Module 10a: Storm Sewer Design

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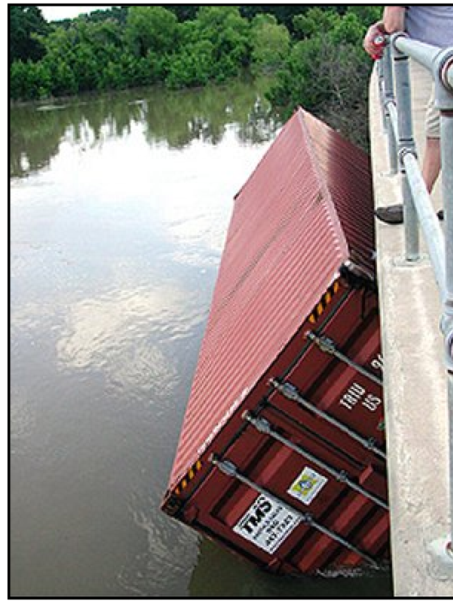

Major floods are dramatic and water flow routes must be recognized when minor drainage systems fail. These types of events are not directly addressed by typical storm drainage systems (the minor systems).

A trailer is trapped under a bridge by floodwaters, Houston, TX.

Photo by Mary Grove.

A sheriff's car is not able to escape rising floodwaters.

Photo by Cindy Cruz.

Siren lights on this submerged firetruck are still flashing on the East Loop at I-10.

Photo by Paul Carrizales.

An unidentified man on a jet ski passes submerged trucks on Interstate 10.

Photo from *Houston Chronicle*.

Ky Calder takes advantage of a break in the rain Saturday morning to take his kayak for a glide down U.S. 59 near the Hazard street overpass. Dave Rossman special to the *Houston Chronicle*.

Storm Drainage System Design

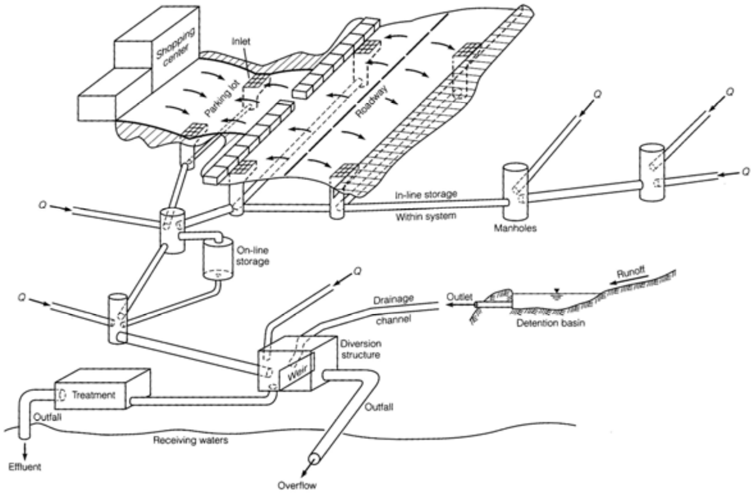

Figure 6.33 ■ Principal Hydraulic Elements in (Minor) Stormwater Management System
Source: ASCE, 1992. Design and Construction of Urban Stormwater Management Systems. p. 114. Reprinted by permission of ASCE.

Chin 2006

CITY OF BIRMINGHAM DEPARTMENT OF PLANNING, ENGINEERING AND PERMITS

ENGINEERING DESIGN GUIDELINES

3. STORM SEWERS

3.01 Method of Determining Runoff

(a) Storm water runoff may be estimated using any accepted method. The two predominant methods presently in use are the Rational Method and the SCS TR55 method, however, runoff estimation is not limited to either method. The owner's engineer shall be responsible for selecting an appropriate method.

(b) Storm sewer systems shall be designed based on future land use.

3.02 Return Periods

(a) The minimum return period used in the design of storm sewer collection systems shall be the 10 year return period.

(b) Box culverts and pipes larger than 60" shall be design using the 25 year return period.

(c) Other return periods may be used as determined by the owner's engineer.

3.03 Storm Sewer Design

(a) In general, the Manning equation shall be used to size drainage structures.

(b) A Manning's "n" of 0.013 shall be used to size concrete pipe and box culverts.

(c) In developed areas, the proposed drainage system shall be compatible with the existing drainage system. The developer may be required to upgrade the existing system to accommodate the proposed development.

(d) The minimum pipe diameter allowed is 18 inches.

(e) Storm sewer pipe shall be a minimum of Class 3 reinforced concrete if the storm sewer is to be maintained by the City of Birmingham, is in an easement, and/or passes runoff through a site. Other pipe materials may be used on systems that will not be turned over to the City of Birmingham for maintenance.

(f) The crowns (inside tops) of pipes shall match wherever practical when changing pipe sizes.

(g) In general, all drainage structures shall be extended to the limits of the development.

(h) Box culverts shall be designed for AASHTO HS-20 loading in vehicular traffic areas (existing or potential) and HS-15 loading in all other areas.

(i) Manholes or inlets shall be placed at changes of direction, changes in grade, junctions with other pipe, where needed to drain an area, or every 400 feet, whichever of these distances is less.

(j) Manholes, inlets, headwalls, etc., shall comply with the details on file in the Department of Planning, Engineering and Permits.

Pipe Size	Maximum Spacing
38 cm or less	122 m
46 cm to 91 cm	152 m
107 cm or greater	183 m

Source: Boulder County (1984).

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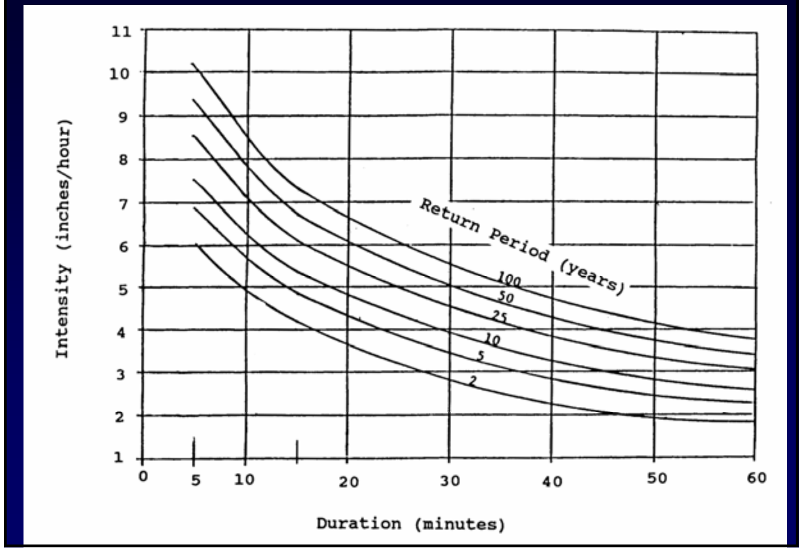
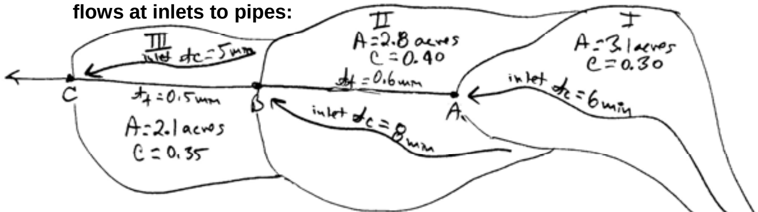
Birmingham Intensity - Duration - Frequency (IDF) Curve



Basic Application of Rational Formula:

Determine "10-yr" (10% probability of being exceeded in any one year) flows at inlets to pipes:

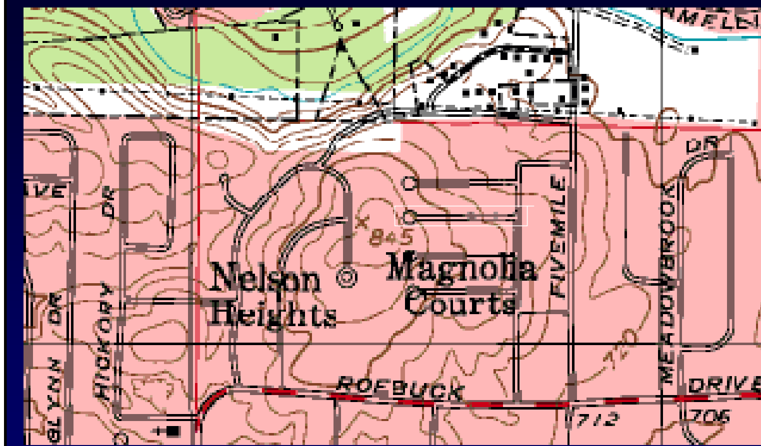


Pipe AB: inlet tc = 6 min; i = 7.4 in/hr; Q = CiA = (0.3)(7.4 in/hr)(3.1 acres) = 6.9 cfs

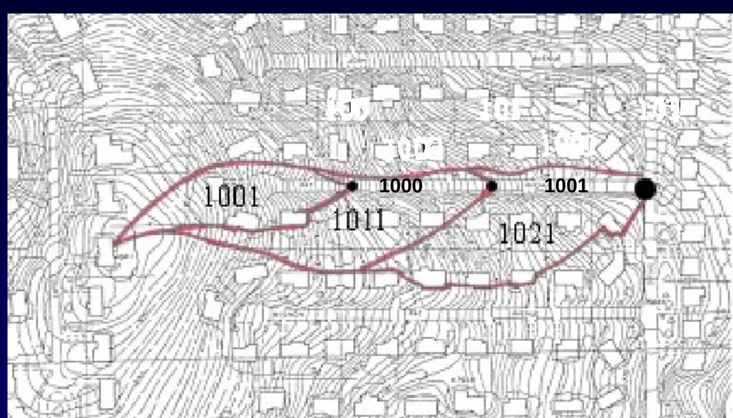
Pipe BC: inlet tc = 8 min vs. 6 +0.6 min, use 8 min; i = 6.6 in/hr;
Q= [(3.1ac)(0.3)+(2.8ac)(0.4)] 6.6 in/hr = 13.5 cfs

Pipe CD: inlet tc = 5 min vs. 6 + 0.6 + 0.5 vs. 8 + 0.5, use 8.5 min; i = 6.3 in/hr;
Q = [(3.1ac)(0.3)+(2.8ac)(0.4)+(2.1ac)(0.35)] 6.3 in/hr = 17.5 cfs

The travel times in the pipes can only be calculated after the pipe sizes are selected and the velocities at the design flows are determined.



Pat Avenue storm sewer example.



Pat Avenue is located in Birmingham, AL. It consists of three subcatchments, three junctions (nodes) and two conduits (pipes) in a residential area. The water collected during a rainstorm is discharged to a main sewer trunk.

Site Information

Sub-catchment	Area (Acres)	Pipe length (ft)	Slope (ft/ft)	Imperv. (%)
1001	1.07	-	0.084	54
1011	1.09	300	0.093	54
1021	1.43	300	0.072	54

Runoff Coefficients for the Rational Formula for Different Hydrologic Soil Groups (A, B, C, D) and Slope Ranges

(from McCuen, Hydrologic Analysis and Design. Prentice-Hall, Inc. 1998)

Land Use	A			B			C			D		
	0–2%	2–6%	6%+	0–2%	2–6%	6%+	0–2%	2–6%	6%+	0–2%	2–6%	6%+
Residential Lot, ⅛ acre	0.25[a]	0.28	0.31	0.27	0.30	0.35	0.30	0.33	0.38	0.33	0.36	0.42
	0.33[b]	0.37	0.40	0.35	0.39	0.44	0.38	0.42	0.49	0.41	0.45	0.54
Residential Lot, ¼ acre	0.22	0.26	0.29	0.24	0.29	0.33	0.27	0.31	0.36	0.30	0.34	0.40
	0.30	0.34	0.37	0.33	0.37	0.42	0.36	0.40	0.47	0.38	0.42	0.52
Residential Lot, ⅓ acre	0.19	0.23	0.26	0.22	0.26	0.30	0.25	0.29	0.34	0.28	0.32	0.39
	0.28	0.32	0.35	0.30	0.35	0.39	0.33	0.38	0.45	0.36	0.40	0.50
Residential Lot, ½ acre	0.16	0.20	0.24	0.19	0.23	0.28	0.22	0.27	0.32	0.26	0.30	0.37
	0.25	0.29	0.32	0.28	0.32	0.36	0.31	0.35	0.42	0.34	0.38	0.48
Residential Lot, 1 acre	0.14	0.19	0.22	0.17	0.21	0.26	0.20	0.25	0.31	0.24	0.29	0.35
	0.22	0.26	0.29	0.2	0.28	0.34	0.28	0.32	0.40	0.31	0.35	0.46
Commercial	0.71	0.71	0.72	0.71	0.72	0.72	0.72	0.72	0.72	0.72	0.72	0.72
	0.88	0.88	0.89	0.89	0.89	0.89	0.89	0.89	0.90	0.89	0.89	0.90

[a] Runoff coefficients for storm recurrence intervals less than 25 years.
[b] Runoff coefficients for storm recurrence intervals of 25 years or longer.

Example of Rational Method Calculation for Area 1001

- Drainage Area (assume: 10-year storm because street is a minor urban street and not a collector street)
  - Drainage Area: 1.07 acres
  - Watershed Slope: 0.084
  - Hydrologic Soil Group C (assume/look up)
  - Land Use Description: ½ acre lots
  - Time of Concentration: 10 minutes

- Using T_c = 10 minutes, i = 6.4 in/hr for 10-year storm
- Using ½-acre lot size, 6+% slope, C soil, C = 0.32

- Peak Discharge = Q_p = CiA
  Q_p = (0.32)(6.4 in/hr)(1.07 acres) = 2.19 cfs

Detailed Site Information

Sub-catchment	Area (Acres)	Slope (ft/ft)	Rational C	Inlet Tc (min)	Travel time in pipe (min)	Total Tc (min)	Intensity (in/hr)	Total Q at bottom of area (cfs)
1001	1.07	0.084	0.32	10.0	-	10.0	6.4	2.19
1011	1.09	0.093	0.32	10.0	0.5	10.5	6.2	4.29
1021	1.43	0.072	0.32	10.0	0.5	11.0	6.1	7.25

Tc gets larger and intensity gets smaller as the total drainage area increases

Conduit Information

Conduit	Shape	Slope	Length (ft)	Manning's n
1000	Circular	0.073	300	0.013
1001	Circular	0.053	300	0.013

Manning's Equation

Diameter of a Pipe Flowing Full Using Manning's Equation for Flow

$$Q = \frac{1.49}{n}\left(\frac{\pi}{4}D^2\right)\left(\frac{D}{4}\right)^{2/3} S^{1/2}$$

$$\frac{nQ}{1.49S^{1/2}} = \left(\frac{\pi}{4}D^2\right)\left(\frac{D}{4}\right)^{2/3}$$

$$\frac{4nQ}{1.49\pi S^{1/2}} = D^2\left(\frac{D}{4}\right)^{2/3}$$

$$\frac{4nQ}{1.49\pi S^{1/2}} = \frac{D^{8/3}}{4^{2/3}}$$

$$\frac{4^{5/3}nQ}{1.49\pi S^{1/2}} = D^{8/3}$$

$$\left(\frac{4^{5/3}nQ}{1.49\pi S^{1/2}}\right)^{3/8} = D$$

These equations are for US Customary units! Use cfs for Q, and ft for D.

Without the 1.49 in the denominator of the last expression, SI units can be used: m^3/sec for Q and m for D.

Storm Sewer Calculations

Conduit	Shape	Slope	Length (ft)	Manning's n	Total Q at end of pipe (cfs)
1000	Circular	0.073	300	0.013	2.19
1001	Circular	0.053	300	0.013	4.29

Conduit	Q (cfs)	Calculated D (ft)	Actual D (ft)	Regulated D (ft)	Q_{full} (cfs)
1000	2.19	0.573	0.667	1.5	28.4
1001	4.29	0.792	0.833	1.5	24.2
At outlet	7.25	0.958	1	1.5	24.2

Sewers Flowing Partly Full

From: Metcalf and Eddy, Inc. and George Tchobanoglous. *Wastewater Engineering: Collection and Pumping of Wastewater*. McGraw-Hill, Inc. 1981.

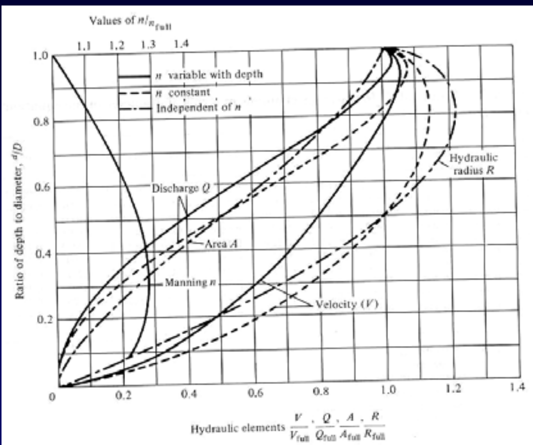


Figure 2-16 Hydraulic elements for circular sewers [10].

Storm Sewer Calculations

Conduit	Q (cfs)	Min. required pipe size (ft)	Q_{full} (cfs)	Q/Q_{full}	d/D
1000	2.19	1.5	28.4	0.077	0.19
1001	4.29	1.5	24.2	0.18	0.29
At outlet	7.25	1.5	24.2	0.30	0.38

Conduit	V/V_{full}	V_{full} (ft/sec)	V at peak flow (ft/sec)	Travel time in pipe (min)
1000	0.59	16.1	9.5	0.5
1001	0.76	13.7	10.4	0.5
At outlet	0.90	13.7	12.3	-

Pipe Sizes

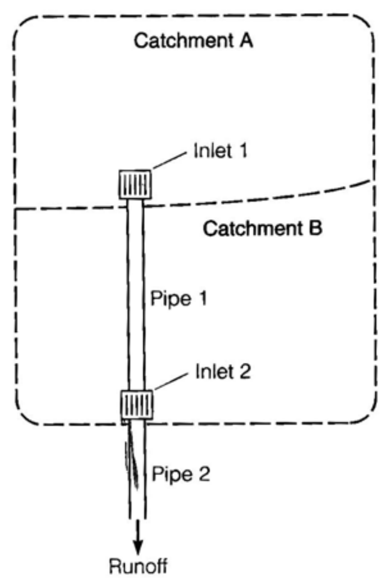
- Minimum size 12 - 18 inches
- In many locations, the minimum size of a storm sewer pipe is regulated

Velocities

- Minimum velocity of 2.0 ft/sec (0.6 m/sec) with flow at ½ full or full depth
- Maximum average velocities of 10-12 ft/sec (2.5-3.0 m/sec) at design depth of flow
- Minimum and maximum velocities may be specified in state and local standards

Slopes

- Sewers with flat slopes may be required to avoid excessive excavation where surface slopes are flat or the changes in elevation are small.
- In such cases, the sewer sizes and slopes should be designed so that the velocity of flow will increase progressively, or at least will be steady throughout the length of the sewer.



Example 5.44 (Chin 2006)

This is another example using the rational formula, but with a further examination of source area flows (paved vs. unpaved area contributions) in an attempt to more accurately consider the independent routing of these flows.

Two pipes and two inlets are shown in the adjoining drawing. Catchment A is 1 ha and is 50% impervious, while catchment B is 2 ha and is 10% impervious. The impervious areas are directly connected to the storm drainage system. The design storm (level of service) has a return frequency of 10 years and the 10-yr IDF curve can be approximated by:

$$i = \frac{7620}{t_c + 36}$$

i, mm/hr
t_c, minutes

Basic watershed data:

Catchment	Surface	C	L (m)	n	S_0	(min) t_c
A 1 ha	pervious 50%	0.2	80	0.2	0.01	45
	impervious 50%	0.9	60	0.1	0.01	11
B 2 ha	pervious 90%	0.2	140	0.2	0.01	71
	impervious 10%	0.9	65	0.1	0.01	12

The effective rainfall rate (i_e) is as follows, using the IDF curve equation and the rational formula:

$$i_e = Ci = C\frac{7620}{t_c + 36}$$

where C is the runoff coefficient. The time of concentration can be estimated using the following equation:

$$t_c = 6.99\frac{(nL)^{0.6}}{i_e^{0.4} S_0^{0.3}}$$

Where n is the Manning's roughness factor for sheetflow conditions, L is the flow length (m) and S_o is the slope of the watershed, as presented in the above data table.

These equations are solved simultaneously to obtain the following time of concentration values for each watershed subarea:

Catchment	Surface	t_c (min)
A	pervious area	46
	impervious area	11
B	pervious	71
	impervious	12

Flows at Inlet 1 and Pipe 1:

Pipe 1 only receives runoff from inlet 1, contributed by catchment A. When the entire catchment A is contributing flow, the time of concentration is 46 minutes (the time needed for both the pervious and impervious areas to be fully contributing). The average rainfall rate corresponding to this time of concentration is therefore 92.9 mm/hr (or 2.58 x 10^{-5} m/sec). The area-weighted runoff coefficient is:

$$\overline{C} = 0.5(0.9) + 0.5(0.2) = 0.55$$

Since the area of the catchment is 1 ha (10,000 m²), the peak runoff rate, Q_p, can be calculated using the rational formula as:

$$Q_p = \overline{C}iA = (0.55)(2.58x10^{-5}\, m/s)(10{,}000m^2) = 0.142m^3/s$$

However, the impervious area should be examined alone, as it may produce a greater peak flow rate than the whole averaged area. This recognizes the separate routing of flows from these greatly different subareas. The time of concentration of the impervious area in catchment A is 11 minutes, and the corresponding rainfall rate averaged for that duration is 162 mm/hr (4.5 x 10^{-5} m/sec). The impervious area runoff coefficient is 0.9 and the area is 0.5 ha (5,000 m²). Therefore, the peak runoff rate, Q_p, can be calculated as:

$$Q_p = \overline{C}iA = (0.9)(4.50x10^{-5}\, m/s)(5{,}000m^2) = 0.203m^3/s$$

This calculated peak runoff rate for the impervious areas alone is therefore greater than the peak runoff rate calculated for the whole catchment averaged conditions, and is therefore controlling. The flow to be handled in Pipe 1 is therefore 0.203 m³/sec.

Flows at Inlet 2:

When the entire catchment B is contributing flow, the inlet time of concentration is 71 minutes. The corresponding averaged rainfall rate for this duration is 71.2 mm/hr (1.98 x 10^{-5} m/sec) and the area-weighted runoff coefficient is:

$$\overline{C} = 0.1(0.9) + 0.9(0.2) = 0.27$$

The catchment B area is 2 ha (20,000 m²) and the peak runoff rate is therefore:

$$Q_p = \overline{C}iA = (0.27)(1.98x10^{-5}\, m/s)(20{,}000m^2) = 0.107m^3/s$$

The impervious area alone has a time of concentration of 12 minutes, and the corresponding averaged rainfall rate for that period is 159 mm/hr (4.41 x 10^{-5} m/sec). The impervious area runoff coefficient is 0.9 and the area is 0.2 ha (2,000 m²). The peak runoff rate just from the impervious area component of catchment B is therefore:

$$Q_p = \overline{C}iA = (0.9)(4.41x10^{-5}\, m/s)(2{,}000m^2) = 0.079m^3/s$$

In this case, the peak flow is greater when the whole catchment conditions are averaged, and the peak flow at inlet 2 is therefore 0.107 m³/sec.

Flow in Pipe 2:

The peak flow for pipe 2 must consider several alternatives. The first case considers the entire 3 ha (30,000 m^2) area of catchments A plus B averaged together (a common way of applying the rational formula, as previously illustrated). The time of concentration for catchment A contributions is the inlet time of concentration of 46 min., plus the travel time of the flow in pipe 1, here assumed to be 2 min. This potential time of travel path therefore totals 48 minutes. This is compared to the inlet time of concentration of catchment B which is 71 min. The 71 min. pathway is therefore the longest and is the time of concentration. The corresponding rainfall rate averaged for this period is 71.2 mm/hr (1.98 x 10^{-5} m/sec). The area-weighted runoff coefficient is therefore:

$$\overline{C} = \frac{1}{3}\left[(0.5+0.2)(0.9)+(0.5+1.8)(0.2)\right] = 0.36$$

and the peak runoff rate is calculated as:

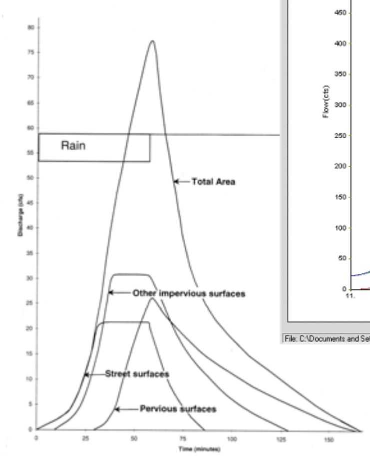
$$Q_p = \overline{C}iA = (0.36)(1.98x10^{-5}\,m/s)(30{,}000m^2) = 0.214m^3/s$$

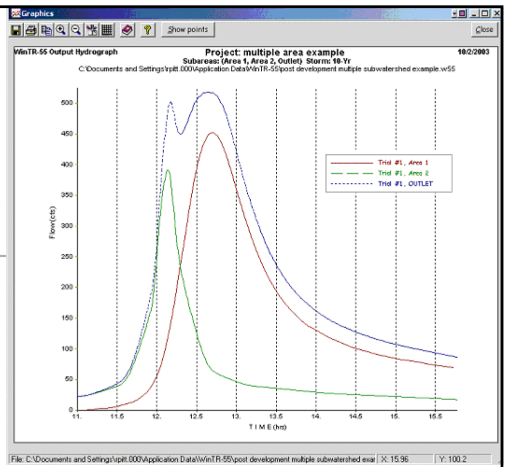
Considering the impervious areas of catchments A and B alone, the area is 0.7 ha (7,000 m^2) and the time of concentration is 13 min. (the 11 min. time of conc. for the impervious areas in catchment A plus the 2 min. travel time in Pipe 1 vs. the 12 min. time of concentration for the impervious areas in catchment B). The corresponding rainfall rate averaged for this time is 156 mm/hr (4.32 x 10^{-5} m/sec), the runoff coefficient is 0.9, and the rational formula provides the peak runoff rate:

$$Q_p = \overline{C}iA = (0.9)(4.32x10^{-5}\,m/s)(7{,}000m^2) = 0.272m^3/s$$

Therefore, the peak flows using the impervious areas alone are controlling for Pipe 2.

In reality, it is likely that the most critical condition would be associated with a combination of conditions, possibly using the impervious area data from catchment A and the entire area from catchment B. It is not easy to tell unless a complete hydrograph routing method that examines the separate subareas is used, such as WinTR-55 for the major drainage areas (or surface drainage), or SWMM5 for any condition. Recall that with WinTR-55, it is necessary to separate subcatchments that differ by a CN of 5, or greater, in each subwatershed.





Routing the separate source area hydrographs results in accurate peak flow predictions.

Pipe Selection (Example 5.45; Chin 2006)

A concrete pipe is to be laid parallel to the ground surface having a slope of 0.5%. The stormwater design peak flow rate is 0.43 m^3/sec.

Using the Manning's Equation (and SI units):

$$D = \left(\frac{3.21Qn}{\sqrt{S_o}}\right)^{3/8} = \left[\frac{3.21(0.43m^3/\sec)(0.013)}{\sqrt{0.005}}\right] = 0.6m$$

However, the Manning's equation is only valid for fully turbulent flow and is only appropriate when the following condition is satisfied: $n^6\sqrt{RS_o} \geq 10^{-13}$ checking:

$$(0.013)^6\sqrt{(0.6m/4)0.005} = 1.3x10^{-13} \geq 10^{-13}$$

Therefore the Manning's equation is valid for this condition.

The velocity in the pipe is:

$$V = \frac{Q}{A} = \frac{0.43m^3/\sec}{\frac{\pi}{4}(0.6m)^2} = 1.52m/\sec$$

This velocity exceeds the minimum velocity necessary to prevent deposition (the minimum is usually considered to be 0.6 to 0.9 m/sec) and is less than the maximum velocity to prevent excess scour (the maximum is usually considered to be 3 to 4.5 m/sec).

Therefore, the selected pipe should be the next commercially available pipe size larger than 60 cm.

Darcy-Weisbach Equation (used if fully turbulent flow conditions are not satisfied):

$$D = \left(\frac{0.811 fQ^2}{gS_o}\right)^{1/5} = \left[\frac{0.811(0.020)(0.43m^3/\sec)^2}{(9.81m/\sec^2)(0.005)}\right] = 0.57m$$

The friction factor, *f*, is assumed to be 0.020, a typical value, for this first trial. The 0.57 m pipe with this discharge has the following velocity:

$$V = \frac{Q}{A} = \frac{0.43m^3/\sec}{\frac{\pi}{4}(0.57m)^2} = 1.69m/\sec$$

The concrete equivalent sand roughness factor, k_s, is in the range of 0.3 to 3.0 mm, and is assumed to be 1.7 mm for this example. With a water temperature of 20ºC, the kinematic viscosity is 1.00 x 10^{-6} m/$\sec^2$. The Reynolds number is therefore:

$$\mathrm{Re} = \frac{VD}{\nu} = \frac{(1.69m/\sec)(0.57m)}{1.00x10^{-6}\,m/s^2} = 9.63x10^5$$

The Jain approximation of the Colebrook equation can be used to estimate *f:*

$$\frac{1}{\sqrt{f}} = -2\log\left[\frac{k_s/D}{3.7} + \frac{5.74}{\mathrm{Re}^{0.9}}\right] = -2\log\left[\frac{1.7mm/57mm}{3.7} + \frac{5.74}{(9.63x10^5)^{0.9}}\right] = 6.16$$

which leads to: *f* = 0.0263. Since this differs from the initial estimated *f* of 0.020, the above computations need to be repeated. The following table summarizes the results from the initial calculations and the next (and final) calculations:

Assumed f	D (m)	V (m/s)	Re	Computed f
0.020	0.57	1.69	9.63×10^5	0.0263
0.0263	0.60	1.52	9.12×10^5	0.0263

Therefore, the Darcy-Weisbach equation also requires that the pipe be at least 60 cm in diameter.

Manhole Head Losses:

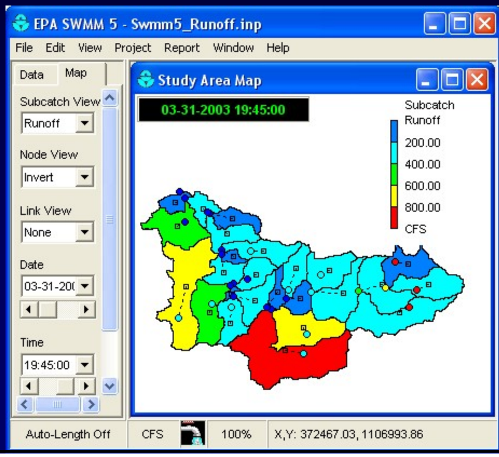
The manholes placed along the pipe will each cause a head loss, h_m:

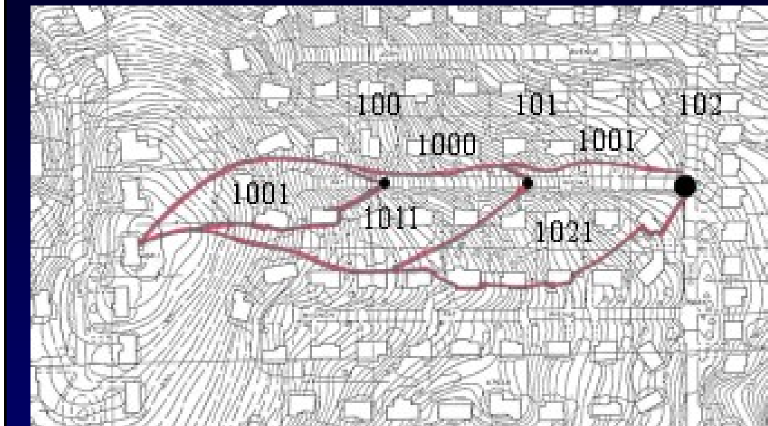
$$h_m = K_c \frac{V^2}{2g} = 0.22\frac{(1.52m/s)^2}{2(9.81m/\sec^2)} = 0.026m$$

K_c is between 0.12 and 0.32 for pipes opposite each other in manholes, and the average value of 0.22 is used in the above example, along with the velocity value calculated with the Darcy-Weisbach equation. This head loss can be reduced with careful grouting of the bottom of the manholes making smooth transitions between the pipe segments. Otherwise, the down-gradient pipe must be lowered about 1 inch to account for this headloss.

Getting Started with Storm and Sanitary Drainage Analysis using SWMM5 (Beta-E 01/23/04)

The model can be downloaded by going to the EPA web site:
http://www.epa.gov/ednnrmrl/swmm/





PAT Avenue subcatchments, joints and conduits (in this example, another link, 1003, was created to allow all subwatershed flows to be combined before the outfall junction, now 103).

Pat Avenue Subcatchment information:

Subcatchment	Area (Acres)	Width (ft)	Slope (ft/ft)	Percentage imperviousness	n Manning impervious	n Manning pervious
1001	1.067	98.3	0.084	54	0.040	0.410
1011	1.087	74.5	0.093	54	0.040	0.410
1021	1.431	109.0	0.072	54	0.040	0.410

Sub-catchment	Horton maximum infiltration rate (in/hr)	Horton minimum infiltration rate (in/hr)	Horton decay coefficient (1/sec)	Horton recovery coefficient (fraction)	Max. volume (inches)
1001	1	0.1	0.002	0.001	0
1011	1	0.1	0.002	0.001	0
1021	1	0.1	0.002	0.001	0

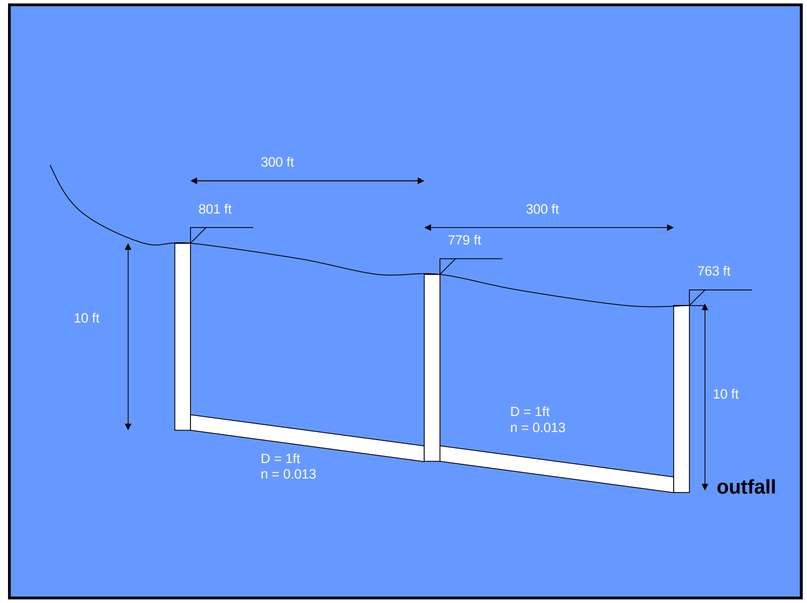
Pat Avenue Junction Information:

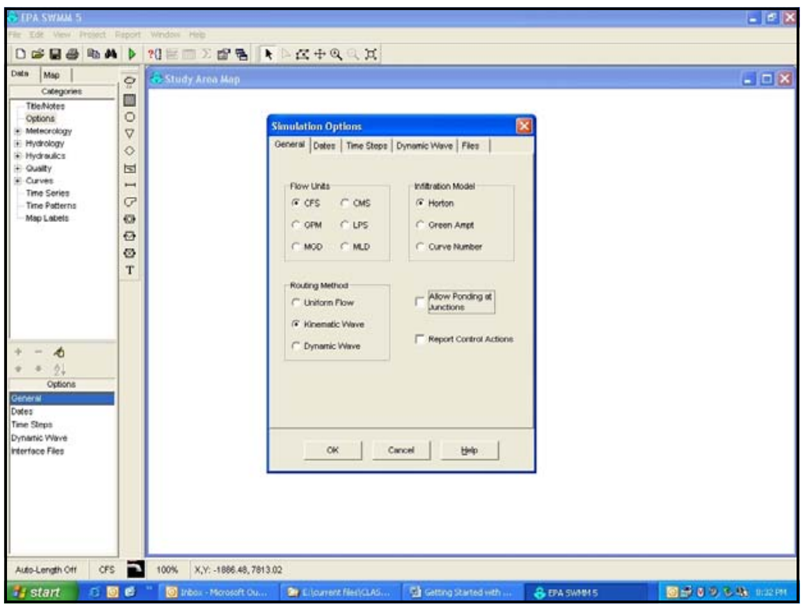
Junction	Invert Elevation (ft)	Maximum Depth (ft)	Initial Depth (ft)	Surcharge Depth (ft)	Ponded Area (ft²)
100	791	10	0	0	0
101	769	10	0	0	0
102	753	10	0	0	0
103 (Outfall)	745	n/a	0	0	0

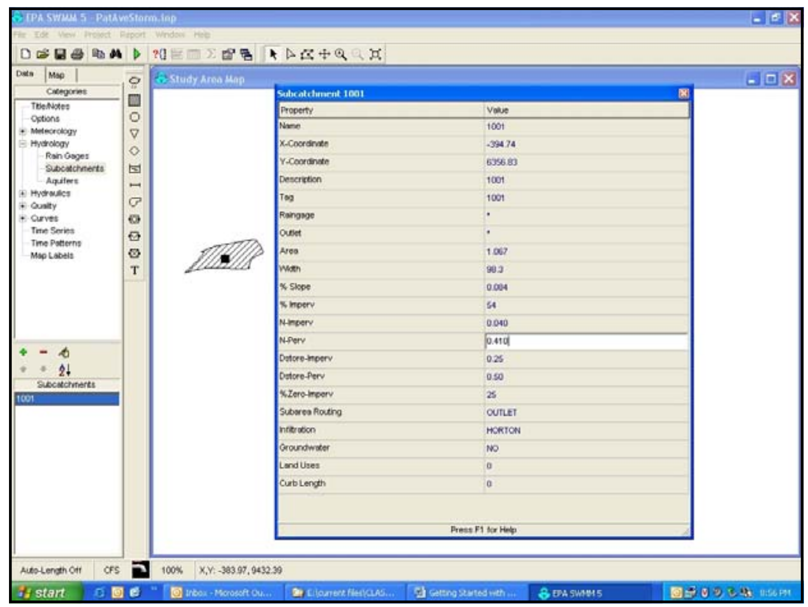
Pat Avenue Conduit Information:

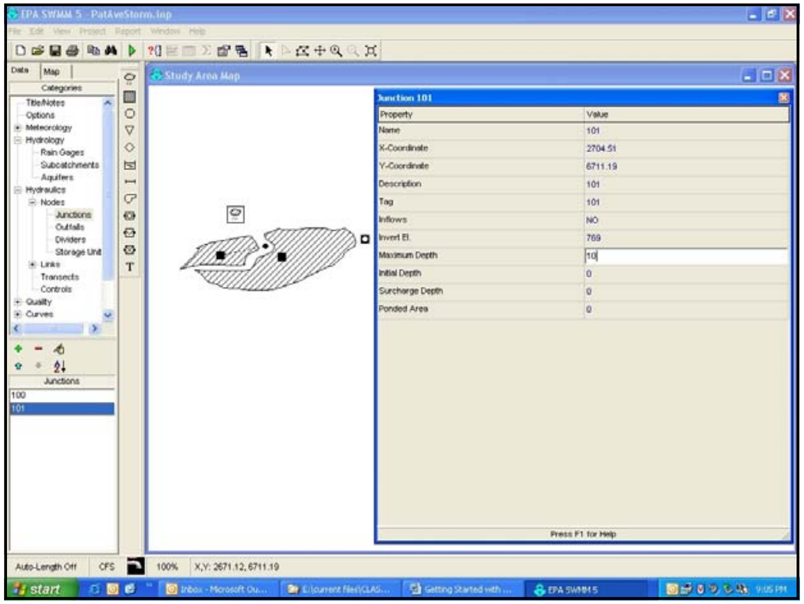
Conduit	Shape	Diameter (ft)	Length (ft)	n Manning	Inlet invert height offset (ft)
1000	Circular	1	300	0.013	0.5
1001	Circular	1	300	0.013	0.5
1003	Circular	1	100	0.013	0.5

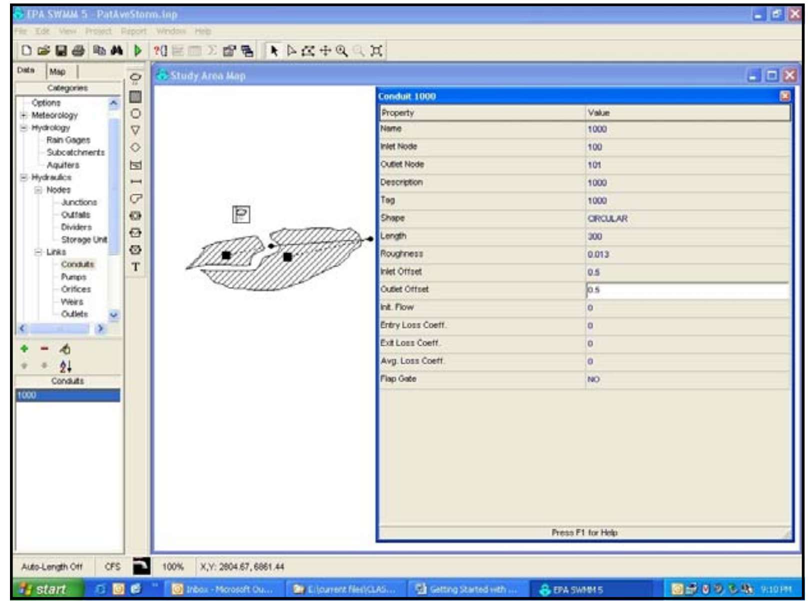
Conduit	Outlet invert height offset (ft)	Initial flow (cfs)	Entry loss coefficient	Exit loss coefficient	Average loss coefficient
1000	0.5	0	0	0	0
1001	0.5	0	0	0	0
1003	0.5	0	0	0	0

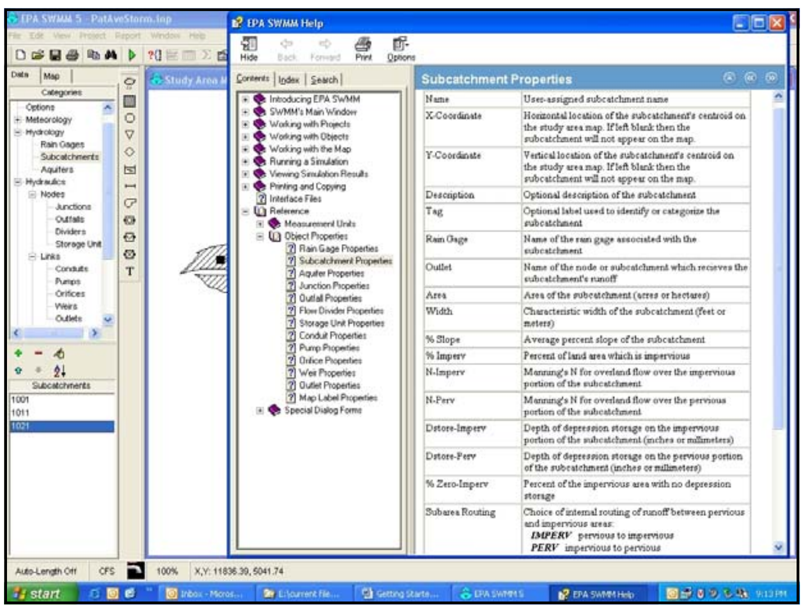


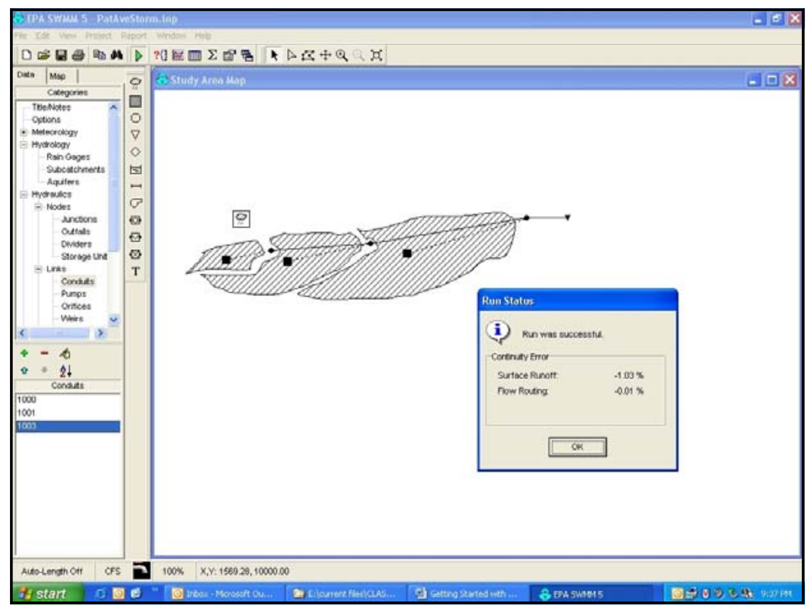


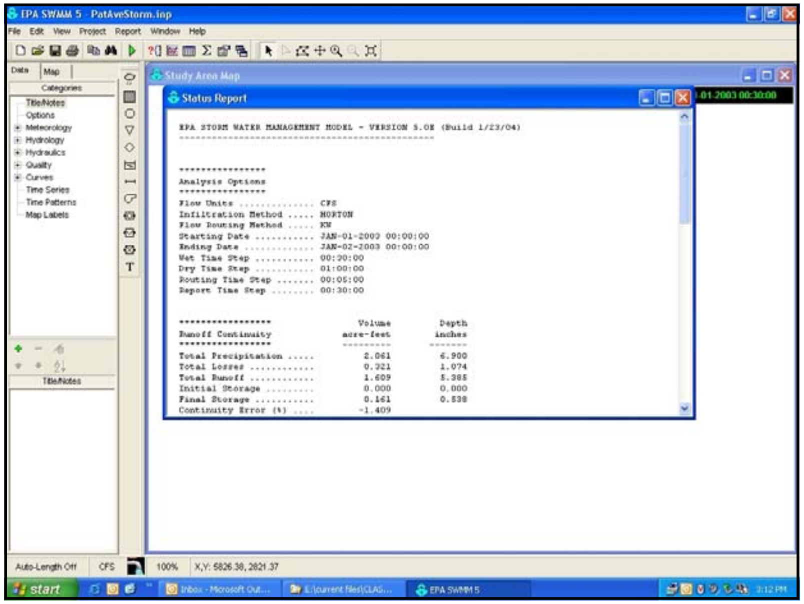


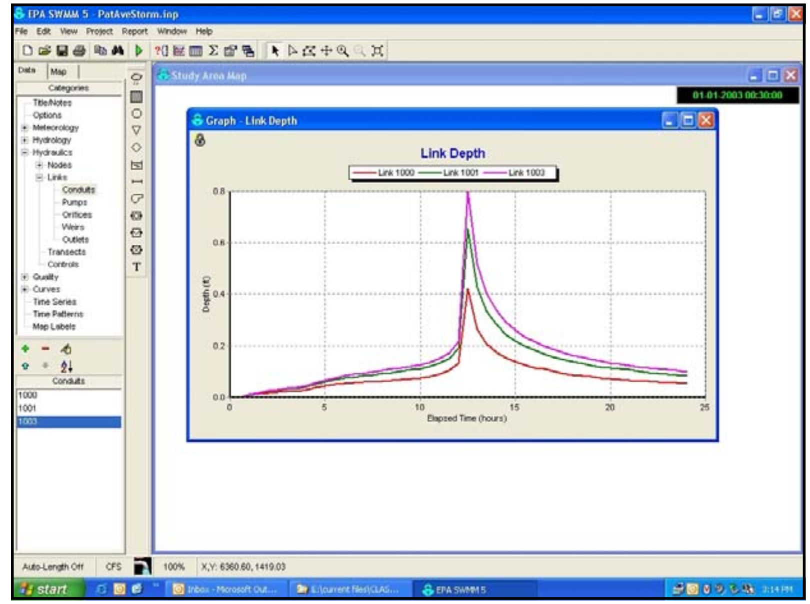

EPA SWMM 5 - PatAveStorm.inp
Study Area Map
Junction 101
Press F1 for Help

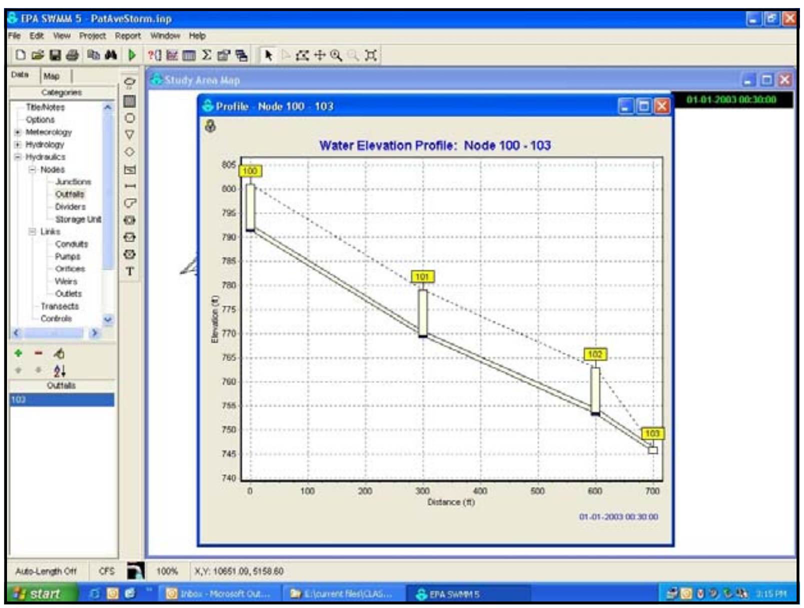

EPA SWMM 5 - PatAveStorm.inp
Study Area Map
Conduit 1000
Press F1 for Help

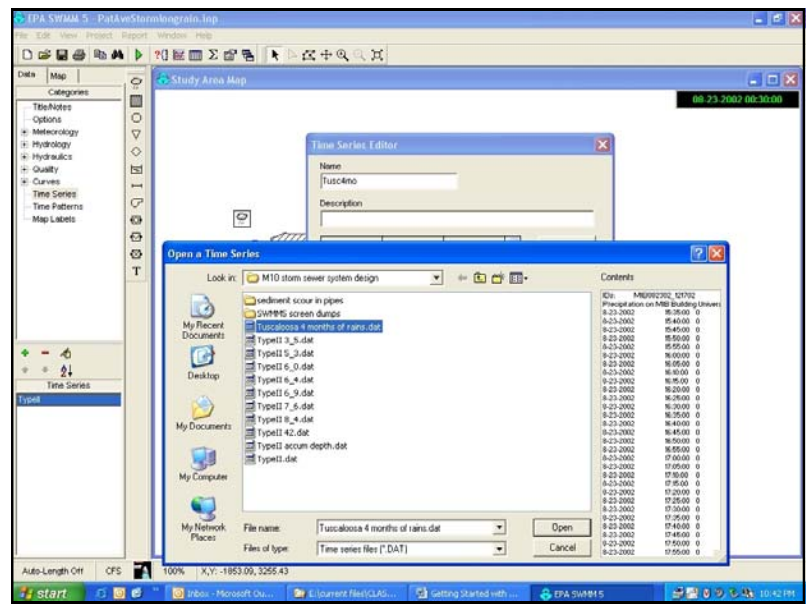

EPA SWMM Help
Subcatchment Properties

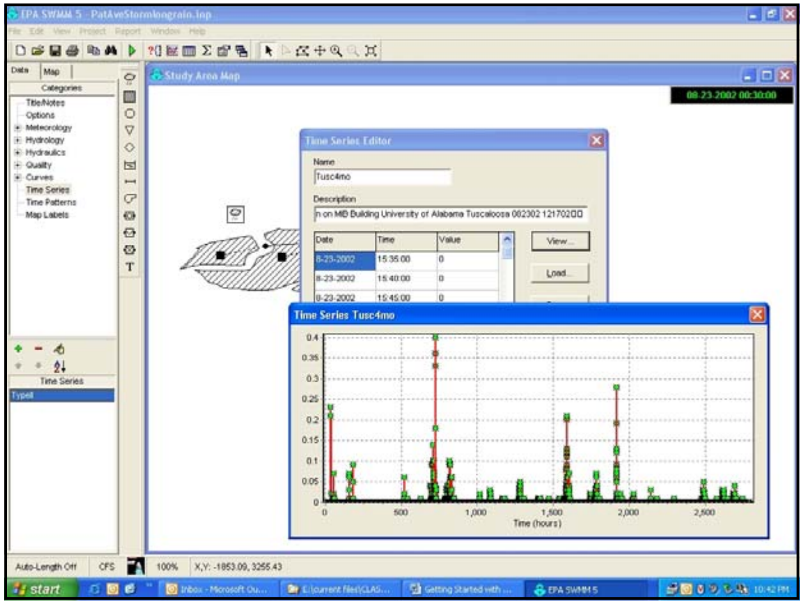

EPA SWMM 5 - PatAveStorm.inp
Study Area Map
Run Status
Run was successful.

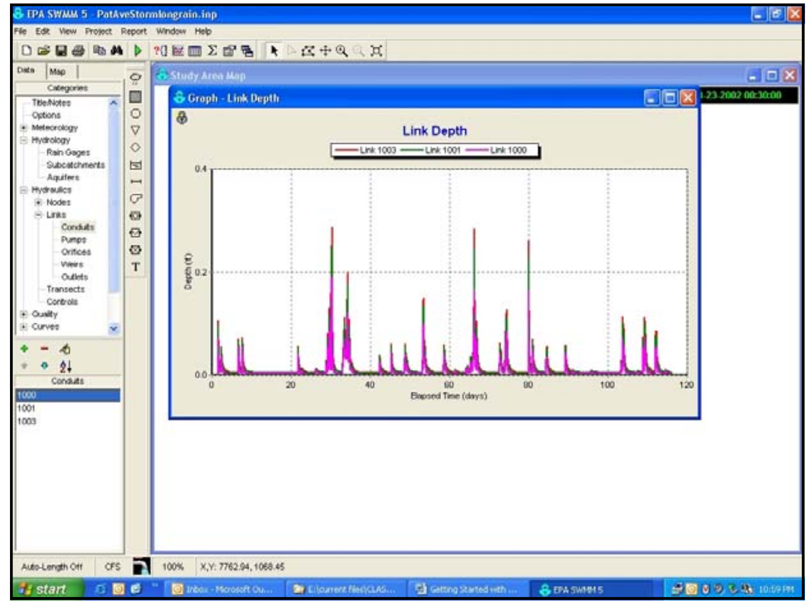

EPA SWMM 5 - PatAveStorm.inp
Study Area Map
Status Report
EPA STORM WATER MANAGEMENT MODEL - VERSION 5.0
Analysis Options
Flow Units .... CFS
Infiltration Method ..... HORTON
Flow Routing Method ..... KW
Starting Date JAN-01-2003 00:00:00
Ending Date JAN-02-2003 00:00:00
Wet Time Step 00:30:00
Dry Time Step 01:00:00
Routing Time Step 00:05:00
Report Time Step 00:30:00
Runoff Continuity
Total Precipitation
Total Losses
Total Runoff
Initial Storage
Final Storage
Continuity Error (%)


EPA SWMM 5 - PatAveStorm.inp
Study Area Map
Graph - Link Depth
Link Depth
Link 1000
Link 1001
Link 1003
Depth (ft)
Elapsed Time (hours)

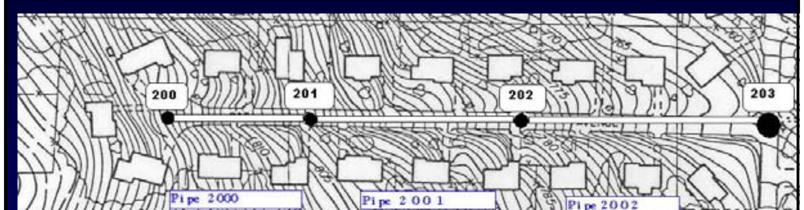

EPA SWMM 5 - PatAveStorm.inp
Study Area Map
Profile - Node 100 - 103
Water Elevation Profile: Node 100 - 103
Elevation (ft)
Distance (ft)
100
101
102
103


Study Area Map
Time Series Editor
Name
Tusc4mo
Description
Open a Time Series
Look in: M10 storm sewer system design
Contents
File name: Tuscaloosa 4 months of rains.dat
Files of type: Time series files (*.DAT)
Open
Cancel





"Hello World" Pat Avenue Sanitary Drainage Design Example

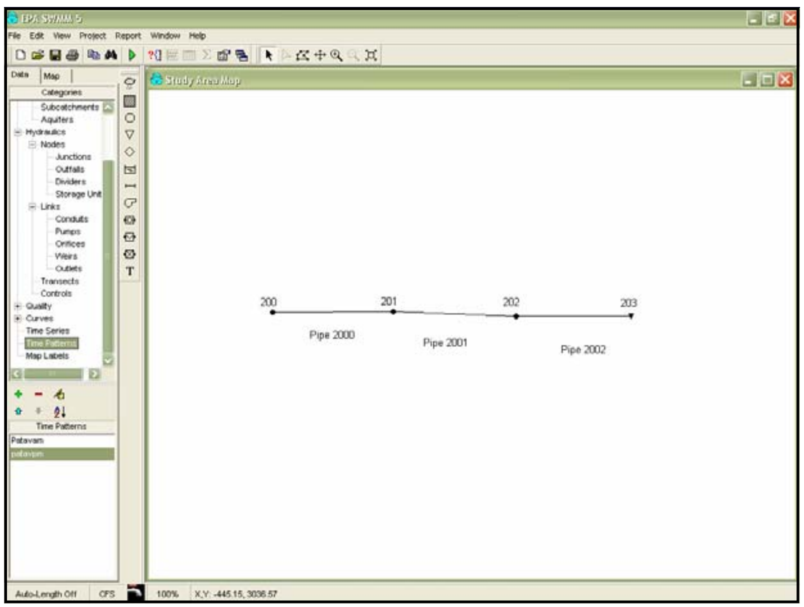


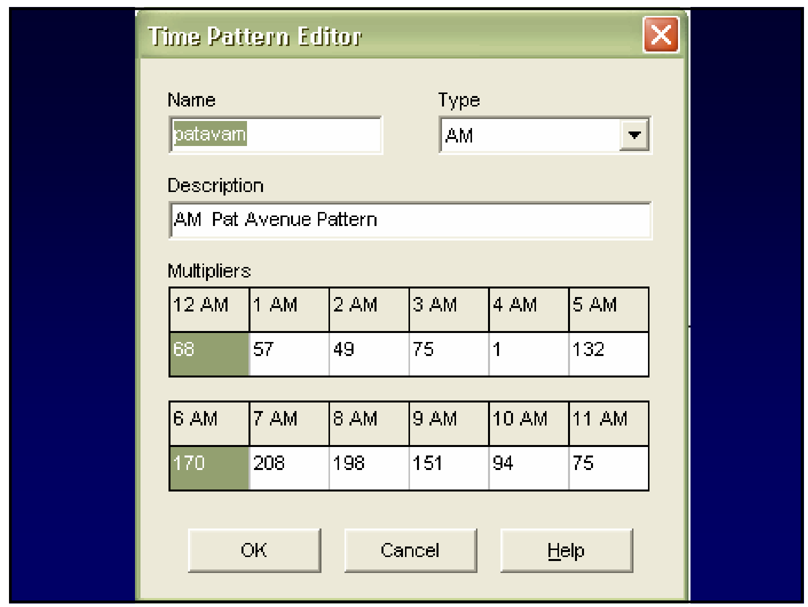
Junction (Node)	Area Served (ac)	# Apt. Build-ings	Population (32 people / building)	Water Use (150 gal / day)	Daily Wastewater Flow (90% of water used)	Sewage (cfs)
200	0.98	3	96	14400	12960	0.020
201	1.63	5	160	24000	21600	0.033
202	2.18	6	192	28800	25920	0.040
203	2.00	4	128	19200	17280	0.027

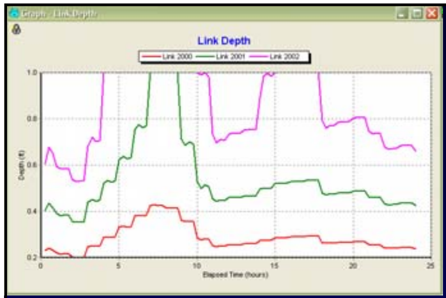
Junction	Invert Elevation (ft)	Maximum Depth (ft)	Initial Depth (ft)	Surcharge Depth (ft)	Ponded Area (ft²)
200	807	13	0	0	0
201	788	13	0	0	0
202	766	13	0	0	0
203 (Outfall)	750	n/a	0	0	0

Conduit	Shape	Diameter (ft)	Length (ft)	n Manning	Inlet invert height offset (ft)
2000	Circular	1	200	0.013	0.5
2001	Circular	1	300	0.013	0.5
2002	Circular	1	300	0.013	0.5

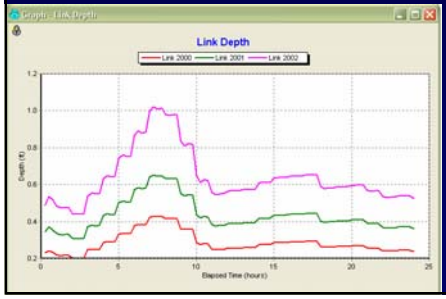
Conduit	Outlet invert height offset (ft)	Initial flow (cfs)	Entry loss coefficient	Exit loss coefficient	Average loss coefficient
2000	0.5	0	0	0	0
2001	0.5	0	0	0	0
2002	0.5	0	0	0	0







Surcharged 1 ft. pipes



Adequate capacity after enlarging pipes 2001 and 2002 to 1.5 ft in diameter