Day 2: Combined Sewer Design
Part two: wet weather flow contributions to combined sewers

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Stormwater Issues in Beijing

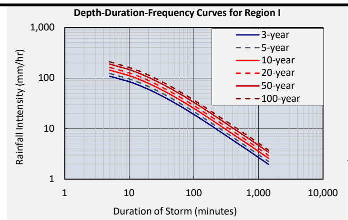
- High seasonal flows, long dry periods
- High nutrient discharges
- First flush investigations
- Distributed infiltration to:
  - Reduce flow discharges to drainage system to reduce overflows and other drainage issues
  - Decreased discharges of nutrients to surface waters
  - Enhance water supply

Example Downtown Commercial and High-Rise Residential Area in Beijing

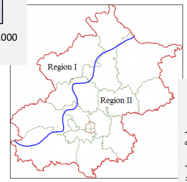
Aerial photo from Google Earth

Example High Density Residential Area in Beijing

Aerial photo from Google Earth

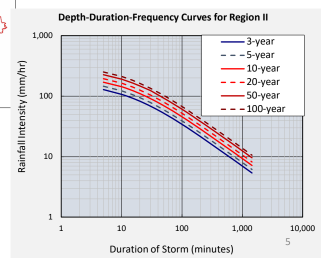


Traditional IDF design storms for design of stormwater collection systems, focusing on 3 to 100-yr reoccurring events (1 to 33% probability of occurring in any one year).



Flatter topography, lower elevations and closer to sea results in greater intensities in Region II compared to Region I, especially for longer duration events.

intensity (mm/hr) associated with:	5 yr rain frequency		20 yr rain frequency		100 yr frequency	
Beijing rain region:	1	2	1	2	1	2
short duration (5 minutes)	124	146	163	194	210	252
one hour (60 minutes)	32	54	43	72	55	92
one day (1440 minutes)	2.6	7.2	3	8.2	3.8	11



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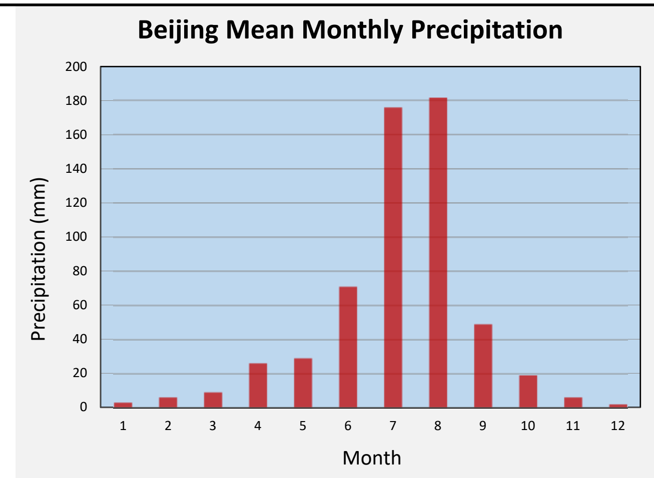

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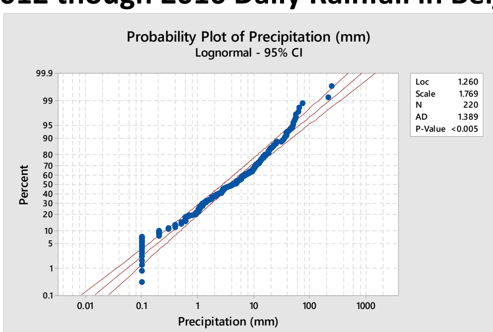
2012 though 2016 Daily Rainfall in Beijing



In this 5-year period, the median rain depth was about 4.4 mm, the 90th percentile rain depth was about 90 mm, and the largest rain depth was about 240 mm (second largest was 207 mm and 3rd largest dropped to 72 mm). Recall the 24-hr/5-yr event was 54-148 mm and the 100 yr/24 hr rain event was 91 to 254 mm).

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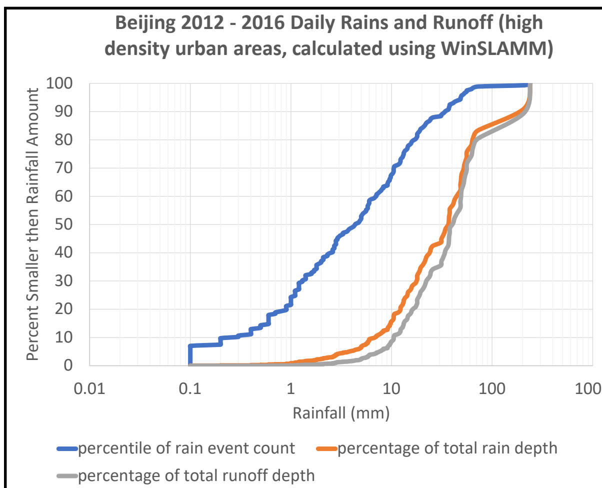

	Beijing 2012 through 2016 rains
% of rain events <10 mm depth:	70%
Rain depths associated with 10% of annual runoff:	<10 mm
Rain depths associated with 50% of annual runoff:	<40 mm
Rain depths associated with 80% of annual runoff:	<70 mm

Rains from 10 to 70 mm produced about 70% of the annual runoff and rains >70 mm produced about 20% of the annual runoff for this 5-year period.

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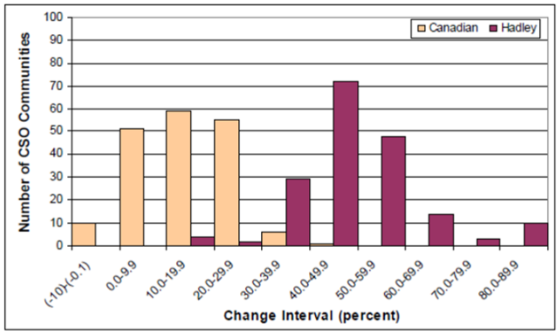
Example Climate Change Effects on Combined Sewers (Northern US Example)



Percent change in frequency of CSO events in the Great Lakes region relative to historical values based on Canadian (CCCM) and Hadley (HADCM2) Model climate projections for the future period from 2060–2099. A 50% change indicates an increase of 4 overflows per year to 6 overflows per year. (EPA 2008)

However, increases in regional populations and associated increased developed areas may cause increased stormwater flows much greater than those associated with climate change. Planning for future conditions must consider both of these potential causes of increased flows.

Summary for Beijing Rainfall and Stormwater Conditions

- Beijing has highly seasonal rains with long interevent periods. Literature suggests nutrient discharges are the greatest concern. First flush investigations of local stormwater, and sources of stormwater pollutants are also described in the literature. Desire to use distributed infiltration to reduce discharges.
- First flushes are not consistent for all land uses and pollutants. Most important for simple drainages that are mostly paved; less obvious for complex drainages with separate source areas.

Summary for Beijing Rainfall and Stormwater Conditions, cont.

- Beijing drainage design events indicate a significant trend across the city, with more severe conditions to the east.
- Conventional drainage design approaches do not work well for the smaller rains that are of most significance in annual pollutant and flow discharges.
- Most of the Beijing rains (by number) are less than 10 mm in depth, while those rains only result in about 10% of the annual runoff. About 80% of the runoff occurs for rains less than 70 mm in depth.

Wet Weather Flow Designs for Combined Sewers

- Basic design criteria for sanitary sewage dry weather flow must be met (minimum pipe diameters, minimum slopes, minimum velocities, etc.).
- Compound pipe shape may be needed to ensure sufficient dry weather sewage velocities, especially if long interevent periods with reduced flushing of settled solids during wet weather. That flushing leads to first flush elevated contaminant levels in discharges to treatment or overflows, desired to be minimized.
- Combined sewers must connect to each building and must be sized large enough to carry increased flows associated with design storms.

Typical Criteria for Wet Weather Flow Drainage Systems
(wet weather conditions)

Pipe Sizes

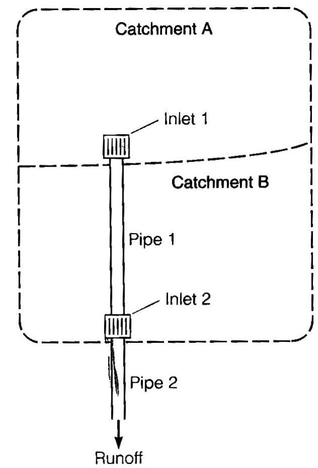
- Minimum size 12 - 18 inches (30 to 45 cm)
- In many locations, the minimum size of a storm sewer pipe is regulated

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 local design 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 (David Chin. *Water-Resources Engineering*. Pearson. 2nd edition 2006)

An example using the rational formula for convenience (Q = CiA), considering separate 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	46
		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 m^2), 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 m^2). 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 m^3/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 m^2) 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 m^2). 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 m^3/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:

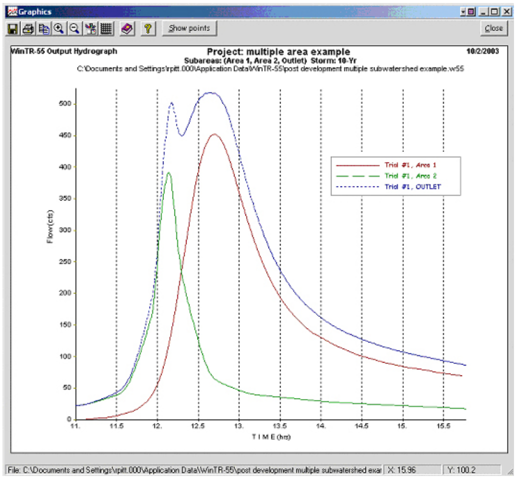
$$Q_p = \overline{C}iA = (0.36)(1.98x10^{-5}\, m/s)(30{,}000m^2) = 0.214m^3/s$$

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.



This is the outcome of a WinTR-55 analysis showing the routing of the separate source area hydrographs resulting in accurate peak flow predictions. This is a suitable method for relatively simple drainage areas. A complete hydrology/hydraulics model is needed for typical combined sewer analyses and designs. The EPA's SWMM model is probably the most popular method for these complete analyses (the extra slides at the end of this presentation illustrates a simple example using SWMM).

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 wet weather design peak flow rate is 0.43 m³/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:

$$n^6\sqrt{RS_o}\geq 10^{-13} \quad \text{checking:}$$

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

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

The water 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 minimize deposition during wet weather (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 damage (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, the exact pipe size determined.

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

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

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/sec². 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:*

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

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:

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

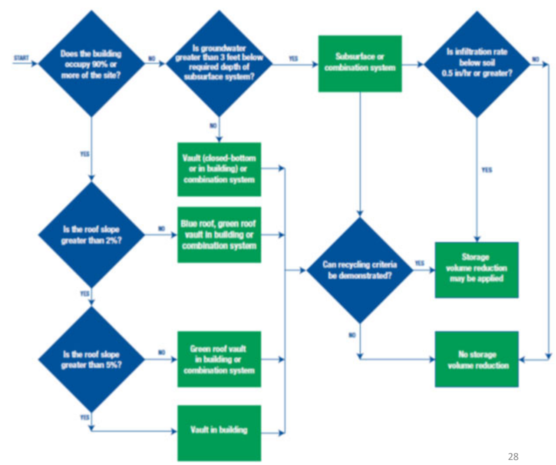
K_c is between 0.12 and 0.32 for pipes opposite each other in manholes, and the average value of 0.22 is used in the above example, along with the velocity value calculated with the Darcy-Weisbach equation. This head loss can be reduced with careful grouting of the bottom of the manholes making smooth transitions between the pipe segments. Otherwise, the down-gradient pipe must be lowered about 2.5 cm (1 inch) to account for this headloss.

New York City (2012) Green Infrastructure Plan to Reduce Peak Discharges to Combined Sewers

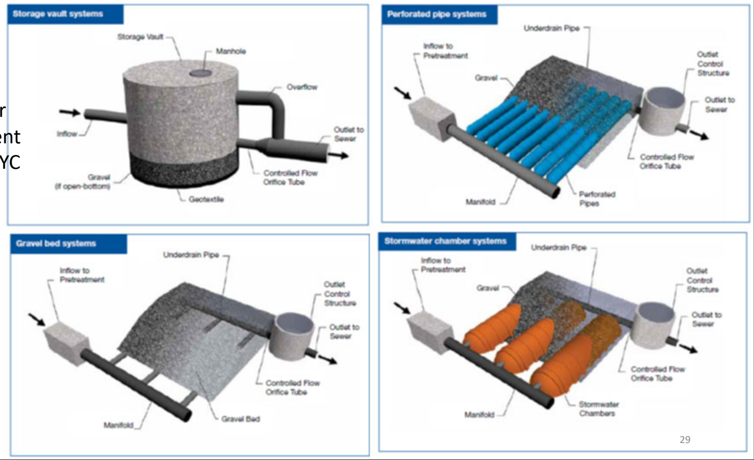
The United States Environmental Protection Agency (EPA) suggests that the use of green infrastructure "can be a cost-effective, flexible, and environmentally-sound approach to reduce stormwater runoff and sewer overflows and to meet Clean Water Act (CWA) requirements. Green infrastructure also provides a variety of community benefits including economic savings, green jobs, neighborhood enhancements and sustainable communities."

Recently, the NYC DEP has revised its stormwater rules for new development and redevelopment in combined sewer areas. The new performance standard is intended to reduce peak discharges to the city's sewer system during rain events by requiring greater onsite storage of stormwater runoff and slower release to the sewer system.

Relevant questions (blue) and conclusions (green) provide general guidance for selecting an appropriate stormwater management system during site planning and building design. (NYC 2012)



Typical subsurface stormwater management systems (NYC 2012)



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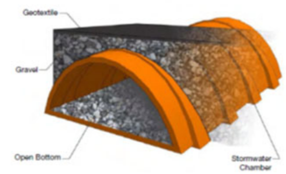
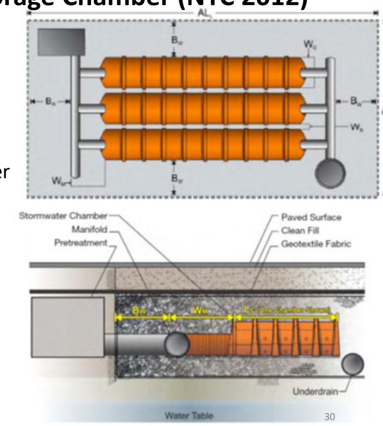
Underground Stormwater Storage Chamber (NYC 2012)



Cross-section of a typical stormwater chamber. Chambers are available in a variety of sizes for different site conditions and stormwater release rates.

Plan and profile views of a typical stormwater chamber layout.



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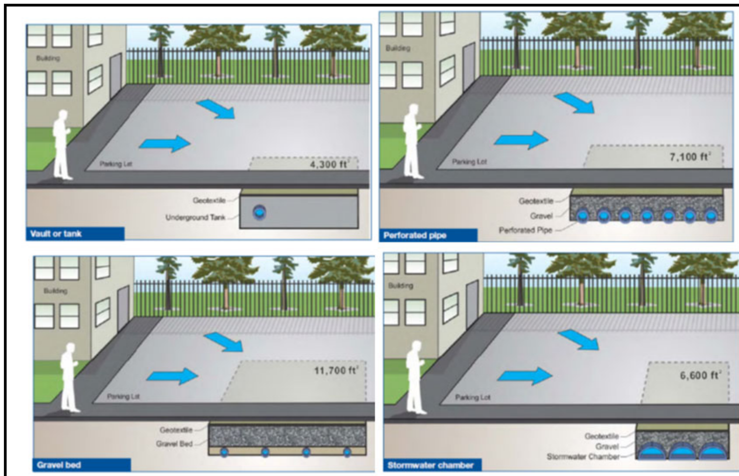

Schematics of four different subsurface systems showing relative footprint areas on a typical one-acre lot (NYC 2012).

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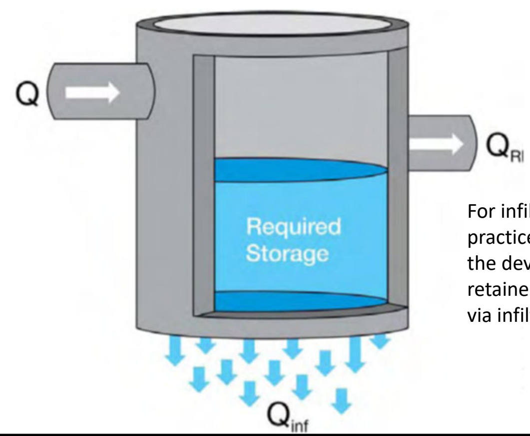
General Infiltration Storage and Head Losses (NYC 2012).



For infiltration practices, a portion of the developed flow is retained on site via infiltration.

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Small and large-scale performance monitoring at Kansas City, MO

Retrofitted controls are limited in the amount of the whole drainage area served.

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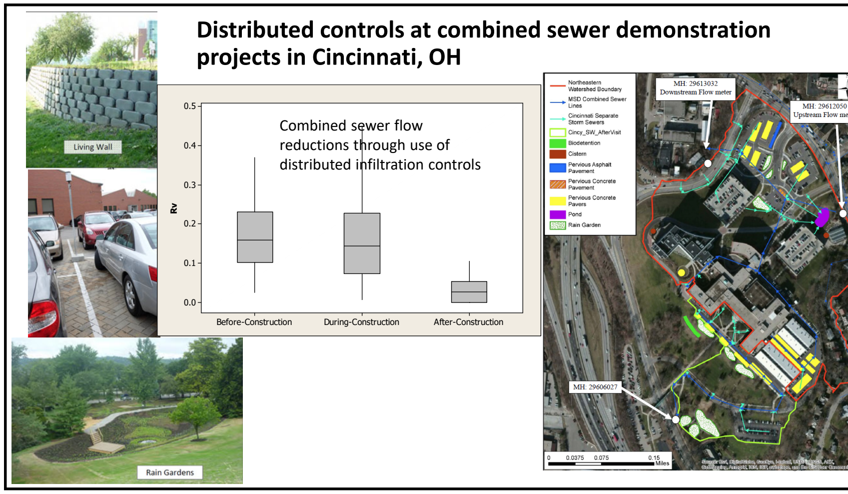
Distributed controls at combined sewer demonstration projects in Cincinnati, OH



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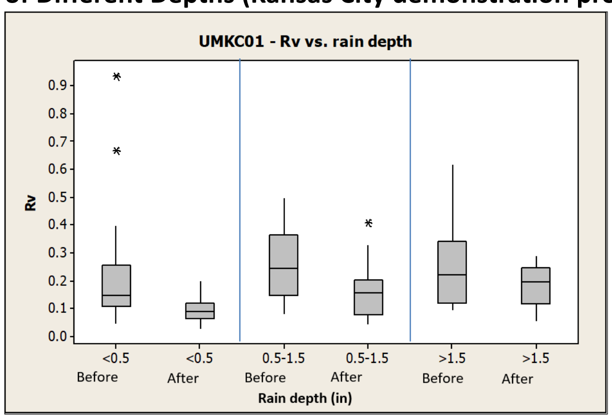
Green Infrastructure Benefits on Combined Sewer Flows for Rains of Different Depths (Kansas City demonstration project



The RV differences for the two smallest rain categories are shown to be significantly different (40 and 33% respectively), but the largest rain category did not show significant differences for the number of observations available.

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Recommended Maintenance Activities for Subsurface Systems (NYC 2012)

Schedule	Activity	Equipment
Seasonally or as needed	• Remove and clean filter bag • Immediately clean up spills on the pavement draining to the green infrastructure • Sweep impervious surfaces that drain to the green infrastructure • Maintain paved cover so that it drains properly to subsurface system • Maintain vegetation cover in good condition with complete coverage (if applicable) • Clean debris from pervious surface over subsurface system, if applicable • Clean perforated pipes (if applicable)	• Broom • Shovel • Replacement filter bags • Jet vacuum
When 25% of the pipe volume has been filled	• Jet-vacuum sediment and debris from the header pipe. Use a high-pressure nozzle with rear-facing jets to wash the sediment and debris into the inlet or pretreatment sump	• Jet vacuum
When sediment buildup reaches half the sump capacity	• Vector sediment and debris from the pretreatment sump. • Apply multiple passes of jet vacuum until backflush water is clean	• Vactor truck • Jet vacuum

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Recommended Maintenance Activities for Subsurface Systems (NYC 2012)

Semi-annually the first year; annually thereafter	• Remove sediment and debris from sumps in pretreatment and outlet control structures using a vacuum truck or similar device, after other system components such as pipes and vaults have been maintained • Replace filter bag	• Shovel • Jet vacuum • Replacement filter bags
Every five to ten years	• Jet-vacuum pipes clear of debris for perforated pipe and gravel bed systems, if scour protection has been installed below the pipes	• Jet vacuum
Winter considerations	• Break up ice formation around inlet hood	• Ice pick, or equivalent tool • Manhole bar

Note: The jet-vacuuming process uses a high-pressure water nozzle to propel itself while scouring and suspending sediment. As such, this process should not be performed in any portions of a subsurface system where scour protection has not been installed.

Combined Sewer System Modeling

The EPA (1999b) believes that continuous simulation models, using historical rainfall data, may be the best way to model sewer systems, CSOs, and their impacts.

Generally, the simplest model that meets the objectives of the modeling effort should be used. Although complex models usually provide greater precision than simpler models, they also require greater expense and effort.

CSS modeling involves hydrology, hydraulics, and water quality:

- Hydrology is the key factor in determining runoff in CSS drainage basins. Hydrologic modeling is generally done using runoff models to estimate flows influent to the sewer system. These models provide input data for hydraulic modeling of the CSS.
- CSS hydraulic modeling predicts the pipe flow characteristics in the CSS. These characteristics include the different flow rate components (sanitary, infiltration, inflow, and runoff), the flow velocity and depth in the interceptors, and the CSO flow rate and duration.
- CSS water quality modeling consists of predicting the pollutant characteristics of the combined sewage in the system.

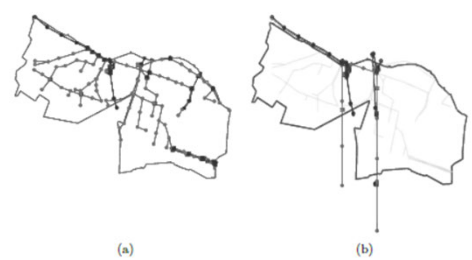
The EPA (1999b) list the following criteria for the selection of a CSS hydraulic mpdels:

1. Ability to accurately represent CSS's hydraulic behavior. For example, a complex, dynamic model may be appropriate when CSOs are caused by back-ups or surcharging. Since models differ in their ability to account for such factors as conduit cross-section shapes, special structures, pump station controls, tide simulation, and automatic regulators, these features in a CSS may guide the choice of one model over another.

2. Ability to accurately represent runoff in the CSS drainage basin. The runoff component of the hydraulic model (or the runoff model, if a separate hydrologic model is used) should adequately estimate runoff flows influent to the sewer system. It should adequately characterize rainfall characteristics as well as hydrologic factors such as watershed size, slope, soil types, and imperviousness.

3. Extent of monitoring. Monitoring usually cannot cover an entire CSS, particularly a large CSS. A dynamic model is more reliable for predicting the behavior of unmonitored overflows, since it can simulate all the hydraulic features controlling the overflow, but it often requires extensive resources for its application.

Drainage System Simplification for Computer Modeling of Combined Sewer Systems (Kroll 2019)



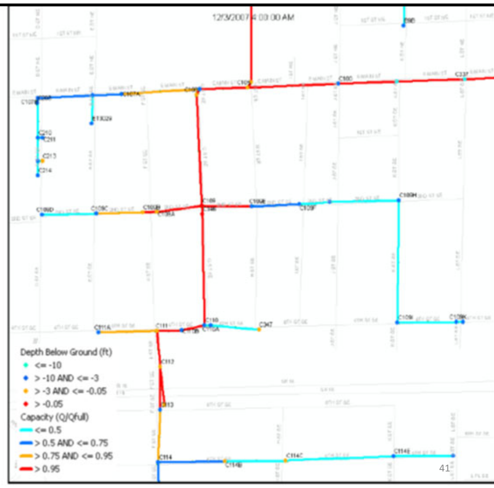
Detailed (a) and simplified network (b)

In this example, a 41% reduction of computer time was realized, along with a 4% difference in the calculated CSO volume.

Capacity Assessment Simulation Results using Computer Analyses of Combined Sewers (Seattle 2020)

Plan and profile plots can identify problem areas, such as:

- Decreasing flow capacities (e.g., smaller or flatter pipes) in the downstream piping.
- Hydraulic restrictions due to special structures, such as gates and weirs.
- Large flows joining the system without a corresponding increase in conveyance capacity.
- Local low spots that only allow for shallow bury pipes.



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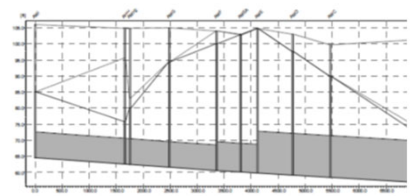
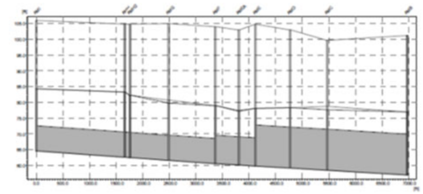
Profile Plots of Modeling Results Identify Problem Areas and Evaluation of Storage Solutions (Witt, *et al.* 2009)



Results showing high HGL due to local instability



Results with storage added where needed for stability

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Conclusions

- Combined sewer designs must consider both dry and wet weather processes, including deposition and scour, corrosion, flow capacities.
- Combined sewers must serve all buildings in an area (like separate sanitary drainage systems). In contrast, separate storm sewers usually do not reach the furthest reaches of a drainage area (relying on surface flows to the first inlets. Therefore, the dry weather sewage flows in a combined sewer in the upper reaches of a watershed also have to address low flushing flows before sufficient stormwater enters the system.
- Computer simulations of combined sewers must accurately include weirs and bypass elevations and locations. Complex hydraulics in combined sewers requires the use of suitable hydraulics models (such as the kinematic wave option in SWMM).
- The combined sewer network can usually be simplified for analyses, but needs to be a long-term continuous simulation considering future rainfall characteristics associated with climate change.
- Green infrastructure can be worthwhile for preventing nuisance drainage problems and reducing overflows during small and intermediate rains, but it has limited benefits in reducing large-scale flooding which may become more common in the future with climate change and population and development increases.

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Extra Slides

Simple Getting Started Example with SWMM ver 5

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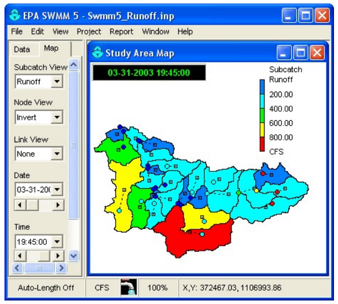
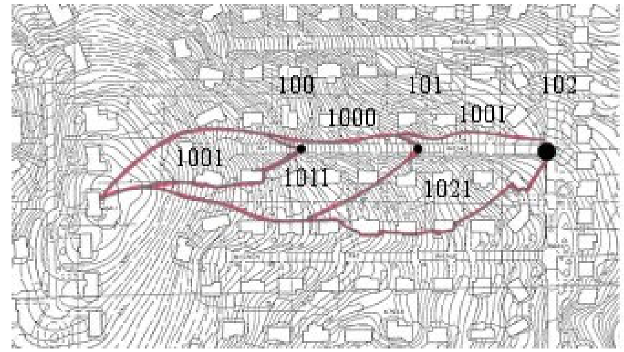
Getting Started with Storm and Sanitary Drainage Analysis using SWMM 5.1.015 (July 20, 2020)

The model can be downloaded by going to the EPA web site:
https://www.epa.gov/water-research/storm-water-management-model-swmm

The user manual can be downloaded at: https://www.epa.gov/sites/default/files/2019-02/documents/epaswmm5_1_manual_master_8-2-15.pdf





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 (ft^2)
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

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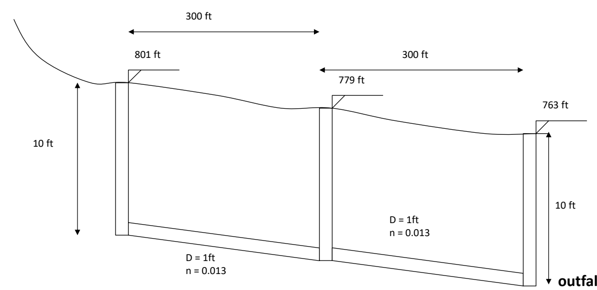

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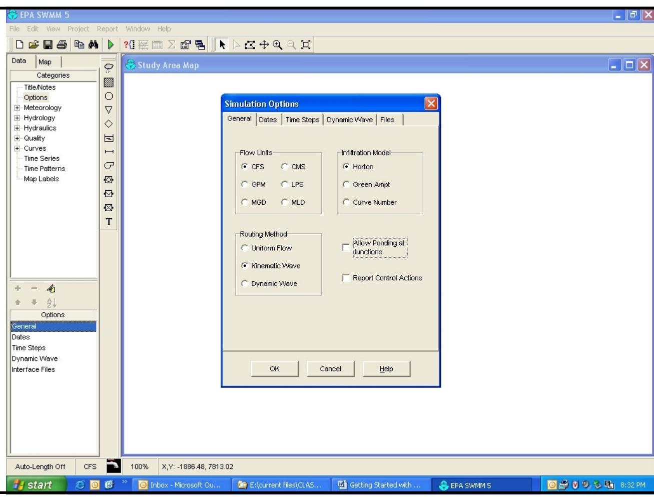
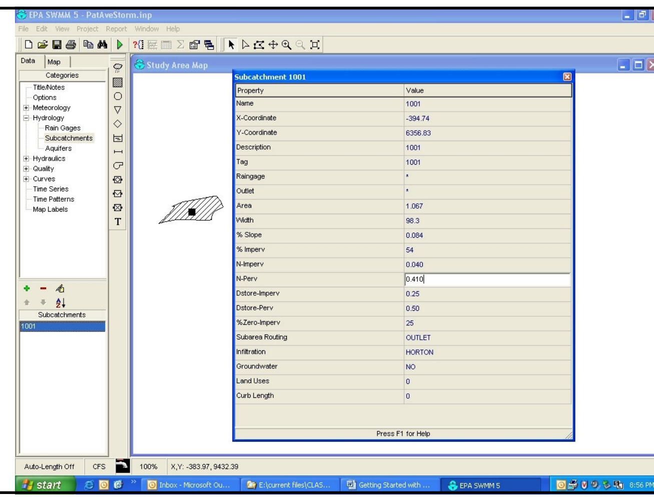


51

51



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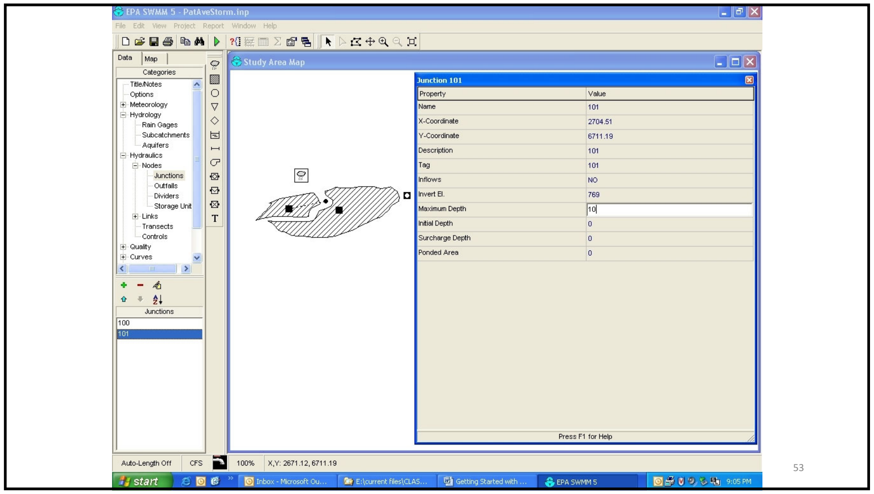
EPA SWMM 5 - PatAveStorm.inp
Study Area Map
Junction 101
Property Value
Name 101
X-Coordinate 2704.51
Y-Coordinate 6711.19
Description 101
Tag 101
Inflows NO
Invert El. 769
Maximum Depth 10
Initial Depth 0
Surcharge Depth 0
Ponded Area 0
Press F1 for Help

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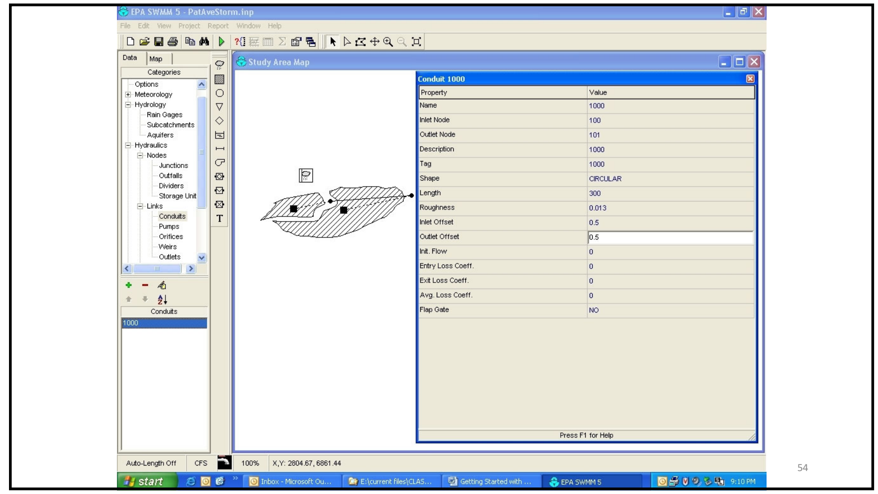
EPA SWMM 5 - PatAveStorm.inp
Study Area Map
Conduit 1000
Property Value
Name 1000
Inlet Node 100
Outlet Node 101
Description 1000
Tag 1000
Shape CIRCULAR
Length 300
Roughness 0.013
Inlet Offset 0.5
Outlet Offset 0.5
Init. Flow 0
Entry Loss Coeff. 0
Exit Loss Coeff. 0
Avg. Loss Coeff. 0
Flap Gate NO
Press F1 for Help

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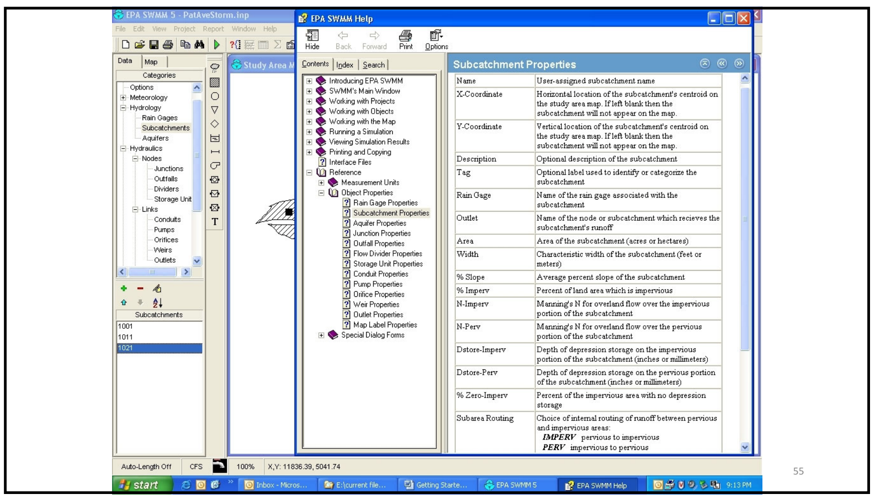
EPA SWMM Help
Subcatchment Properties
Name User-assigned subcatchment name
X-Coordinate Horizontal location of the subcatchment's centroid on the study area map. If left blank then the subcatchment will not appear on the map.
Y-Coordinate Vertical location of the subcatchment's centroid on the study area map. If left blank then the subcatchment will not appear on the map.
Description Optional description of the subcatchment
Tag Optional label used to identify or categorize the subcatchment
Rain Gage Name of the rain gage associated with the subcatchment
Outlet Name of the node or subcatchment which recieves the subcatchment's runoff
Area Area of the subcatchment (acres or hectares)
Width Characteristic width of the subcatchment (feet or meters)
% Slope Average percent slope of the subcatchment
% Imperv Percent of land area which is impervious
N-Imperv Manning's N for overland flow over the impervious portion of the subcatchment
N-Perv Manning's N for overland flow over the pervious portion of the subcatchment
Dstore-Imperv Depth of depression storage on the impervious portion of the subcatchment (inches or millimeters)
Dstore-Perv Depth of depression storage on the pervious portion of the subcatchment (inches or millimeters)
% Zero-Imperv Percent of the impervious area with no depression storage
Subarea Routing Choice of internal routing of runoff between pervious and impervious areas:
IMPERV pervious to impervious
PERV impervious to pervious

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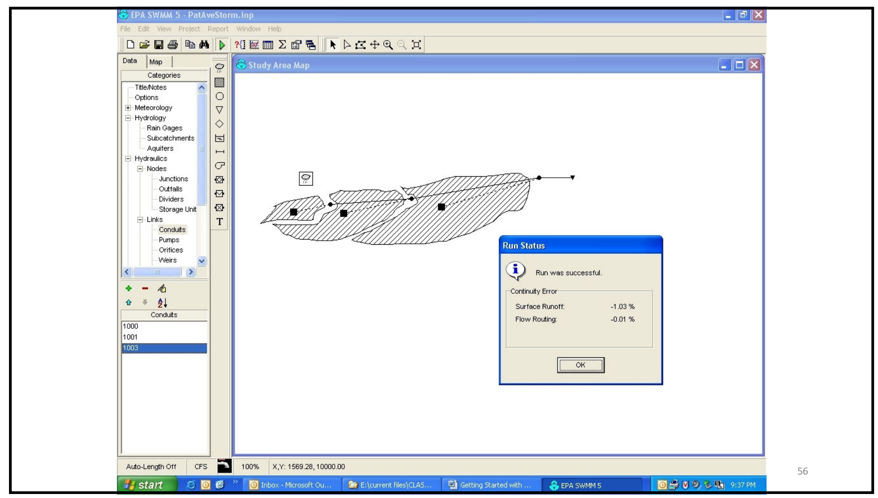
EPA SWMM 5 - PatAveStorm.inp
Study Area Map
Run Status
Run was successful.
Continuity Error
Surface Runoff: -1.03 %
Flow Routing: -0.01 %
OK

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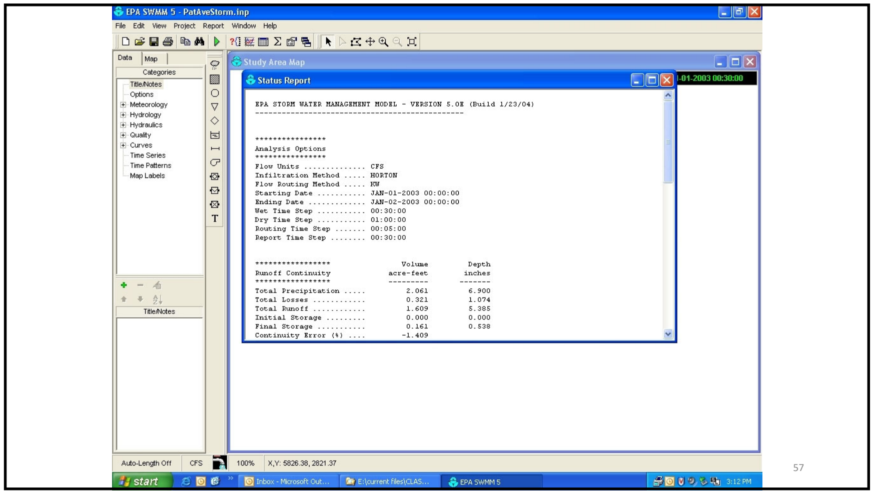
EPA SWMM 5 - PatAveStorm.inp
Study Area Map
Status Report
EPA STORM WATER MANAGEMENT MODEL - VERSION 5.0E (Build 1/23/04)
Analysis Options
Flow Units .... CFS
Infiltration Method ..... HORTON
Flow Routing Method ..... KW
Starting Date JAN-01-2003 00:00:00
Ending Date JAN-02-2003 00:00:00
Wet Time Step 00:30:00
Dry Time Step 01:00:00
Routing Time Step 00:05:00
Report Time Step 00:30:00
Runoff Continuity Volume acre-feet Depth inches
Total Precipitation 2.061 6.900
Total Losses 0.321 1.074
Total Runoff 1.609 5.385
Initial Storage 0.000 0.000
Final Storage 0.161 0.538
Continuity Error (%) .... -1.409
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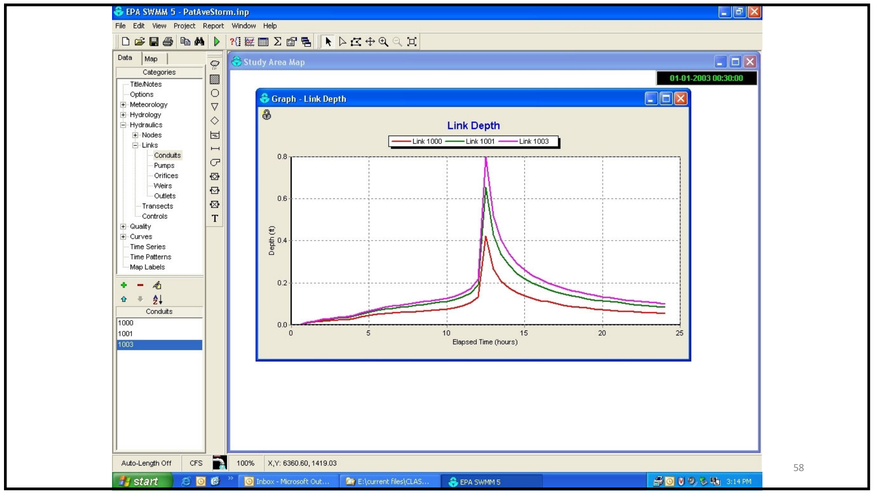
EPA SWMM 5 - PatAveStorm.inp
Study Area Map
Graph - Link Depth
Link Depth
Link 1000
Link 1001
Link 1003
Depth (ft)
Elapsed Time (hours)
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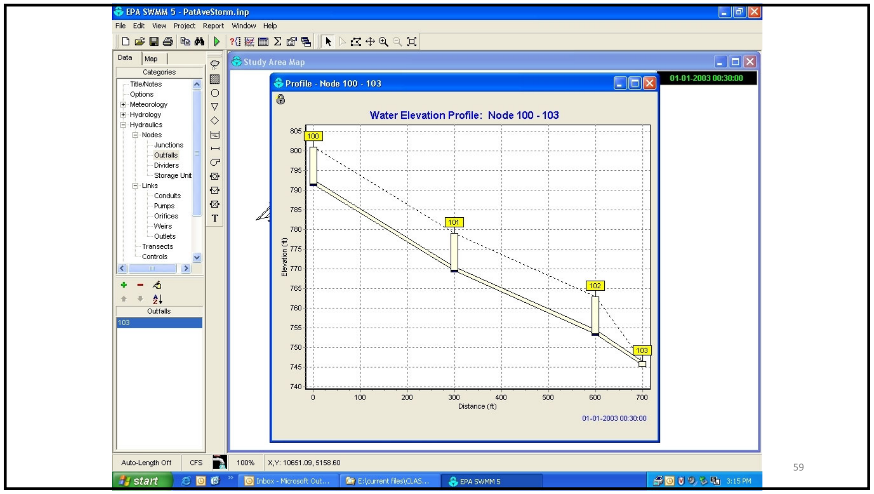
EPA SWMM 5 - PatAveStorm.inp
Study Area Map
Profile - Node 100 - 103
Water Elevation Profile: Node 100 - 103
Elevation (ft)
Distance (ft)
01-01-2003 00:30:00
59

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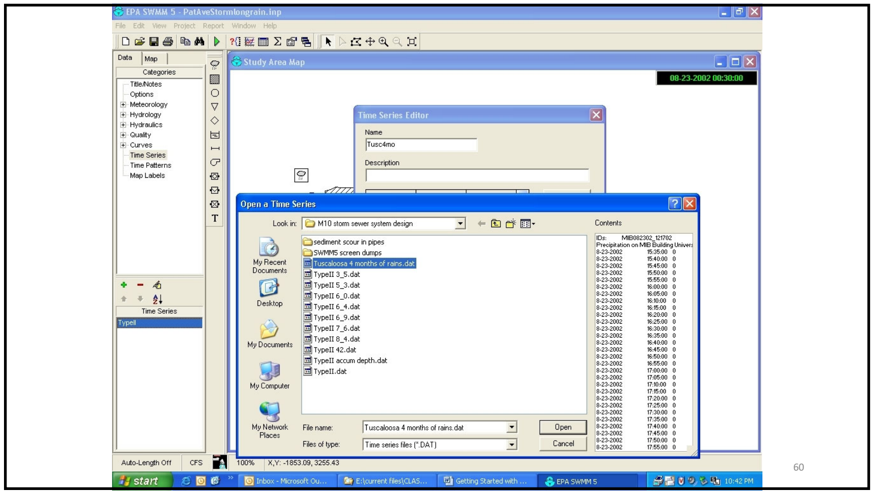
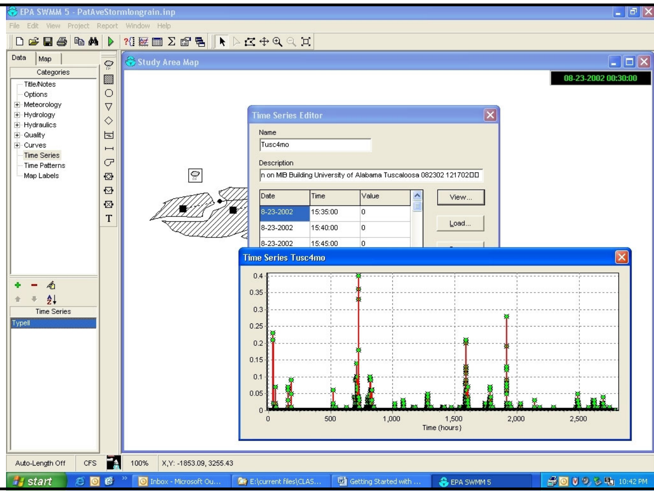

EPA SWMM 5 - PatAveStormlongrain.inp
Study Area Map
Time Series Editor
Open a Time Series
Look in: M10 storm sewer system design
sediment scour in pipes
SWMM5 screen dumps
Tuscaloosa 4 months of rains.dat
File name: Tuscaloosa 4 months of rains.dat
Files of type: Time series files (*.DAT)
Open
Cancel
60

60



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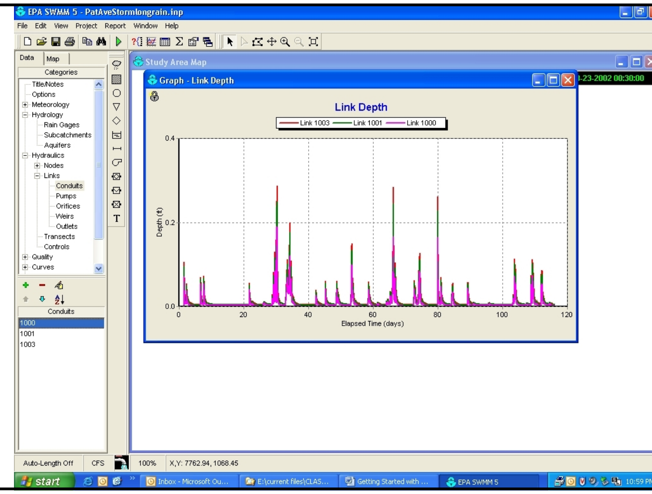

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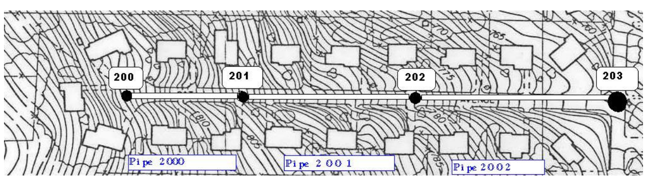
"Hello World" Pat Avenue Sanitary Drainage Design Example



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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

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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

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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

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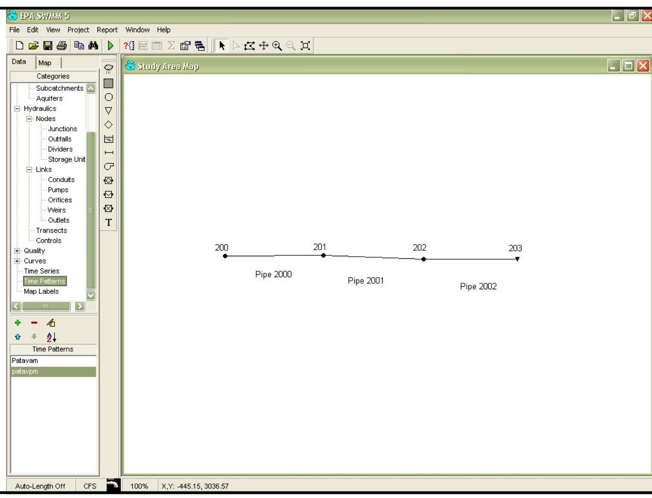

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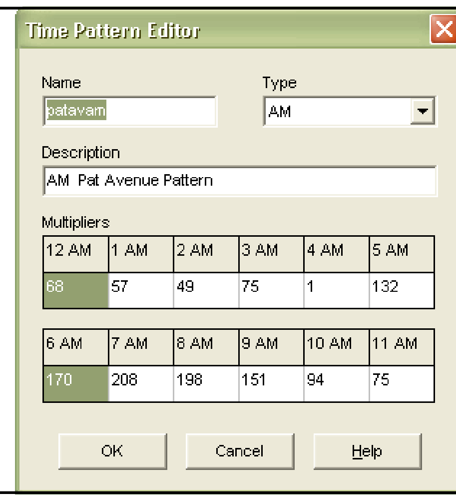
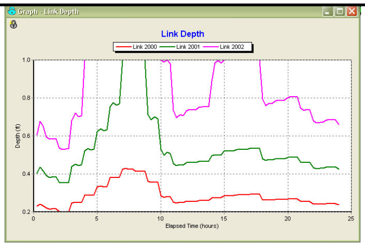
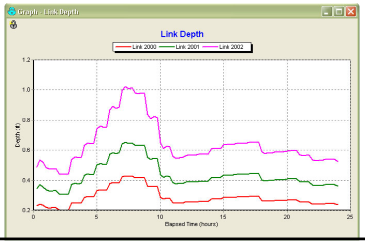


68



Surcharged 1 ft. pipes



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

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