollutant contribution from the major CSO areas was conducted.

Description of CSO Areas

As depicted in Figure 4.1, the shaded areas represent regions served by combined sewers. There are two major CSOs in the City of Hackensack; one located at Anderson Avenue serving 510 acres and the other locates at Court Street serving 478 acres. There are five CSOs in Ridgefield Park serving a total area of 452 acres. Among 17 CSO areas located in North Bergen, only 523 Acres are served by CSOs which discharge to tributaries of the Hackensack River. There are 8 major CSOs in Jersey City discharging into tributaries of Hackensack River, These are designated as regulator number RW1-RW8 in the 201 study.

Methodology of Calibration and Verification

Data collected from three CSO monitoring stations were used for calibration and verification purposes. These stations include: Court street CSO in the city of Hackensack, Sip Avenue CSO and St. Paul Avenue CSO in Jersey City. Each CSO comprised of two monitoring locations: the overflow station at the outfall from the regulator and the "land-use" station at a manhole upstream of the regulator. The overflow stations designated C3, C9, and C12 were installed to capture the actual overflow from the regulator. The land-use stations, designated C3A, C9A, and C12A, were used to measure the combined flow and water quality from land surface runoff/washoff and the sanitary sewage upstream from the regulator. Data collected at land-use stations were utilized to calibrate and verify the SWMM model in CSO areas. These collected data were not impacted by surcharging in the interceptors and regulators, thus deeming them more reliable in estimating loads from the CSO areas. Dry weather flow data collected during the period of October 18, and 19 of 1988 were synthesized to produce a daily cycle of flow and pollutant concentrations, to represent the baseflow component in model simulations.

CSO area boundaries that contributed to the monitoring stations were delineated and schematized similar to the methodology adopted in storm sewer areas. The runoff computed from these drainage areas were then merged with dry weather flows and compared with data collected at corresponding stations. The results of calibration and verification of all the CSO areas are shown in Figures 4.14 - 4.22. The computed hydrographs and pollutographs generally match well with the observed data. The higher concentrations shown before and after a storm event reflect the dry weather flow (sanitary) concentrations in the combined sewers.

Calibration/Verification of the Court Street CSO

The drainage basin contributing to the Court Street CSO is shown in Figure 4.14. The combined sewer monitoring station C3A is located at a manhole located in the parking lot of the Country Court House. The combined sewer system at this location drains an area of 367.2 acres with an overall percentage of imperviousness of 40.5%. The land use composition is: medium/low density residential, 48.3%, high density residential, 17.7%, industrial, 8.3%, commercial, 5.4%, and open space, 20.3%.

The storm event on August 29, 1988 was used to calibrate the SWMM model for the Court Street CSO. The results of model calibration are shown in Figure 4.15. Model simulations of observed hydrographs and pollutographs indicate a good match for the three variables of concern. The model reproduced the NH_3-N concentrations with a high degree of accuracy. The high concentrations prior to the beginning of the storm reflected the dry weather sewage flow while the lower concentrations observed during the storm are a result of dilution due to stormwater runoff.

Model simulations were repeated on the Court Street CSO during the October 21, 1988 storm event as a verification. The results are presented in Figures 4.15 and 4.16. Here again, the model reproduced the hydrograph and pollutographs remarkably well. Two extremely high BOD concentrations measured at the beginning of the storm of 600 and 850 mg/l for BOD_5, were considered unrealistic and not considered in the analysis. Except for this inconsistency, model calibration and verification of the Court Street CSO was considered excellent.

Calibration/Verification of the Sip Avenue CSO

The drainage basin contributing to the Sip Avenue CSO is shown in Figure 4.17. The combined sewer monitoring station C9A located at the manhole on Sip Avenue, Jersey City, is located immediately upstream of the Freeman Avenue intersection. The combined sewer at this location serves a drainage area of 246.1 acres with an overall percentage of imperviousness of 60%. The land use composition is: medium/low density residential, 16.4%, high density residential, 58.5%, industrial, 10%, commercial, 10%, and open space, 5.1%.

The storm event on August 29, 1988 was used to calibrate the SWMM model. The results are shown in Figure 4.16. Here again, the model simulated the observed hydrographs and pollutographs very well. A single value of 190 mg/l for BOD_5 was considered unrealistic and deleted from the analysis.

Model simulations were repeated for the Sip Avenue CSO during the May 1, 1989 storm event as a verification. The results are presented in Figures 4.18 and 4.19. The model performed well and reasonably reproduced the hydrograph and pollutographs in both timing and magnitude. These results indicate a successful calibration/verification of the Sip Avenue CSO by the SWMM-4 model.

Calibration/Verification of the St. Paul Avenue CSO

The drainage basin contributing to the St. Paul Avenue CSO is shown in Figure 4.20. The CSO monitoring station C12A located at the manhole on St. Paul Avenue, Jersey City, is located upstream of the Charlotte Avenue intersection. The combined sewer at this location serves a drainage area of 261 acres with an overall percentage of imperviousness of 67%. The land use composition is: medium/low density residential, 11.5%, high density residential, 9.2%, industrial, 74.3%, commercial, 3.8%, and open space, 1.2%.

The storm event on October 21, 1988 was used to calibrate the SWMM model. The results of the model calibration are shown in Figure 4.21 and 4.22. The model simulated the observed hydrographs and pollutographs fairly well. The collected data appears to have missed the first flush for both BOD and NH_3-N. Model simulations were repeated at the St. Paul Avenue CSO during the May 1, 1989 storm event as a verification. The results are presented in Figure 4.19. The model generated excellent results, reproducing observed hydrographs and pollutographs both in timing and magnitude. The results confirm that the loading rates adopted in the calibration effort are representative of the St. Paul Avenue CSO sub-basin.

4.2.3 Review of Model Calibration and Verification

In the previous sections of this chapter a detailed descripiton was presented of the SWMM-4 model and its application in both storm sewer and CSO areas within the lower Hackensack River Watershed. The selection of loading rates and different parameters used in the calibration and verification effort were based on **local, rather than literature values** generated through the comprehensive data collection effort in 1988 and 1989. Using these values, the model was successfully calibrated and verified over five storm sewered catchments and three CSO catchments.

The results of this calibration and verification effort gives confidence to the adopted SWMM-4 model as a reliable tool in computing stormwater runoff loadings into the lower Hackensack River. Thus, the adopted model can now be extrapolated to the entire lower Hackensack River Basin, using the results of the model calibration and verification effort. This expanded model application is presented in Chapter 6 of the report.

4.3 Long-Term Modeling of CSO Areas

4.3.1 Descriptions of CSO Sub-Basins

For long-term simulations of the CSO areas, the CSO outfalls within the watershed were aggregated into 6 CSO sub-basins. The City of Hackensack has two sub-basins, namely the Anderson Street CSO and the Court Street CSO. CSOs in Ridgefield Park were lumped into a single sub-basin. CSOs in North Bergen were also lumped into a single sub-basin. The Jersey City West CSO system consists of 13 CSOs, of which only 8 CSOs discharge into the study area. The 8 CSOs in Jersey City West were lumped into two sub-basins: Jersey City North, including regulators RW1 and RW2; and Jersey City South, which includes regulators RW3 through RW8. The sub-basin characteristics are shown in Table 4.5.

4.3.2 Dry Weather Flows

Dry weather flows and water quality data were collected for two consecutive days in most of the CSO monitoring stations. These data were used to construct diurnal flow variations in each sub-basin. The water quality concentrations did not show a significant variation, and thus, the concentrations were averaged and held as constant. The summary of the statistical analyses of dry weather flows are shown in Appendix A-1.

4.3.3 Simulations of CSO Outfalls

The storm water runoff from the CSO sub-basins were first simulated using the Runoff Block of SWMM-4 to generate the land surface build-up and wash-off similar to the methodology adopted for the storm sewered areas. The Transport Block was then used to combine the land surface runoff with the dry weather flow developed previously. Each of these conbined flows were then routed to a flow divider element to simulate the CSO regulator. The interceptor capacities obtained from the previous I/I and 201 Studies were used to define the characteristics of the flow divider elements. Flows exceeding the interceptor capacities were then input to the receiving water model. Table 4.5 shows summaries of dry weather flows and interceptor capacities for all the CSO sub-basins.

Table 4.5 - Summary of CSO Sub-Basins

Sub-Basin Name	Dry Weather Flow Min.	Ave. (cfs)	Max.	Concentrations CBOD (ppm)	NBOD (ppm)	Interceptor Capacity (cfs)
Anderson Street	0.61	1.0	3.325	136.5	13.6	4.66
Court Street	1.53	2.4	4.375	243.5	17.9	7.3
Ridgefield Park	0.786	1.57	2.092	207.3	15.8	4.72
North Bergen	0.62	1.709	3.457	109.2	11.1	7.2
Jersey City North	2.155	6.378	12.774	111.8	5.59	11.32
Jersey City South	3.154	8.467	15.741	130.0	11.8	20.1