4.0 Hydrologic and Hydraulic Modeling

4.1 Overview and Modeling Approach

In accordance with AECOM's scope of services for Phase 2, a hydrologic and hydraulic model was prepared for the Wellington Avenue Tributary Catchment Area, which is comprised of six subcatchments, to simulate the sanitary sewer flows under dry weather and wet weather conditions under various system configurations. AECOM prepared three technical memoranda presenting: the calibration of the Wellington Avenue Tributary Area; a limited area tributary to the Washington Street CSO Facility; the wet weather recalibration of the Wellington Avenue CSO Facility Tributary Area and Washington Street CSO Facility; and the calibration of the Newport WPCP and CSO overflows at each of the CSO facilities. The following Technical Memoranda (TM) are included in the Appendix.

- Appendix A - Model Setup and Data Collection, June 2008
- Appendix B - Flow Metering Investigation for Hydraulic Model Calibration/Verification, January 7, 2008
- Appendix C - Wellington Avenue CSO Facility Model Calibration, May 2, 2008
- Appendix D - Washington Street CSO Facility Model Calibration, September 11, 2008
- Appendix E - Alternative Analysis, December 10, 2008

The TMs present a summary of the model calibration/verification process and documents the methodology, parameters used and results. The validation of the model under various configurations indicates how well the model predicts dry and wet-weather flows compared to actual measurements and flow meter data, and discusses the results as they relate to the objectives of the scope of work.

The following sections present the approach utilized to develop the hydrologic and hydraulic parameters for the existing conditions model.

4.1.1 Model Background and Approach

The MIKE URBAN (MU) model by DHI was recommended to the City by AECOM during the preparation of the Phase 1 Part 2 Report.

The MU model consists of two modules; (1) a hydrologic module, and (2) a hydraulic module (i.e. MU Pipe Flow Module). The hydrologic module characterizes the generation of base wastewater flows as well as the storage and transfer of rainfall and runoff to the wastewater collection system within a given catchment area. A catchment area in MU is a geographical feature which represents hydrological urban catchment areas or wastewater drainage areas. The hydraulic module simulates the routing of flows, over time, through the wastewater conveyance system – through the pipes, storage components, pump stations and CSO control structures that make up the system.

The following summarizes the setup and data requirements for the model. More detailed descriptions are provided in Appendix A.

Hydrologic Module

The hydrologic module requires input on the sewer catchment area physical characteristics, such as: area, runoff time of concentration, percentage of total Catchment Area contributing extraneous flow, and rainfall. These inputs are developed for each catchment area and the hydrologic module is used to produce a flow hydrograph (output) for each catchment area. The measured rainfall events and corresponding flow data collected in the metering program are then used to calibrate the model.

The hydrologic module consists of a fast response component (FRC) and a slow response component (SRC). Rainfall that falls onto the ground surface but does not infiltrate into the soil is represented as the surface runoff (i.e. FRC). This FRC flow can include, but is not limited to, directly connected catch basins or roof leaders. MIKE URBAN simulates this fast response flow using the Model A module. Rainfall that infiltrates the soil and enters the system through leaks or other delayed methods, such as sump pumps, are represented by the SRC. In MU, the rainfall dependent inflow and infiltration (RDII) module controls subsurface infiltration and is represented in the resultant hydrographs as overland flow, interflow, and baseflow, which are referred as *reservoirs* by MU. These methods work together as a water balance, and distribute I/I to the collection system through either surface runoff (FRC) or through 3 connected reservoirs: surface storage, unsaturated zone storage, and groundwater storage zone (SRC). This is depicted schematically in Figure 4.1.

Rainfall that infiltrates into the soil column first begins filling the upper soil storage reservoir. Overland flow is the portion of the water that is sent out of the upper soil storage reservoir into the pipe network. Flow that does not discharge to the collection system from the upper storage reservoir seeps further into the soil column and begins filling the lower zone storage reservoir. Water not discharging to the collection system from the lower zone storage is called interflow. Flow that does not discharge to the collection system from this middle reservoir seeps further down into the soil and adds to ground water storage or baseflow. RDII parameters control the size of the reservoirs and the rate at which flow is released from each reservoir, into the pipe network. It is important to remember that the various components of this subsurface system are all interrelated; all the reservoirs are connected. This means that changing any of the parameters for both reservoir size and discharge rate (out of the reservoirs) will have an impact on the discharge out of all other reservoirs.

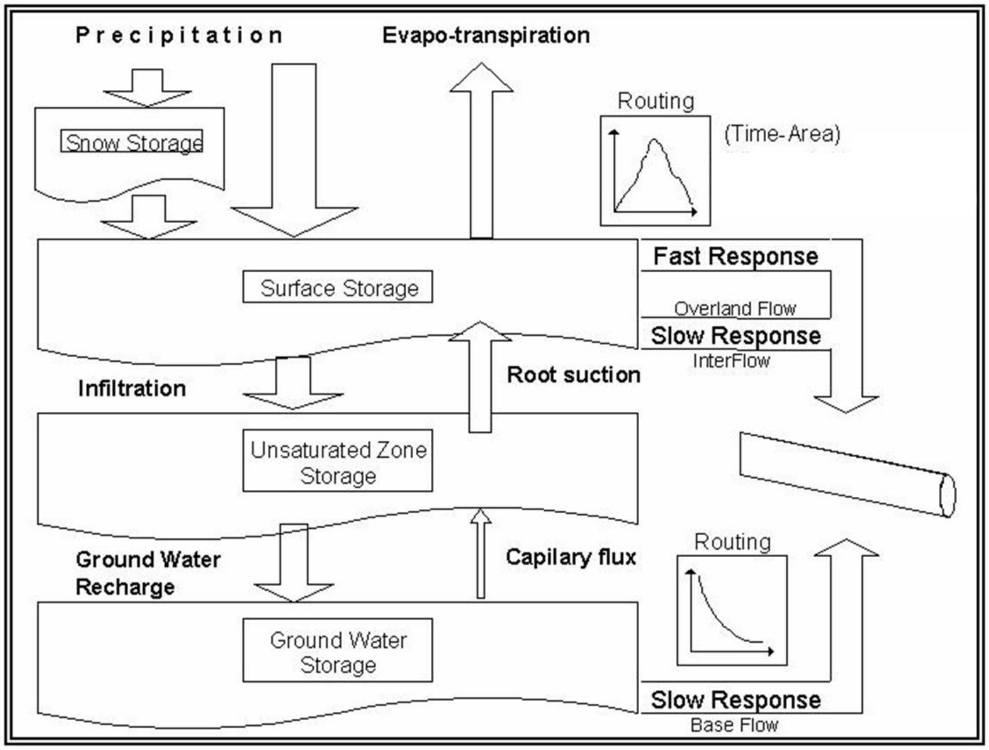

Figure 4.1
View of RDII Hydrologic Model

While there are several options available to generate runoff hydrographs (i.e. FRC) in MU, the common method is referred to as Model A, which is similar to the Rational Method. Input data for the Model A module include the catchment area, percent of total area that is contributing flow, and time of concentration. Model A treats the catchment area as a sloped rectangular ground surface, a specified fraction of which is connected to the sewer system. The shape of the runoff is controlled by the catchment area's time-of-concentration and time-area curve, which simulates the catchment area's contributing portion as a function of time. These two parameters represent a conceptual description of the catchment area reaction speed and the catchment area shape. The model utilizes three default time-area curves that simulate the rainfall runoff response of the catchment area and the curves are shown in Figure 4.2.

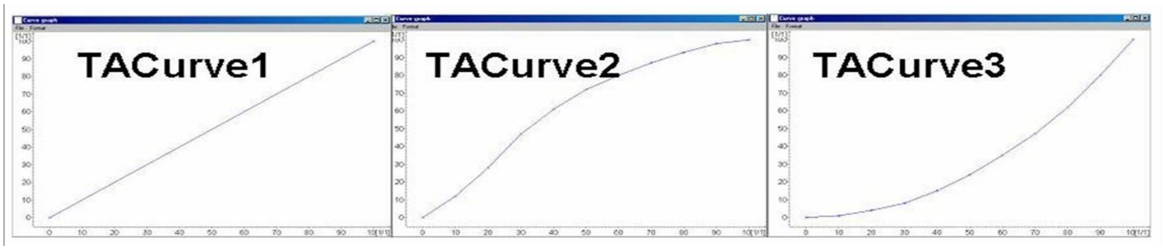

Figure 4.2
Time – Area (TA) Curves Used in MIKE URBAN

Hydraulic Module

For the hydraulic module, the physical characteristics of the pipes, manholes and other collection system features must be described, as well as their connectivity. Information needed for the hydraulic model includes the pipe network, storage conduits, pumping stations, control structures, and boundary conditions at model boundaries. The model includes features to check for errors in the data such as missing pipes or manholes and disconnected networks.

The hydraulic module uses the St. Venant equations to calculate one-dimensional, gradually varied, unsteady flows. The flow rate, depth, and velocity vary at each cross section at each time step throughout the modeled system, in pipes, manholes, storage conduits, wet-wells, and other hydraulic structures. The Model Development and Data Collection Technical Memorandum in Appendix B describes these evaluations in more detail.

MU refers to pipes (gravity and force main) as *links* and manholes as *nodes*. Hydraulic control structures such as weirs, orifices, gates, and pumps are also treated as links.

The hydraulic module supports the following features:

- Multiple pipe cross-section shapes
- Circular manholes
- Storage elements
- Detention basins
- Orifices
- Overflow weirs
- Pump operations
- Flow regulation
- Constant or time variable outlet water level (boundary condition)

- Constant or time variable inflows into the network (boundary condition)
- Non-standard head losses at manholes and basins
- Depth-variable friction coefficients
- Real time controls

The hydraulic module uses boundary conditions to simulate conveyance in the system. Upstream and downstream boundary conditions are either user specified or automatically generated by the hydrologic module.

In addition to flow rate, depth, and velocity, the hydraulic module also tracks how much volume has overflowed from each node. This is particularly useful in determining amount of overflow from each CSO location.

4.1.2 Model Development

The model development process included the establishment of physical characteristics, boundary conditions and simulation control parameters. The Wellington Avenue CSO Facility Model Technical Memorandum (Appendix C) and Washington Street CSO Calibration Facility Model Calibration Technical Memorandum (Appendix D) include more detail on these parameters.

Physical Characteristics

Hydrologic Module

The Wellington Avenue CSO Facility tributary service area and limited Washington Street CSO Facility Tributary area, as shown in Figure 4.3 defines the area that contributes wastewater flow to the Long Wharf Pump Station. The six original sewer catchment areas for the Wellington Avenue CSO Facility tributary area were imported for the modeling effort. Further refinement, modification, and addition to the original catchment area boundaries were required to provide for more detail in critical areas such as the Washington Street CSO Facility.

Base sanitary and diurnal flow patterns were designated for each of the catchment areas to generate the service baseflow component of the system. Recorded rainfall data were also linked to the catchment areas to generate wet weather flows in the system.

In addition to sanitary baseflow and rainfall, MU also allows the user to adjust groundwater levels, input soil characteristics, indicate land use, and account for transpiration and evaporation to simulate infiltration. Groundwater levels were used to provide a portion of the infiltration base flow and approximate antecedent moisture conditions. Transpiration and evaporation were assumed to be minor and were established during the calibration period using the meter flow data. Land use and soil characteristics are accounted for in the Module A and RDII parameters on a per catchment area basis.

The model includes default parameters for surface storage, root zone storage, overland flow and groundwater flow coefficients as part of the RDII module. These parameters were adjusted during the calibration process to account for slow infiltration flows such as those associated with connected sump pumps (due to increased groundwater conditions), rainfall dependent infiltration, and for surface flows associated with wet or dry antecedent conditions.

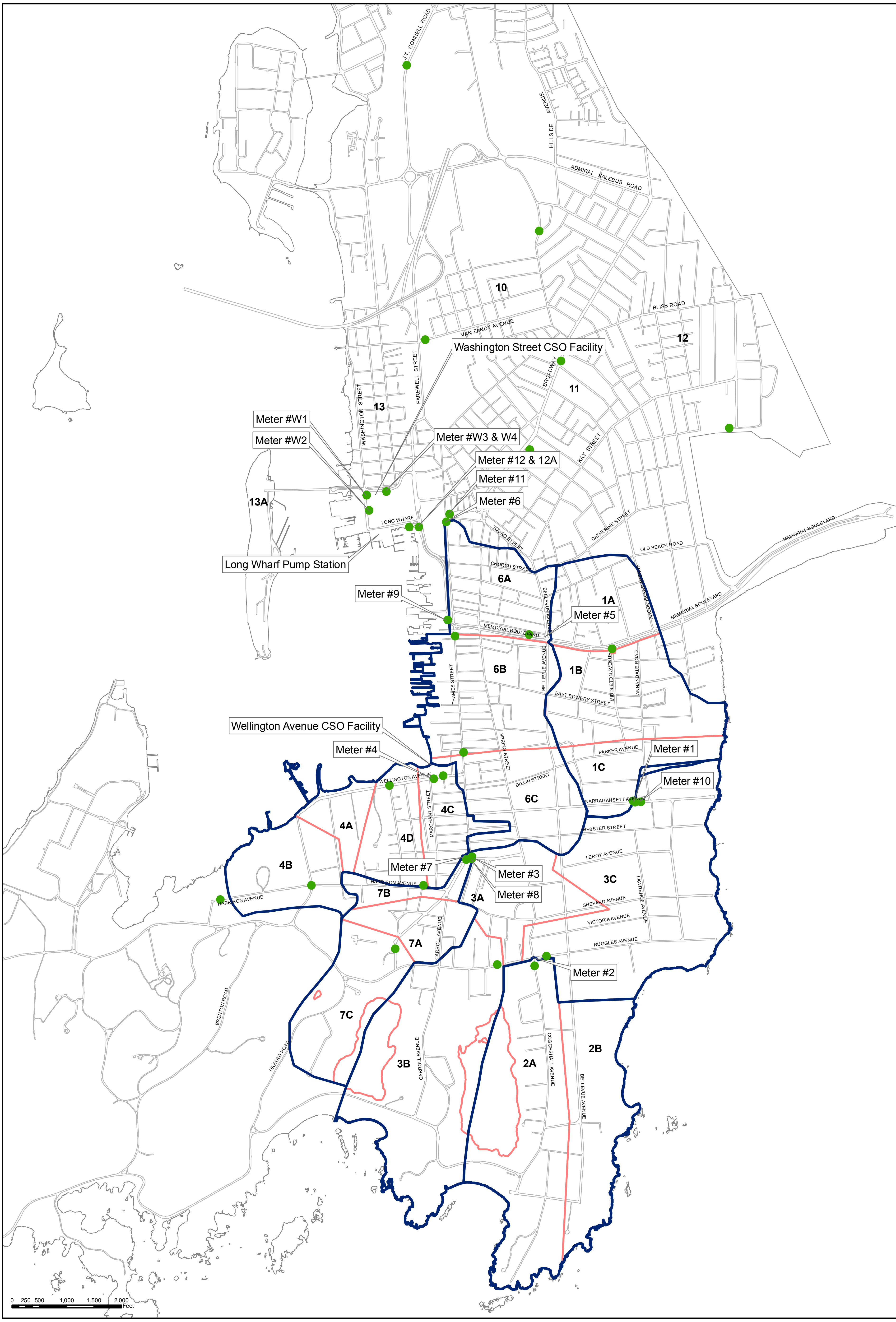

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Figure 4.3
Tributary Catchment Areas
Phase 2 CSO Control Plan

Sewer Catchment
Sub Catchment Area
Meter Locations

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Hydraulic Module

The development of the existing conditions model included utilizing the data from the City's current geographical information system (GIS). Use of these data required reorganizing and reformatting the existing GIS data in order to input the data into the required fields in the model database. In addition to the available GIS information, data from the manhole inspections performed during Phase 1 of the CSO program were used to provide elevation, pipe size, and material inputs to the model. If the data were not available in the GIS or manhole inspection information, field investigations were performed to collect any missing physical pipe information and to confirm network connectivity.

The modeled pipe network tributary to the Wellington Avenue CSO Facility includes the Thames Street Interceptor, pipes greater than 12-inches in diameter, and select local interceptors less than 10-inches in diameter. The modeled pipe network in the vicinity of the Washington Street CSO Facility includes the twin 54-inch conduits from Washington Square and the Thames Street Interceptor, the 42-inch America's Cup Relief Sewer, and the Long Wharf Pump Station. Figure 4.4 presents a map of the Wellington Avenue and Washington Street modeled pipe network.

Importing the nodes or manholes was the first step in building the model. All data were imported in the Rhode Island State Plane Coordinate System (NAD83, units: feet) and vertical datum used was NGVD 29.

The model network consists of 285 nodes (manholes), 271 links (pipe segments) measuring approximately 12 miles. The model simulates the flow contribution of approximately 6,300 acres (9.85 square miles) of the City of Newport or approximately 87 percent of the City. The City's GIS database and plans were used to establish connectivity. The key data for pipes and manholes in the model are shown in Table 4.1.

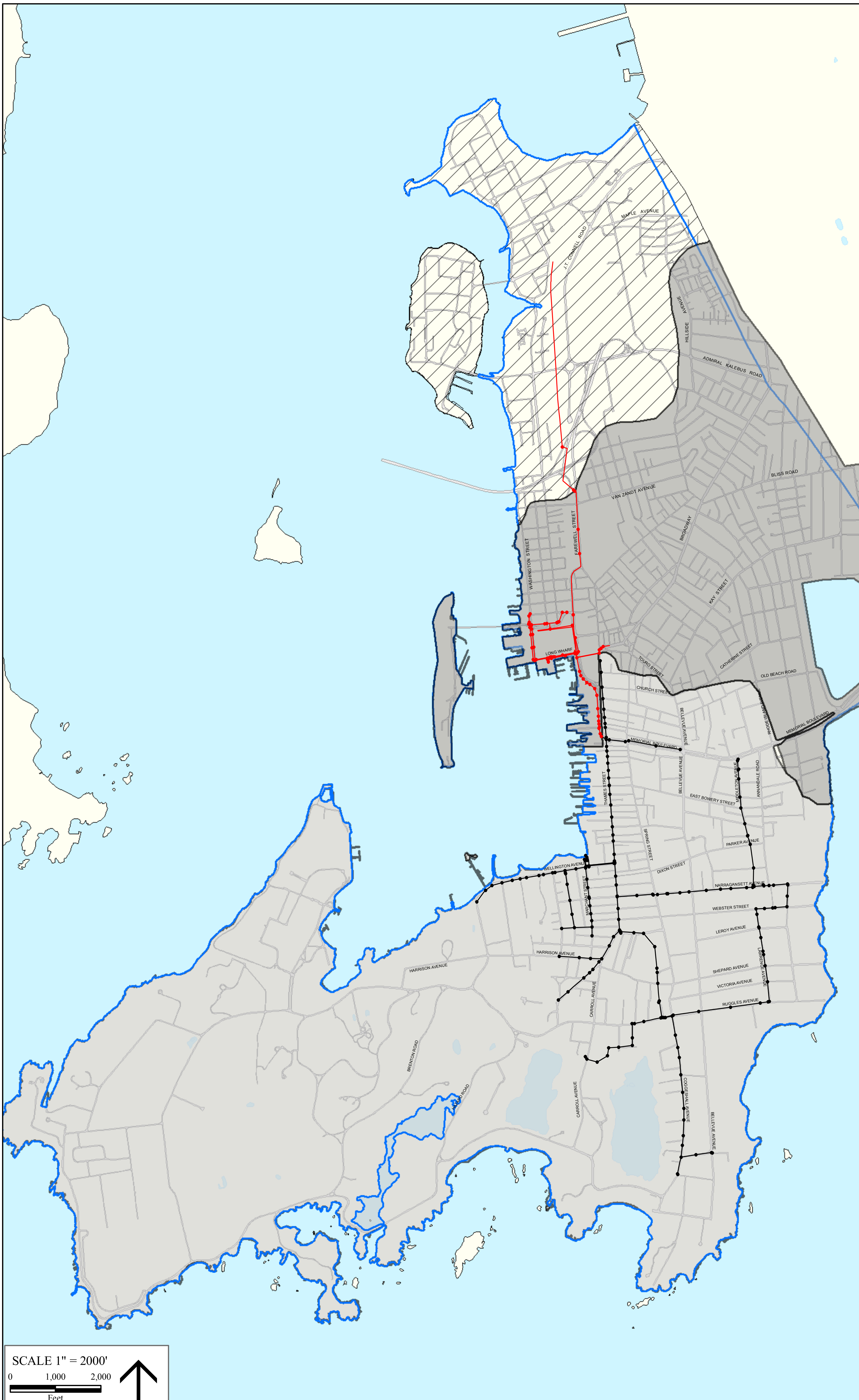
TABLE 4.1
HYDRAULIC MODULE NETWORK DATA

Pipes	***Manholes***
Pipe ID	Manhole ID (current ID in City GIS)
Upstream/Downstream Manhole ID	Node type (e.g. manhole, wet well, storage tank, etc.)
Cross Section Geometry	Manhole Rim Elevation
Diameter (feet)	Manhole Invert Elevation
Length (feet)	Manhole Diameter (feet)
Upstream/Downstream Invert	X and Y GIS Coordinates
Manning's Roughness Coefficient	

Nodes

AECOM used existing GIS elevation data when assigning rim and invert elevations to the MU nodes. For manholes missing required ground surface elevation data, an elevation was calculated by interpolation between the nearest surveyed features including catch basins, manholes, or other mapped features. The criteria for assigning the invert elevation fell into three categories:

- Direct import of invert elevation data from as built construction plans, survey data, and other historic plans or previous measurements. It was assumed that the elevations were correct;



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Washington St. Tributary Area
Wellington Tributary Area
Area Directly Tributary to WPCP

Wellington Area
Sanitary Pipes >= 12"
Manholes

Washington St. Area
Sanitary Pipes Washington St.
Manhole

Figure 4.4
Wellington Avenue and Washington Street Modeled Pipe Network
Phase 2 CSO Control Plan

- Calculation of invert elevations obtained by subtracting the manhole depth as measured during the Phase 1 Manhole Inspections from the existing ground surface elevations in the Newport GIS map; and

- Interpolation between known invert elevations upstream and downstream of the missing data point. If available, pipe slope information was used in the interpolation calculation.

Links (Pipes and Force Mains)

The selection of the manholes and pipes to be included in the model was based on capturing flows from each catchment area. Pipes or links in the model are listed below.

- All lines tributary to the Thames Street Interceptor greater than twelve inches in diameter,

- Select lines less than twelve inches in diameter that function as interceptor or transmission lines,

- Force mains between the Wellington Avenue CSO Facility and the Thames Street Interceptor, and

- The 84-inch Narragansett Avenue storage conduit.

- The duel 54-inch conduits conveying flow from Washington Square and the Thames Street Interceptor to Long Wharf Pump Station and the Washington Street CSO Facility.

- The 42-inch America's Cup Relief sewer,

- The 36-inch force main from the Long Wharf Pump Station to the WPCP, and

- Sewers in the immediate vicinity of the Washington Street CSO Facility.

The majority of editing for the links was done in MU. Quality control was performed by such MU features as viewing longitudinal profiles to help identify invert errors and an error log feature was used to identify pipes with lower invert elevations than their corresponding manhole elevations. This error log also indicated negative slopes in the pipe network and velocity anomalies thus providing a check list of possible errors.

The pipe diameter values in the City's GIS database are in inches. MU requires that the pipe diameter be in feet, therefore, all dimensions in the GIS database were converted to feet prior to import into MU.

MU pipe segments were drawn based on the 'From' and 'To' manhole (which are located by the X and Y coordinates in the manhole database) and MU automatically calculated pipe length. Pipe friction and pipe condition are simulated using Manning's 'n' Values for open channel flow calculations. Manning's 'n' values used in the model for all gravity pipes and force mains are shown in Table 4.2. All pipes that were not field inspected were assumed to be in fair condition and a corresponding 'n' value was assigned based on the pipe material. For pipes in poor condition, the Mannings "n" value was adjusted to account for increased roughness.

TABLE 4.2
PIPE ROUGHNESS

Pipe Material	*Manning's n Value*
Asbestos Cement (AC)	0.014
Brick	0.015
Iron	0.014
Normal Concrete	0.013
Plastic (PVC)	0.013
Vitrified Clay (VC)	0.015

Minor head losses associated with bends in pipes are accounted for automatically in MU. The equations used to estimate the head loss for each bend can be found in the MU Reference Manual (Pipe Flow).

Force mains are defined as Pressure Mains (links) and Pressure Nodes (nodes) in MU. At the end of each force main, a 'Receiving Manhole' needed to be defined, which defines the transition point between pressure flow and gravity flow (despite the fact that MU does not use pressure flow equations for the pressure mains – see the MU Reference Manual (Pipe Flow) for a description of how it treats pressure flow). A Receiving Manhole elevation needed to be defined at this location and acts as a boundary condition for the transition between pressure and gravity flow. This elevation was usually set between the invert and crown elevations of the downstream gravity pipe.

Pump Stations

The sanitary pump station at the Wellington Avenue CSO Facility, the Long Wharf Pump Station and the dewatering and overflow pumps at the Washington Street CSO Facility were included in the model, with 14 individual pumps being modeled. In MU, a pump is represented as a link between two nodes. The upstream node is the wet-well basin at the pump station; and the downstream node is the recipient. The pumps, with the exception of pumps at the Long Wharf Pump Station, were modeled as constant rate pumps controlled by start and stop levels in the wet well. Pump curve information was used to simulate the operation of the pumps at the Long Wharf Pump Station.

The pumps modeled at the Wellington Avenue CSO Treatment Facility are the Sanitary Wet Well (SW) pumps and the microstrainer chamber backwash (BW) pumps. The SW pumps discharge daily sanitary flows from the facility to the Thames Street Interceptor. The microstrainer chamber backwash pumps convey debris captured by the microstrainer screens back to the sanitary collection system. Since the removal of the microscreens, the backwash pumps have been used to pump additional flow to the Thames Street Interceptor during wet weather. The calibrated model includes the backwash pumping conditions that were in effect during the flow metering period in October 2007.

Levels specific to each pump were obtained from City plans and are presented in Table 4.3.

TABLE 4.3
PUMP OPERATION MODEL PARAMETERS

Pump#	*Location*	*Start (Elev.)*	*Stop (Elev.)*	*Capacity*
SW#1	Sanitary Well	-8.0	-11.5	640 gpm
SW#2	Sanitary Well	-7.2	-9.0	640 gpm
SW#3	Sanitary Well	-7.0	-8.5	640 gpm
BW#1	Microstrainer Backwash	-5.7	-6.5	250 gpm
DW#1	Settling Basin (WSCSO)	N/A	N/A	830 gpm
DW#2	Settling Basin (WSCSO)	Standby	Standby	830 gpm
EP#1	Effluent Basin (WSCSO)	-4.8	-6.6	10,400 gpm
EP#2	Effluent Basin (WSCSO)	-4.3	-6.0	10,400 gpm
EP#3	Effluent Basin (WSCSO)	-3.8	-5.3	10,400 gpm
EP#4	Effluent Basin (WSCSO)	Standby	Standby	10,400 gpm
SS#1	Sanitary Wet Well (LWPS)	-2.75	-6.25	5,600 gpm
SS#2	Sanitary Wet Well (LWPS)	-1.25	-3.25	5,600 gpm
SS#3	Sanitary Wet Well (LWPS)	Standby	Standby	5,600 gpm

Gates and Inverted Weirs

All gates and inverted weirs in the City's system were simulated in MU with the 'Orifices/ Gates' option. This allows the modeler to either keep a gate opened, closed, or partially opened. An equivalent rectangular orifice / gate was specified in all cases (as opposed to circular) in order to take advantage of MU's Real Time Control (RTC) module, which may be important for future planning and analysis.

The model contains one RTC operated gate at the terminus of the Narragansett Avenue Relief Sewer. This 18-inch gate is operated by sensors in the sanitary wet well at the Wellington Avenue CSO Treatment Facility. The gate closes when the sanitary wet well at the Wellington Avenue CSO Facility reaches Elev. -7.0 feet. The gate reopens when the wet well level recedes to Elevation -8.5 feet. Due to the sensitivity of the system and limited storage capacity of the sanitary wet well, the gate typically opens and closes over a period of time after a wet weather event to reintroduce stored flow.

Weirs

Two weirs were utilized to simulate the influent channel between the sanitary wet well and former microstrainer chambers and the effluent channel from the microstrainer chamber to the storm well at the Wellington Avenue CSO Facility.

Boundary Conditions

Hydrologic Module

Flow meter data used to calibrate the model were collected during the following time periods:

- March 28 to May 2, 2005 (Wellington Avenue Tributary Area)
- April 18 to May 30, 2006 (Catchment Area 6 Supplemental Flow Metering)
- October 17 to November 20, 2007 (Wellington Avenue Tributary Area)
- April 22 to May 15, 2008 (Washington Street Tributary Area)

Flow meter data collected in 2005 is included in the Phase 1 Part 1 Report. Flow meter data collected in 2006 is included in the Phase 1 Part 2 Report. Flow meter data collected in 2007 is provided in Appendix C – Wellington Avenue CSO Facility Model Calibration. Flow meter data collected in 2008 is provided in Appendix D – Washington Street CSO Facility Model Calibration. Figure 4.5 presents the meter locations for each of the flow metering periods.

Metered rainfall and tidal measurement data are the only hydrologic boundary conditions for the model. The data were collected at 15-minute increments during all metering programs. Rain data were collected by a tipping bucket rain gauge located on the roof of the Long Wharf Pump Station. It was assumed that this gauge location accurately represented the rainfall throughout the City and that there was no temporal or spatial variation between the Wellington Avenue CSO Facility and Washington Street CSO Facility tributary service areas. The tidal data were gathered via a tidal elevation gauge installed at the Long Wharf Pump Station and the baseflow was determined using the discharge readings from the meters. Additional tidal data was obtained from the National Oceanic and Atmospheric Administration (NOAA). It was assumed that the algorithm used to calculate tidally-influenced inflow varied linearly between low and high tide and was compared to flow meter data variations to verify. The assumed baseflow in the model is based on a rate per acre per day.

Hydraulic Module

Boundary conditions were developed using the data collected in the flow metering program, from the Wave Avenue Pump Station, and the meter data collected at the twin 54-inch junction structure on America's Cup Avenue. In addition, flow data from the Wellington Avenue CSO Facility were evaluated in conjunction with further evaluation of the metered data during the calibration and verification process.

A series of inflow points were designated in each catchment area. Inflow points are locations where inflow from the catchment area is introduced (injected) into the model. Figure 4.6 presents the location of the inflow points in each of the catchment areas.

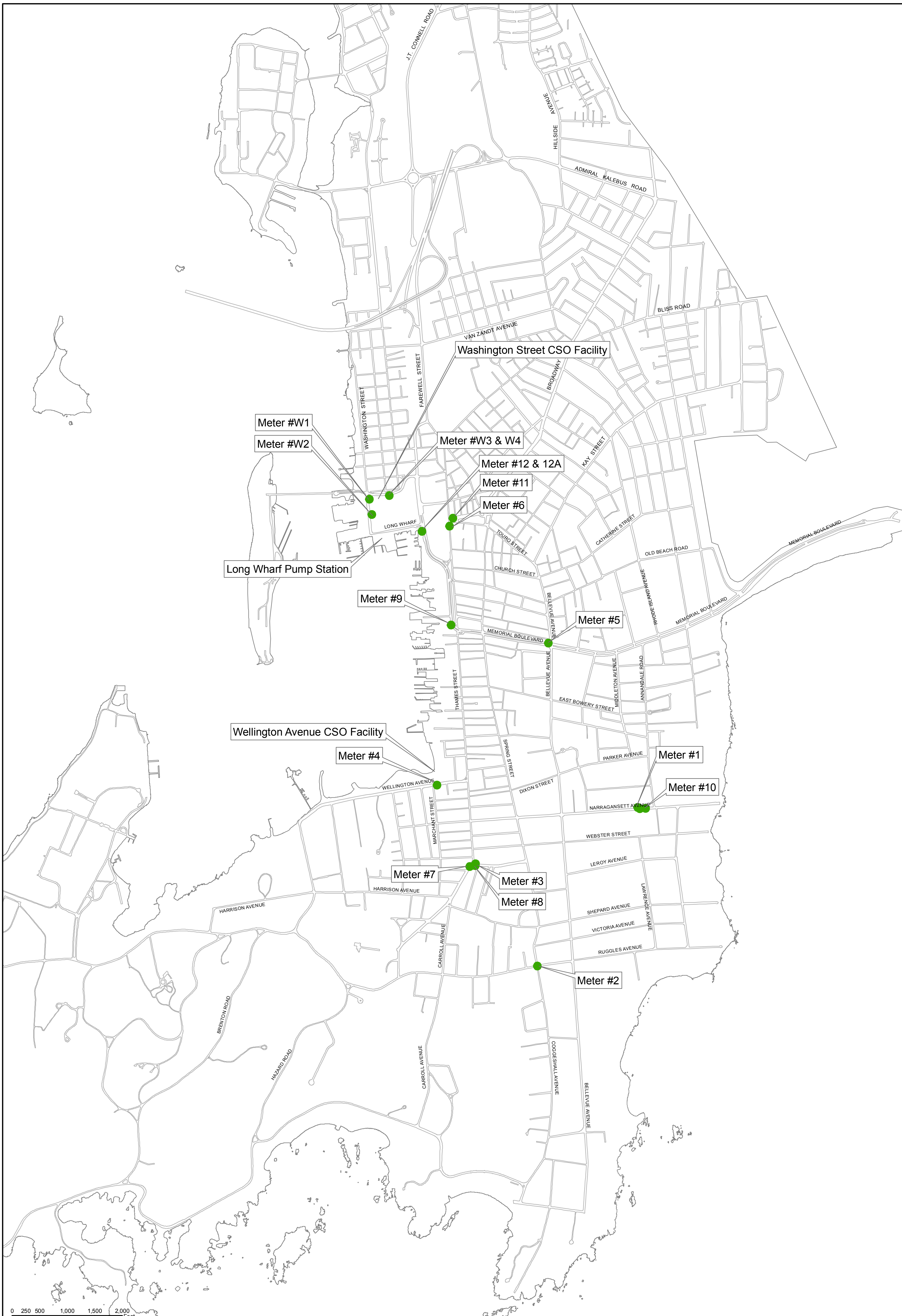

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Figure 4.5
Meter Locations
Phase 2 CSO Control Plan

● Meter Locations

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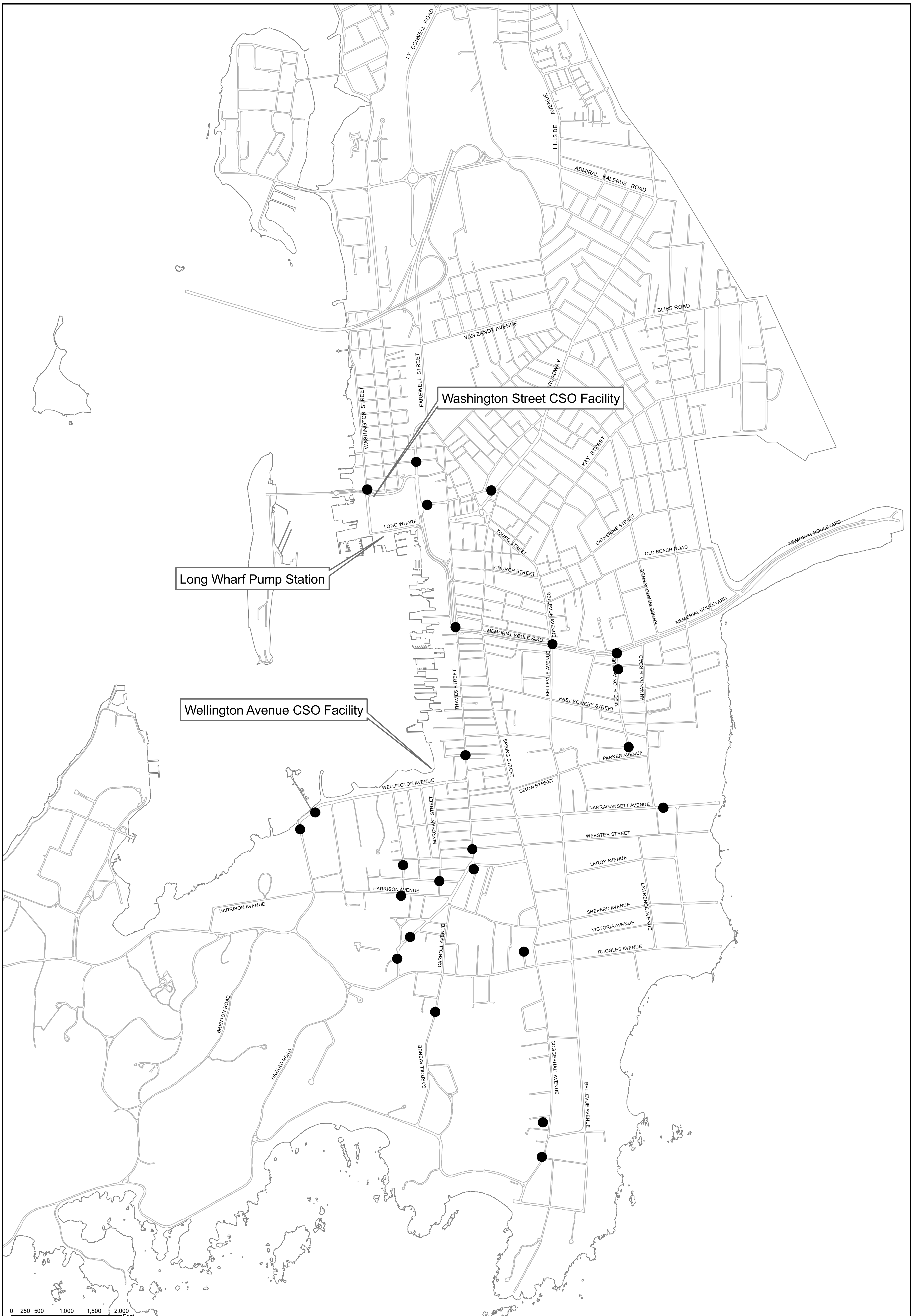

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Figure 4.6
Model Baseflow Input Locations
Phase 2 CSO Control Plan

● Input Locations

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Simulation Control Parameters

The MU model uses two separate result files, the runoff simulation, and the network simulation. Table 4.4 and Table 4.5 present the time step information used for the model computation for both the runoff and network simulations, respectively. All other MU default simulation control parameters were unchanged.

TABLE 4.4
HYDROLOGIC MODULE SIMULATION CONTROL PARAMETERS

Hydrologic Module	
Fixed Time Step	60
Time Step Dry Weather	60
Time Step Wet Weather	300
Time Step FRC	60
Time Step SRC	4.00

TABLE 4.5
HYDRAULIC MODULE SIMULATION CONTROL PARAMETERS

Hydraulic Module	
Model Type	Dynamic Wave
Time Step (min)	4 seconds
Time Step (max)	60 seconds
Factor	1.30
Save rate	Every 5 minutes

4.1.3 Model Calibration Approach

Calibration is used for nearly every kind of scientific modeling. Physically based models generally have some parameters that can be directly measured and others that cannot. Calibration determines the values of non-measurable parameters that satisfy the input/output relationship of the modeled system. This is accomplished by running the model using incremental iterations of values for one or more of the unknown parameters. The outputs from each of the model iterations are compared with measured values for the output parameters (such as flow, for a hydrologic model). When the modeled output closely and consistently matches the measured output, the model is considered calibrated.

The procedure for selecting parameter values to calibrate each of the flow components is complex. It requires a detailed understanding of the relationship between parameter values defined in MU and the resulting simulated flow response. The calibration procedure typically begins by first defining the less variable components of flow, such as base wastewater flow and dry weather infiltration. Therefore, the initial steps of calibration involve comparing and calibrating model simulations to records collected during periods of dry weather. After dry weather calibration is completed, the effort focuses on matching simulation results to recorded wet weather flows. In general, the procedure first involves targeting particular periods of the observed flow record to match hydrograph volume and then peak flow and shape.

Dry Weather

The first step in the calibration process for each tributary basin is to match simulated flows with flows measured during dry weather. The dry weather flows used for calibration were the minimum flows measured over the course of the metering period.

Dry weather flow estimations quantify the sanitary load to the sewer system, while the repetitive profiles describe any temporal dry weather loading variations. The dry weather flow and repetitive profiles are calculated using measured flow during non-rainfall periods.

Dry weather flows in MU include two components: the daily diurnal pattern, which consists of wastewater generated by domestic, commercial or industrial development; and a constant flow that represents dry weather infiltration. After wastewater is accounted for, the remaining flow, represented as a constant flow in the model, is assumed to be dry weather infiltration. For this calibration effort, base infiltration was not estimated separately from wastewater flow. Therefore, in the model, the parameters used to estimate wastewater flow include base infiltration.

The pattern shape is derived directly from dry weather flow monitoring data available for the basin being calibrated. Diurnal patterns are daily, repeatable flow patterns normally found during periods of dry weather. They usually include a component of dry weather infiltration (assumed constant) in addition to the municipal waste stream. Diurnal patterns were determined from measured flow data at each isolated flow meter location for each sewer catchment area.

Wet Weather

Rain data were collected at 15-minute increments for all metering programs by a tipping bucket rain gauge located on the roof of the Long Wharf Pump Station. All designated catchment areas for the City of Newport were assigned rainfall data as recorded at the Long Wharf Pump Station.

Wet weather calibration was performed by comparing the modeled response to the data observed by the meters. This is discussed in Sections 4.2 and 4.3.

Fast Response Component (FRC) - Surface Runoff

As noted in Section 4.1.1, the Model A parameters were used to generate surface runoff. Two main parameters are adjusted in Model A: percent impervious area and time of concentration. Adjustment of the FRC parameters were used in the calibration to match the inflow peak(s) observed in the flow meter data. Time-of-concentration was used to improve the timing of the peak. This was one of the last calibration parameters to be adjusted, as the overall water balance (total I/I volume using Model A and RDII) needed to be achieved first. The calibrated hydrologic parameters for each catchment area are shown in Table 4.6.

TABLE 4.6
CALIBRATION HYDROLOGIC PARAMETERS FOR FRC

Catchment Area	Percentage of Total Area Contributing Flow to RDII (%)	Time of Concentration (Tc) (Min.)	Initial Loss (in.)
1	6.0	90	0.05
2	3.0	90	0.024
3	0.50	90	0.024
4	6.0	90	0.024
6	6.0	90	0.024
7	4.0	90	0.05
10	3.0	7	0.05
11	5.0	20	0.1
12	5.0	20	0.1
13	6.0	90	0.024
WM	6.0	7	0.05

* - WM represents a small unmetered catchment area in the vicinity of West Marlborough Street.

Slow Response Component (SRC) - Rainfall Dependent Infiltration and Inflow

The first step in calibration was to adjust the RDII area percent to balance the total volume of water in the system. An easy way to determine this is to set interflow storage (Lmax) artificially high to send all flow to baseflow, and then match this volume to "dry" periods. Also, the baseflow time constant (CK_{bf}) was set artificially high so baseflow was constant.

RDII parameters that were used to adjust hydrograph magnitude and shape included:

- Magnitude parameters: Umax, Lmax
- Shape/Timing parameters: CK, CK_{IF}, CK_{bf}
- Others: CQof, thresholds

The overland flow component is mainly controlled through the parameters Umax and CK. Umax mainly controls the magnitude of overland flow, but can also affect interflow. Umax can be thought of as the size of the soil storage reservoir. Typically, the larger the reservoir, the less overland flow is generated (and vice versa). The magnitude can also be increased by lowering the overland time constant (CK). The overland time constant will affect the shape of the overland flow – lower values are used for obtaining sharper peaks in the hydrograph.

The overland coefficient (CQof) also has a significant impact on overland flow. The CQof is a value between 0 and 1, where the higher the number, the more flow is sent to the pipe network. The lower the number, more flow is sent into the soil reservoirs. The CQof is very sensitive and can quickly change the simulation results. In most cases, this value was kept at 0.5 until it became apparent that the other parameters could not complete the calibration.

Interflow is the most difficult flow component to calibrate. The interflow flow component is mainly controlled through the parameters Lmax and CK_{IF}. Lmax mainly controls the magnitude of interflow; however, it can also affect overland flow and baseflow. Lmax can be thought of as the size of the lower zone storage reservoir. The larger the lower zone storage reservoir, the more interflow is available to the collection system. To increase the magnitude of interflow, a higher Lmax value should be used. The magnitude can also be increased by lowering the interflow time constant (CK_{IF}). The interflow time constant will affect the shape of the interflow, where lower values will produce sharper spikes in the hydrograph.

The threshold parameters (overland, Tof, and interflow, Tif) can be used to help with the timing of the relative water content in the root zone. It is used to build up antecedent soil moisture before allowing water to get into the collection system (an analogy would be a stormwater detention basin that would fill up to a specified level before it is allowed to discharge to the downstream surface water). No other RDII parameters were adjusted. Calibrated hydrologic parameters are shown in Table 4.7.

TABLE 4.7
CALIBRATION PARAMETERS FOR SRC

Catchment Area	Percentage of Total Area Contributing Flow to RDII (%)	Umax	CK	Lmax	CKif	CQof
1	75	0.394	15	3.94	150	0.3
2	15	0.394	15	3.94	150	0.3
3	60	0.394	15	3.94	150	0.3
4	15	0.394	15	3.94	150	0.3
6	50	0.394	15	3.94	150	0.3
7	25	0.394	15	3.94	150	0.3
10	50	0.394	15	3.94	150	0.3
11	8	0.394	15	3.94	150	0.3
12	8	0.394	15	3.94	150	0.3
13	85	0.394	15	3.94	150	0.3
WM	80	0.394	15	3.94	150	0.3

The following sections present the calibration results of the Wellington Avenue and Washington Street model based on the parameters presented in Tables 4.6 and 4.7.

4.2 Wellington Avenue CSO Facility Tributary Area Calibration

Calibration results for the Wellington Avenue CSO Facility and Washington Street CSO Facility tributary area models are presented in this section and in Section 4.3.

Generally accepted calibration criteria were established at the outset of the project. At each calibration location for dry and wet weather, calibration included volume and peak error within ±10%. The correlation coefficient (R^2) is also an indicator of similar trends between the modeled and metered flow data. While an R^2 value of 0.7 or greater is preferred, variations in diurnal flow or erratic flow patterns such as an upstream pumping station can reduce the R^2 parameter's effectiveness. For the objectives of this study, volume and peak errors were used as calibration criteria, and R^2 was used as a secondary indicator.

As presented in the TM for Calibration of the Wellington Avenue CSO Facility Tributary Area (Appendix C), the area tributary to the Wellington Avenue CSO Facility was initially calibrated using data acquired by flow metering during the fall of the 2007. The sewage flows observed during 2007 were generally low because of minimal rainfall observed over the metering period and dry antecedent conditions. The generally low system flows during the period resulted in no overflows at the Wellington Avenue CSO Facility.

Due to the limited wet weather flow data collected during the dry flow metering period, the initial calibration did not adequately simulate wet weather conditions for evaluation of CSO controls. . The Wellington Tributary area was recalibrated by selecting runoff and rainfall dependent inflow and infiltration (RDII) parameters to match flows to the 2005 flow metering period. Since improvements to the sewer system were made in some of the catchment areas tributary to the Wellington Avenue CSO Facility, such as disconnections of roof leaders, a catchment area that did not have changes needed to be selected for the recalibration. As no sewer system improvements were made in Catchment Area 2, this area was selected to be calibrated to both the 2005 and 2007 flow metering periods. Runoff and RDII parameters were reselected to match the volumes observed during the 2005 metering period and adjusted to reproduce the flow volumes and peaks observed during 2007. Once the RDII and Runoff parameters were established for Catchment Area 2, those parameters were used in each of the other Wellington Avenue CSO facility tributary areas. Adjustments were made to the percentage of contributing area for Model A Runoff and RDII to calibrate each of the areas to the flows observed in 2005 and in 2007. In some areas, flows in 2005 and 2007 diverged during calibration. In areas were this divergence in flows was observed, it was assumed that the flows in 2007 would control and the percentage of flow from Runoff and RDII was adjusted accordingly to simulate the inflow observed during the 2007 period. Figures 4.7a and b and 4.8a and b present a comparison of simulated and observed flows for representative areas. Appendix F includes calibration plots for each of the metered catchments.

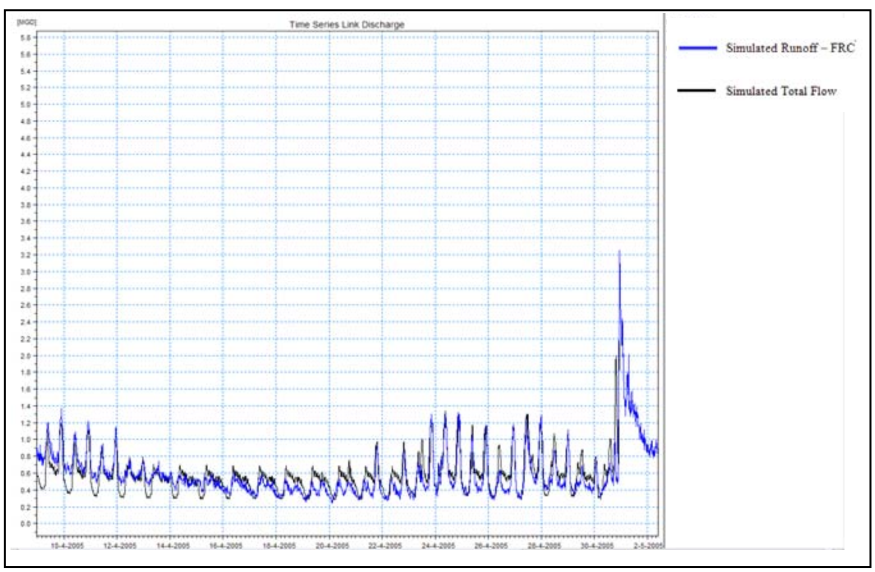

Figure 4.7a
Sample Comparison of 2005 Simulated and Observed Flows
2005 Catchment Area 4 – Wellington Avenue

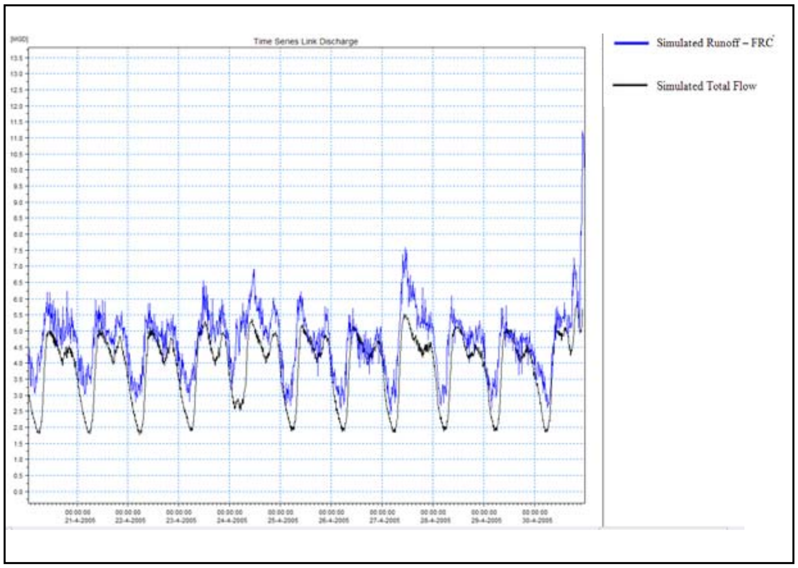

Figure 4.7b
Sample Comparison of 2005 Simulated and Observed Flows
2005 Catchment Area 6 – Terminus of the Thames Street Interceptor

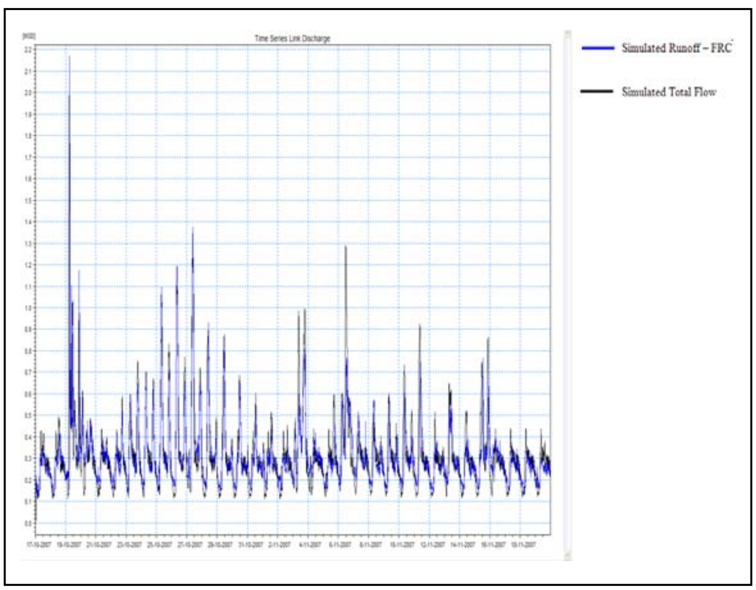

Figure 4.8a
Sample Comparison of 2007 Simulated and Observed Flows
2007 Catchment Area 4 – Wellington Avenue

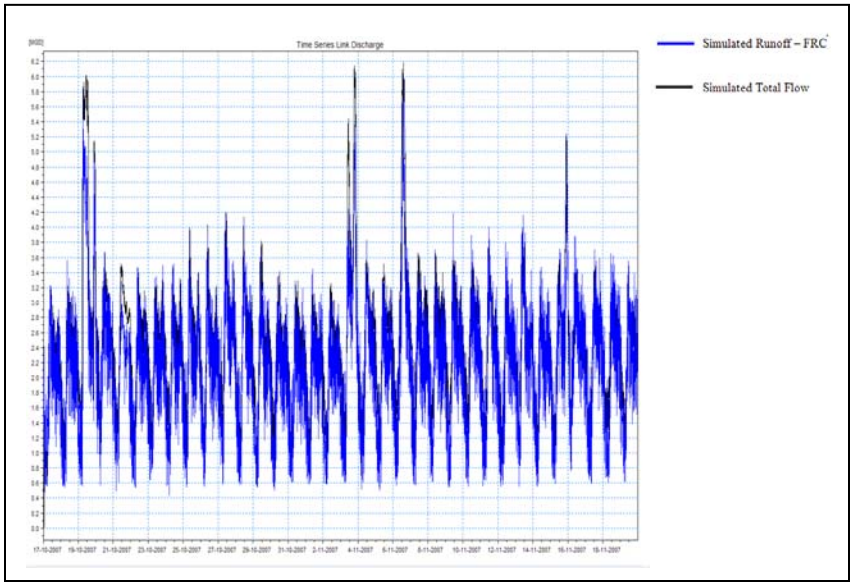

Figure 4.8b
Sample Comparison of 2007 Simulated and Observed Flows
2007 Catchment Area 6 – Terminus of the Thames Street Interceptor

The simulated flows were also calibrated for depth of flow at each meter location to ensure that the simulated hydraulic grade line was similar. The calibration of flow depth is particularly important in a CSO system where sedimentation or other conditions that increase the hydraulic grade line can reduce available capacity, result in additional overflows, and overflow volumes. Complete model output for the calibration of Wellington Avenue CSO Facility Tributary Area is presented in Appendix F.

4.2.1 Wave Avenue Pump Station Flow

Flows from the Wave Avenue Pump Station were metered in 2005, 2006, and 2007. For the calibration of the Wellington Avenue CSO Facility Tributary Area, flows from the Wave Avenue Pump Station metering were directly input into the model as boundary condition flows.

4.3 Washington Street CSO Facility Tributary Area Calibration

Similar to the calibration for the Wellington Avenue CSO Facility Tributary Area in the spring of 2008, the Washington Street CSO Facility Tributary Area flow metering program was generally dry with minimal rainfall and no overflows at the CSO facility. In addition to the limited wet weather flows observed during the period, a series of anomalous flows were observed. These anomalies were generally periods of no or minimal flow with continuously increasing depth measurements. These anomalies were observed in Meter W2, located on the sewer line carrying flows from the Hillside Avenue and Prescott Hall neighborhoods with an overflow to the Washington Street CSO facility on Marsh Street. The meter is located on Washington Street directly west of the Washington Street CSO Facility.

Due to the limited wet weather flow meter data for the Washington Street Tributary area, the calibrated wet weather parameters obtained in Catchment Area 2 of the Wellington Avenue CSO Facility Tributary area were also extended to the Washington Street area. Using a similar methodology as described in the previous section, the percentage of contribution area for the Model A runoff and RDII flows were adjusted using the wet weather parameters to calibrate the flow volumes and peaks observed in the spring 2008 flow metering period. Using the calibrated wet weather parameters, simulated flow volumes generally correlated with observed flows, however, it was noted that the operations of the Long Wharf Pump Station greatly affect flow patterns (i.e. flow reversal or stagnant flow).

Figure 4.9 presents a representative plot of the simulated and observed flows from Catchment Area 10 – Prescott Hall/Hillside Avenue. Complete model output for the Washington Street CSO Facility Tributary Area is included in Appendix F.

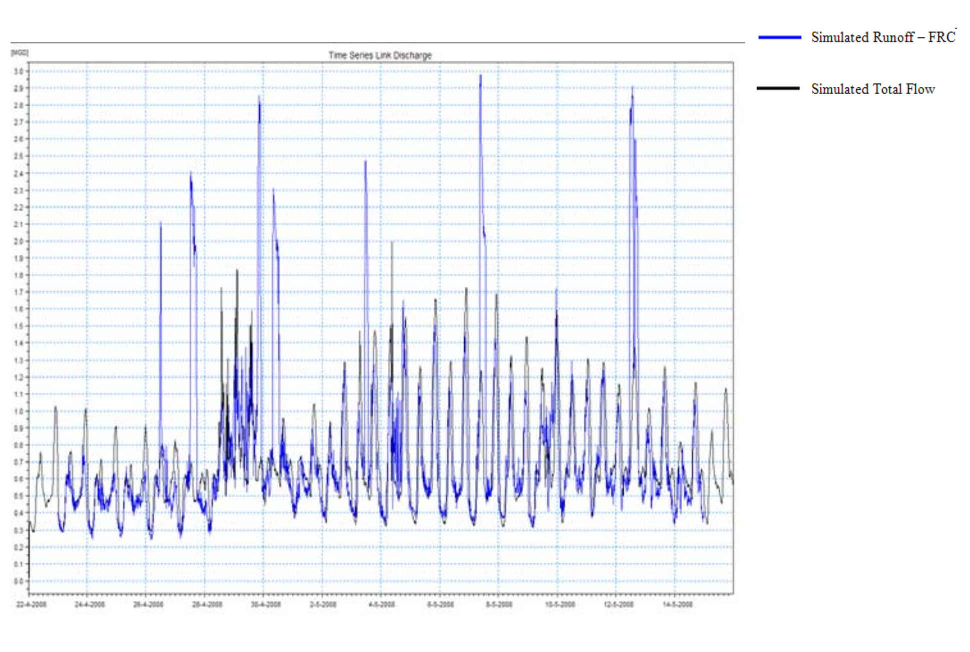

Figure 4.9
Sample Comparison of 2007 Simulated and Observed Flows
2008 Catchment Area 10 – Prescott Hall/Hillside Avenue/Railroad Easement 18-inch Sewer

4.4 Calibration Of Flows To The WPCP

Once the tributary areas were calibrated to one or more sets of flow metering data, flows and pumps were adjusted to accurately simulate flow at the Water Pollution Control Plant (WPCP). Flows were calibrated using flow data recorded at the WPCP during the flow metering periods used in the calibration of the Wellington and Washington CSO Tributary areas and other select periods. The simulated flows for each metering period were compared to flows observed at the WPCP. Initially, flows were determined to be lower than observed at the WPCP, however, flows exiting the Wellington Tributary areas correlated well to the respective meter data, therefore, all flows reaching the WPCP were adjusted using runoff and RDII parameters in the Washington Street CSO tributary areas, and using the Long Wharf Pump Station to convey flow to the WPCP. By adjusting the wet weather contribution from the Washington Street Area and the pump levels at the Long Wharf Pump Station, flows at the WPCP generally correlated with the flow patterns observed at the WPCP.

The runoff and RDII parameters for the Washington Street tributary areas and the pumps at the Long Wharf Pump Station were adjusted to accurately simulate flow volumes to the WPCP. The adjustments to the catchment areas were generally minor. Flows to the WPCP from the Long Wharf Pump Station and Washington Street CSO facility (via dewater pumps during dry weather) account for approximately 85% of total flow at the WPCP. The remaining 15% of flows contributed by the US Navy properties in Newport, Portsmouth, and Middletown, Middletown's Coddington Highway Pump Station, and a small portion of Newport's gravity sewer system directly tributary to the WPCP. Table 4.8 below presents the observed flows received at the WPCP from the Navy and Middletown. The observed flows from the Navy and from Middletown's Coddington Highway Pump Station typically vary between 10 and 13.5 percent of total WPCP flow. Please note that this calculation does not include additional flows from the Newport system that are conveyed directly to the WPCP. Based on these data, the 15 percent estimate is conservative. This gravity system accepts flow from the commercial district adjacent to the treatment plant in addition to pumped flows from the Maple Avenue Pump Station. The Dyre Street Pump Station also pumps directly to the WPCP.

Table 4.8
Comparison of Flows Originating from the Navy and Middletown
Direct to Newport WPCP
April - May 2006

Date	WPCP Influent Flow		Coddington Cove/Navy	Coddington Point (Calc'd)	Training Station Pump Station*	Coddington Highway (Middletown) (Calc'd)		% of Total Flow*
	MGD	gal	gal	gal	gal	Kgal	gal	
Apr-06	197.5	197,500,000	13,760,600	2,962,500	2,547,900	5142	5,142,000	11.1%
May-06	373.6	373,640,000	28,358,600	5,604,600	4,038,000	11176	11,176,000	12.1%

* Excluding flows entering the WPCP from the WPCP Tributary Newport Gravity Sewer

After calibration, the simulated system flows were generally within 10% of the observed flow at the WPCP during the 2006, 2007, and 2008 evaluation periods.

4.5 CSO Overflow Calibration at the Wellington Avenue CSO Facility

The final calibration step was to calibrate the CSO facilities to the observed CSO overflows. Due to ongoing system improvements in the Wellington Tributary area, a recent large storm generating a CSO was selected. The September 26 and 27, 2008, storms were selected as a calibration CSO because it was the most recent and was in a period of normal rainfall and antecedent moisture conditions. The storm resulted in an overflow of approximately 900,000 gallons at the Wellington Avenue Facility and approximately 550,000 gallons at the Washington Avenue Facility over the period. The September 26 storm had a total rainfall of 2.53 inches with a peak hour intensity of 0.59 inches per hour and a duration of approximately 20 hours. The weekend total was 3.73 inches of rainfall with a peak hour intensity of 0.59 and a total duration of 77 hours.

The model simulated an overflow at Wellington Avenue of approximately 2,000,000 gallons and approximately 600,000 gallons at the Washington Street Facility. Similar to the divergence in flow experienced during the calibration of the Wellington Avenue CSO Facility Tributary Area, the reductions in flow did not have a proportionate change at each facility. While the September storm was over predicted, other storms that generated CSO events were simulated to observe the behavior of the simulated CSO facilities. Data recorded during CSO events from 2005 and 2006 were compared to simulated flows under various wet and dry antecedent conditions and it was observed that the overflows typically fell between the two conditions. These results generally indicate that the model was producing accurate results for each of the antecedent conditions and when run for the typical year, the rainfall record input simulates the antecedent conditions. Table 4.9 presents a comparison of observed and simulated overflow volumes at the Wellington Avenue CSO Facility. Complete model output for the Wellington CSO Facility overflow calibration is included in Appendix E.

TABLE 4.9
COMPARISON OF OBSERVED AND SIMULATED CSO VOLUMES AT WELLINGTON AVENUE CSO FACILITY

Date of Observed Overflow	Measured Volume of Overflow (MGD)	Simulated Overflow Volume – (MGD)
May 2006	16,051,536	17,436,790
April 2005	5,383,580	5,440,353

4.6 Rainfall Analysis Methodology

The calibrated model requires rainfall data input to simulate CSO events and to evaluate CSO control alternatives. To develop the rainfall database to be used for alternatives analysis, rainfall data records were researched through the National Climatic Data Center (NCDC) and also via local agencies like the United States Geologic Survey, the U.S. Navy and local universities. The research yielded three data sets: 1) T.F. Green Airport in Providence, 2) Newport Rose Island, and 3) the University of Rhode Island at Kingston, RI. The rest of the NCDC data available for any rain gauges within the Newport area were significantly incomplete or lacked rainfall data completely and therefore were not selected to be analyzed any further. NCDC rainfall data collected at Rose Island in Newport is geographically closest to the project area, however, it does not encompass a long enough period to support long-term statistical analysis needed to determine the representative (i.e., average) rainfall year. Therefore, a rainfall data set with a longer period of record was required. Both data sets from Providence and Kingston were analyzed to determine which data set would be appropriate to use as a surrogate rainfall data set for Newport.

4.6.1 Methodology

Data Sources

The data sets from Providence and Kingston were analyzed to determine which one would be better suited to use as a surrogate rainfall data for Newport. A summary of the three data sets used in the rainfall analysis is shown in Table 4.10.

TABLE 4.10
SUMMARY OF RAINFALL DATA SETS

Station ID (Location)	Data Source	Type of Data	Years on Record	Total Days of Data Available
Newport Rose (Newport, RI)	NCDC	Hourly, recorded in tenths of inches	1996 to 2002	500
T.F. Green Airport (Providence, RI)	NCDC	Hourly, recorded in hundredths of inches	1948 to 2008	8,472
URI - Kingston (Kingston, RI)	University of Rhode Island	Daily, recorded in hundredths of inches	1893 to 2007	35,099

Data Preparation

As Table 4.10 indicates, all three data sets differed in the type of data, years on record and total days of data available. In order to perform the rainfall analysis, all three sets were converted into a consistent format for an overlapping time period. The Newport Rose and T.F. Green Airport data were converted to daily rainfall data sets by using only the daily rainfall totals, which are provided in NCDC's hourly rainfall records. Newport's rainfall data was stored in hundredths of inches; however it is observed and available to tenths only. The other two data sets, which are provided in hundredths of inches, were rounded to the nearest tenths to create a consistent record type.

The three data sets were refined by eliminating daily records that did not appear in all three data sets. The resulting data sets were checked for major periods of inconsistency. For example, the Newport Rose data had a few unrealistically prolonged periods in which zero rainfall was recorded when the other two stations contained records of rainfall. It was assumed that the Newport station was not functioning properly during these periods and these days were removed from consideration in the analysis. At the end of the data refining process, a total of 445 days had rainfall records for all three stations. The three 445-day data sets were used for the rainfall analysis.

Statistical Analysis

Using Microsoft Excel's correlation data analysis tool, the Newport data set was compared to the other two data sets. The correlation analysis tool examines each pair of data sets to determine whether the two data sets show a tendency to vary together. If two data sets are closely correlated to each other, their correlation will approach 1 and if the two data sets are unrelated, their correlation will approach 0. The correlation factors betweens the three rainfall data sets are shown in Table 4.11.

TABLE 4.11
CORRELATION FACTORS OF RAINFALL DATA SETS

	Newport	Providence	Kingston
Newport	1.00	-	-
Providence	0.83	1.00	-
Kingston	0.63	0.67	1.00

Figure 4.10 presents a comparison scattergraph of the Newport data set to the Providence and Kingston data sets. The Newport data is represented on the horizontal axis and the comparative data set, either Providence or Kingston, is represented on the vertical axis. A thick diagonal line labeled "Perfect Agreement" represents the ideal correlation between the two data sets, meaning that if the data sets were perfectly correlated, their corresponding daily rainfall totals would fall along this line. As the correlation factors in Table 4.11 suggest, the Newport and Providence comparison generally falls closer to the line than the Newport and Kingston comparison.

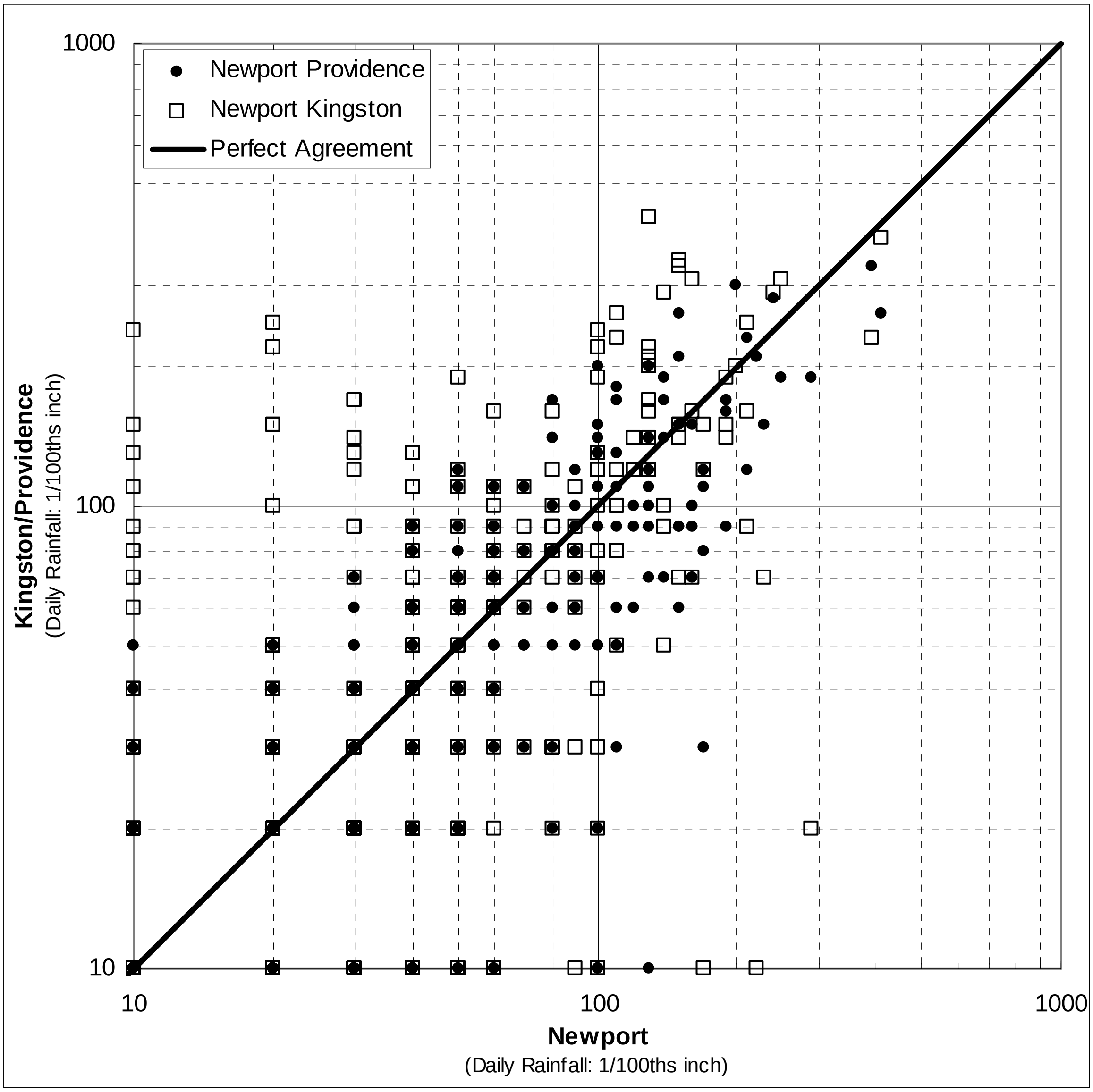

Figure 4.10
Rainfall Data Set Comparisons: Newport to Providence and Newport to Kingston

4.6.2 Selection of Rainfall Data Source

Based on the incompleteness of rainfall data sets available for Newport, a surrogate rainfall data set is required for the determination of the typical year to use in the hydraulic modeling analysis of CSO control alternatives. The T.F. Green Airport in Providence has a complete data set dating back to 1948 and is more closely correlated to the available Newport rainfall data than the Kingston rainfall data. As a result of the close correlation of the T.F. Green Airport rainfall data with the Newport rainfall data, the rainfall data from the T.F. Green Airport will be utilized in the hydraulic model to evaluate CSO events for existing conditions and proposed CSO control alternatives.

4.7 Long Term Rainfall Analysis

Narragansett Bay Commission (NBC) conducted an extensive rainfall analysis using the data from T.F. Green as part of the Combined Sewer Overflow Control Facilities Program. Design storms were synthetically developed based on statistical analysis of the rainfall data. In addition, the period of record (1948 through 1982) was analyzed to determine the typical average year of rainfall. The NBC's analysis concluded that the years of 1951 and 1978 would be used to develop annual statistics of CSO volumes and frequencies for CSO control alternatives evaluations.

4.7.1 Updated Rainfall Analysis

AECOM performed an updated analysis of the full rainfall database for the T.F. Green Airport to determine if 1951 and 1978 would be utilized for the evaluation of CSO control alternatives.

Analysis of 1951 and 1978

As noted above, a typical period analysis was conducted by the Narragansett Bay Commission (NBC) for use in their CSO Control Facilities Program (NBC, 1997). This analysis selected the year 1951 as the typical year based on total annual volume and total number of storms. Table 4.12 is from the NBC report and compares the number of storms in 1951 to another possible year, 1978.

TABLE 4.12
COMPARISON OF 1951 AND 1978 FROM NBC REPORT

Year	Total Precipitation (in)	Number of Storms	Average Storm depth (in)	> 1 Year	> 3 Month	> 1 Month
1951	45.60	96	0.48	1	7	16
1978	47.01	72	0.65	2	10	20

The rainfall from T.F. Green Airport rain gage was analyzed to characterize the storms within the year 1951. The inter-event time defines the amount of dry weather in between storm events. A minimum inter-event time of 10 hours was used to define storm events in order to obtain the same number of storms as cited in Table 4.12 (96 storms). For each storm the total depth, intensity, and duration was calculated.

Statistics for various design storms (1 year, 3 month, etc.) were estimated from a graph provided in the NBC report. This figure is reproduced in Figure 4.11.

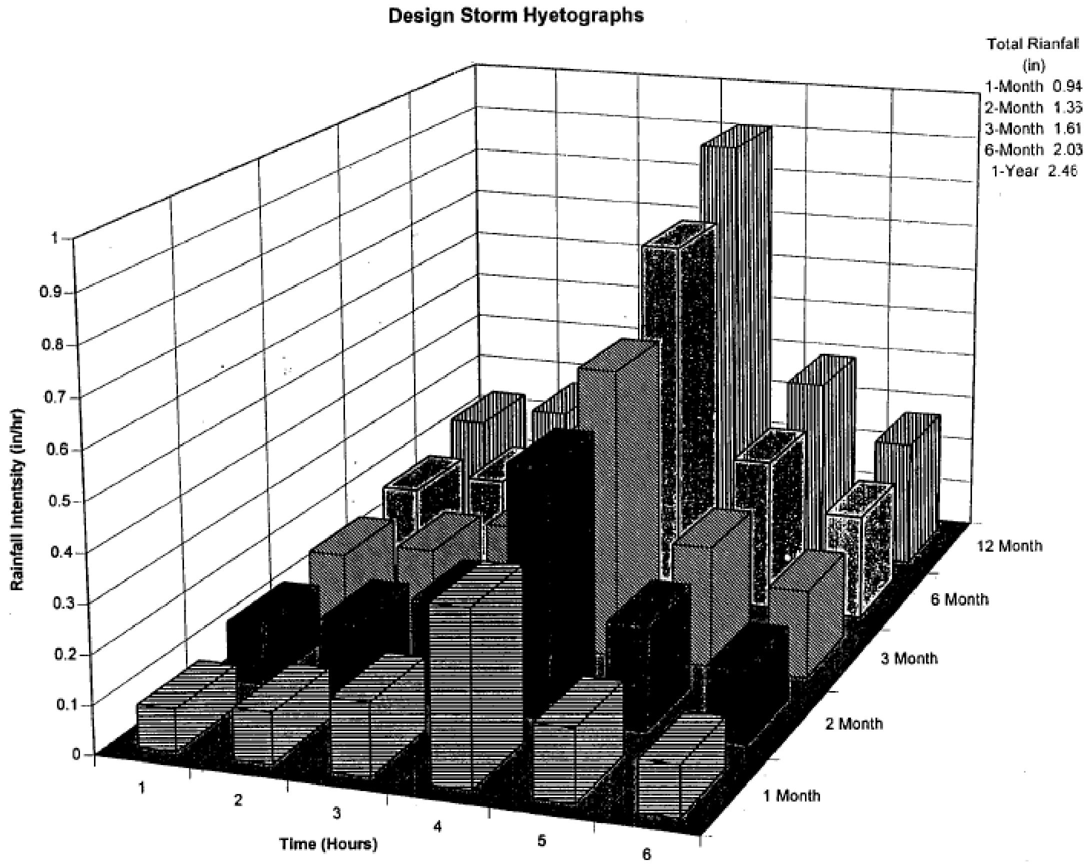

Figure 4.11
Design Storms From NBC CSO Facilities Plan Report

The statistics for storms in 1951 were compared to the design storms from the NBC report. Table 4.13 summarizes this comparison.

TABLE 4.13
SUMMARY OF STORMS IN 1951 BASED ON NBC DESIGN STORMS

Return Period	Total Depth (in)	Peak Intensity (in/hr)	# Storms in 1951 based on Intensity	# Storms in 1951 based on Depth
1 month	0.94	0.38	8	8
2 month	1.36	0.55	1	3
3 month	1.61	0.62	0	7
6 month	2.03	0.78	0	0
1 year	2.46	0.90	1	0

It should be noted that the comparison of total storm depth to design storm depth is not completely valid since 1951 storm durations may not necessarily be the same as the design storms. However, the comparison is still useful. As noted in Table 4.13, the rainfall data presented in the NBC report includes 16 one-month storms, 7 three-month storms, and 1 one-year storm within this period. However, overflows are, in many cases, driven by storm peak intensities. In terms of peak intensity there is only one storm greater than a 1 year storm in peak intensity and no others greater than a 3- or 6-month. It is understood that for the NBC CSO Control Plan, CSO controls were designed to capture up to a 3-month storm. However, since the requirements for the CSO controls for the Wellington Avenue CSO Facility require zero discharges in a typical year, AECOM determined that analysis of the entire period of record at the T.F. Green Airport rain gage was required to develop another typical period that included one-year storms for depth and a one-year storm for peak intensity.

4.7.2 Selection of Typical Year for Long Term Rainfall Analysis

The first step in selecting a typical period is to determine the definition of “typical.” Average annual rainfall statistics are compared to potential years. Hourly rainfall data is available from T.F. Green Airport starting in May 1, 1948 to present day. Storms in all complete years at T.F. Green Airport (1949-2007) were analyzed to characterize typical annual rainfall.

The 59 years of rainfall data were analyzed to determine the minimum inter-event time that should be used to distinguish rainfall events. The inter-event time is defined as the number of hours between storm events. It is common practice to define a minimum inter-event time such that the coefficient of variation of inter-event times is equal to one. This is done to achieve statistical independence in rainfall events. A minimum inter-event time of 6 hours was used for this analysis. Storm events were identified and summarized based on total duration, peak intensity, and total depth. It is important that the selected typical year contain storms in a range of depth and intensity ranges. Therefore, the number of storms in various intensity and depth ranges was calculated for each year.

Table 4.14 is a summary of the long-term statistics for T.F. Green Airport. To assess the effect of potential climate change, the long-term averages for the period of record were compared to the averages for the past 10 and 30 years. According to Table 4.14, there are small differences in statistics if using the past 10 years, 30 years, or the period of record. Therefore, it was determined that the typical year would be selected based on a comparison to statistics describing the entire period of record.

TABLE 4.14
LONG-TERM STATISTICS FOR T.F. GREEN AIRPORT RAIN GAGE

Statistic		Entire Period Average	Entire Period Std. Dev	Last 30 Years Average	Last 30 Years Std. Dev	Last 10 Years Average	Last 10 Years Std. Dev
Total # Storms		105.8	9.2	105.6	9.8	107.9	11.0
Total Depth (in)		45.0	7.6	45.8	7.2	46.4	6.0
Count of Storms with Depths	LT 0.25	60.6	8.5	60.3	9.4	64.1	10.1
	0.25 to 0.5	16.1	4.2	15.7	4.3	14.6	4.2
	0.5 to 1	15.7	3.3	16.2	2.9	15.3	2.2
	1 to 2	9.6	3.4	9.4	3.4	9.8	3.1
	2 to 2.5	1.8	1.3	1.8	1.4	1.3	0.9
	GT 2.5	1.9	1.4	2.2	1.3	2.8	1.5
Count of Storms with Intensities	LT 0.1	61.7	8.7	60.6	9.5	63.4	9.9
	0.1 to 0.25	26.3	4.3	26.7	5.3	25.7	4.9
	0.25 to 0.5	13.6	3.9	14.2	3.9	14.9	3.7
	0.5 to 1	3.6	2.1	3.6	1.7	3.4	1.8
	GT 1	0.5	0.7	0.5	0.7	0.5	0.7

Each year was scored based on how far the year's statistics deviated from the long-term average. For each year, the number of standard deviations away from the long-term average was calculated for various statistics such as total depth and number of storms in various intensity and depth ranges. These numbers were added together to develop a score; the year with the lowest score is defined to be most typical. Table 4.15 illustrates the statistical categories used and the score for the typical year selected by NBC, 1951.

TABLE 4.15
ILLUSTRATION OF SCORING SYSTEM

Statistic		Period of Record Avg	Period of Record Std Dev (σ)	1951 Count	Number of σ away from avg
# Storms		105.8	9.2	105	0.1
Total Depth		45.0	7.6	45.6	0.1
Count of Storms with Depths	0.25 to 0.5	16.1	4.2	13	0.7
	0.5 to 1	15.7	3.3	19	1.0
	1 to 2	9.6	3.4	15	1.6
	2 to 2.5	1.8	1.3	0	1.4
	GT 2.5	1.9	1.4	0	1.4
Count of Storms with Intensities	0.1 to 0.25	26.3	4.3	22	1.0
	0.25 to 0.5	13.6	3.9	19	1.4
	0.5 to 1	3.6	2.1	3	0.3
	GT 1	0.5	0.7	1	0.7
SCORE FOR 1951 =					**9.6**

Each year in the dataset was scored following the methodology illustrated in Table 4.15. Table 4.16 summarizes the scores for the top ten years in the period of record.

TABLE 4.16
SUMMARY OF TOP TEN TYPICAL YEARS IN PERIOD OF RECORD

Statistic		**Ave**	**1996**	**1991**	**1994**	**1973**	**1974**	**1960**	**1984**	**1990**	**1971**	**2004**
# Storms		105.8	112	99	101	104	111	107	99	110	96	109
Total Depth		45.0	44.61	45.69	45.23	48.12	40.79	40.08	48.74	44.57	38.42	43.49
Count of Storms with Depths	0.25 to 0.5	16.1	20	17	12	16	19	12	18	20	14	10
	0.5 to 1	15.7	17	18	16	19	15	18	19	16	17	15
	1 to 2	9.6	9	12	10	9	10	8	9	13	9	10
	2 to 2.5	1.8	2	1	2	1	2	2	2	1	2	1
	GT 2.5	1.9	2	2	3	3	0	1	1	1	0	3
Count of Storms with Intensities	0.1 to 0.25	26.3	27	28	26	28	28	24	30	28	24	25
	0.25 to 0.5	13.6	14	14	11	15	12	13	13	14	14	16
	0.5 to 1	3.6	4	3	5	5	4	3	4	5	3	2
	GT 1	0.5	0	1	0	0	0	0	0	0	1	0
	Score	**---**	**3.6**	**4.6**	**4.8**	**5.4**	**5.4**	**5.5**	**5.6**	**5.7**	**6.1**	**6.2**

Based on this analysis, the year 1996 was selected as the most typical year in the period of record at T.F. Green airport. However, the year does not match long-term averages exactly. In particular, the year 1996 does not have any storms with a peak intensity greater than 1 inch/hour though there is often one such storm according to long term averages. Therefore, the year 1996 was "typicalized" in order to make a period more reflective of long-term averages. There is precedence for this in other EPA-approved studies, most notably the typical year developed for the Massachusetts Water Resources Authority's CSO Facilities Plan (MWRA, 1997) in which several storms were inserted and removed to create a typical year. The June 11, 2001 storm event was inserted into the 1996 rainfall time series. This storm had a 1.07 in/hr peak intensity, 2.02 inches total depth and lasted 11 hours. Analysis of all storms within the period of record indicates that the average inter-event time for storm events is approximately 76 hours. In addition, most of the storms with peak intensities greater than 1 inch/hr (78%) occurred during the summer months (June, July, and August). Therefore, the June 11, 2001 storm was inserted in June 13, 1996. This period in 1996 allows for approximately 76 hours of inter-event time before and after the storm and occurs during a period where high intensity storms are more likely to occur.

Table 4.17 is a summary of all the storms within the selected typical year, 1996, which was selected for the long term analysis of CSO control alternatives.

TABLE 4.17
SUMMARY OF STORM EVENTS IN THE TYPICAL YEAR SELECTED FOR LONG TERM RAINFALL ANALYSIS

Number	Start	End	Depth	Intensity	Duration	Notes
1	01/02/1996 8:00	01/03/1996 10:00	0.29	0.04	26	
2	01/07/1996 18:00	01/08/1996 11:00	0.18	0.03	17	
3	01/10/1996 0:00	01/10/1996 2:00	0.05	0.02	2	
4	01/12/1996 14:00	01/12/1996 22:00	1.08	0.31	8	
5	01/17/1996 1:00	01/17/1996 7:00	0.03	0.01	6	
6	01/19/1996 14:00	01/19/1996 19:00	0.98	0.5	5	
7	01/24/1996 10:00	01/24/1996 22:00	0.85	0.29	12	
8	01/27/1996 7:00	01/27/1996 22:00	1.42	0.48	15	
9	01/29/1996 21:00	01/30/1996 0:00	0.13	0.06	3	
10	01/31/1996 10:00	01/31/1996 10:00	0.01	0.01	0	
11	02/02/1996 7:00	02/02/1996 10:00	0.02	0.01	3	
12	02/03/1996 0:00	02/03/1996 4:00	0.1	0.03	4	
13	02/09/1996 0:00	02/09/1996 4:00	0.12	0.05	4	
14	02/11/1996 10:00	02/11/1996 14:00	0.27	0.11	4	
15	02/14/1996 9:00	02/14/1996 13:00	0.16	0.07	4	
16	02/16/1996 14:00	02/16/1996 18:00	0.05	0.02	4	
17	02/21/1996 7:00	02/21/1996 7:00	0.01	0.01	0	
18	02/21/1996 15:00	02/21/1996 23:00	0.68	0.3	8	
19	02/23/1996 1:00	02/23/1996 1:00	0.01	0.01	0	
20	02/23/1996 9:00	02/23/1996 9:00	0.01	0.01	0	
21	02/24/1996 6:00	02/24/1996 15:00	0.57	0.27	9	
22	02/27/1996 23:00	02/27/1996 23:00	0.02	0.02	0	
23	02/28/1996 7:00	02/28/1996 10:00	0.13	0.11	3	
24	03/02/1996 10:00	03/02/1996 18:00	0.3	0.07	8	
25	03/03/1996 6:00	03/03/1996 6:00	0.01	0.01	0	
26	03/03/1996 13:00	03/03/1996 13:00	0.01	0.01	0	
27	03/05/1996 20:00	03/07/1996 1:00	1.08	0.08	29	
28	03/07/1996 8:00	03/07/1996 21:00	0.42	0.06	13	
29	03/08/1996 6:00	03/08/1996 14:00	0.03	0.01	8	
30	03/15/1996 13:00	03/15/1996 23:00	0.17	0.05	10	
31	03/19/1996 19:00	03/20/1996 3:00	0.68	0.49	8	
32	03/26/1996 0:00	03/26/1996 0:00	0.01	0.01	0	
33	04/01/1996 22:00	04/02/1996 7:00	0.59	0.2	9	
34	04/07/1996 12:00	04/08/1996 7:00	0.45	0.06	19	
35	04/09/1996 16:00	04/10/1996 16:00	0.44	0.08	24	
36	04/10/1996 23:00	04/10/1996 23:00	0.01	0.01	0	
37	04/12/1996 8:00	04/12/1996 9:00	0.03	0.02	1	
38	04/12/1996 23:00	04/13/1996 7:00	0.05	0.02	8	
39	04/14/1996 8:00	04/14/1996 8:00	0.01	0.01	0	
40	04/16/1996 4:00	04/16/1996 17:00	2	0.38	13	
41	04/23/1996 23:00	04/24/1996 3:00	0.08	0.03	4	
42	04/26/1996 20:00	04/26/1996 22:00	0.03	0.02	2	
43	04/29/1996 14:00	04/29/1996 19:00	0.69	0.24	5	
44	04/30/1996 4:00	04/30/1996 4:00	0.01	0.01	0	
45	04/30/1996 11:00	05/01/1996 1:00	0.51	0.18	14	
46	05/02/1996 17:00	05/02/1996 17:00	0.02	0.02	0	
47	05/03/1996 11:00	05/03/1996 17:00	0.25	0.13	6	
48	05/05/1996 23:00	05/06/1996 14:00	0.29	0.05	15	
49	05/08/1996 2:00	05/08/1996 8:00	0.18	0.06	6	
50	05/10/1996 7:00	05/10/1996 11:00	0.27	0.16	4	
51	05/11/1996 21:00	05/12/1996 7:00	0.25	0.09	10	

Number	Start	End	Depth	Intensity	Duration	Notes
52	05/16/1996 13:00	05/17/1996 5:00	0.61	0.08	16	
53	05/21/1996 15:00	05/21/1996 15:00	0.18	0.18	0	
54	05/30/1996 6:00	05/30/1996 16:00	0.36	0.14	10	
55	06/03/1996 8:00	06/04/1996 5:00	0.63	0.11	21	
56	06/05/1996 5:00	06/05/1996 6:00	0.07	0.06	1	
57	06/10/1996 4:00	06/10/1996 12:00	0.05	0.02	8	
58	**06/11/2001 14:00**	**06/12/2001 0:00**	**2.02**	**1.07**	**11**	**Start storm on 06/13/1996 at 16:00**
59	06/17/1996 5:00	06/17/1996 5:00	0.03	0.03	0	
60	06/18/1996 0:00	06/18/1996 4:00	0.06	0.03	4	
61	06/19/1996 16:00	06/20/1996 5:00	0.52	0.17	13	
62	06/20/1996 23:00	06/21/1996 1:00	0.18	0.08	2	
63	06/22/1996 19:00	06/22/1996 19:00	0.02	0.02	0	
64	06/24/1996 21:00	06/25/1996 2:00	0.39	0.28	5	
65	06/30/1996 21:00	06/30/1996 23:00	0.22	0.2	2	
66	07/03/1996 3:00	07/03/1996 3:00	0.08	0.08	0	
67	07/03/1996 10:00	07/03/1996 19:00	0.44	0.22	9	
68	07/04/1996 11:00	07/04/1996 12:00	0.33	0.31	1	
69	07/13/1996 1:00	07/13/1996 3:00	1.4	0.92	2	
70	07/19/1996 7:00	07/19/1996 7:00	0.06	0.06	0	
71	07/19/1996 17:00	07/19/1996 17:00	0.03	0.03	0	
72	07/23/1996 10:00	07/23/1996 19:00	0.41	0.2	9	
73	07/26/1996 7:00	07/26/1996 8:00	0.05	0.03	1	
74	07/31/1996 14:00	07/31/1996 19:00	0.42	0.21	5	
75	08/01/1996 2:00	08/01/1996 3:00	0.05	0.03	1	
76	08/01/1996 12:00	08/01/1996 13:00	0.22	0.19	1	
77	08/10/1996 1:00	08/10/1996 2:00	0.07	0.05	1	
78	08/13/1996 2:00	08/13/1996 14:00	0.93	0.15	12	
79	08/23/1996 19:00	08/23/1996 19:00	0.05	0.05	0	
80	08/24/1996 9:00	08/24/1996 10:00	0.38	0.21	1	
81	08/28/1996 2:00	08/28/1996 12:00	0.42	0.12	10	
82	09/01/1996 21:00	09/02/1996 7:00	0.25	0.08	10	
83	09/07/1996 7:00	09/07/1996 22:00	1.16	0.36	15	
84	09/12/1996 11:00	09/12/1996 11:00	0.03	0.03	0	
85	09/13/1996 7:00	09/13/1996 10:00	0.06	0.02	3	
86	09/14/1996 5:00	09/14/1996 5:00	0.02	0.02	0	
87	09/17/1996 3:00	09/18/1996 9:00	2.78	0.7	30	
88	09/22/1996 15:00	09/23/1996 3:00	0.64	0.18	12	
89	09/24/1996 23:00	09/25/1996 1:00	0.21	0.09	2	
90	09/28/1996 23:00	09/29/1996 6:00	0.24	0.08	7	
91	10/08/1996 14:00	10/09/1996 3:00	2.36	0.41	13	
92	10/19/1996 21:00	10/20/1996 14:00	3.05	0.63	17	
93	10/22/1996 6:00	10/22/1996 7:00	0.09	0.06	1	
94	10/23/1996 21:00	10/23/1996 22:00	0.31	0.17	1	
95	10/28/1996 11:00	10/28/1996 13:00	0.21	0.15	2	
96	10/30/1996 12:00	10/30/1996 14:00	0.07	0.03	2	
97	11/07/1996 2:00	11/07/1996 11:00	0.14	0.04	9	
98	11/08/1996 4:00	11/08/1996 4:00	0.06	0.06	0	
99	11/09/1996 7:00	11/09/1996 13:00	0.52	0.11	6	
100	11/19/1996 14:00	11/19/1996 17:00	0.12	0.06	3	
101	11/26/1996 3:00	11/26/1996 19:00	1.4	0.37	16	
102	12/01/1996 17:00	12/02/1996 8:00	1.28	0.46	15	
103	12/06/1996 6:00	12/06/1996 13:00	0.85	0.14	7	
104	12/07/1996 15:00	12/08/1996 2:00	1.5	0.3	11	
105	12/11/1996 20:00	12/11/1996 21:00	0.07	0.04	1	
106	12/12/1996 14:00	12/12/1996 14:00	0.02	0.02	0	

Number	Start	End	Depth	Intensity	Duration	Notes
107	12/13/1996 23:00	12/13/1996 23:00	0.02	0.02	0	
108	12/14/1996 13:00	12/14/1996 18:00	0.14	0.05	5	
109	12/17/1996 1:00	12/17/1996 1:00	0.02	0.02	0	
110	12/17/1996 8:00	12/17/1996 17:00	0.59	0.13	9	
111	12/19/1996 3:00	12/19/1996 20:00	1.12	0.14	17	
112	12/24/1996 20:00	12/25/1996 2:00	0.54	0.22	6	
113	12/29/1996 17:00	12/29/1996 17:00	0.04	0.04	0	

4.7.3 Model Results of Typical Year 1996 under Existing System Conditions

The model was used to simulate flows under existing conditions for the typical year to determine if the year 1996 reasonably generated the CSO volume and number of events in a typical year at the Wellington Avenue CSO Facility. The simulation resulted in the following predicted CSO activity:

- 14 overflows at the Wellington Avenue CSO Facility totaling approx. 22.8 Million Gallons presented in Table 4.18.

These results correlate well with the statistical analysis performed on the CSO overflows at the Wellington Avenue CSO Facility from 2002 through 2008 (period selected based on CSO events recorded after implementation of 2001 operational improvements at the Wellington Avenue CSO Facility and Narragansett Storage Conduit), which resulted in an average number of overflows of 17 per year and an annual volume of approximately 22 million gallons at the Wellington Avenue CSO Facility.

TABLE 4.18
SUMMARY OF PREDICTED OVERFLOWS DURING SIMULATED TYPICAL YEAR 1996
WELLINGTON AVENUE CSO FACILITY
EXISTING SYSTEM CONFIGURATION

CSO Event No.	Total Rainfall (in.)	Peak Intensity (in./hr)	Date of CSO Discharge	CSO Volume (gal)
1	1.08	0.31	1/12	56,510
2	0.98	0.5	1/19	160,236
3	0.85	0.29	1/24	9,272
4	1.42	0.48	1/27	1,070,870
5	2.0	0.38	4/16	2,255,678
6	1.4	0.92	7/13	333,189
7	2.78	0.7	9/18	3,805,481
8	2.36	0.41	10/8	3,434,380
9	3.05	0.63	10/20	6,917,520
10	1.4	0.37	11/26	320,776
11	1.28	0.46	12/2	243,025
12	0.85	0.14	12/6	7,100*
13	1.5	0.3	12/7	3,188,094
14	1.12	0.14	12/19	950,806

* Duration of overflow less than 15 minutes.

Based on the above analysis, it was concluded that the year 1996 would generate representative CSO activation frequency and volumes for evaluation of CSO control alternatives.

4.8 Design Storm Analysis

Synthetic design storms will be used as input to simulate the existing system's reaction to various larger intensity and duration storms and also to evaluate CSO control alternatives for events that exceed a one-year frequency. The synthetic design storms that were selected are from the *TP 40: Rainfall Frequency Atlas of the US* - Weather Bureau Technical Paper No. 40 (1961) available from the National Climatic Data Center, part of the National Oceanic and Atmospheric Administration. Table 4.19 summarizes the storms to be used in evaluation of CSO control alternatives.

TABLE 4.19
SUMMARY OF SYNTHETIC STORM DATA
(DURATION OF EACH STORM = 24 HOURS)

Frequency of Storm (Year)	Peak Intensity (in/hour)	Total Rainfall (in)
1	1.1	2.7
2	1.3	3.4
5	1.7	4.4
10	2.1	5.0
20*	2.23	5.5
25	2.3	5.8
50	2.6	6.4
100	3.0	7.2

* Interpolated from 10- and 25-year storm data

The calibrated model will be utilized for the following:

- Evaluation of existing system flows and CSOs for the typical year 1996;
- Evaluation of CSO control alternatives for the Wellington Avenue CSO Facility and any impacts to the Washington Street CSO Facility for both the typical year 1996 and the synthetic design storms;
- Evaluation of system optimization such as modifications to existing weir plates; and
- Evaluation of impacts to the Newport system from flows from Middletown's Wave Avenue Pump Station.

These evaluations are presented in detail in Section 7.