

# **STORM WATER MANAGEMENT MODEL**

## **APPLICATIONS MANUAL**

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## **FOREWORD**

The U.S. Environmental Protection Agency is charged by Congress with protecting the Nation's land, air, and water resources. Under a mandate of national environmental laws, the Agency strives to formulate and implement actions leading to a compatible balance between human activities and the ability of natural systems to support and nurture life. To meet this mandate, EPA's research program is providing data and technical support for solving environmental problems today and building a science knowledge base necessary to manage our ecological resources wisely, understand how pollutants affect our health, and prevent or reduce environmental risks in the future.

The National Risk Management Research Laboratory is the Agency's center for investigation of technological and management approaches for reducing risks from threats to human health and the environment. The focus of the Laboratory's research program is on methods for the prevention and control of pollution to the air, land, water, and subsurface resources; protection of water quality in public water systems; remediation of contaminated sites and ground water; and prevention and control of indoor air pollution. The goal of this research effort is to catalyze development and implementation of innovative, cost-effective environmental technologies; develop scientific and engineering information needed by EPA to support regulatory and policy decisions; and provide technical support and information transfer to ensure effective implementation of environmental regulations and strategies.

Water quality impairment due to runoff from urban and developing areas continues to be a major threat to the ecological health of our nation's waterways. The EPA Stormwater Management Model is a computer program that can assess the impacts of such runoff and evaluate the effectiveness of mitigation strategies. This manual presents a number of worked-out examples that shows new users how to set up and apply SWMM to the most common types of stormwater management and design problems encountered in practice.

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## **ABSTRACT**

The EPA Storm Water Management Model (SWMM) is a dynamic rainfall-runoff simulation model that computes runoff quantity and quality from primarily urban areas. This manual is a practical application guide for new SWMM users who have already had some previous training in hydrology and hydraulics. It contains nine worked-out examples that illustrate how SWMM can be used to model some of the most common types of stormwater management and design problems encountered in practice. These include: computing runoff for both pre- and post development conditions; analyzing the hydraulics of simple collection systems; designing a multi-purpose detention pond; modeling distributed low impact runoff controls; simulating the buildup, washoff, transport and treatment of stormwater pollutants; analyzing both dual drainage and combined sewer systems; and running long-term continuous simulations. Each example is accompanied by a complete SWMM input data file.

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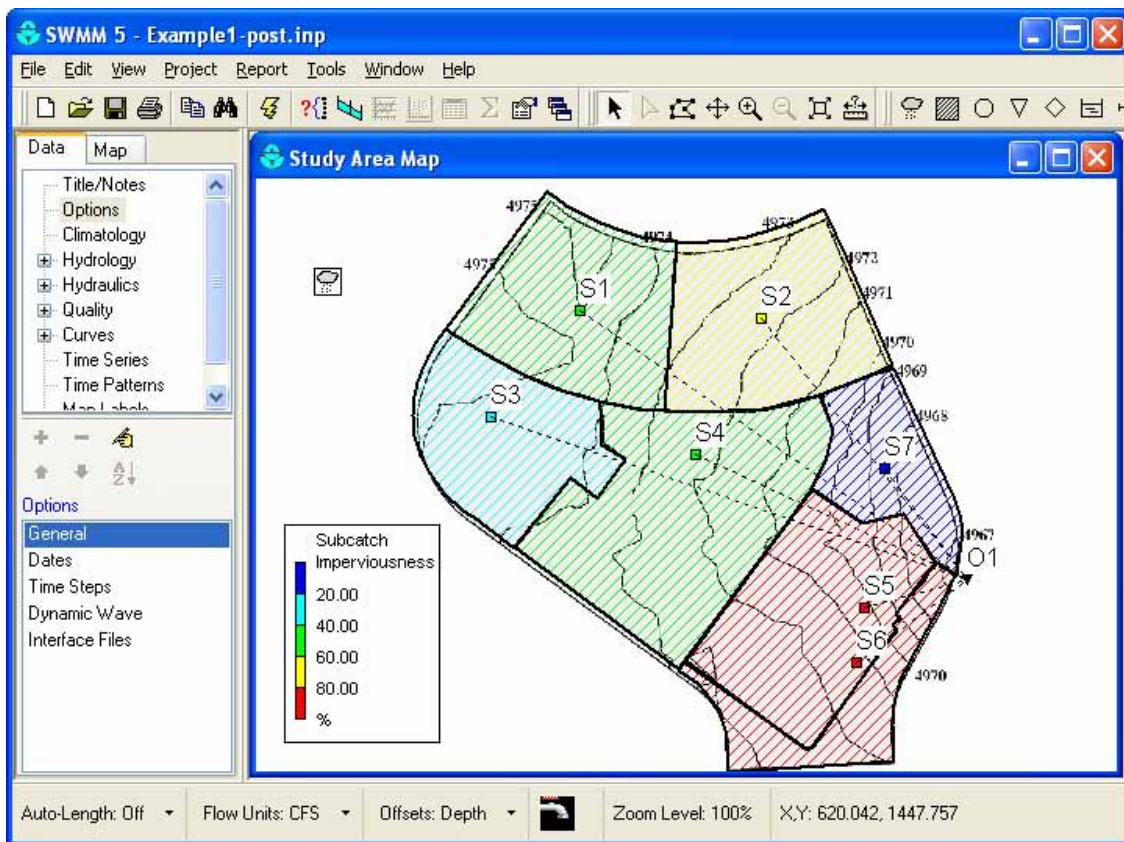
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# Introduction

The EPA Storm Water Management Model (SWMM) is a dynamic rainfall-runoff simulation model that computes runoff quantity and quality from primarily urban areas. The runoff component of SWMM operates on a collection of subcatchment areas that receive precipitation and generate runoff and pollutant loads. The routing portion of SWMM transports this runoff through a system of pipes, channels, storage/treatment devices, pumps and regulators. SWMM tracks the quantity and quality of runoff generated within each subcatchment and the flow rate, flow depth, and quality of water in each pipe and channel during a simulation period comprised of multiple time steps.

SWMM was first developed in 1971 and since then has undergone several major upgrades. It continues to be widely used throughout the world for planning, analysis, and design related to stormwater runoff, combined sewers, sanitary sewers, and other drainage systems in urban areas and has also been used for modeling non-urban areas. The most current implementation of the model is version 5.0 which was released in 2005. It has modernized both the model's structure and its user interface, making SWMM easier to use and more accessible to a new generation of hydrologists, engineers, and water resources management specialists.



The objective of this manual is to serve as a practical application guide for new SWMM users who have already had some previous training in hydrology and hydraulics. It contains nine worked-out examples that illustrate how SWMM can be used to model some of the most common types of stormwater management and design problems encountered in practice. The following applications are discussed in the manual:

1. *Post-Development Runoff*. Surface runoff from a 29 acre residential site is computed for several design-storm events under both pre- and post-development conditions.
2. *Surface Drainage Hydraulics*. A surface runoff conveyance network is added to the post-development catchment area of Example 1 and is analyzed with SWMM's various hydraulic routing options.
3. *Detention Pond Design*. A detention pond and outlet structure are designed for the post-development condition of Example 2 that meets both water quality control and peak flow reduction criteria.
4. *Low Impact Development*. Two typical Low Impact Development (LID) controls, filter strips and infiltration trenches, are placed within the post-development catchment modeled in Example 2.
5. *Runoff Water Quality*. The buildup, washoff and routing of total suspended solids (TSS) is simulated within the post-development catchment modeled in Example 2.
6. *Runoff Treatment*. The removal of TSS in the LID controls and detention pond of Examples 3 and 4 is modeled.
7. *Dual Drainage Systems*. The surface drainage system of Example 2 is converted into a parallel system of below ground storm sewers and above ground streets and gutters that is subjected to both surcharged flow and street flooding.
8. *Combined Sewer Systems*. The sewer system of Example 7 is converted into a combined system that carries both dry weather sanitary flow and wet weather runoff and includes various flow diversion structures and a pumped force main.
9. *Continuous Simulation*. SWMM's statistical tools are used to analyze the performance of the detention pond designed in Example 3 over a continuous ten year period of historical rainfall.

All the examples are developed for the same catchment area and each one builds in some degree on the results of a previous example. Therefore, it is recommended that the reader begin with Example 1 and work sequentially through Example 9, while hopefully building the required input data files and running them with SWMM for each example. They can then compare the input files they build with the files created by the authors. These files, as well as the backdrop image files that are needed to complete the examples, are available in a compressed file named **epaswmm5\_apps\_manual.zip**. It can be downloaded from the EPA SWMM web site at: <http://www.epa.gov/endnrml/models/swmm>.

This manual assumes that readers have a basic knowledge of how to run EPA SWMM 5 and perform such functions as opening a new project, setting project defaults, adding drainage system objects to a project, editing the properties of these objects, and viewing simulation results. These topics and more are covered in the *SWMM Users Manual* which is also available from the EPA SWMM web site. That manual includes a tutorial example that leads new users through each of these steps.

# Example 1. Post-Development Runoff

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This first example demonstrates how to construct a hydrologic model of an urban catchment and use it to compare stormwater runoff under both pre- and post-development conditions. It illustrates the process of spatially dividing a catchment into smaller computational units, called subcatchments, and discusses the characteristics of these subcatchments that SWMM uses to transform rainfall into runoff. This example considers runoff only. Flow routing of runoff through the drainage pipes and channels contained within the catchment is addressed in Example 2.

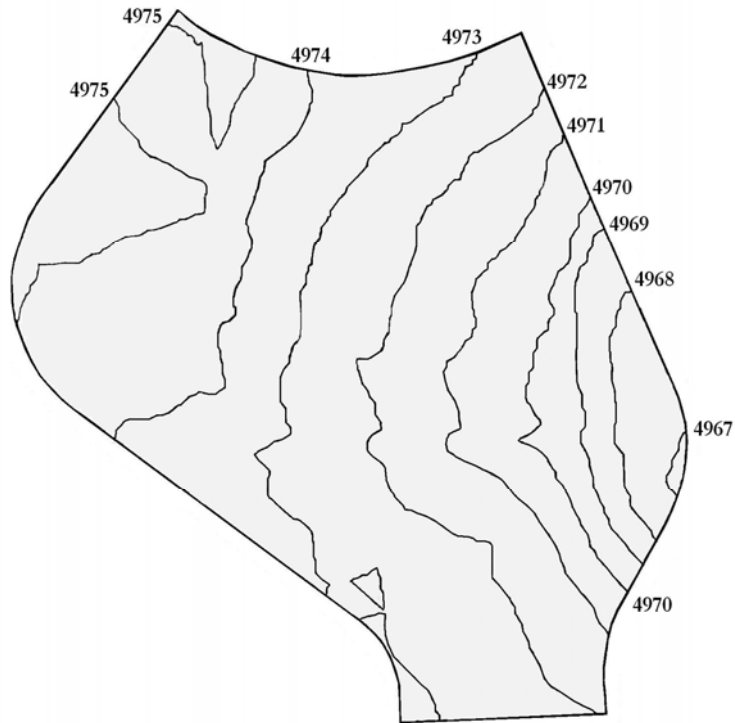
Models of this type are very common in practice. Many local stormwater ordinances and agencies require that new developments limit peak runoff flows relative to those under pre-development conditions. To meet environmental sustainability objectives, similar criteria are being applied to total runoff volume as well.

## 1.1 Problem Statement

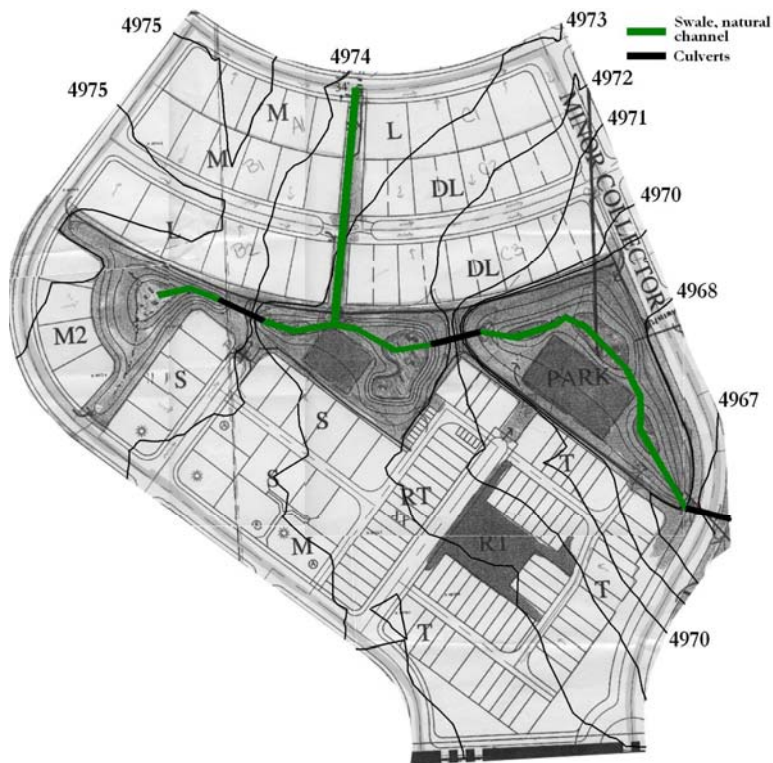
Figure 1-1 is a contour map of a 29 acre natural catchment area where a new residential development is planned. This undeveloped area is primarily pasture land that has a silt loam soil type. Figure 1-2 shows the proposed development for this site. With the exception of the depressions located in the parkland area, no major changes in topography are expected. This implies that future streets will, in general, follow the natural slope. However, the residential lots will be graded toward the street at a slope of 2% so they can drain easily. The developed site will drain to a stream through a culvert under the street located on the southeast side of the site, which is considered to be the outlet point of the catchment.

The objective is to estimate the stormwater discharges at the catchment's outlet and compare them to the ones generated prior to urbanization. The approach typically employed in stormwater drainage manuals will be used, which is to compute the hydrologic response of the catchment to a series of synthetic design storms associated with different return periods. The design storms used here will be for a 2-hour event with return periods of 2, 10, and 100 years. Most of the parameter values used in this example were taken from tables published in the *SWMM User's Manual* (Rossman, 2008). These were supplemented with design guidelines published by the Denver Urban Drainage and Flood Control District (UDFCD) (UDFCD, 2001).

Two models will be built: one that represents the catchment in its current undeveloped condition and one that represents the catchment after it is fully developed. Because this is an initial estimation of the discharges at the outlet of the catchment under its current and future conditions, no channelized flows will be defined and only runoff as overland flow will be simulated. Example 2 in this manual will add a conveyance system of swales, channels, and culverts to this model.



**Figure 1-1. Undeveloped site**



**Figure 1-2. Developed site**



## 1.2 System Representation

SWMM is a distributed model, which means that a study area can be subdivided into any number of irregular subcatchments to best capture the effect that spatial variability in topography, drainage pathways, land cover, and soil characteristics have on runoff generation. An idealized subcatchment is conceptualized as a rectangular surface that has a uniform slope and a width  $W$  that drains to a single outlet channel as shown in Figure 1-3. Each subcatchment can be further divided into three subareas: an impervious area with depression (detention) storage, an impervious area without depression storage and a pervious area with depression storage. Only the latter area allows for rainfall losses due to infiltration into the soil.

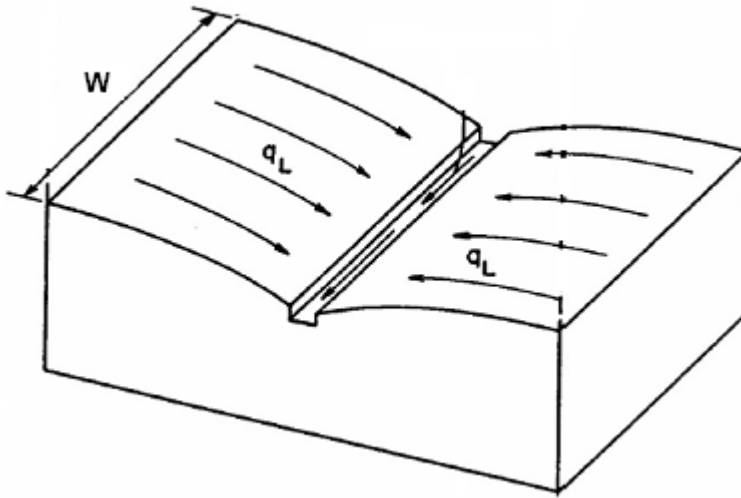


Figure 1-3. Idealized representation of a subcatchment

The hydrologic characteristics of a study area's subcatchments are defined by the following set of input parameters in SWMM:

- **Area**

This is the area bounded by the subcatchment boundary. Its value is determined directly from maps or field surveys of the site or by using SWMM's Auto-Length tool when the subcatchment is drawn to scale on SWMM's study area map.

- **Width**

The width can be defined as the subcatchment's area divided by the length of the longest overland flow path that water can travel. If there are several such paths then one would use an average of their lengths to compute a width.

In applying this approach one must be careful not to include channelized flow as part of the flow path. In natural areas, true overland flow can only occur for distances of about 500 feet before it begins to consolidate into rivulet flow. In urbanized catchments, true overland flow can be very short before it is collected into open channels or pipes. A maximum overland flow length of 500 feet is appropriate for non-urban catchments while the typical overland flow length is the

length from the back of a representative lot to the center of the street for urban catchments. If the overland flow length varies greatly within the subcatchment, then an area-weighted average should be used.

Because it is not always easy to accurately identify all of the overland flow paths within a subcatchment, the width parameter is often regarded as a calibration parameter whose value can be adjusted to produce a good match between observed and computed runoff hydrographs.

- **Slope**

This is the slope of the land surface over which runoff flows and is the same for both the pervious and impervious surfaces. It is the slope of what one considers to be the overland flow path or its area-weighted average if there are several such paths in the subcatchment.

- **Imperviousness**

This is the percentage of the subcatchment area that is covered by impervious surfaces, such as roofs and roadways, through which rainfall cannot infiltrate. Imperviousness tends to be the most sensitive parameter in the hydrologic characterization of a catchment and can range anywhere from 5% for undeveloped areas up to 95% for high-density commercial areas.

- **Roughness Coefficient**

The roughness coefficient reflects the amount of resistance that overland flow encounters as it runs off of the subcatchment surface. Since SWMM uses the Manning equation to compute the overland flow rate, this coefficient is the same as Manning's roughness coefficient  $n$ . Separate values are required for the impervious and pervious fractions of a subcatchment since the pervious  $n$  is generally an order of magnitude higher than the impervious  $n$  (e.g., 0.8 for dense wooded areas versus 0.012 for smooth asphalt).

- **Depression Storage**

Depression storage corresponds to a volume that must be filled prior to the occurrence of any runoff. Different values can be used for the pervious and impervious areas of a subcatchment. It represents initial abstractions such as surface ponding, interception by flat roofs and vegetation, and surface wetting. Typical values range between 0.05 inches for impervious surfaces to 0.3 inches for forested areas.

- **Percent of Impervious Area Without Depression Storage**

This parameter accounts for immediate runoff that occurs at the beginning of rainfall before depression storage is satisfied. It represents pavement close to the gutters that has no surface storage, pitched rooftops that drain directly to street gutters, new pavement that may not have surface ponding, etc. By default the value of this variable is 25%, but it can be changed in each subcatchment. Unless special circumstances are known to exist, a percent imperviousness area without depression storage of 25% is recommended.

- **Infiltration Model**

Three different methods for computing infiltration loss on the pervious areas of a subcatchment are available in SWMM. They are the Horton, Green-Ampt and Curve Number models. There is no general agreement on which model is best. The Horton model has a long history of use in dynamic simulations, the Green-Ampt model is more physically-based, and the Curve Number model is derived from (but not the same as) the well-known SCS Curve Number method used in simplified runoff models.

The Horton model will be used in the current example. The parameters for this model include:

- *Maximum infiltration rate*: This is the initial infiltration rate at the start of a storm. It is difficult to estimate since it depends on the antecedent soil moisture conditions. Typical values for dry soils range from 1 in/h for clays to 5 in/h for sands.
- *Minimum infiltration rate*: This is the limiting infiltration rate that the soil attains when fully saturated. It is usually set equal to the soil's saturated hydraulic conductivity. The latter has a wide range of values depending on soil type (e.g., from 0.01 in/hr for clays up to 4.7 in/hr for sand).
- *Decay coefficient*: This parameter determines how quickly the infiltration rate “decays” from the initial maximum value down to the minimum value. Typical values range between 2 to 7 hr<sup>-1</sup>.

- **Precipitation Input**

Precipitation is the principal driving variable in rainfall-runoff-quantity simulation. The volume and rate of stormwater runoff depends directly on the precipitation magnitude and its spatial and temporal distribution over the catchment. Each subcatchment in SWMM is linked to a Rain Gage object that describes the format and source of the rainfall input for the subcatchment.

### 1.3 Model Setup – Undeveloped Site

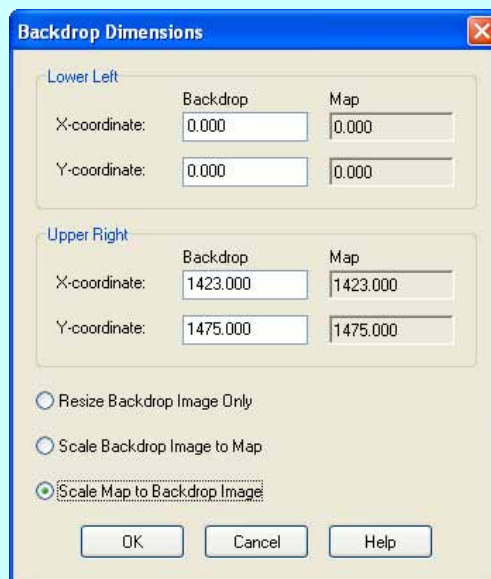
The SWMM model for the undeveloped site is depicted in Figure 1-4. It consists of a rain gage *RI* that provides precipitation input to a single subcatchment *S1* whose runoff drains to outfall node *O1*. Note that the undeveloped site contour map has been used as a backdrop image on which the subcatchment outline has been drawn. The SWMM input file for this model is named **Example1-Pre.inp**.

## Utilizing a Backdrop Image in SWMM

To help facilitate the placement of drainage-system objects, SWMM can utilize an image as a backdrop behind a project's study area map. This image is typically a site map of some kind with known dimensions. Any bitmap image file (**BMP** extension), Windows metafile (**WMF** or **EMF** extension) or JPEG image file (**JPG** or **JPEG** extension) can be used as a backdrop. These images would typically come from a CAD or GIS drawing of the site or perhaps from an electronically published or scanned topographic or street map.

Before adding the backdrop image, the actual horizontal and vertical dimensions represented in the image must be known in order to scale the map correctly. To add a properly scaled backdrop image to a SWMM project do the following:

1. Select **View | Backdrop | Load** from the main menu.
2. Enter the name of the backdrop image file to be loaded in the *Backdrop Image Selector* dialog that appears. After closing the dialog, the backdrop will appear on the Study Area Map with a default set of dimensions.
3. To properly scale both the backdrop and the study area map, select **View | Backdrop | Resize** from the main menu. In the *Backdrop Dimensions* dialog select the "Scale Map to Backdrop Image" checkbox. This will automatically adjust the dimensions of the map to be the same as the backdrop. Then enter **0,0** for the lower left coordinates and the backdrop's width and height for the upper right coordinates.



4. Finally, select **View | Dimensions** from the main menu and select the appropriate units for the map's dimensions (typically feet or meters).

At times one will want to lighten the backdrop image so that the added drainage system objects will stand out more clearly on the map. This can be done by toggling the **View | Backdrop | Watermark** option on the main menu.

### Subcatchment Properties

According to the site's contour map, its topography is fairly homogenous and no well-defined channels exist within the basin which means that mainly overland flow takes place. There are no roads or other local impervious areas and the type of soil is similar throughout the watershed (Sharpsburg silt loam). Therefore, no disaggregation is required based on the spatial distribution of catchment properties. The single subcatchment *S1* drains to the free outfall node *O1* whose elevation is **4967** ft.

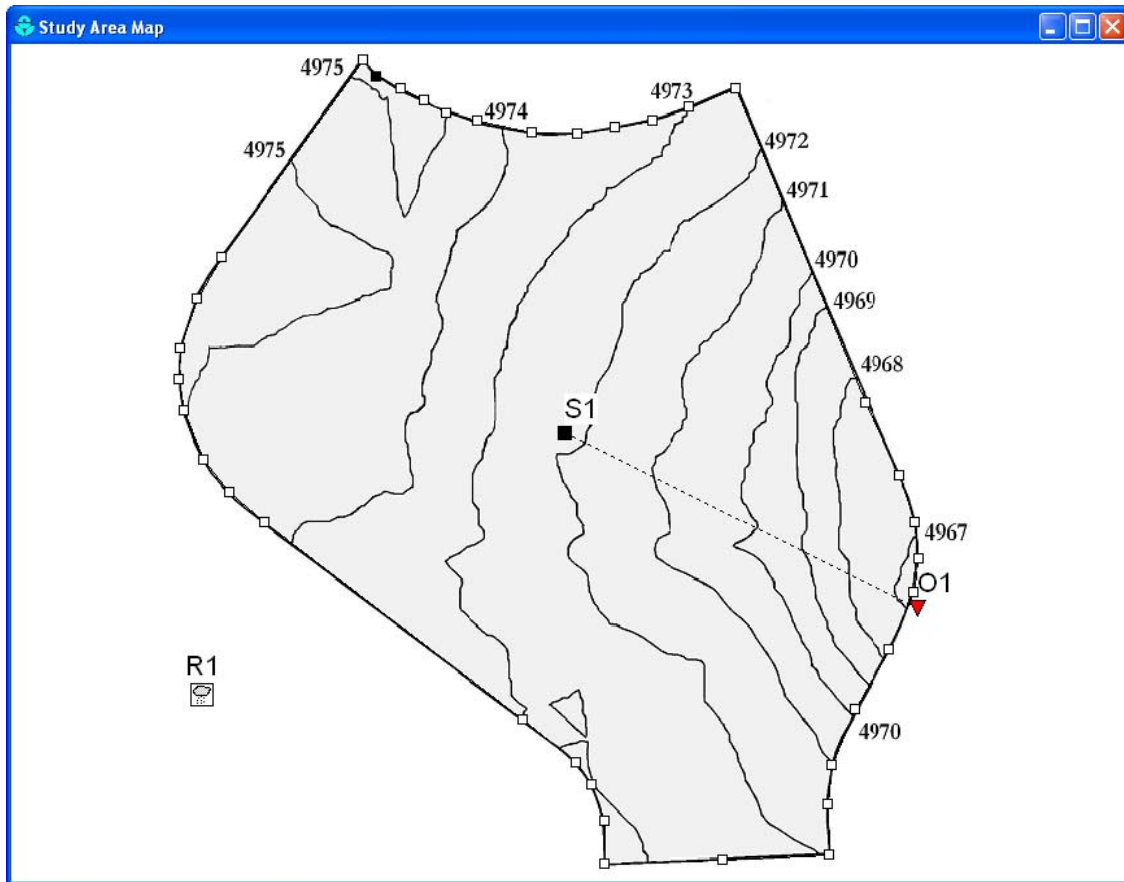


Figure 1-4. SWMM representation of the undeveloped study area

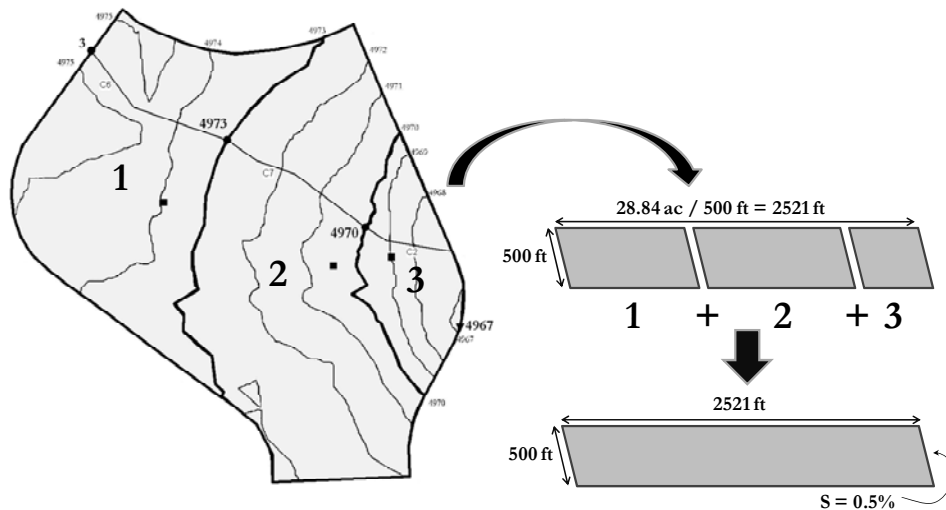
The area shown in Figure 1-4 is not the entire pre-development natural catchment. It has been bounded by the post-development roadways to come so that comparisons between the two conditions (developed and undeveloped) can be made.

The properties assigned to the single subcatchment *S1* are summarized in Table 1-1. Their values were developed on the basis of the undeveloped site being primarily pasture land containing a silt loam soil. Parameter values for this soil type can be found in the *SWMM User's Manual* and the *UDFCD Guidance Manual*.

**Table 1-1. Properties of the undeveloped subcatchment**

Property	Value	Property	Value
Area	28.94 ac	Depression storage, pervious areas	0.3 in.
Width	2521 ft	Depression storage, impervious areas	0.06 in.
Slope	0.5 %	% of impervious area without depression storage	25%
Imperviousness	5 %	Maximum infiltration rate	4.5 in./hr
Roughness coefficient, impervious areas	0.015	Minimum infiltration rate	0.2 in./hr
Roughness coefficient, pervious areas	0.24	Infiltration decay coefficient	6.5 hr <sup>-1</sup>

The subcatchment's area was determined using SWMM's *Auto-Length* tool. The subcatchment width was arrived at by first assuming a maximum overland-flow length of 500 ft, as recommended for undeveloped areas. By the time runoff has travelled this distance it has consolidated into rivulets and therefore no longer behaves as overland flow over a uniform plane. Based on this assumption, the subcatchment was divided into subareas with flow-path lengths of 500 feet or less. Figure 1-5 shows that there were three such areas whose flow-path lengths are each 500 ft. Thus, the average flow length is also 500 ft. When the total subcatchment area of 28.94 acres (1,260,626 ft<sup>2</sup>) is divided by the average flow length the resulting subcatchment width is 2,521 ft. The average slope of the subcatchment was derived from the area-weighted average of the slopes of the three sub-areas that comprise the overland flow paths as shown in Figure 1-5 and Table 1-2. Its value is 0.5.

**Figure 1-5. Computation of width of the undeveloped subcatchment**

**Table 1-2. Flow lengths and slopes of the undeveloped subcatchment**

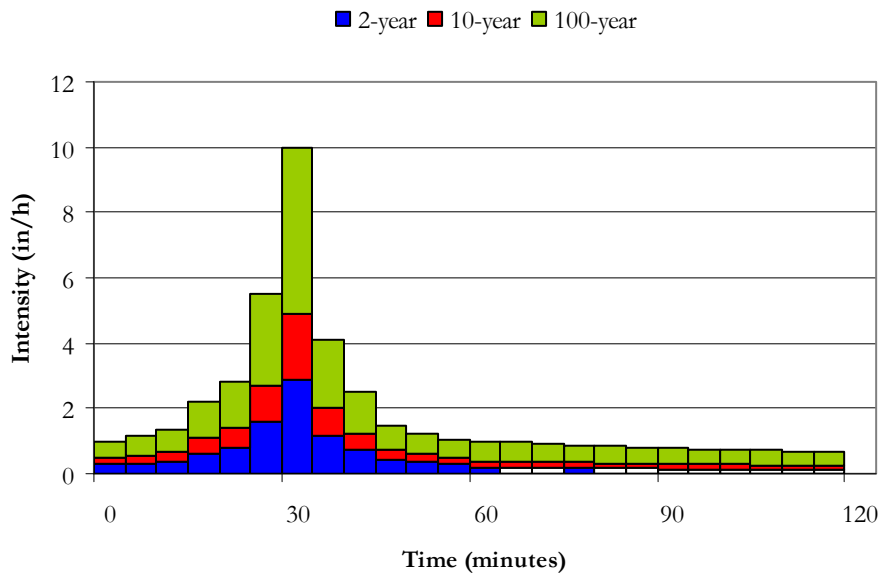
Sub-Area	Flow Path Length (ft)	Associated Area ( $A_i$ ) (ac)	Upstream Elevation (ft)	Downstream Elevation (ft)	Elevation Difference (ft)	Slope ( $S_i$ ) (%)
1	500	11.13	4974.8	4973	1.8	0.4%
2	500	14.41	4973	4970	3	0.6%
3	500	3.4	4970	4967	3	0.6%
<b>Average</b>	<b>500</b>					<b>0.5<sup>1</sup></b>

<sup>1</sup> Weighted average corresponding to  $\sum_{i=1}^{i=3} \frac{S_i \cdot A_i}{\sum A_i}$

The Horton method was selected as the infiltration model for this analysis. The values assigned to its parameters are typical of those for a silt loam soil as found in this watershed and are listed in Table 1-1. It is strongly recommended to use any available site-specific data from the study area BEFORE relying on values from the literature.

### Rain Gage Properties


The properties of rain gage *RI* describe the source and format of the precipitation data that are applied to the study area. In this example, the rainfall data consist of three synthetic design events that represent the 2-, 10- and 100-year storms of 2-hour duration. Each storm is represented by a separate time series object in the SWMM model that consists of rainfall intensities recorded at a 5 minute time-interval. The time series are named 2-yr, 10-yr and 100-yr, respectively and are plotted in Figure 1-6. The total depth of each storm is 1.0, 1.7, and 3.7 inches, respectively. These design storms were selected by the City of Fort Collins, CO to be used with SWMM (City of Fort Collins, 1999).

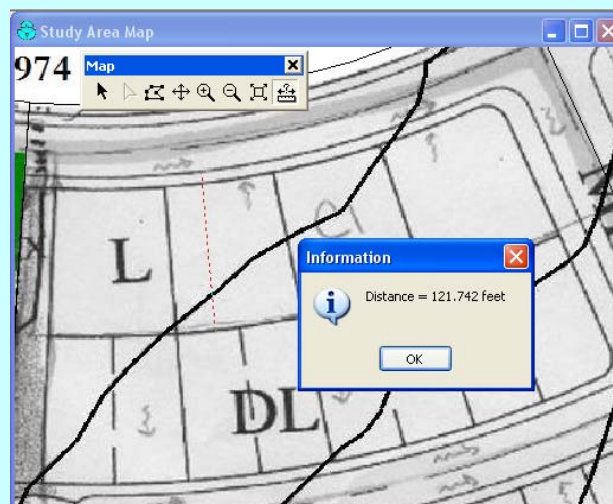
**Figure 1-6. Design storm hyetographs**

## Measuring Tools Available in SWMM

SWMM's graphical user interface has several tools that can assist in measuring distances and areas on a project's study area map. One such tool is the *Auto-Length* option. If *Auto-Length* is turned on, any subcatchment's polygon outline that is drawn or edited will have its area computed automatically and stored in its *Area* property. The same holds true for conduit lengths. The current status of the *Auto-Length* option is shown in the status bar at the bottom of SWMM's main window. It can be switched on/off by clicking the drop-down arrow next to the **Auto-Length** display.

Another feature of *Auto-Length* is its ability to re-compute the areas of all subcatchments and the lengths of all conduits in a project at once. This capability proves useful when a user changes the map scaling, switches between using US and metric units, or has done a lot of map editing with *Auto-Length* turned off. To implement this feature, make sure *Auto-Length* is turned on and select **View | Dimensions** from the main menu. In the *Map Dimensions* dialog that appears, check on the option to re-compute all lengths and areas. There is no need to change any of the other settings in the dialog unless one so desires. Once **OK** is clicked, all of the subcatchments and conduits on the map will have their areas and lengths updated.

A second measurement tool is the *Ruler* tool. It is used to measure the distance along a polyline as well as compute an area if the polyline is closed to form a polygon. To activate the *Ruler* tool select the  button on the *Map Toolbar*. Then click the mouse on the first point to begin measuring from, and continue to click at intermediate points along the path being measured. To complete the path and determine its length, right click (or press **Enter**) at the terminating point. To measure the perimeter and area of a polygon, make the terminating point the same as the initial point of the path.





## 1.4 Model Results – Undeveloped Site

### Analysis Options

Table 1-3 shows the analysis options used to run the model. Three runs of the model were made, one for each design storm event. To analyze a particular storm event one only has to change the *Series Name* property of the rain gage to the rainfall time series of interest. The total discharges to the outfall for each storm were then plotted on one graph for comparison.

**Table 1-3. Analysis options**

Option	Value	Explanation
Flow units	CFS	U.S customary units used throughout
Routing Method	Kinematic wave	Must specify a routing method, but it is not used in the overland flow computations in this example
Start analysis time and date	01/01/07 - 00:00	Not important for single event simulation
Start reporting time and date	01/01/07 - 00:00	Start reporting results immediately
End analysis time and date	01/01/07 - 12:00	12 h of simulation (storm duration is 2 h)
Reporting time step	1 minute	Good level of detail in results for a short simulation
Runoff dry-weather time step	1 hour	Not important for single event simulation
Runoff wet-weather time step	1 minute	Should be less than the rainfall interval
Routing time step	1 minute	Should not exceed the reporting time step

### Simulation Results

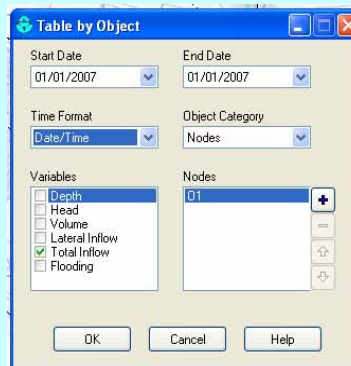
Figure 1-7 shows the outlet hydrographs obtained for each of the design storms. It was created by following the procedure outlined in the sidebar titled *Exporting Data from SWMM*. Note the significant increase in the peak discharge as the return period increases and how sensitive it is to the rainfall intensity. (The 2-yr storm hyetograph is plotted for comparison.) The rate at which the discharge volume increases is much greater than the rate at which the rainfall volume changes for different return periods. This is because the soil becomes more saturated during larger storms resulting in more of the rainfall becoming runoff.

Table 1-4 compares the peak rainfall intensity, total rainfall, total runoff volume, runoff coefficient, peak runoff discharge and total infiltrated volume for each design storm. Additionally, the last column provides the percent of the rainfall that is infiltrated in each case. These values came directly from the Subcatchment Runoff Summary table that appears in the Status Report of a SWMM run.

## Exporting Data from SWMM

While data can be plotted in the SWMM interface for one run's results, it is not possible to plot the results of an additional run with those of an older run. To do this the results from each run must be exported to a spreadsheet or other plotting software. Data from both plots and tables can be easily exported in SWMM. The steps below explain how this is done for the runoff seen at the watershed outlet for the 2- and 10-yr storms.

1. Run the 2-yr simulation. Click on the watershed outlet *O1* and then select the Table icon (📊) on the *Standard Toolbar*. For this example, choose “**by Object...**”
2. In the *Table by Object* dialog box select **Date/Time** for the *Time Format* and for the variable select **Total Inflow**. Click **Ok**.



3. A table of runoff flow rates and their times will appear. Select **Edit | Select All** and then **Edit | Copy to...** from the main menu. In the *Copy Table* dialog box you have the choice of copying the data to the Clipboard and pasting it from there directly into the spreadsheet or saving it to a text file. Choose **Clipboard** for this example. Open a spreadsheet document, and paste the Clipboard contents into it.
4. Return to SWMM and run the 10-yr storm and repeat steps 1 to 3. Paste the data into the same spreadsheet, next to the data from the 2-yr storm.
5. Now use the Scatter plot and the formatting tools of your spreadsheet to plot the *Total Inflow* data for both runs on the same graph.

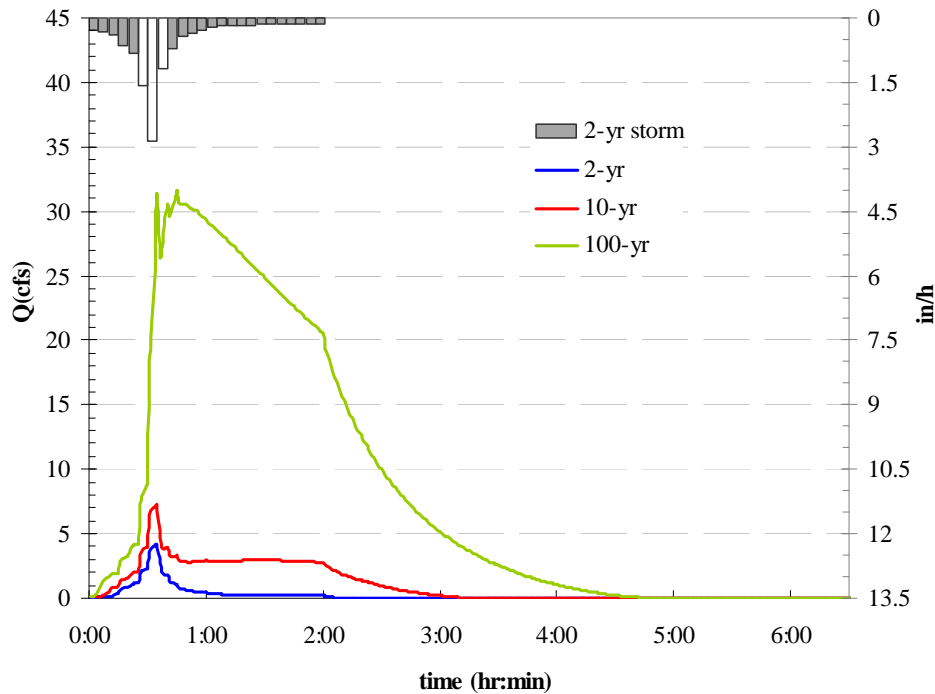


Figure 1-7. Runoff hydrographs (flow Q versus time) for the undeveloped site

Table 1-4. Summary of results for the undeveloped site

Design Storm	Peak Rainfall (in./h)	Total Rainfall (in.)	Runoff Volume (in.)	Runoff Coeff. (%)	Peak Runoff (cfs)	Total Infiltration (in.)	% of Rainfall Infiltrated
2-yr	2.85	0.978	0.047	4.8	4.14	0.93	95.1
10-yr	4.87	1.711	0.22	13.1	7.34	1.48	86.5
100-yr	9.95	3.669	1.87	50.8	31.6	1.80	49.1

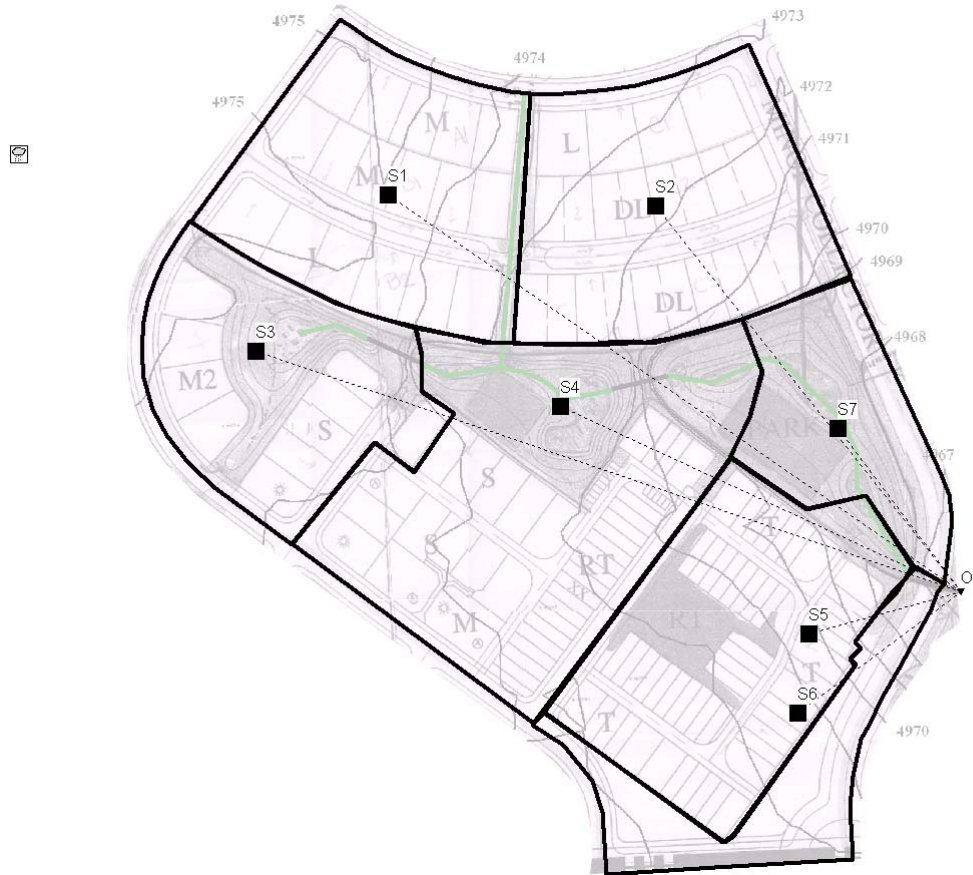
## 1.5 Model Setup – Developed Site

The increase in impervious surface and reduction of overland flow length are the main factors affecting the hydrologic response of a catchment when it becomes urbanized. The reduction in infiltrative surface creates additional surface runoff as well as higher and faster peak discharges. In this section, the runoff hydrology of our example site will be modeled in its post-development condition. The SWMM input data for this model is named **Example1-Post.inp**. Again, the focus will be on just the rainfall-runoff transformation and overland flow processes. Routing through channelized elements will be covered in Example 2.

### Catchment Discretization

In the urbanized catchment there are channelized elements (gutters and swales) that conduct runoff to the site's outlet. The partitioning of the study area into individual

subcatchments depends not only on the spatial variability in land features but also on the location of the channelized elements. Inspection of the developed site plan for the example study area (see Figure 1-2) suggests that a total of seven subcatchments would be sufficient to represent both the spatial differences in planned land uses and the location of channelized elements within the site. The subcatchment boundaries were determined by aggregating together sub-areas whose potential overland flow paths share a common direction and drain to the same collection channel. The resulting discretization is shown in Figure 1-8.



**Figure 1-8. Discretization of the developed site into subcatchments**

Figure 1-8 shows all of the subcatchments discharging their overland flow directly into the watershed's outlet node, *O1*. In reality, the discharge outlet of each subcatchment should be the point where its runoff enters the channelized drainage system. However, since this example **does not** consider routing through any channelized elements in the watershed (Example 2 covers this issue) it is acceptable to use the study area's outlet node (*O1*) as a common outlet for all of the subcatchments. The elevation of this point is **4962** ft, which corresponds to the bottom elevation of a planned culvert under the street.

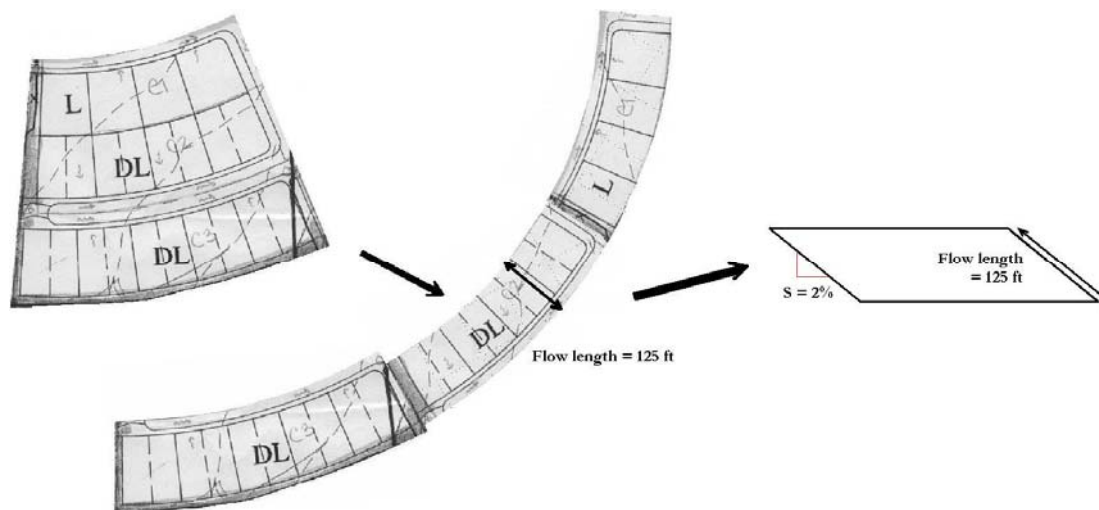
## Geometric Parameters

Table 1-5 lists the area, flow path length, width, slope and imperviousness of each subcatchment. The areas were computed using SWMM's *Auto-Length* tool as the outline of each subcatchment was traced on the scaled backdrop image. (See the sidebar "[Measuring Tools Available in SWMM](#)" for more information.)

**Table 1-5. Geometric properties of the subcatchments in the developed site**

Subcatchment	Area (ac)	Flow Length (ft)	Width (ft)	Slope (%)	Percent Impervious
1	4.55	125	1587	2.0	56.8
2	4.74	125	1653	2.0	63.0
3	3.74	112	1456	3.1	39.5
4	6.79	127	2331	3.1	49.9
5	4.79	125	1670	2.0	87.7
6	1.98	125	690	2.0	95.0
7	2.33	112	907	3.1	0.0

Figure 1-9 illustrates how the overland flow path length was estimated for subcatchment S2 which consists entirely of residential lots. This subcatchment can be represented as a rectangular area with an overland flow length equal to the distance from the back of a typical lot to the middle of the street (125 ft in this case). SWMM's width parameter can then be computed as the area ( $4.74 \text{ ac} = 206,474.4 \text{ ft}^2$ ) divided by the overland flow length, which results in a value of **1650 ft**.



**Figure 1-9. Definition of overland flow length and slope for subcatchment S2**

In contrast to  $S_2$ , subcatchments  $S_3$  and  $S_4$  contain both residential lots and grass-covered areas. Their overland flow lengths are computed as an area-weighted average of the flow lengths across each type of area as shown in Figure 1-10. Their widths are then found by dividing their areas by their overland flow lengths.

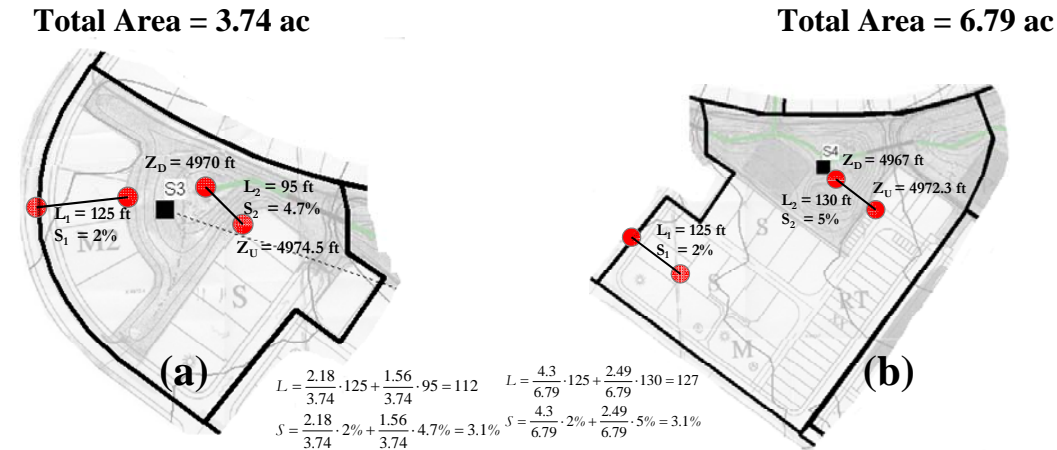


Figure 1-10. Width and slope computation for subcatchments  $S_3$  (a) and  $S_4$  (b)

Slopes characterizing overland flow in mostly urbanized subcatchments will be the lot slope, which is usually about 2%. By way of illustration, Figure 1-10 shows how the slopes of subcatchments  $S_3$  and  $S_4$  correspond to the area-weighted average of the slope of the overland flow paths over both the residential lots and the grass-covered areas.

## Imperviousness

The imperviousness parameter in SWMM is the *effective* or *directly connected* impervious area, which is typically less than the total imperviousness. The effective impervious area is the impervious area that drains directly to the stormwater conveyance system, e.g. a gutter, pipe or swale. Ideally, the imperviousness should be measured directly in the field or from orthophotographs by determining the percent of land area devoted to roofs, streets, parking lots, driveways, etc. When these observations are not available, it is necessary to use other methods. A conservative approximation that tends to overestimate runoff discharges is to use runoff coefficients as the imperviousness value. A runoff coefficient is an empirical-constant value that represents the percentage of rainfall that becomes runoff. Using the runoff coefficient to represent the percent imperviousness of a subcatchment results in a higher estimate of impervious area because the value is calculated from the runoff of both the impervious and pervious areas of the subcatchment. For purely illustrative purposes, runoff coefficients will be used in this example to estimate the imperviousness for each subcatchment within the developed watershed. The steps involved are as follows:

1. Identify all of the major land uses that exist within the subcatchment.
2. Compute the area  $A_j$  devoted to each land use  $j$  in the subcatchment.

3. Assign a runoff coefficient  $C_j$  to each land use category  $j$ . Typical values are available in drainage criteria and basic literature (see for example UDFCD, 2001; Akan, 2003). Pervious areas are assumed to have a runoff coefficient of 0.
4. Compute the imperviousness  $I$  as the area weighted average of the runoff coefficients for all of the land uses in the subcatchment,  $I = (\sum C_j A_j)/A$ , where  $A$  is the total area of the subcatchment.

When this approach is applied to the current example the results listed in Tables 1-6 and 1-7 are obtained. Table 1-6 displays the various land-use categories that appear in the developed site along with their runoff coefficients. The latter were obtained from the City of Fort Collins Storm Drainage Design Criteria and Construction Standards (City of Fort Collins, (city of Fort Collins, 1984 and 1997). Table 1-7 lists the amount of area devoted to each land use within the site's subcatchments. These areas were used to compute a weighted-average runoff coefficient that is used as a surrogate for the imperviousness of the given subcatchment.

**Table 1-6. Land use categories in the developed site**

<b>Id</b>	<b>Land Use</b>	<b>Runoff coefficient (C)</b>
M	Medium density	0.65
L	Low density	0.45
DL	Duplex	0.70
M2	Medium density	0.65
S	Apartment, high density	0.70
RT	Commercial	0.95
T	Commercial	0.95
P	Natural (park)	0

**Table 1-7. Land use coverage (ac) and imperviousness for subcatchments in the developed site**

<b>Sub-catchment</b>	<b>Total Area (ac)</b>	<b>Area M</b>	<b>Area L</b>	<b>Area DL</b>	<b>Area M2</b>	<b>Area S</b>	<b>Area RT</b>	<b>Area T</b>	<b>Area P</b>	<b>Imperviousness (%)</b>
S1	4.55	2.68	1.87	0	0	0	0	0	0	56.8
S2	4.74	0	1.32	3.42	0	0	0	0	0	63
S3	3.74	0	0	0	1	1.18	0	0	1.56	39.5
S4	6.79	0.61	0	0	0	2.05	1.64	0	2.49	49.9
S5	4.79	0	0	0	0	0	0.7	3.72	0.37	87.7
S6	1.98	0	0	0	0	0	0	1.98	0	95
S7	2.33	0	0	0	0	0	0	0	2.33	0



## Remaining Parameters

The remaining subcatchment properties for the developed site (roughness coefficients, depression storages, and infiltration parameters) are kept the same as they were for the undeveloped condition. Likewise, the same analysis options were used to run the simulations. Refer to Tables 1-1 and 1-3 for a listing of the parameter values used in the undeveloped condition.

## 1.6 Model Results – Developed Site

### Outlet Hydrographs

Figure 1-11 shows the outlet hydrographs (the Total Inflow to node *OI*) obtained for each of the design storms under post-development conditions in the study site. As with the pre-development hydrographs, the peak runoff flow occurs close to when the peak rainfall does and there is a significant increase in peak discharge as the return period increases. Unlike the pre-development case, the post-development's hydrographs show a more rapid decline once the rainfall ceases. This behavior can be attributed to the much larger amount of imperviousness under the post-development condition (57%) as compared to pre-development (5%). Table 1-8 summarizes the results obtained for each design storm in the same fashion that Table 1-4 did for the pre-development condition.

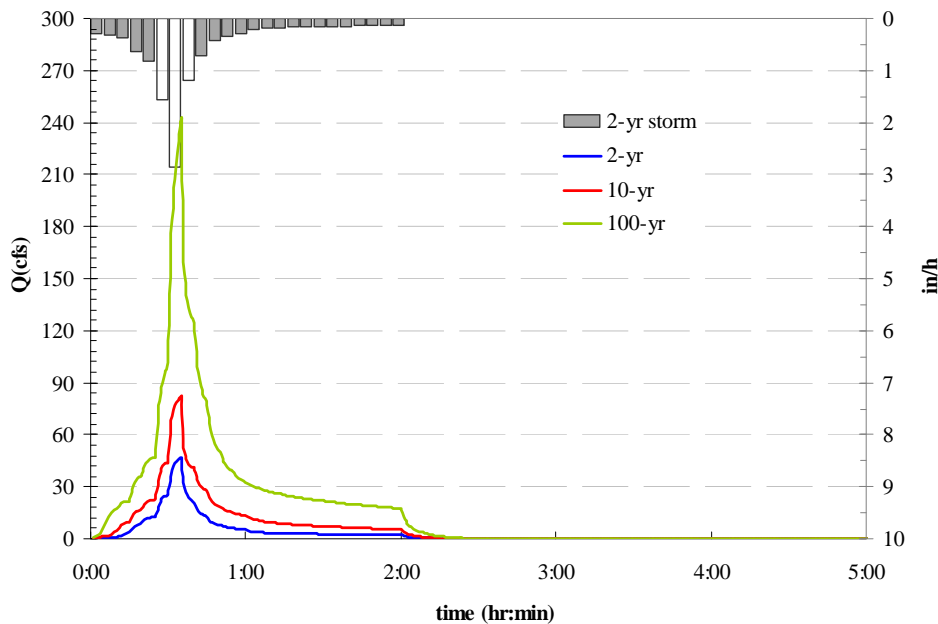


Figure 1-11. Runoff hydrographs (flow  $Q$  versus time) for the developed site



**Table 1-8. Summary of results for post-development conditions**

Design Storm	Peak Rainfall (in./h)	Total Rainfall (in.)	Runoff Volume (in.)	Runoff Coeff. (%)	Peak Runoff (cfs)	Total Infiltration (in.)	% of Rainfall Infiltrated
2-yr	2.85	0.978	0.53	54.5	46.7	0.42	42.9
10-yr	4.87	1.711	1.11	64.7	82.6	0.58	33.8
100-yr	9.95	3.669	3.04	82.7	241	0.61	16.6

## Pre- and Post-Development Comparison

Table 1-9 compares total runoff volumes, runoff coefficients, and peak discharges computed for both the pre- and post-development conditions. For larger storm events, where infiltration plays a minor role in the runoff generation, the responses become more similar between the two cases. Total runoff volume under post-development conditions is approximately 10, 5, and 2 times greater than under pre-development conditions for the 2-yr, 10-yr, and 100-yr storms, respectively. Peak flows are about 10 times greater for both the 2-yr and 10-yr storms but only 7 times greater for the 100-yr event.

**Table 1-9. Comparison of runoff for pre- and post-development conditions**

Design Storm	Total Rainfall (in.)	Runoff Volume (in.)		Runoff Coeff. (%)		Peak Runoff (cfs)	
		Pre	Post	Pre	Post	Pre	Post
2-yr	0.978	0.047	0.53	4.8	54.50	4.14	46.74
10-yr	1.711	0.24	1.11	13.1	64.70	7.34	82.64
100-yr	3.669	1.87	3.04	50.8	82.70	31.6	240.95

## 1.7 Summary

This example used SWMM to estimate the runoff response to different rain events for a 29 ac development that will be built in a natural area. Comparisons were made between the runoff peaks and total volume for each event for both pre- and post-developed conditions. The key points illustrated in this example were:

1. Building a SWMM model for computing runoff requires that a study area be properly partitioned into a collection of smaller subcatchment areas. These can be determined by examining the potential pathways that runoff can travel as overland flow and the location of the collection channels, both natural and constructed, that serve to intercept this runoff.
2. Initial estimates of most subcatchment parameters can be based on published values that are tabulated for various soil types and land uses. The primary exception to this is the width parameter which should be based on the length of the overland-flow path that the runoff travels.

3. Path lengths for true overland flow should be limited to about 500 ft or to the distance at which a collection channel/pipe is reached if it is less than 500 ft.
4. Urban development can create large increases in the imperviousness, peak-runoff flow rate, and total-runoff volume.

The next case study, Example 2, will further refine the model built in this example by adding a stormwater collection system to it and routing the runoff flows through this system.

## Example 2. Surface Drainage Hydraulics

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Example 1 showed how to construct a hydrologic model of an urban catchment that compared stormwater runoff under both pre- and post-development conditions. Hydraulic routing was not considered. This example will demonstrate how SWMM's hydraulic elements and flow routing methods are used to model a surface drainage system. A conveyance network will be added to the post-development model built in Example 1 and be sized to pass the 2 hour synthetic storm events with return periods of 2-, 10-, and 100-years. For simplicity, open channels (e.g. swales or gutters) will be used to convey flow. The simple routing network developed in this example will be built upon further in Example 7 where additional open channels and below-ground pipes will be added that are designed according to typical drainage design criteria.

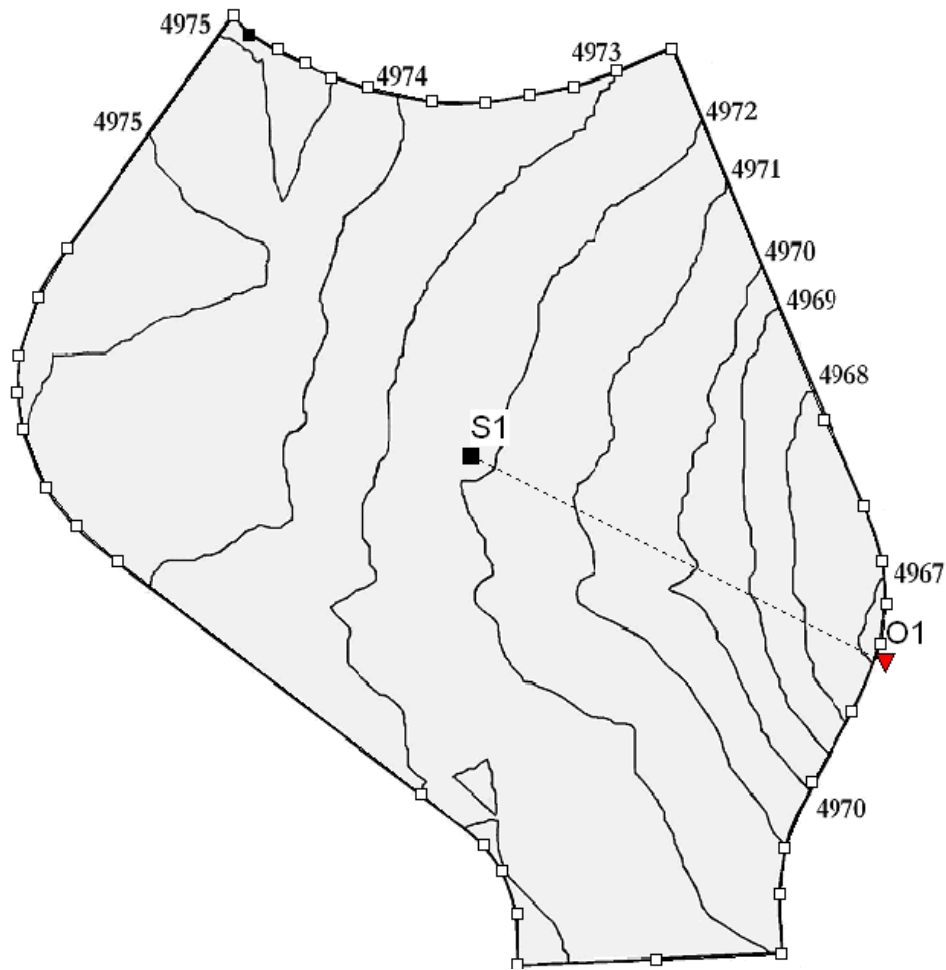
### 2.1 Problem Statement

Figures 2-1 and 2-2 show the SWMM model layouts of the pre- and post-development study area developed in Example 1. In Figure 2-1 the pre-development site was represented by a single subcatchment whose width parameter was determined by assuming a maximum overland flow length of 500 ft, as recommended for undeveloped areas. For the developed case (Figure 2-2), the post-development site was discretized into seven subcatchments, the subcatchment widths were computed using an area-weighted average of the flow lengths across each type of area, and all subcatchments discharged their overland flow directly to the site's outlet, node *O1* (see Section 1.5 of Example 1).

The objective of this example is to add a simple surface drainage system to the post-development site. A system of gutters, grass swales, and culverts will be designed and sized to convey the 100-yr storm. Runoff from three design storms (the 2-, 10- and 100-yr storms) will be routed through this system using the three alternative hydraulic routing methods available in SWMM. The resulting outflow hydrographs at the site's outlet will be compared to those generated in Example 1 where no hydraulic routing was used.

### 2.2 System Representation

SWMM models a conveyance network as a series of nodes connected by links (Figure 2-3). Links control the rate of flow from one node to the next and are typically conduits (e.g. open channels or pipes) but can also be orifices, weirs or pumps. Nodes define the elevation of the drainage system and the time-varying hydraulic head applied at the end of each link it connects. The flow conveyed through the links and nodes of the model is ultimately discharged to a final node called the outfall. Outfalls can be subjected to alternative hydraulic boundary conditions (e.g. free discharge, fixed water surface, time varying water surface, etc.) when modeled with Dynamic Wave. The properties of these drainage system elements are explained in detail in the sidebar "[\*Hydraulic Elements in SWMM\*](#)".



**Figure 2-1. Pre-development site**

Hydraulic routing is the process of combining all inflows that enter the upstream end of each conduit in a conveyance network and transporting these flows to the downstream end over each instance of time. The resulting flows are affected by such factors as conduit storage, backwater, and pipe surcharging. SWMM can perform hydraulic routing by three different methods: *Steady Flow*, *Kinematic Wave* and *Dynamic Wave*. These three methods are summarized below.

- **Steady Flow**

*Steady Flow* routing is an instantaneous translation of a hydrograph from the upstream end of a conduit to the downstream end with no time delay or change in shape due to conduit storage. *Steady Flow* routing will simply sum the surface runoff from all subcatchments upstream of the selected node through time.

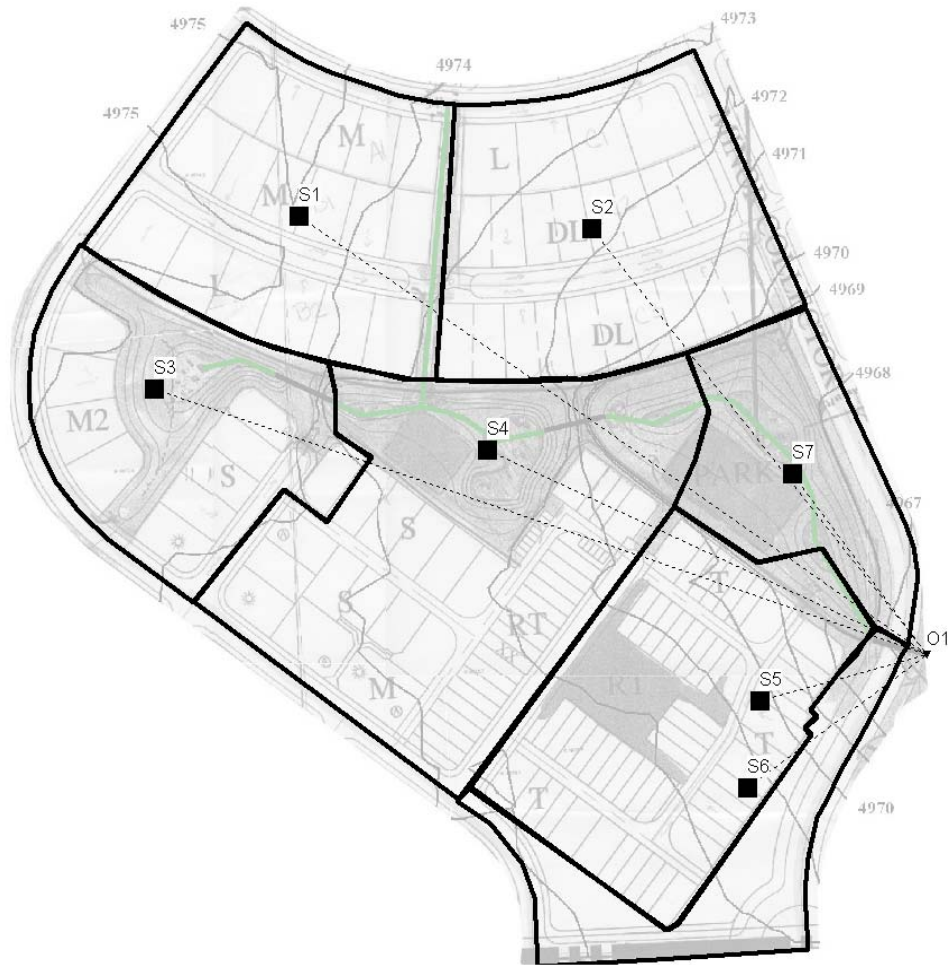


Figure 2-2. Post-development site without conveyance system

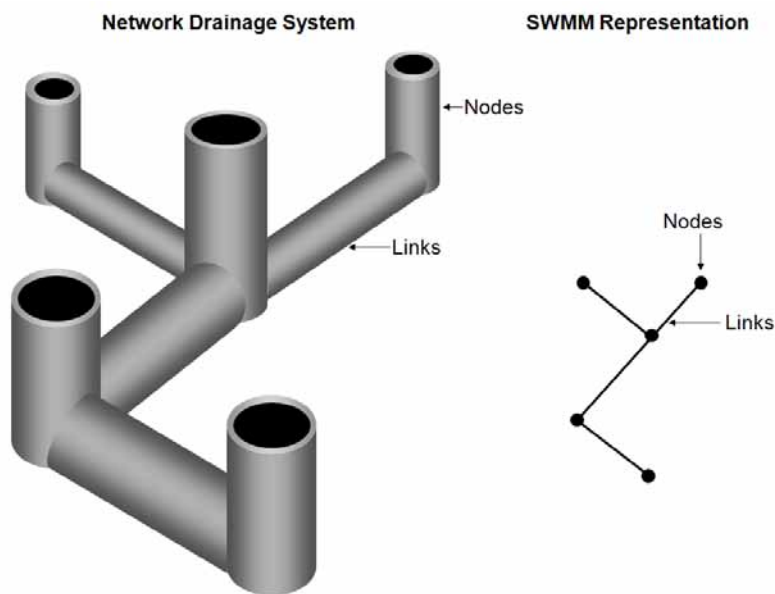











Figure 2-3. Links and Nodes

## Hydraulic Elements in SWMM

All hydraulic elements modeled in SWMM are classified as either nodes or links. The hierarchy of these elements is shown below along with both the required and optional properties that characterize each element's hydraulic behavior.

Type of Hydraulic Element	Category of Element	Examples or Types	Required Properties	Optional Properties
Nodes	Junction 	-Manholes -Points of change in conduit slope or cross section	-Invert elevation -Maximum depth -Initial depth	-Surcharge depth -Treatment -Inflows -Ponded area
	Divider 	-Cutoff -Tabular -Weir -Overflow	-Invert elevation -Diversion link -Type -Maximum depth -Initial depth	-Ponded area -Surcharge depth -Treatment -Inflows
	Storage Unit 	-Reservoirs -Peak shaving detention ponds -BMPs	-Invert elevation -Storage curve -Maximum depth -Initial depth	-Treatment -Inflows -Evaporation factor
	Outfall 	-Free -Normal -Fixed -Tidal -Time series	-Invert elevation -Type	-Inflows -Treatment -Tide gate
Links	Conduit 	-Natural channels -Closed conduits -Open Channels	-Inlet node -Outlet node -Shape and section geometry -Length -Roughness -Inlet offset -Outlet offset	-Initial flow -Maximum flow -Entry loss coefficient -Exit loss coefficient -Average loss coefficient
	Pump 	-Off-line -In-line incremental -Variable speed in-line	-Inlet node -Outlet node -Pump curve	-Initial status -Startup depth -Shutoff depth
	Orifice 	-Circular orifice -Rectangular orifice	-Inlet node -Outlet node -Type -Shape and geometry -Inlet offset -Discharge coefficient	-Flap gate -Time to open/close
	Weir 	-Transverse -Side-flow -V-notch -Trapezoidal	-Inlet node -Outlet node -Type -Geometry -Inlet offset -Discharge coefficient	-Flap gate -End contractions -End coefficient
	Outlet 	-Used to model special head-discharge relationships	-Inlet node -Outlet node -Inlet offset -Rating curve	-Flap gate

- **Kinematic Wave**

*Kinematic Wave* uses the normal flow assumption for routing flows through the conveyance system. In *Kinematic Wave* routing, the slope of the hydraulic grade line is as the same as the channel slope. *Kinematic Wave* routing is most applicable to the upstream, dendritic portions of drainage systems where there are no flow restrictions that might cause significant backwater or surcharging. It can be used to approximate flows in non-dendritic systems (i.e., those that have more than one outflow conduit connected to a node) only if “flow divider” nodes are used.

- **Dynamic Wave**

*Dynamic Wave* routing is the most powerful of the flow routing methods because it solves the complete one-dimensional Saint Venant equations of flow for the entire conveyance network. This method can simulate all gradually-varied flow conditions observed in urban drainage systems such as backwater, surcharged flow and flooding. *Dynamic Wave* can simulate looped conduit systems and junctions with more than one link connected downstream (bifurcated systems). The ability to simulate bifurcated systems allows one to model pipes and gutters in parallel; this more advanced level of modeling will be described in Example 7.

## **2.3 Model Setup**

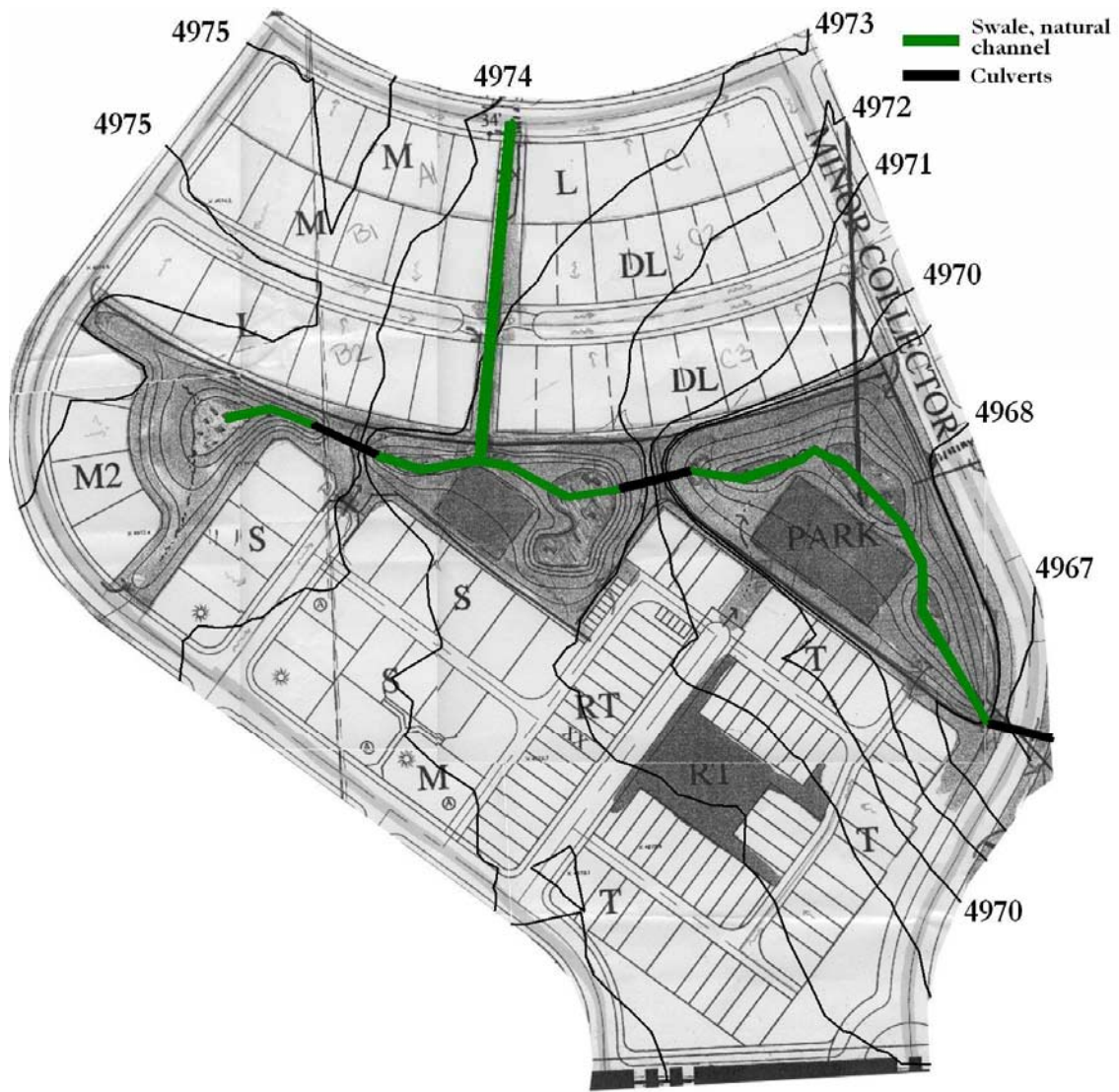
### **System Layout**

Figure 2-4 shows the layout of the runoff conveyance system that will be added to the developed site. It consists of 7 grass swales, 3 culverts, and one street gutter. The objective in this example is to estimate the discharges at the outlet of the site and not design all of the elements within the entire drainage system. For this reason, only the main surface conduits that route runoff to the outlet in the aggregated model will be considered. These will purposely be oversized to ensure that all the generated runoff is conveyed to the outlet and that no flooding occurs within the site (see Example 7 for an analysis of surcharged pipes and flooded junctions). The starting point for building this model is the input file Example1-Post.inp that was created in Example 1.

In Example 1, the subcatchment widths were set to properly represent the overland flow process. All the subcatchments were directly connected to the outlet of the study area; flows through channels were not modeled. In this example, the subcatchment properties will remain the same as defined previously, but conduits representing the channelized flow throughout the site will be added into the model.

The definition of the conveyance system begins by specifying the location of its nodes (or junctions). A node is required wherever runoff enters the conveyance system, whenever two or more channels connect and where the channel slope or cross section changes significantly. They are also required at locations with weirs, orifices, pumps, storage, etc. (see Example 3 where orifices and weirs are used as outlets to a storage unit). The locations of the nodes for this example are shown in Figure 2-5. They are labeled *J1* through *J11*. The invert elevation of each node (i.e., the elevation at the bottom of the lowest connecting channel) is shown in Table 2-1.





**Figure 2-4. Post-development site with runoff conveyance added**





The definition of the conveyance system continues by adding the feeder channels *C1*, *C2* and *C6* that convey runoff into the main drainage way that runs through the undeveloped park area of the site. Conduit *C1* is a grass swale that drains subcatchment *S1*'s runoff to the watershed's main drainage way; Conduit *C2* is a gutter that carries subcatchment *S2*'s runoff to the upstream end of the culvert (*C11*) that discharges to the site's outlet (*O1*); conduit *C6* carries runoff from subcatchment *S4* into culvert *C7*. At this point, the elevations of the bottom bed of these channels correspond to the invert elevations of their respective upstream and downstream junctions. Their lengths are automatically determined by drawing them with Auto-Length turned on. SWMM uses this information to compute the slope of each channel.

Finally, the remaining conduits *C3* through *C11* that comprise the main drainage way through the park to the outlet need to be defined. As before, they connect to their end nodes with no vertical offset and the Auto-Length tool is used to compute their lengths. Subcatchments *S3* through *S7* drain to different locations of the main drainage channel. *S3* drains to the culvert (*C3*) at the beginning of the main drainage channel, *S4* drains to the swale *C6* that connects to the second culvert (*C7*) on the main drainage channel, *S7* and *S5* drain to the main drainage channel at *J10*, and *S6* drains directly to the last culvert (*C11*) on the main drainage channel. Table 2-2 summarizes the outlet junction and conduit associated with each subcatchment.

**Table 2-2. Subcatchment outlets**

Subcatchment	Outlet Junction	Outlet Conduit
S1	J1	C1(Swale)
S2	J2	C2 (Gutter)
S3	J3	C3 (Culvert)
S4	J7	C6 (Swale)
S5	J10	C10 (Swale)
S6	J11	C11 (Culvert)
S7	J10	C10 (Swale)




Note that the conveyance system modeled in this example (Figure 2-5) ignores the storage and transport provided by the street gutters within each subcatchment. In some applications, however, these conveyance elements can play a significant role and should be represented, perhaps by adding a channel within each subcatchment that represents the aggregated effects of routing through all of its street segments. To keep the example simple, this level of detail is not included and only the major drainage channels within the site are considered.

## System Properties

Properties can now be assigned to the conduits and junctions that have been defined. Table 2-3 shows the cross-sectional shapes of the three conduit types used in this example. The swale side slopes (*Z1* and *Z2* (horizontal:vertical)), roughness coefficient (*n*), bottom width (*b*) and height (*h*) of the swale section are as recommended by the

UDFCD Manual (2001). The gutter cross-slopes ( $Z_1$  and  $Z_2$ ), roughness coefficient ( $n$ ), bottom width ( $b$ ) and height ( $h$ ) are based on typical design practice. The culvert diameters will be sized as described in the next section to convey runoff from the 100-year storm.

**Table 2-3. Characteristics of the conduits used in the model**

Type of Shape	Cross Section	Representing	D (ft)	$Z_1$	$Z_2$	b (ft)	h (ft)	n
Trapezoidal		Swales	-	5	5	5	3	0.05
Trapezoidal		Gutters	-	0.0001 <sup>1</sup>	25	0	1	0.016
Circular		Culverts	4.75 <sup>2</sup>	-	-	-	-	0.016

<sup>1</sup> SWMM cannot accept a slope of zero

<sup>2</sup> This is the initial diameter of the culverts, not the final value

Table 2-4 shows the SWMM properties assigned to each conduit. Conduit lengths were computed using the *Auto-Length* option as described in the previous section. The inlet and outlet offset of all the conduits, with one exception, were set to zero which means that the bottom elevation of each conduit coincides with the elevation of its inlet and outlet junctions. The exception is conduit C2 (the gutter), whose outlet offset of 4 ft represents the difference in elevation between the bottom of the gutter and the channel bed in the park. The diameters of the three circular culverts will be determined in the next section.

**Table 2-4. Conduit properties**

Conduit ID	Type of Conduit	Inlet Node	Outlet Node	Length (ft)	h (ft) or D (ft)	b (ft)	$Z_1$	$Z_2$
C1	Swale	J1	J5	185	3	5	5	5
C2	Gutter	J2	J11	526	1	0	0.0001	25
C3	Culvert	J3	J4	109	<b>TBD</b>	-	-	-
C4	Swale	J4	J5	133	3	5	5	5
C5	Swale	J5	J6	207	3	5	5	5
C6	Swale	J7	J6	140	3	5	5	5
C7	Culvert	J6	J8	95	<b>TBD</b>	-	-	-
C8	Swale	J8	J9	166	3	5	5	5
C9	Swale	J9	J10	320	3	5	5	5
C10	Swale	J10	J11	145	3	5	5	5
C11	Culvert	J11	O1	89	<b>TBD</b>	-	-	-

**TBD = To Be Determined**

Because the conduits are all surface channels and not buried pipes, it is sufficient to set the maximum depth of all the junctions as zero. This will cause SWMM to automatically set the depth of each junction as the distance from the junction's invert to the top of the highest conduit connected to it. Thus, junction flooding (the only flooding allowed by SWMM) will occur when the channel capacity is exceeded. Finally, the outfall node *O1* is defined as a **free outfall** (see the sidebar “*Hydraulic Elements in SWMM*”) with an elevation of **4962** ft. The resulting SWMM input file has been named **Example2-Post.inp**.

## 2.4 Model Results

### Culvert Sizing

Before SWMM's alternative routing methods are compared, the diameters of the three culverts in the conveyance system must be determined. This is done by finding the smallest available size for each culvert from those listed in Table 2-5 that will convey the runoff from the 100-year, 2-hr design storm without any flooding. This process involves the following steps:

1. Start with each culvert at the largest available diameter.
2. Make a series of SWMM runs, reducing the size of conduit *C3* until flooding occurs. Set the size of *C3* to the next larger diameter.
3. Repeat this process for conduit *C7* and then for *C11*.

Note that one proceeds systematically from upstream to downstream, making sure that each culvert in turn is just big enough to handle the flow generated upstream of it. This procedure is appropriate because there is no flooding under the baseline condition (with the culverts at their maximum possible size). The approach should not necessarily be applied when pipe diameters have to be enlarged (or when there is flooding under the baseline condition). It is very common to find design situations in which changes downstream have significant effects upstream, so that a minor change in the diameter of a pipe located downstream may solve flooding problems upstream.

**Table 2-5. Available culvert sizes**

Diameter (ft)	Diameter (in)	Diameter (ft)	Diameter (in)	Diameter (ft)	Diameter (in)	Diameter (ft)	Diameter (in)
1	12	2	24	3	36	4	48
1.25	15	2.25	27	3.25	39	4.25	51
1.5	18	2.5	30	3.5	42	4.5	54
1.75	21	2.75	33	3.75	45	4.75	57

These culvert-sizing runs are made using the **Example2-Post.inp** file with the routing method set to **KW** (*Kinematic Wave*), the Rain Gage's time series set to **100-yr**, and the following set of time steps: **1** minute for reporting, **1** h for dry weather, **1** minute for wet weather and **15** s for routing. Note that the routing time step is somewhat stringent for the drainage system being modeled and the routing method used (*KW*). It is

used here, however, because *Dynamic Wave* routing will be used later in this example and it typically requires smaller time steps than *Kinematic Wave* to produce stable results. If just *KW* routing were used for this model, the routing time step could probably be safely set to 1 minute (see sidebar “[A Note About Time Steps](#)”). The presence or absence of flooding is determined by examining the *Node Flooding Summary* section of a run’s *Status Report*.

### **A Note About Time Steps**

SWMM requires that four time steps be specified: runoff time steps for both wet weather and dry weather, a flow routing time step, and a reporting time step. The most common error new users make is to use time steps that are too long. The runoff wet weather time step should not exceed the precipitation recording interval. The flow routing time step should never be larger than the wet weather time step, and in most cases should be 1 to 5 minutes (or less) for *Kinematic Wave* routing and 30 s (or less) for *Dynamic Wave* routing. *Dynamic Wave* routing can also employ a *Variable Time Step* option that automatically lowers the time step during periods when flows change rapidly. High continuity errors typically result when runoff or routing time steps are too large. If the reporting time step is set too high important details in the output results might be missed. Setting the reporting time step equal to the routing time step helps prevent this, but can generate very large output files. Starting with smaller time steps, users can experiment with larger time steps to find one that produces acceptably accurate results most efficiently.

After making these sizing runs with **Example2-Post.inp**, the final diameters selected for the culverts are **2.25** ft for *C3*, **3.5** ft for *C7*, and **4.75** ft for *C11*. Table 2-6 lists the fraction that each conduit is full and the fraction of its full Manning flow that is reached under peak flow conditions for the 100-year event. These values are available from the *Link Flow Summary* table of SWMM’s *Status Report*.

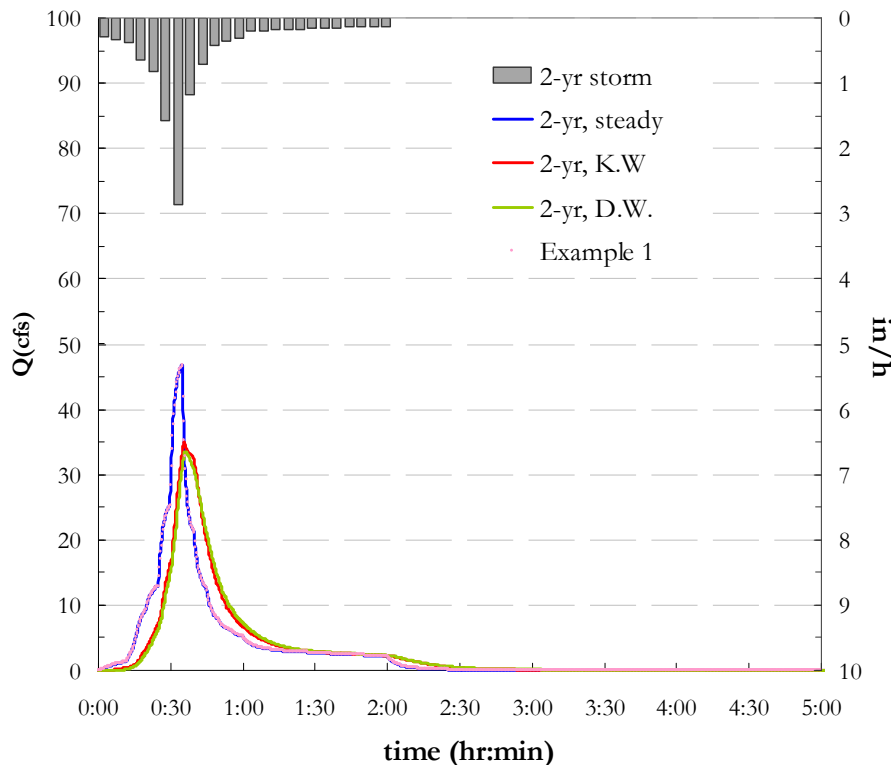
**Table 2-6. Maximum depths and flows for conduits during the 100-yr event**

<b>Conduit ID</b>	<b><u>Maximum Depth</u> Full Depth</b>	<b><u>Maximum Flow</u> Full Flow</b>
C1	0.37	0.11
C2	0.96	0.94
C3	0.70	0.83
C4	0.38	0.12
C5	0.67	0.41
C6	0.44	0.16
C7	0.71	0.85
C8	0.70	0.44
C9	0.88	0.76
C10	0.88	0.74
C11	0.78	0.95

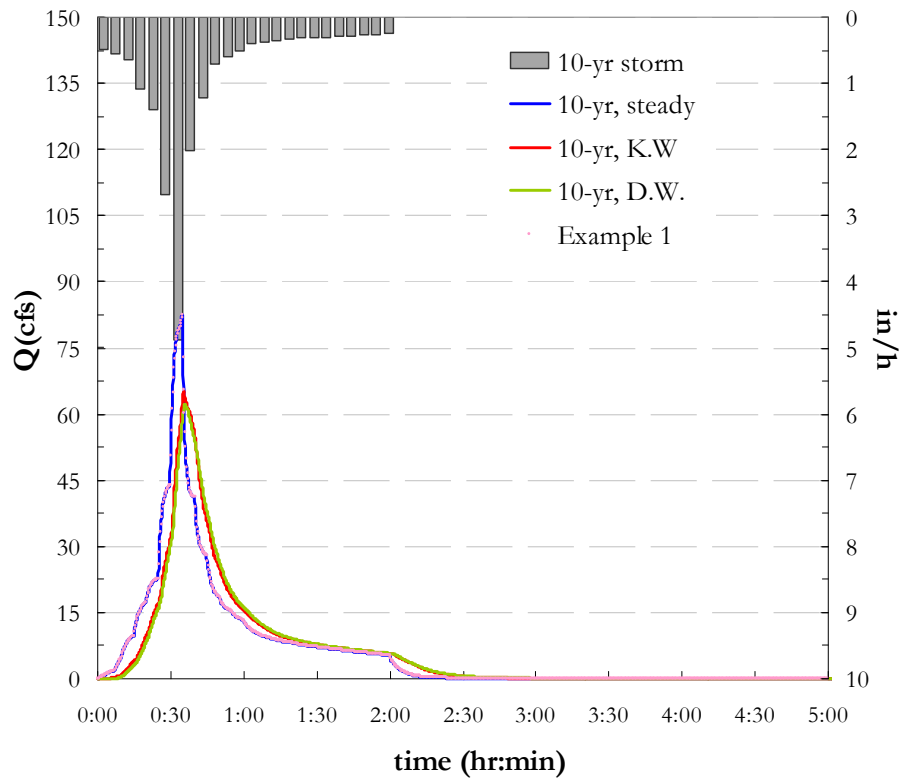
## Comparison of Routing Methods

Having adequately sized the culverts, the model is next run using all three routing methods (*Steady Flow*, *Kinematic Wave*, and *Dynamic Wave*) to obtain the outlet discharges which are then graphed by design storm along with the discharges found from Example 1. Figures 2-6, 2-7 and 2-8 show the outflow hydrographs (Total Inflow to node *O1*) generated for each design storm using all three hydraulic routing methods. As in Example 1, these graphs were created by first exporting the pertinent SWMM results for each run to a spreadsheet and then using the spreadsheet's graphing tools. For *Steady Flow* routing the outlet flows are identical to the flows generated in Example 1 (shown as a dotted line) except for the case of the 100-yr design storm which produced flooding within the system. This is because the discharge that appears at the outlet of each subcatchment in *Steady Flow* routing instantaneously appears at the site outlet where it is added to the discharges of the other subcatchments of the watershed. Thus, *Steady Flow* routing produces results at the site outlet that would have been generated had no channels been simulated in the model.

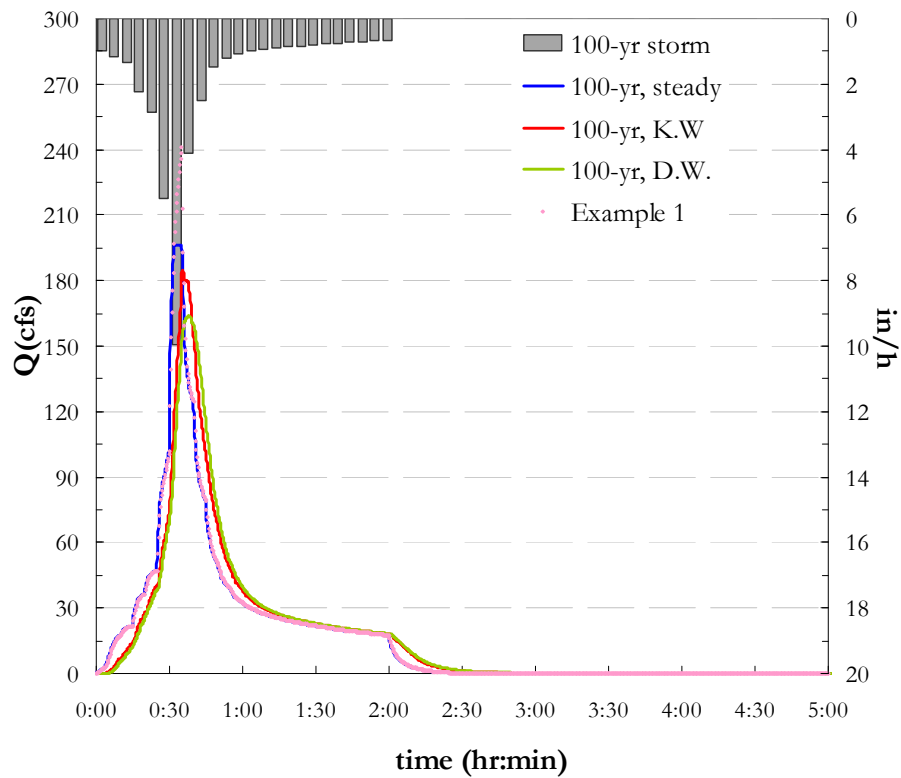
Regarding flooding, the *Steady Flow* method indicates potential for flooding in the system by computing the flow depth in the conduits using Mannings equation; if this depth exceeds the channel capacity, the flow is truncated to the full-flow capacity of the conduit and flooding is reported. Figure 2-8 shows this difference between the outlet discharges generated by the *Steady Flow* routing method and the simulation without routing when flooding does occur.



**Figure 2-6. Post-development outflow hydrographs for 2-yr storm**



**Figure 2-7. Post-development outflow hydrographs for 10-yr storm**



**Figure 2-8. Post-development outflow hydrographs for 100-yr storm**

The other two routing methods, *Kinematic* and *Dynamic Wave*, both produce a time-lag and a reduction in the peak flow, spreading the volume of the outlet hydrograph out over time. These effects are more pronounced under *Dynamic Wave* routing because it accounts for backwater that can increase even further the storage utilized within the conveyance system.

Table 2-7 compares total runoff volumes, runoff coefficients, and peak discharges at the outlet computed for the post-development model without routing (from Example 1) and with *Dynamic Wave* (DW) routing from this example. These values come directly from SWMM's *Status Report*. In terms of runoff volumes and coefficients, the results obtained with routing are identical to those found in Example 1 where no hydraulic routing was considered. The effects of routing are observed in the comparison of peak flows, which decrease when routing is considered. In the case of this example, peaks produced with *Dynamic Wave* routing are 28.7% smaller for the 2-yr storm, 24.8% smaller for the 10-yr storm and 32.4% smaller for the 100-yr storm in comparison to the peaks produced when no routing was considered.

**Table 2-7. Comparison of runoff for post-development conditions with and without routing**

Design Storm	Total Rainfall (in.)	Runoff Volume (in.)		Runoff Coeff. (%)		Peak Runoff (cfs)	
		No Routing <sup>1</sup>	DW	No Routing	DW	No Routing	DW
2-yr	0.98	0.53	0.53	54.5	54.5	46.7	33.4
10-yr	1.71	1.11	1.11	64.7	64.7	82.6	62.2
100-yr	3.67	3.04	3.04	82.7	82.7	241	164.1

<sup>1</sup>Results from Example1-Post.inp

## 2.5 Summary

This example introduced the use of hydraulic routing in SWMM. It demonstrated how a surface runoff collection system is laid out, how to size the elements of this system, and the effect that routing of runoff flows through this system has on the catchment's outlet hydrograph. Comparisons were made between the runoff peaks and total volume for different design storm events using each of SWMM's three routing options as well as with no routing. The key points illustrated in this example are:

1. A runoff collection system can be represented as a network of links and nodes, where the links are conduits (such as grass swales, street gutters, and circular culverts) and the nodes are the points where the conduits join to one another.
2. An iterative process that proceeds from upstream to downstream can be used to determine the minimum conduit size needed to prevent flooding under a particular extreme design event.
3. *Steady Flow* hydraulic routing produces outlet discharges identical to those produced without routing unless there is flooding in the drainage system. The method instantaneously translates hydrographs from the upstream end of a conduit to the downstream end, with no delay or change in shape.



4. *Dynamic Wave* and *Kinematic Wave* routing produce smaller peak runoff discharges than models without routing (Example 1) due to storage and possible backwater effects within the channels. Routing with *Dynamic Wave* resulted in a decrease of 32.4% for the 100-yr peak outlet flow.
5. Except for flooding, the choice of routing method (*Steady Flow*, *Kinematic Wave* or *Dynamic Wave*) does not affect the total volume of runoff that leaves the study area through the outlet.

In the next case study, Example 3, a storage unit will be added to the post-development drainage system developed in this example to mitigate the effects that urbanization has on receiving streams.

## Example 3. Detention Pond Design

This example illustrates how to define, design, and evaluate a detention pond using SWMM. Storage units, orifices and weirs will be used to model a multi-purpose detention pond built to detain a water quality capture volume (WQCV) and control peak post-development release rates to their pre-development levels. The urban catchment studied in Examples 1 and 2 will also be used in this example.

Storage is widely used in urban runoff quantity and quality control, providing both peak flow reduction and suspended solids removal. The design criteria for storage structures have changed over time due to an improved understanding of the effects that urban runoff has on the environment. Facilities must control not only the extreme runoff events to prevent flooding, but also the more common smaller events that produce a “first flush” pollution phenomenon and thereby impact the quality of receiving water bodies.

### 3.1 Problem Statement

In Example 1 a model was built to estimate the pre-development runoff from a 29 acre site. Additional models were constructed to estimate the site’s post-development runoff both without flow routing (Example 1) and with routing through a surface collection system (Example 2). Total site runoff for the 2-, 10- and 100-yr design storms was computed for both the pre- and post-development conditions. Based on the results of these models, it is now required to design a detention pond immediately downstream of the planned urban development to both prevent flooding and protect water quality in a receiving stream. It is required that the pond reduce the peak discharges of the 2-, 10- and 100-yr storms to those of the undeveloped site, and that extended detention be provided for a specific water quality capture volume.

Table 3-1 shows the discharges to be controlled by the pond. The pre-development peaks were determined in Example 1 (Table 1-9 in Section 1.5) and post-development peaks were determined in Example 2 (Table 2-7 in Section 2.4). The SWMM input files that produced these results are **Example1-Pre.inp** and **Example2-Post.inp**, respectively. The 2-, 10- and 100-yr rainfall hyetographs are included in both these input files.

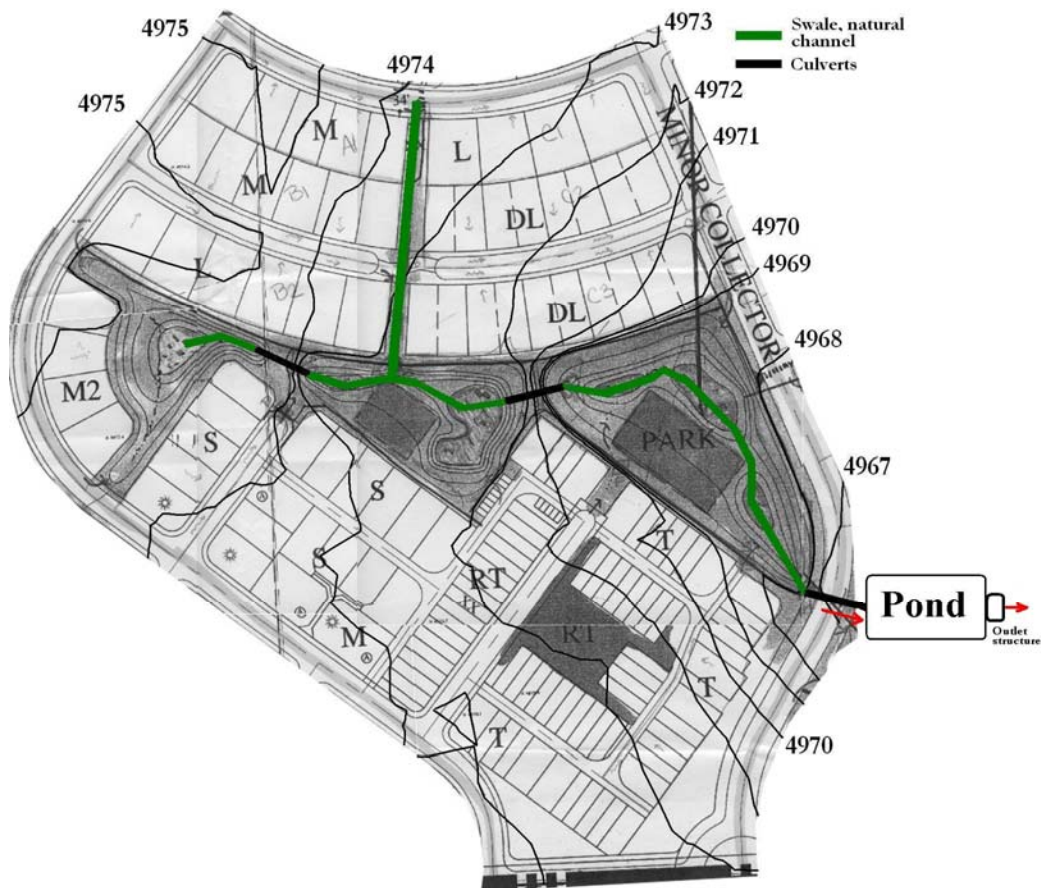
**Table 3-1. Pre- and post-development peak discharges**

Return Period (yr)	Rainfall Depth (in.)	Pre-development Peak Discharge (cfs) <sup>1</sup>	Post-development Peak Discharge (cfs) <sup>2</sup>
2	0.98	4.14	33.5
10	1.71	7.34	62.3
100	3.67	31.6	163.8

<sup>1</sup> From Example1-Pre.inp

<sup>2</sup> From Example2-Post.inp

In addition to controlling the discharges listed, it is required that a Water Quality Capture Volume (WQCV) be controlled. The WQCV is defined as a suitable volume expressed in units of watershed depth that is detained for a long enough period of time to achieve a targeted level of pollutant removal. The required volume and drawdown time vary under different stormwater control policies (Akan and Houghtalen, 2003). In this example, the WQCV must be released over **40** hours, during which a significant portion of particulate pollutants found in urban stormwater are removed. Finally, for safety reasons, a maximum storage depth of **6** ft is considered for the final design. In this example the minor storms ( the WQCV and 2-yr storm) and the major storms (the 10- and 100-yr storms) runoff will be detained in separate sections of the detention pond. Both sections will have the shape of a trapezoidal prism. The location of the pond within the developed study area is shown in Figure 3-1.



**Figure 3-1. Detention pond location for the post-development site**

## 3.2 System Representation

The main elements used to design detention ponds in SWMM are storage units with orifice and weir outlets. These three elements are described below:

### 1. Storage units

Storage units in SWMM are modeled as nodes. They are similar to the conveyance-system junction nodes introduced in Example 2 but have some fundamental differences: storage volume is described by a *Storage Curve*, an *Evaporation Factor* can be specified, and a *Maximum Depth* of storage must be defined.

- *Storage Curve*: This curve defines the shape of a storage unit by describing how the surface area within the unit varies with water depth. This curve is integrated by SWMM to compute stored volume as a function of depth. It can be specified to the model as either a functional equation or as a tabular curve (area-depth pair data).
- *Evaporation Factor*: To allow evaporation from the surface of a storage unit one sets its *Evaporation Factor* to 1 and provides evaporation data to the model using SWMM's *Climatology Editor*. The default value for this parameter is 0, implying that evaporation is ignored.
- *Maximum Depth*: The maximum depth of a storage unit must be defined and should not be left at the default zero value. If the storage unit's depth is not defined, the model will assume the storage unit has a zero depth even if a storage curve has been assigned or a conduit is connected to the storage unit. If the largest depth on the storage curve is below the maximum depth, the last area value on the curve will be extended outward.

### 2. Orifices

SWMM's orifice-type link can be used to represent the opening along the side or bottom of the storage unit that serves as an outlet. The upstream node of the orifice is the storage unit while its downstream node would be a junction that connects it to a downstream conduit. Orifice properties that need to be defined include its height above the bottom of the storage unit (invert offset), its type (side or bottom orifice), its geometry (rectangular or circular shape and the respective dimensions) and its hydraulic properties (discharge coefficient and the presence/absence of a flap gate to prevent backflows).

### 3. Weirs

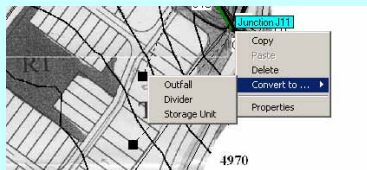
SWMM's weir-type link can be used to represent the opening at the top of the storage unit that serves as an overflow structure. As with the orifice, the upstream node of the weir is the storage unit while its downstream node connects it to a downstream conduit. Weir properties that need to be defined include the weir's height above the bottom of the storage unit, its type (transverse, V-notch or trapezoidal), its geometry and its hydraulic properties (discharge coefficients, end contractions and the presence/absence of a flap gate to prevent backflows).

## Converting Node and Link Elements

Nodes need not be deleted and links redrawn to replace a node with a storage unit or to define orifices and weirs in SWMM. Instead, they can be converted. For example, a node can be converted to a storage unit by following the procedure outlined below.

### *Convert a Node to a Storage Unit*

1. Right-click on the node to be converted and choose “**Convert to...**” from the pop-up menu as show in the figure below.
2. Select “**Storage Unit**” from the sub-menu that appears.
3. Open the *Property Editor* for the new storage unit and define its new name (e.g. *SUI*). The new unit is given the invert elevation and maximum depth of the node from which it was converted.
4. Enter any additional properties needed to define the behavior of the storage unit (such as its *Storage Curve*).



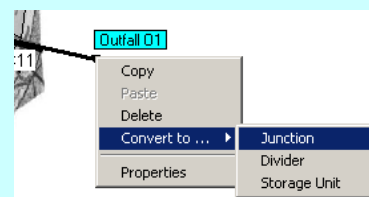
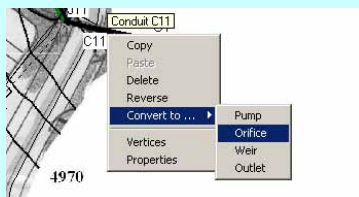
### *Convert a Conduit to an Orifice*

Like nodes, links can be converted to other types of links. Follow the steps below to convert the conduit connecting a storage unit to an outfall into an orifice.

1. Right-click on the conduit downstream of the storage unit and choose “**Convert to...**” from the pop-up menu.
2. Select “**Orifice**” from the sub-menu that appears..
3. Open the *Property Editor* of the orifice and define its dimensions, invert offset and discharge coefficient.

### *Convert an Outfall to a Node*

4. If the storage unit (*SUI*) is to have more than one orifice, then the orifices cannot be connected directly to an outfall (*OI*). The outfall must be converted into a node (*OI*) in this case and a new outfall created (*O2*). Convert the outfall using the same procedure described above.



### 3.3 Model Setup

SWMM can be used to model storage facilities that capture runoff from different design storms and release it to a receiving channel at a controlled rate. This example demonstrates how the design of a storage pond is an iterative process in which the dimensions of the pond and its outlets are changed to satisfy the design criteria and constraints for the design storms considered. The three main steps used to design the storage pond are:

1. Estimate the water quality capture volume (WQCV).
2. Size the storage volume and the outlet to control the release rate of the WQCV.
3. Size the storage volume and the outlet to control the peak runoff rates from the 2-, 10- and 100-yr design storms.

The final design will be a storage unit with a shape specific to its location, rainfall and climate conditions; a defined relationship between its surface area and storage depth; and a multi-outlet structure designed to control different runoff events. Figure 3-2 shows the schematic of a detention pond and its outlets designed to control a WQCV and the peak discharges for three design storms. The stacked trapezoidal prism shape shown in this figure will be used in this example; the upper prism will control the major storms (10- and 100-yr) while the lower prism will control the minor storms (WQCV and 2-yr).

Note that the discharge for different storms is controlled by a combination of orifices and weirs rather than a single unique outlet. Orifice 1 in Figure 3-2 controls the release of the WQCV; orifices 1 and 2 control the release of the 2-yr storm; orifices 1, 2 and 3 control the release of the 10-yr storm and all the orifices combined together with the weir (4) control the release of the 100-yr storm.

#### Estimation of the Water Quality Capture Volume

The WQCV is the critical runoff volume to be used in the design of stormwater quality enhancement facilities. Detailed investigation based on calibrated long-term runoff simulations is the preferred method to determine this volume for a given site (Guo and Urbonas, 1996). However, several methodologies or “rules of thumb” have been proposed to estimate the WQCV that are simpler to use but still reliable when long-term records are not available (see for instance Guo and Urbonas, 1995, 1996 and 2002; Water Environment Federation, 1998). This example will use the methodology proposed by the UDFCD (2001). Figure 3-3 shows the curves defined in this methodology to estimate the WQCV as a function of the tributary catchment’s total imperviousness and the drain time of the capture volume. The following steps are used to estimate the WQCV for the detention basin being designed in this example:

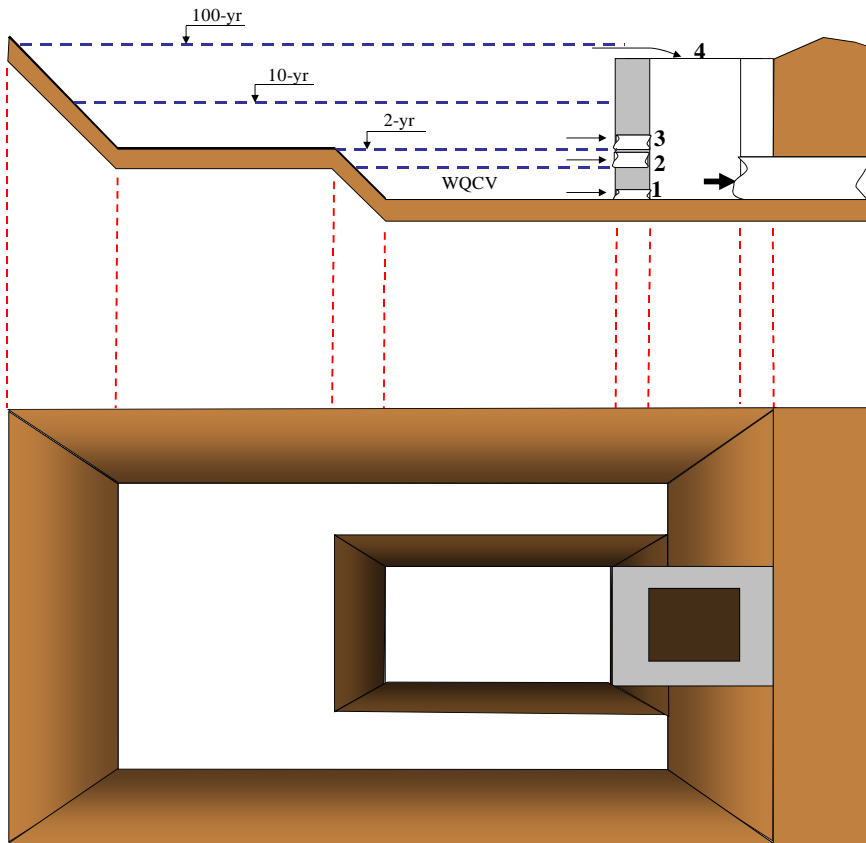


Figure 3-2. Schematic representation of a detention pond

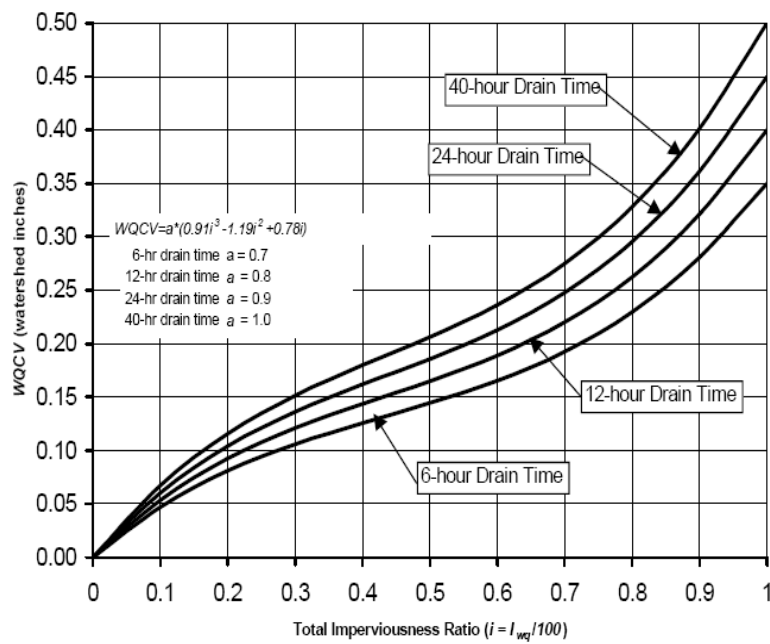


Figure 3-3. Water quality capture volume (UDFCD, 2001)

1. First, determine the developed site's average Directly Connected Impervious Area (DCIA). DCIA is the impervious area that is directly connected to the stormwater drainage system; it does not include rooftops, patios, etc. that drain to lawns or other pervious areas, and is smaller than the gross or total impervious area that is typically estimated through aerial photography. These areas were previously estimated for each of the seven subcatchments defined for the post-development site condition in Example 1 and are presented in Table 3-2.

**Table 3-2. Post-development subcatchment data**

Subcatchment	S1	S2	S3	S4	S5	S6	S7
Area (ac)	4.55	4.74	3.74	6.79	4.79	1.98	2.33
Imperviousness (%)	56.8	63	39.5	49.9	87.7	95	0

2. Next calculate the site's average percent imperviousness by weighting the imperviousness of each subcatchment by its area and dividing by the total area (28.94-ac) of the study area. The average percent imperviousness of the site determined by this method is  $57.1\% \approx 57\%$ .
3. The next step is to determine the WQCV in watershed inches. Assume that the example site is located in Colorado's high plains near the foothills and that the storage unit is to have a 40 hr drain time. From Figure 3-3 the corresponding WQCV in watershed inches is 0.23 in. Thus the total water quality control volume is  $28.94\text{-ac} \cdot 0.23 \text{ in}/12 = 0.555 \text{ acre-ft}$  or 24,162 ft<sup>3</sup>.
4. If the design location were not in Colorado's high plains near the foothills, one would need to adjust the WQCV determined from Figure 3-3. The curves shown in Figure 3-3 are defined to control the 80th percentile runoff event and are appropriate for use in Colorado's high plains near the foothills. For other locations, the WQCV from Figure 3-3 can be adjusted to obtain an appropriate volume, WQCV<sub>o</sub>, using Equation 3-1. In this equation,  $d_6$  is the average precipitation depth of the runoff-producing storms. Storm events for Equation 3-1 were defined for a 6-hour inter-event period and have a minimum depth of 0.1 in. Figure 3-4 shows estimates of  $d_6$  for the contiguous United States (UDFCD, 2001).

$$WQCV_o = d_6 \frac{WQCV}{0.43} \quad (3-1)$$

### Pond Geometry and Dimensions

The shape of the storage unit will depend on the regulations in the location where the structure will be constructed. Generally, it is recommended that the distance between the inlet and outlet of the facility be maximized; a length to width ratio of 2:1 to 3:1 is adequate. This example will use a length to width ratio of 2:1, a WQCV depth ( $h_l$ ) of 1.5 ft, and a side slope of 4:1 (H:V). Figure 3-5 shows the geometry of the WQCV and equations developed based on the length to width ratio (2:1) and the storage unit side slope (4:1) that describe the unit's geometry. The steps used to determine the dimensions of the WQCV are:



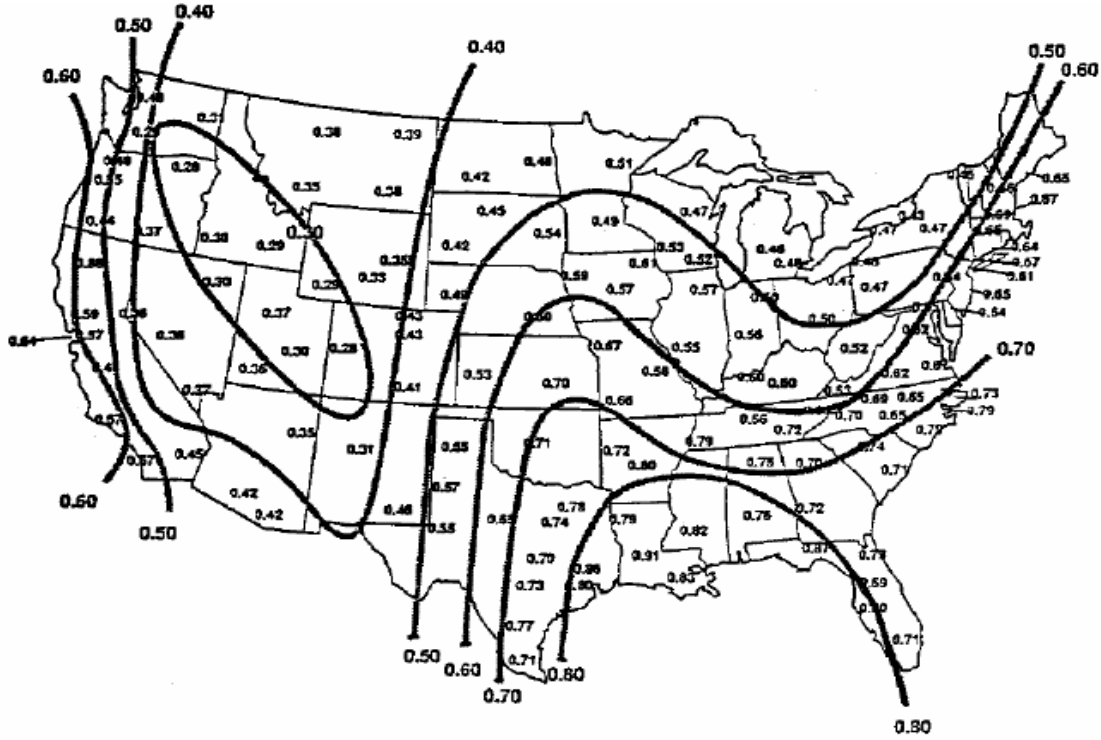


Figure 3-4. Average depth (inches) of runoff producing storms in the US (Driscoll, et al., 1989)

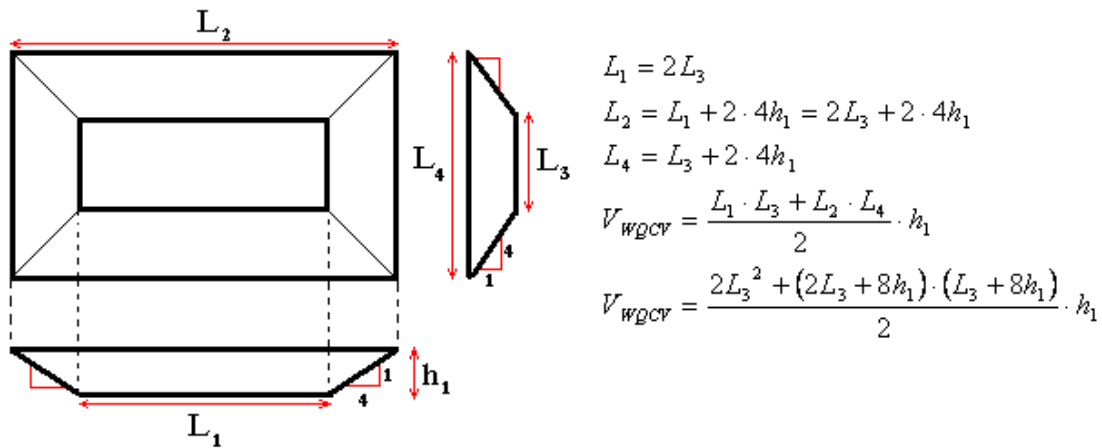


Figure 3-5. Geometry of the pond's WQCV

1. Solve for  $L_3$  using the WQCV found in the previous section (24,162 ft<sup>3</sup>) and  $h_1$  equal to the WQCV depth (1.5 ft). Rearranging the fifth equation listed in Figure 3-5 yields the following quadratic equation for  $L_3$ :

$$4L_3^2 + 24h_1L_3 + (64h_1^2 - 2V_{WQCV} / h_1) = 0$$

Solving for  $L_3$  gives  $L_3 = 85.15 \text{ ft} \approx 86 \text{ ft}$ .

2. Next solve for the other dimensions of the WQCV using  $L_3$  and  $h_1$ . From the first equation from the top in Figure 3-5,  $L_1 = 170.3 \text{ ft} \approx 171 \text{ ft}$ , from the second equation  $L_2 = 184 \text{ ft}$ , and from the third equation  $L_4 = 98 \text{ ft}$ .
3. Then define the storage curve for the WQCV portion of the storage unit. At 0 depth the area is  $L_1 \cdot L_3 = 14,706 \text{ ft}^2$  while at the full depth of 1.5 ft the area is  $L_2 \cdot L_4 = 18,032 \text{ ft}^2$ . These pairs will be entered into the model in the following section together with new points in the surface area-depth curve representing the shape shown in Figure 3-2 to control larger volumes.

### Adding a Storage Unit to the Model

The **Example2-Post.inp** file will be used as a starting point to add a storage unit into the model that represents the detention pond. The following steps are taken to define the storage unit.

1. A new *Storage Curve* object named *SU1* is created to represent the shape of the storage unit.
2. The two previously determined depth-area points are entered into the *Curve Editor* dialog for curve *SU1*. These two points are  $d_1 = 0$ ,  $A_1 = 14706 \text{ ft}^2$  and  $d_2 = 1.5 \text{ ft}$ ,  $A_2 = 18032 \text{ ft}^2$ .
3. A new storage unit node, also named *SU1*, is placed onto the study area map as shown in Figure 3-6, and is left disconnected from the drainage system. The following properties are assigned to *SU1*; *Storage Curve* = **Tabular**; *Curve Name* = **SU1**; *Invert Elevation* = **4956** ft (six feet lower than the outfall node elevation defined in the previous examples); *Maximum Depth* and *Initial Depth* = **1.5** ft (the maximum allowable depth defined to control the WQCV).
4. An additional node (*J\_out*), conduit (*C\_out*) and outfall node (*O2*) are added that will connect the orifices and weirs draining the storage unit (*SU1*) to the outfall node (*O2*). This must be done because it is not possible to connect more than one hydraulic link to an outfall node in SWMM. The invert elevations of *J\_out* and *O2* are set to **4954** ft to avoid backwater effects (again, details in the geometric definition of the storage unit will depend on the local conditions and it is not the intention of this example to cover these details) and *C\_out* is given a length of **100** ft and a roughness of **0.01**. Figure 3-7 shows the independent storage unit system, the tabular storage curve *SU1* for the WQCV and the storage unit's Properties table.

Initially, the storage unit and its WQCV orifice are modeled independent of the watershed to size the WQCV orifice to drain in 40 hours. Although the storage unit and the watershed are shown in the same input file in Figure 3-6, they will run as independent systems in the model because they are not hydraulically connected. The location of the pond in Figure 3-6 will be its final location in the model. The pond could have been placed in the park area since there is significant open space for it, but for clarity it was placed at the downstream end of the park.

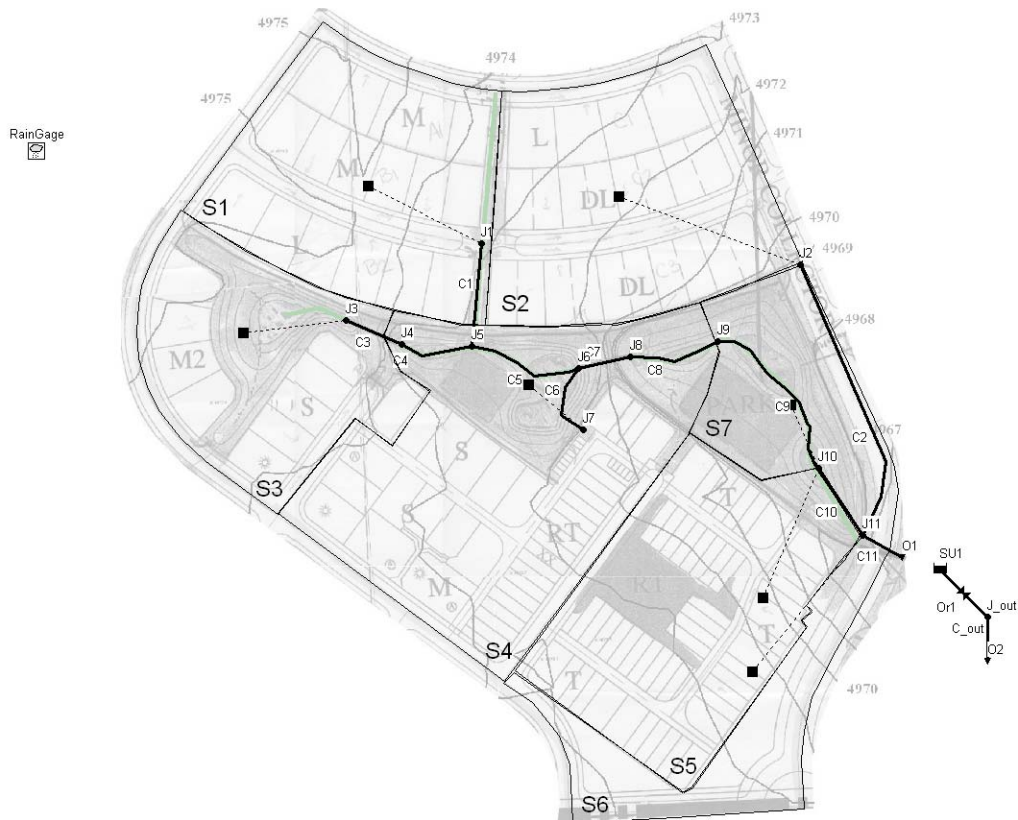


Figure 3-6. Study area map with storage unit SU1

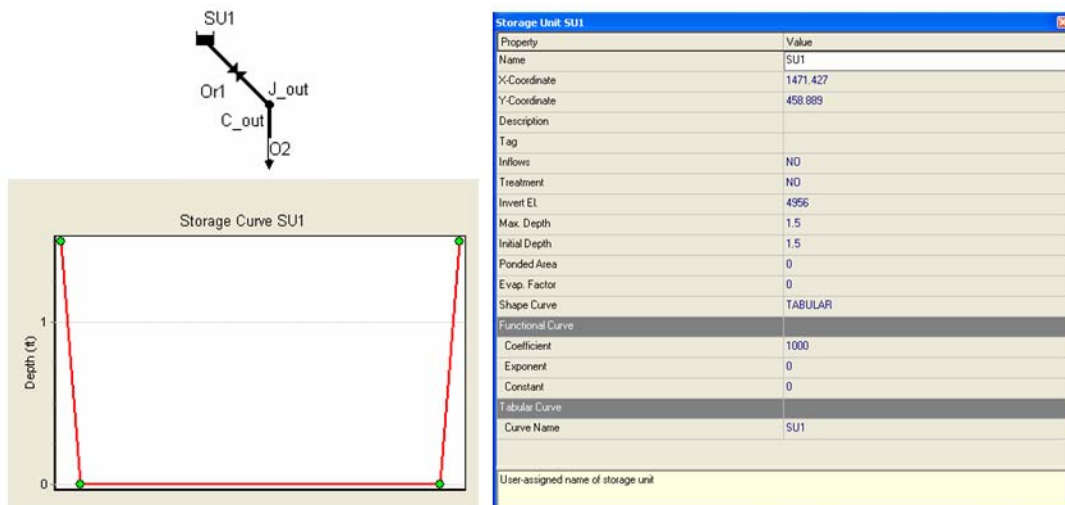


Figure 3-7. Properties of storage unit SU1

## Sizing the WQCV Orifice

The next step is to design the pond outlet so that the entire WQCV is released within 40 hours. The outlet will be an orifice connecting the storage unit to the downstream outfall *O2*. This orifice could be located at the bottom or side of the storage unit and be either circular or rectangular in shape. The following steps are used to size the orifice so the WQCV drains in 40 hours.

1. A side orifice (*Or1*) is added between the storage unit (*SU1*) and the node (*J\_out*) leading to the outfall node. It is given a rectangular shape and assigned an *inlet offset* of **zero** so that its invert is the same as the storage unit. Its *discharge coefficient* is assumed to be the default value of **0.65**.
2. The simulation time step options are set as follows: *reporting*, *wet-weather* and *routing* time steps to **15** seconds and the *dry-weather* time steps to **1** hr. The *simulation duration* must be longer than 40 hours so that the performance of the orifice can be properly evaluated; this example uses **72** hours.
3. The final dimensions of orifice *Or1* are determined by running SWMM several times using *Dynamic Wave* flow routing while iteratively changing the orifice dimensions until a size is found that drains the WQCV in approximately 40 hours. For each run, the dimensions of the orifice are varied while keeping the initial water depth in the storage unit at the depth of the WQCV, 1.5 ft. One can assume that the basin is essentially empty once the water depth is 0.05 ft. Note that the runoff discharge generated by the rainfall falling on the subcatchments does not affect the storage unit during this part of the example because it is not connected to the drainage system.

Figure 3-8 shows the drainage time for three iterations as well as that for the final design. Table 3-3 shows the dimensions assigned to the orifice by iteration. The final orifice design has a height of **0.3** ft and a width of **0.25** ft. This small size is typical of a WQCV orifice. That is why the orifice must be protected by a screen to prevent plugging during the storm and maintenance must be done regularly to ensure the screen remains free of debris.

**Table 3-3. Design of the WQCV outlet (*Or1*)**

<b>Iteration</b>	<b>1</b>	<b>2</b>	<b>3</b>	<b>Final</b>
Height (ft)	0.166	0.25	0.25	<b>0.3</b>
Width (ft)	0.25	0.25	0.4	<b>0.25</b>
Inlet Offset (ft)	0	0	0	<b>0</b>
Discharge Coefficient	0.65	0.65	0.65	<b>0.65</b>
Drainage time (hr:min)	53:58	43:21	27:07	<b>40:12</b>

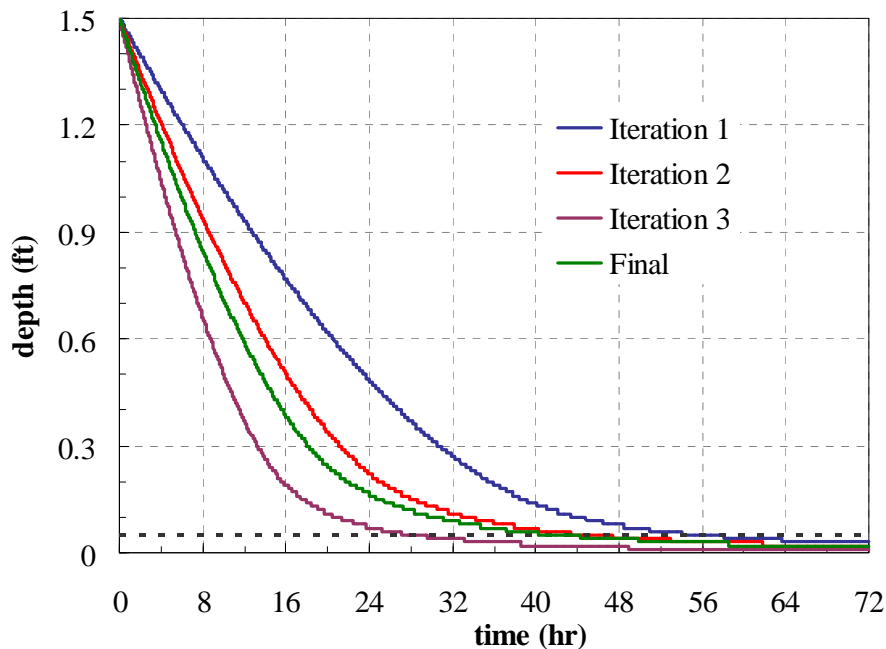


Figure 3-8. WQCV drainage times for the iterations shown in Table 3-3

### Sizing the 2-yr Design Storm Orifice

The runoff volume generated by the 2-yr storm will be larger than the WQCV volume designed for in the previous section. The volume of the storage unit must now be enlarged and a new outlet must be defined. This new outlet, which will be placed at a height of 1.5 ft above the basin floor, will begin to discharge when the runoff volume from any storm just exceeds the WQCV. This outlet will control not only the peak runoff rate of the 2-yr storm but also partially control the runoff rate from storms greater than the 2-yr storm. The required increase in storage volume will be achieved by extending the sides of the storage unit above the WQCV depth while keeping a lateral slope of 4:1 (H:V) as shown in the basin schematic, Figure 3-2. The following steps outline how the storage unit is sized for the 2-yr design storm orifice.

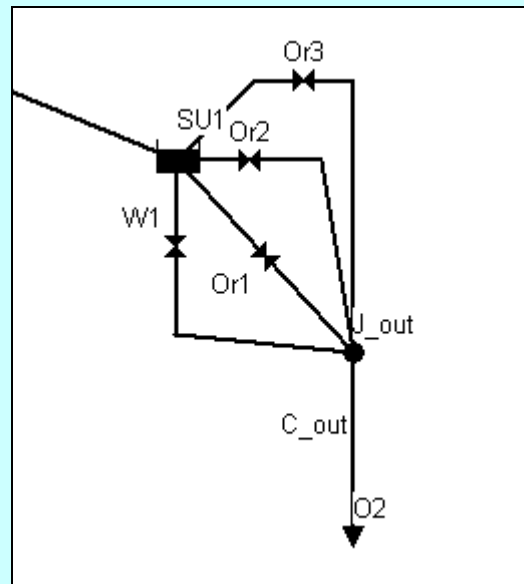
1. The storage unit is first connected to the rest of the drainage system. This can be done by changing culvert *C11*'s outlet to *SU1* and deleting the original outfall node *O1*. Culvert *C11* is given a downstream offset of 1ft so that for minor storms it has no backwater but still has its crown below the top of the storage pond.
2. Next the size of the pond is enlarged for flood control by expanding its height while keeping a constant slope (refer to Figure 3-2 for an illustration). This is done by entering a new pair of surface area-depth pairs to the storage curve *SU1*. The values for this new pair are:  $d_3 = 6$  ft,  $A_3 = 29583$  ft<sup>2</sup>. The initial depth of the storage unit is set to **zero** and its maximum depth to 6 ft to account for the new volume.
3. The model is then run for the 2-yr storm using only the WQCV orifice to determine the maximum depth in the storage unit, the peak discharge of the WQCV orifice (*Or1*), and the time it takes the storage unit to empty. The results show a maximum

storage unit depth of 2.82 ft, a maximum orifice discharge of 0.64 cfs and an emptying time of 56:23 (hr:min).

4. Based on the results in step 3, the peak outflow for the 2-yr storm can be increased because the pre-development 2-yr peak runoff (4.14 cfs from Table 3-1) is larger than the discharge through the WQCV orifice (0.64 cfs). Increasing this discharge is advantageous because it will reduce the final volume of the storage required and save in costs. To increase the 2-yr storm pond outflow, a second orifice (*Or2*) is added directly above the WQCV depth (inlet offset = 1.5 ft) as illustrated in Figure 3-2. This orifice is assigned a rectangular shape with an inlet offset of **1.5** ft and a discharge coefficient of **0.65**. It should be drawn on the map with at least one intermediate vertex so that it can be distinguished from the existing orifice *Or1* (see the sidebar “Drawing Links as Polylines”).

### Drawing Links as Polylines

SWMM allows links to be drawn as polylines containing any number of straight-line segments that define the alignment or curvature of the link. Once a link has been drawn its interior points can be added, deleted, and moved. This feature is especially useful when two or more links share the same set of end nodes and would otherwise appear directly on top of one another on the map. The figure on the right shows how polylines were used to draw the various orifices and weirs that comprise the outlet structure of a storage unit so that they could be distinguished from one another.



5. An initial estimate of *Or2*'s area *A* is made using the orifice equation:

$$Q = CA(2gh)^{1/2} \quad (3-2)$$

with  $C = 0.65$ ,  $Q = (4.14 - 0.64)$  cfs = 3.5 cfs and  $h = (2.84 - 1.5)$  ft = 1.34 ft. This produces an orifice area of 0.58 ft<sup>2</sup>. Assume *Or2* has an initial height of 0.58 ft and a width of 1 ft.

6. Running the model with these dimensions for *Or2* produces a discharge of 2.84 cfs. This value is less than the target discharge (4.14 cfs). Therefore, iterations must again be used to size orifice *Or2* as was done for *Or1* until the combined peak discharge of the two orifices is equal to or a little less than the 2-yr pre-development peak discharge (4.14 cfs).

7. To simplify the iterations for sizing *Or2* its height is fixed at **0.5** ft and its width is varied in 0.05 ft increments until the combined discharge of both orifices is close to 4.14 cfs. A size of **0.5** by **2** ft produces a peak discharge of 4.11 cfs and a maximum storage unit depth of 2.21 ft. These dimensions will thus be used for the 2-yr orifice.

### Sizing the 10-yr Design Storm Orifice

Up to this point the storage unit has been represented as a single trapezoidal prism. This shape was determined for the WQCV back in the “Pond Geometry and Dimensions” section and increased in size (keeping the 4:1 side slope) to contain the 2-yr storm runoff. In this section the storage unit shape is redefined by adding an additional trapezoidal prism over the minor storm prism to contain the 10- and 100-yr storm volumes (See Figure 3-2). The steps below show how this is done to size the 10-yr storm orifice:

1. The storage curve *SUI* is modified by replacing the surface area-depth pairs  $d_2, A_2$  and  $d_3, A_3$  with the following three surface area-depth pairs:  $d_2 = 2.22$  ft,  $A_2 = 19659$  ft<sup>2</sup>;  $d_3 = 2.3$  ft,  $A_3 = 39317$  ft<sup>2</sup> and  $d_4 = 6$  ft,  $A_4 = 52644$  ft<sup>2</sup>. Note that the new  $d_2, A_2$  point is the highest point of the original shape (the maximum height of the 2-yr storm). The area of point 3 ( $A_3 = 39317$  ft<sup>2</sup>) doubles the area of point 2 ( $A_2 = 19659$  ft<sup>2</sup>). The depth,  $d_3$  is set 0.1 feet above  $d_2$  so that the change in cross sectional area is not too abrupt. In practice this transitional area would have a slope of 2% so that water in storage will drain into the smaller basin after the storm. The area of point 4 is computed by extending the sides of the storage unit above the point 3 while keeping a lateral slope of 4:1 (H:V).
2. The model is run with the 10-yr storm and the existing orifices to determine if a 10-yr storm orifice is needed. The resulting maximum water depth is 3.20 ft and the combined peak discharge from both existing outlets is 6.96 cfs. The pre-development peak discharge for the 10-yr storm is 7.34 cfs which means that the storage unit volume can again be decreased by adding another orifice.
3. A new 10-yr storm orifice (*Or3*) is added directly above the depth of the volume designed to control the 2-yr storm runoff (inlet offset = **2.22** ft). As with *Or2*, *Or3* is drawn with intermediate vertices so that it can be seen easily on the system map. The orifice equation (3-2) is used to estimate its required area. For  $C = 0.65$ ,  $Q = (7.34 - 6.96)$  cfs = 0.38 cfs and  $h = (3.20 - 2.22)$  ft = 0.98 ft the resulting orifice area is 0.073 ft<sup>2</sup>. A height of 0.25 ft and a width of 0.25 ft are used as an initial estimate of the orifice's size.
4. When the model is run with the 10-yr storm for this size of *Or3*, the combined discharge is 7.22 cfs. Because this discharge is less than the pre-development discharge (7.34 cfs), the orifice's width is increased to 0.35 ft and the model is re-run. The new combined discharge is 7.32 cfs and the maximum depth in the storage unit is 3.17 ft. This is sufficiently close to the target discharge to accept this orifice size (height = **0.25** ft, width = **0.35** ft).

## Designing the 100-yr Weir

The model can now be run with the 100-yr storm using the combined WQCV, 2-yr and 10-yr orifices to determine if the 100-yr weir is needed. The peak discharge of these combined orifices for the 100-yr storm is 12.57 cfs, which is not enough to pass the 100-yr storm's runoff (31.6 cfs), and the storage unit floods. A weir will be designed to control this extreme event so that the pre-development discharge is matched and the total water depth in the pond does not exceed the 6 ft depth that was given as the maximum depth for safety reasons. This is accomplished as follows:

1. A new weir link *W1*, drawn with intermediate vertices, is added between the storage unit and the node (*J\_out*) leading to the final outfall. It is specified as a transverse weir whose inlet offset is **3.17** ft above the storage unit bottom (the maximum depth reached by the volume controlling the 10-yr storm runoff), and whose discharge coefficient is **3.3**. The height of the weir opening is set at **2.83** ft which is the distance between the volume controlling the 10-yr storm runoff and the maximum depth of the storage unit.
2. The weir equation (3-3) is used to determine an initial width *L* for the weir:

$$Q = CLh^{3/2} \quad (3-3)$$

Using  $Q = (31.6 - 12.57)$  cfs = 19.03 cfs,  $C = 3.3$  and  $h = 2.83$  ft produces a width of 3.43 ft  $\approx$  3.45 ft.

3. With weir dimensions of height = 2.83 ft, width = 3.45 ft and invert offset = 3.17 ft the model is run for the 100-yr storm. The resulting peak total discharge from the storage unit is 42.4 cfs which exceeds the target flow of 31.6 cfs.
4. Step 3 is repeated with successively smaller weir widths until a combined discharge close to 31.6 cfs is obtained. A width of **1.75** ft produces a combined 100-yr discharge of 31.2 cfs and a maximum depth of 5.42 ft in the storage unit.
5. The final step is to insure that adequate freeboard is maintained in the storage unit. The current design provides  $6.0 - 5.43 = 0.53$  ft. The required amount will depend on local design guidelines. For example, the UDFCD (2001) requires a freeboard of 1 foot above the maximum water surface elevation when the weir is conveying the maximum discharge.

## 3.4 Model Results

The final SWMM model for the post-development site with the detention pond is shown in Figure 3-9 and its input file is named **Example3.inp**. Table 3-4 summarizes the characteristics of the different discharge elements included in the pond's outlet, which are illustrated in Figure 3-10.



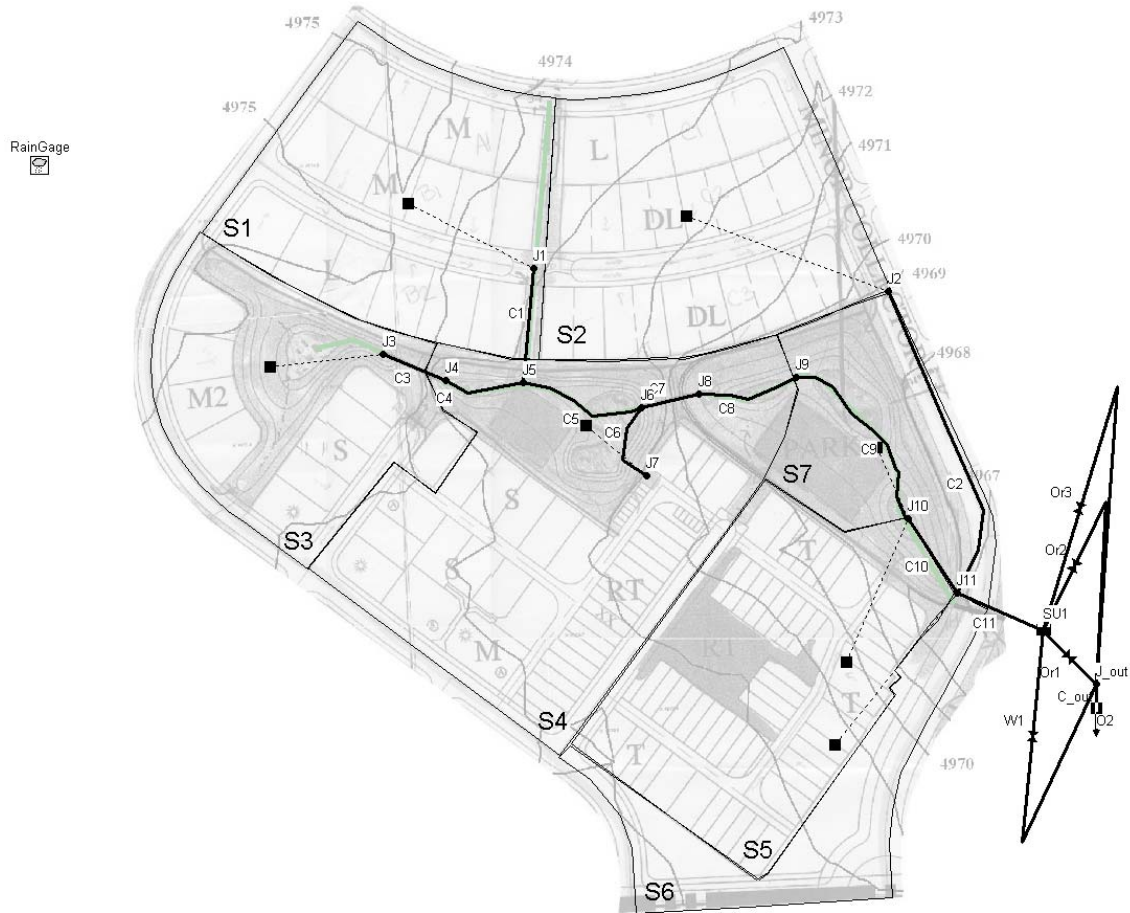


Figure 3-9. Final design of the detention pond and outlet structure

### Orifice & Weir Dimensions

### Offsets

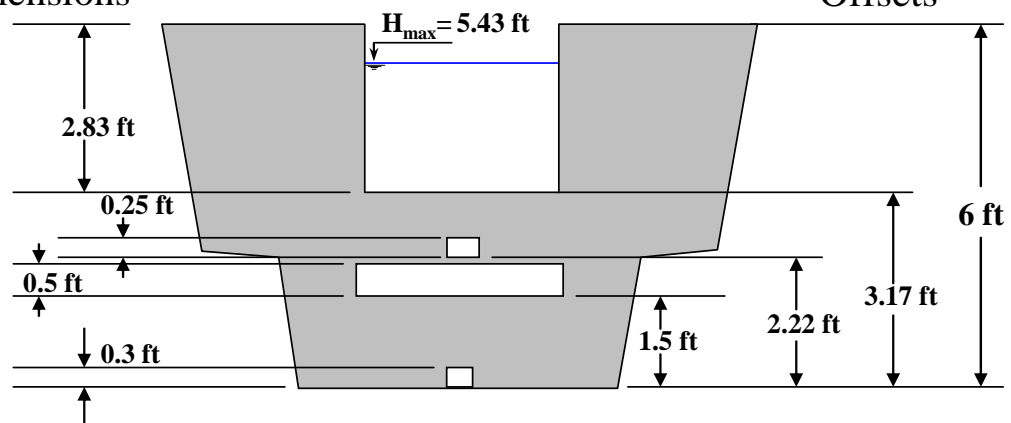


Figure 3-10. Detail of the pond outlet structure

**Table 3-4. Characteristics of the pond's outlet structure**

<b>ID</b>	<b>Type of Element</b>	<b>Event Controlled</b>	<b>Shape</b>	<b>Height h (ft)</b>	<b>Width b (ft)</b>	<b>Invert Offset Z (ft)</b>	<b>Discharge Coefficient</b>
Or1	Orifice	WQCV	Side Rectangular	0.3	0.25	0	0.65
Or2	Orifice	2-yr	Side Rectangular	0.5	2	1.5	0.65
Or3	Orifice	10-yr	Side Rectangular	0.25	0.35	2.22	0.65
W1	Weir	100-yr	Rectangular	2.83	1.75	3.17	3.3

The outflow hydrographs produced by this final design for the detention pond were compared against those resulting with no control as well as against the pre-development discharge targets. Those target discharges were 4.14, 7.34 and 31.6 cfs for the 2-, 10- and 100-yr storms, respectively. The post-development controlled hydrographs were generated using the input file for the final design of this example (**Example3.inp**), while the post-development uncontrolled discharges were generated using the final design developed in Example 2 (**Example2-Post.inp**). The pre-development hydrographs were generated using the final design developed in Example 1 for the pre-development watershed (**Example1-Pre.inp**). The models were run using a **15** second time step for reporting, wet-weather runoff and flow routing and a **1** hr dry weather runoff time step.

The resulting sets of hydrographs are shown in Figures 3-11 through 3-13. Once again a spreadsheet program was used to combine results from different SWMM runs onto one graph. They verify that the detention pond was able to control post-development peak discharges from the site to their pre-development levels. Note, however, that the storage unit had no effect on reducing the total volume of post-development runoff that resulted from the large increase in impervious area.

### 3.5 Summary

This example showed how SWMM could be used to design a detention pond and its outlet structure to provide both a water quality capture volume (WQCV) and peak runoff control. The WQCV was designed to provide a 40 hour drawdown time to satisfy water quality treatment requirements while the peak runoff goal was to limit the maximum post-development discharges for the 2-, 10- and 100-yr storms to their pre-development values. The key points illustrated in this example were:

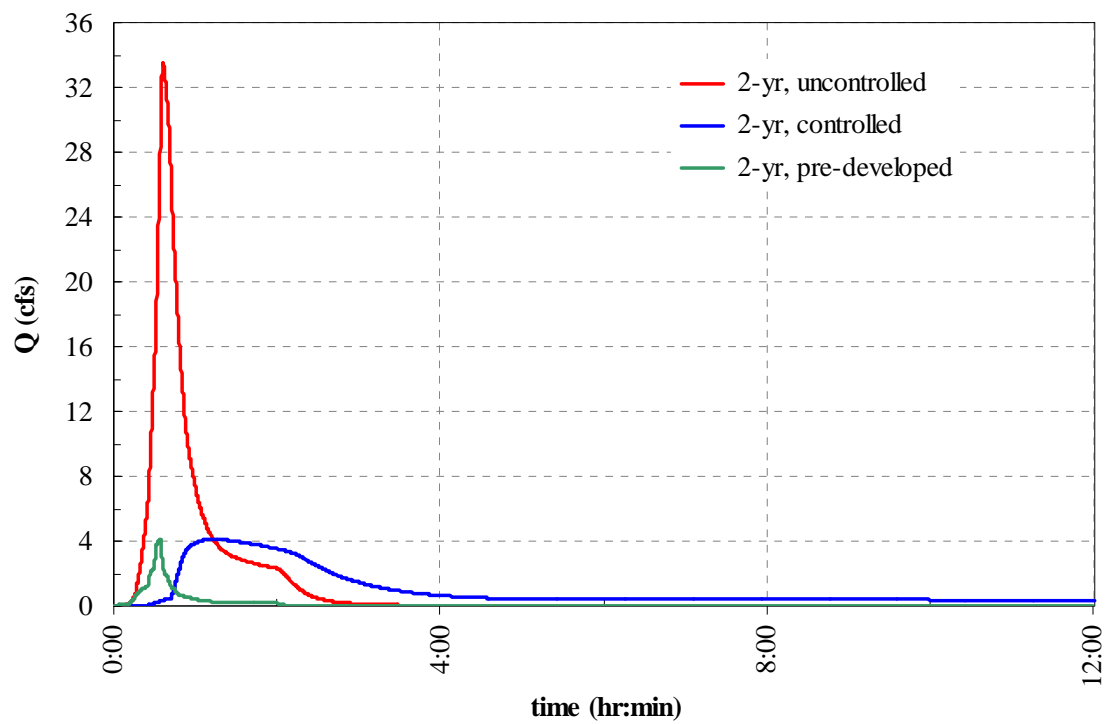


Figure 3-11. Outlet hydrographs for the 2-yr storm

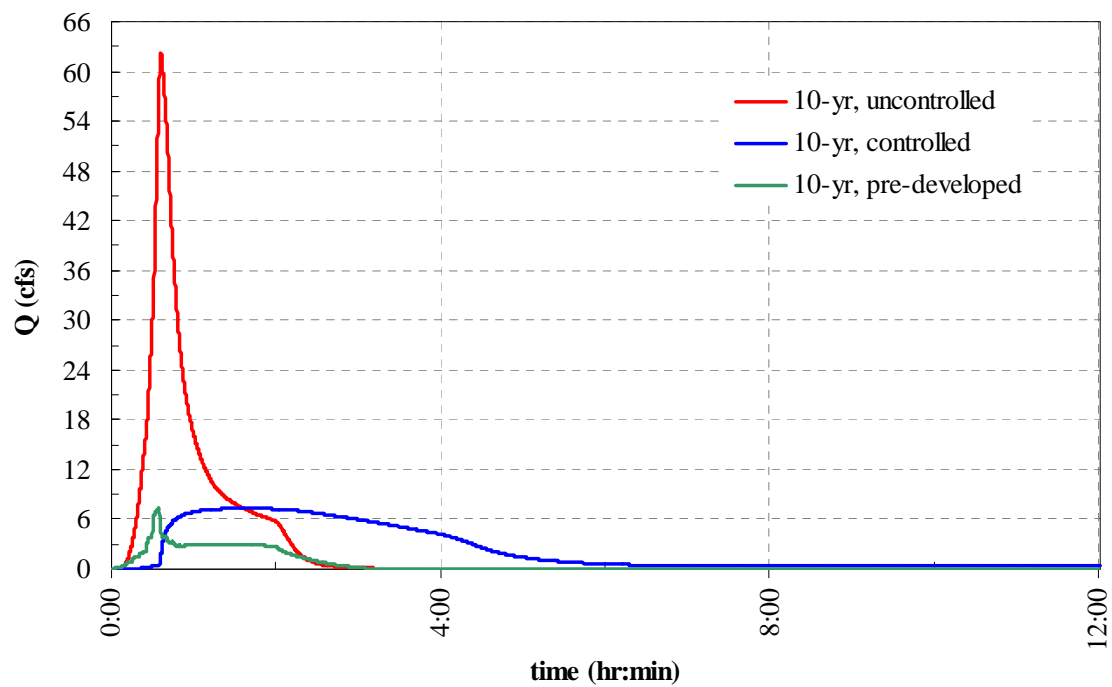
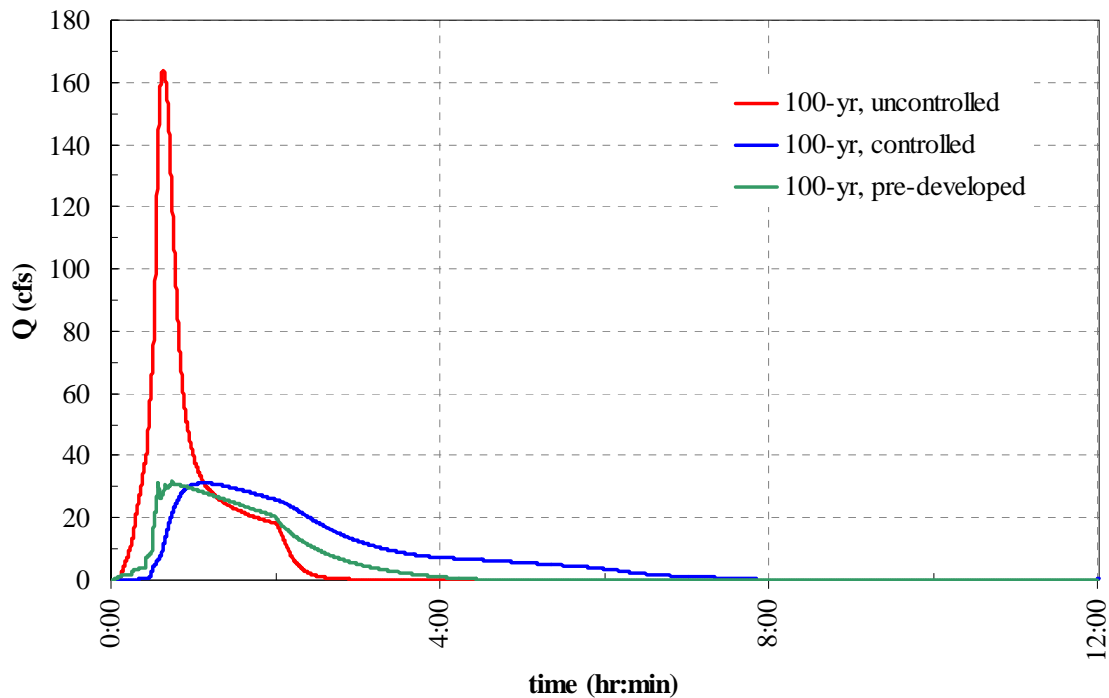


Figure 3-12. Outlet hydrographs for the 10-yr storm



**Figure 3-13. Outlet hydrographs for the 100-yr storm**

1. The WQCV's outlet structure can be designed by using a full storage unit (volume = WQCV) disconnected from the drainage system and an orifice whose dimensions are varied until the storage is drained in a time equal to that defined by local regulation (40 hr in this example).
2. The dimensions of the other component of the outlet structures (e.g. orifices and weirs) used to control peak flows can be designed sequentially. The maximum water depth reached using one design storm is the location of the invert offset of the orifice or weir used to control the next larger design storm.
3. The orifice and weir equations are useful for making initial estimates of an outlet's dimensions.
4. Although detention storage is effective in controlling peak runoff rates it has no effect on reducing runoff volume.

This example will be extended to include water quality treatment associated with the storage pond in Example 6 and continuous simulation in Example 9.

## Example 4. Low Impact Development

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This example demonstrates how to model two Low Impact Development (LID) control alternatives, filter strips and infiltration trenches. The detention pond modeled in Example 3 is an example of a subdivision-scale drainage control structure. The LIDs in this example are hydrologic source controls that operate on a smaller scale and rely heavily on infiltration and distributed small-scale storage to reduce the overall runoff volume from a watershed and control water quality.

SWMM is better suited to simulating some types of hydrologic source control techniques than others. Filter strips and infiltration trenches are two such source controls. This example will illustrate how they can be represented in SWMM by applying them to the same catchment area studied in the previous Examples 1, 2 and 3.

### 4.1 Problem Statement

In Examples 1 and 2, runoff estimates for the 29 acre residential development shown in Figure 4-1 were made without any source controls in place. In this example, the effects that infiltration trenches (*IT*) and filter strips (*FS*), two commonly used LIDs, have on the site's runoff will be examined. As illustrated in Figure 4-1, four infiltration trenches will be placed at each side of the east-west street in the upper part of the study area. Additionally, filter strips will be used to control the runoff from lots *S*, *M* and *M2*, located in the southwest section of the site. These strips will be built along the sidewalks so they control runoff from the lots before it reaches the gutters.

In Example 3 the quantitative objective for the design of a detention pond was to reduce the post-development watershed discharge to pre-development levels. The LIDs modeled here will not be designed to accomplish a specific quantitative objective but rather to achieve a general reduction in runoff volume to help meet sustainability goals and place a lower burden on stormwater controls further downstream in the basin. The performance of the LIDs for the 2-, 10- and 100-yr storms will be analyzed.

### 4.2 System Representation

The two LIDs modeled here are filter strips and infiltration trenches. Guidance in the representation of these and other LIDs can be found in Huber et al. (2006).

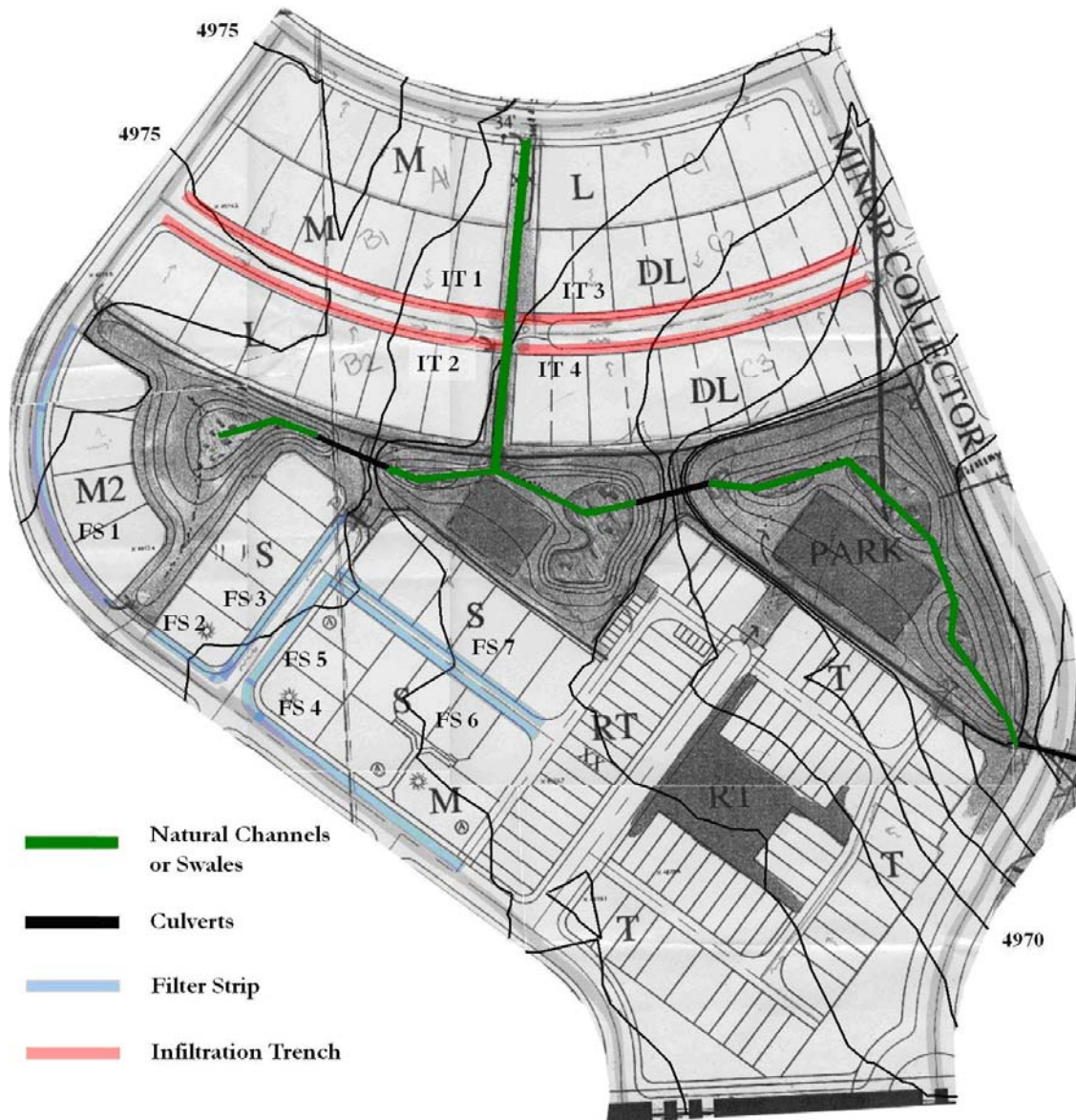


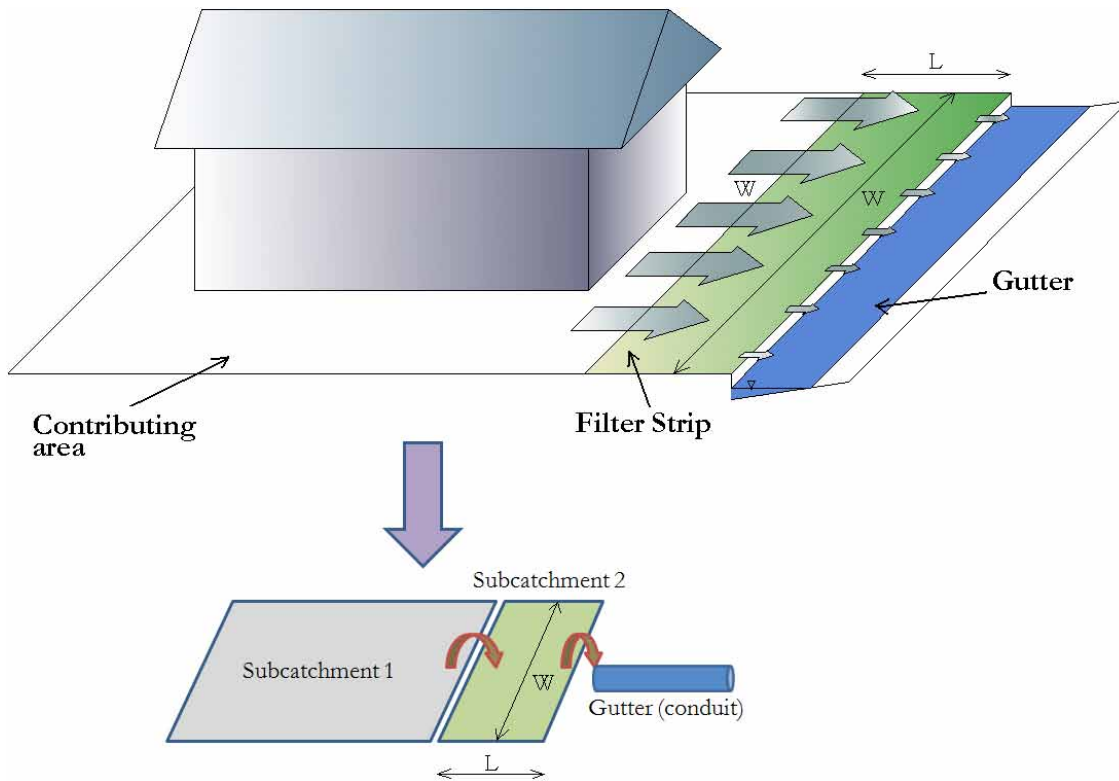
Figure 4-1. Post-development site with LIDs in place

### Filter Strips

Filter strips are grassed or vegetative areas through which runoff passes as sheet flow. They do not effectively reduce peak discharges but are effective in removing particulate pollutants for small storms ( $< 1$  year storm) (Akan and Houghtalen, 2003). Flat slopes ( $< 5\%$ ) and low to fair permeability (0.15 to 4.3 mm/h or 0.006 to 0.17 in/h) of natural subsoil are required for their effective operation (Sansalone and Hird, 2003). SWMM does not have a unique visual object to represent filter strips but they can be represented as a pervious subcatchment that receives runoff from an upstream subcatchment as illustrated in Figure 4-2. The two most important processes that must be simulated with filter strips are infiltration and storage. A filter strip can be simulated as a 100% pervious subcatchment whose geometry (area, width and slope) is obtained directly



from the field. This subcatchment receives water from an upstream contributing area (impervious or semi-impervious) and drains to a conduit representing the gutter or street. The infiltration for a filter strip can be simulated using any of SWMM's infiltration options.



**Figure 4-2. Schematic representation of a filter strip**

## Infiltration Trenches

Infiltration trenches are excavations backfilled with stone aggregate used to capture runoff and infiltrate it to the ground (Guo, 2001). Subsoil with a minimum permeability of 13 mm/h (0.5 in/h) is required for a good performance (U.S. EPA, 1999). The most important processes that must be simulated for an infiltration trench are infiltration, storage and the water flow along the trench. Figure 4-3 shows a workable SWMM representation of an infiltration trench. It consists of a rectangular, fully pervious subcatchment whose depression storage depth equals the equivalent depth of the pore space available within the trench.

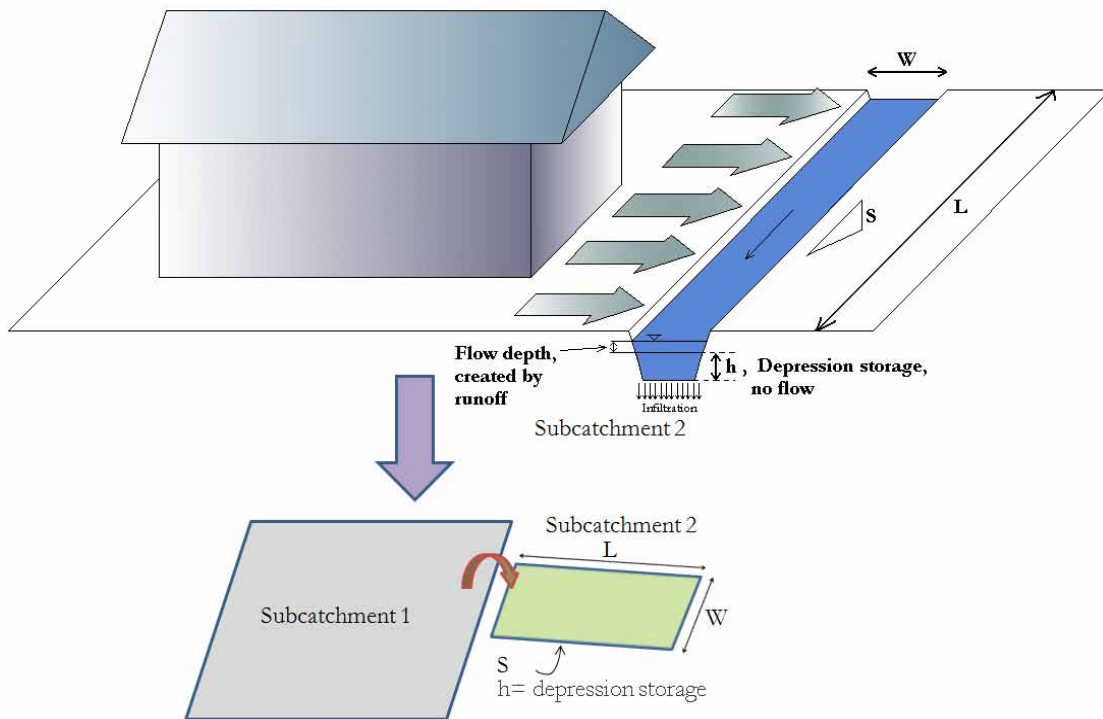


Figure 4-3. Schematic representation of an infiltration trench

### 4.3 Model Setup – Filter Strips

Figure 4-1 shows the locations of the filter strips (*FS*) to be modeled in this example. The initial SWMM file to which the LIDs will be added is **Example2-Post.inp**, which includes both subcatchments and a runoff conveyance system in it. Figure 4-1 will be used as the backdrop of the new model to help facilitate the placement of the filter strips. The image file for this backdrop is named **Site-Post-LID.jpg**. Table 4-1 lists the subcatchment to which each filter strip belongs and the length of each strip.

Table 4-1. Subcatchments containing filter strips

Subcatchment controlled	Filter Strip	Filter Strip Length (ft)
S3	FS 1	410
S3	FS 2	105
S3	FS 3	250
S4	FS 4	359
S4	FS 5	190
S4	FS 6	345
S3 and S4	FS 7	375

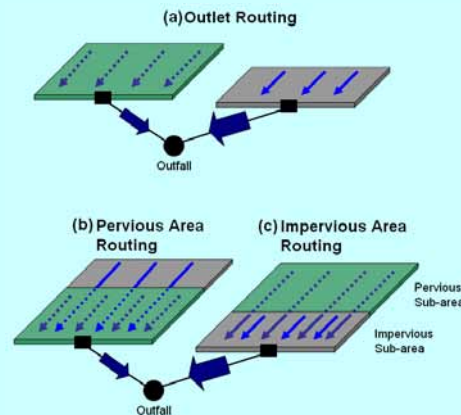
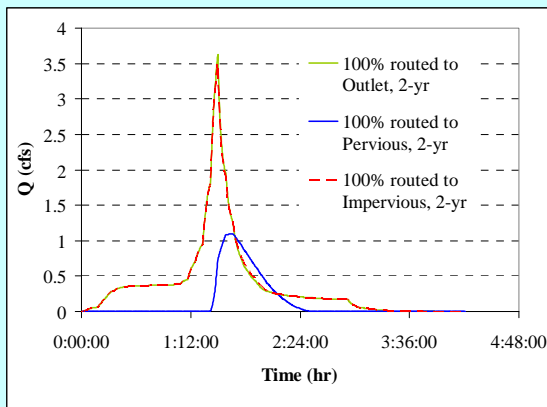


## Sub-Area Routing in SWMM

An alternative method to modeling LIDs, other than creating additional subcatchments as used in this example, is to use *Sub-Area Routing*. Each subcatchment modeled in SWMM is composed of two sub-areas, an impervious sub-area and a pervious sub-area, and options called *Sub-Area Routing* and *Percent Routed* appear in their Properties tables.

*Sub-Area Routing* has three options: **Outlet**, **Impervious** and **Pervious**. The Outlet option ((a) in the figure below) routes the runoff from both sub-areas directly to the subcatchment's outlet. The Pervious option ((b) in the figure below) routes the runoff from the impervious sub-area across the pervious sub-area and then to the outlet, and the Impervious option ((c) in the figure below) routes the runoff from the pervious sub-area across the impervious sub-area and then to the outlet.

When the runoff from the impervious surface is routed across the pervious surface ((b) in the figure below), some of the runoff is lost to infiltration and depression storage in the pervious sub-area. The graph below shows the typical effect on runoff these three sub-area routing methods have when applied to a single subcatchment. Note that because the runoff produced by the pervious area in this case is negligible, the "100% routed to Outlet" and the "100% routed to Impervious" cases are practically the same.



The *Sub-Area Routing - Pervious* option can be used to model LIDs. This is done by representing the LID as the pervious sub-area, setting the subcatchment's pervious values to those of the LID, routing the runoff from the impervious sub-area of the subcatchment to the pervious subarea, and defining the *Percent Routed* to represent the percentage of impervious surface connected to the LID.

Modeling LIDs with this method implies that the entire subcatchment's pervious surface represents the LID. This approach is not as flexible as the one presented in this example because the slope and width of the LID must be those of the subcatchment, which is not always the case.

The filter strips to be constructed are identified in Table 4-1 with letters **FS** and a number. In the model being built some of the strips will be aggregated together as they are represented as new subcatchments. These filter-strip subcatchments will be identified as “*S\_FS\_number*”.

To better estimate the runoff treated by the filter strips, subcatchments *S3* and *S4* will be discretized further. *S3* will be divided into three subcatchments *S3.1*, *S3.2* and *S3.3* and *S4* into four subcatchments *S4.1*, *S4.2*, *S4.3* and *S4.4* as shown in Figure 4-4. It will be necessary to re-estimate the average overland runoff length and new subcatchment widths, slopes and percents imperviousness to place in the model. Refer to Example 1 for help in determining the new subcatchments’ hydrologic properties. The slopes of the new subcatchments representing lots will be **2%**. The same Horton infiltration rates already defined in the previous examples will be used here: **4.5** in/hr for the maximum, **0.2** in/hr for the minimum and **6.5** hr<sup>-1</sup> for the decay constant. With the addition of these new subcatchments and the re-discretization of some of the original ones, it is necessary to define new channels and junctions to connect them to the drainage system. These new elements are illustrated in Figure 4-4 (pipes in red and junctions in blue). Table 4-2 lists the properties of the new junctions and Table 4-3 does the same for the new conduits.

Table 4-4 summarizes the properties of the new subcatchments. The outlets of the new subcatchments (*S3.2*, *S3.3*, *S4.2*, and *S4.3*) are not junctions but filter strips represented as subcatchments. This cascade layout used in the SWMM model is shown in Figures 4-5 for subcatchments *S3.1*, *S3.2* and Figure 4-6 for subcatchments *S4.1*, *S4.2*, *S4.3* and *S4.4*.

The length of the filter strips (i.e., the overland flow length existing between lots and streets) will be **4** feet and the slope of each strip along this length is the same as the typical lot slope of **2%**. The widths of the strips (perpendicular to the overland flow direction) is computed directly from the map with the *Ruler* tool as explained in Example 1 and are given in the last column of Table 4-5.

**Table 4-2. Properties of the new junctions**

New Junction	Invert Elevation (ft)
J15	4974.5
J16	4973.5
J17	4973.5

**Table 4-3. Properties of the new conduits**

New Conduit	Inlet Node	Outlet Node	Length (ft)	Type of Section <sup>1</sup>	Manning Coefficient	Maximum Depth (ft)	Bottom Width (ft)	Left Slope	Right Slope
C15	J15	J3	444.75	Swale	0.05	3	5	5	5
C16	J17	J5	200.16	Swale	0.05	3	5	5	5
C17	J16	J7	300.42	Gutter	0.016	1.5	0	0.0001	25

<sup>1</sup> Type of section based on those defined in Example 2

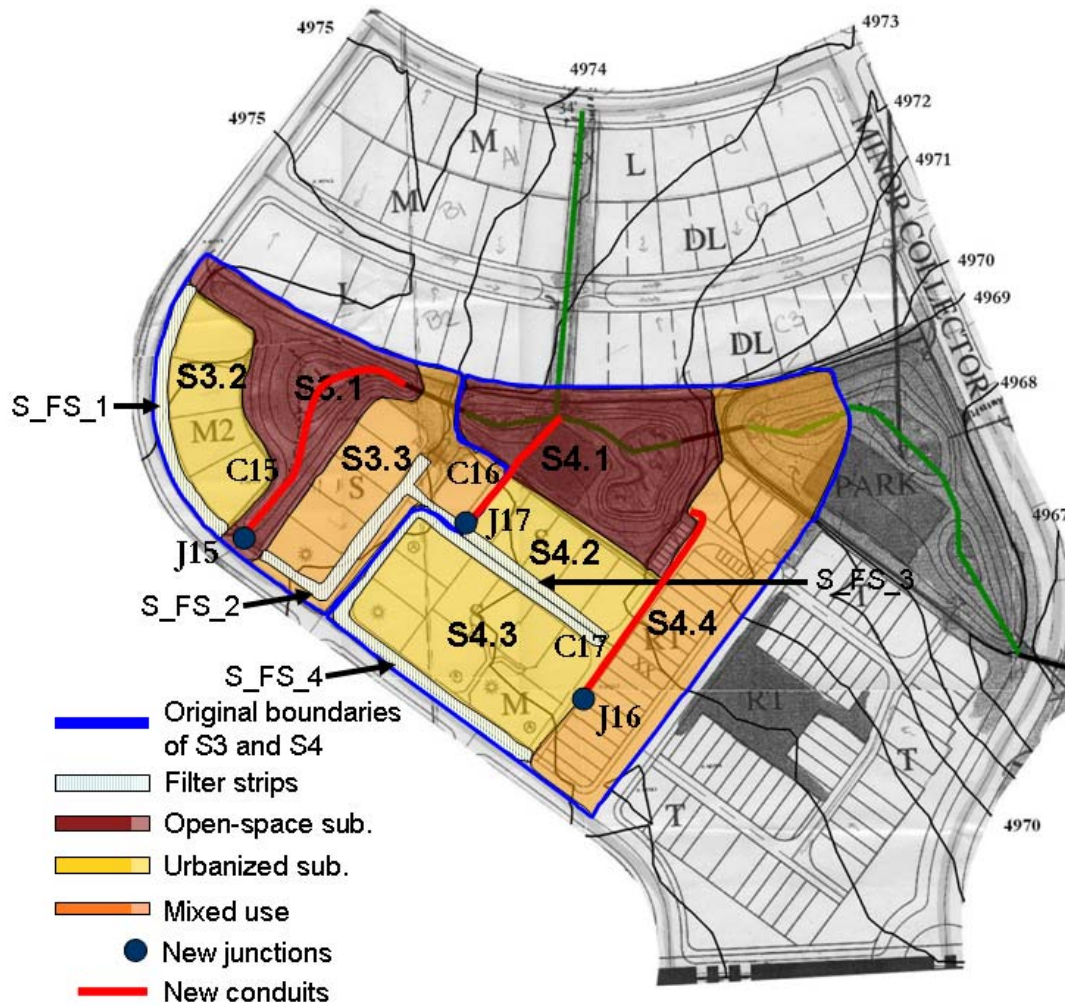


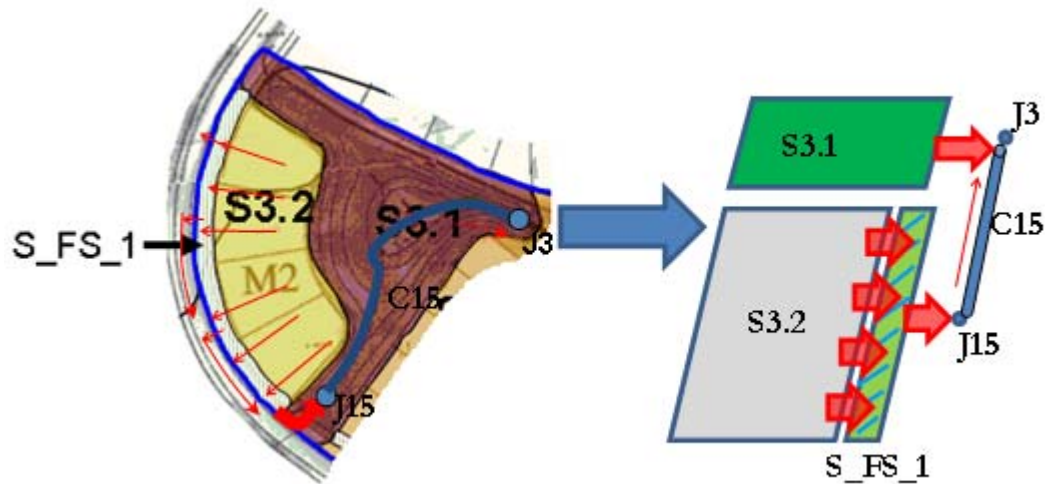
Figure 4-4. Re-discretization of subcatchments S3 and S4

Table 4-4. Properties of the subcatchments derived from S3 and S4

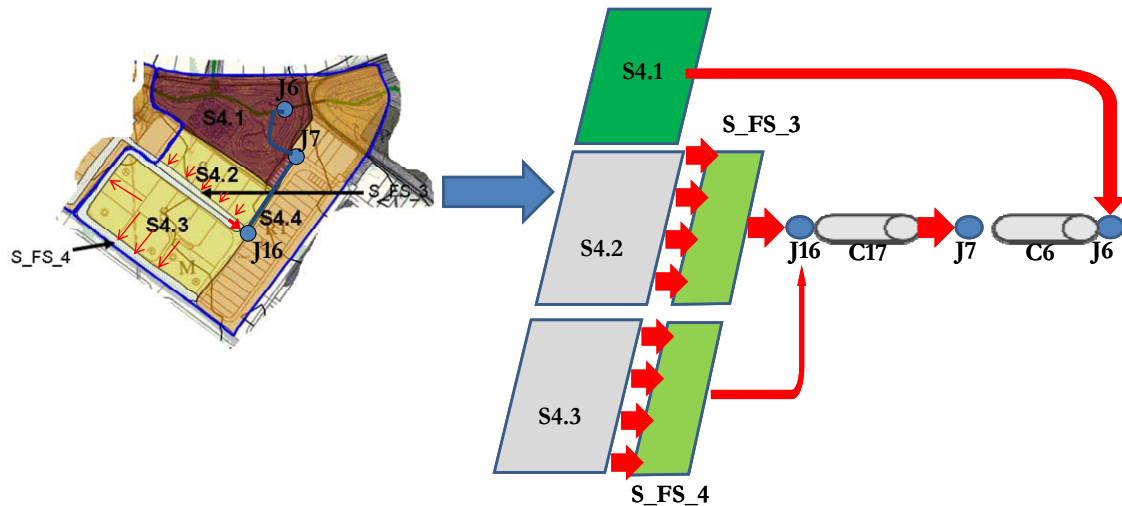
New Subcatchment	Outlet	Area (ac)	Width (ft)	Slope (%)	Imperviousness (%)
S3.1	J3	1.29	614	4.7	0
S3.2	S_FS_1	1.02	349	2	65
S3.3	S_FS_2	1.38	489	2	58
S4.1	J6	1.65	580	5	0
S4.2	S_FS_3	0.79	268	2	70
S4.3	S_FS_4	1.91	657	2	65
S4.4	J9	2.40	839	2	69

**Table 4-5. Widths of the filter strip subcatchments**

Filter Strip	Subcatchment Controlled	Composition	Partial Widths (ft)	Total Widths (ft)
S_FS_1	S3.2	FS 1	410	410
S_FS_2	S3.3	FS 2 + FS 3 + part FS 7	105 + 250 + 108	463
S_FS_3	S4.2	Part FS 7	267	267
S_FS_4	S4.3	FS 4 + FS 5 + FS 6	359 + 190 + 345	894



**Figure 4-5. Representation of subcatchments S\_FS\_1, S3.1, and S3.2**



**Figure 4-6. Representation of subcatchments S\_FS\_3, S\_FS\_4, S4.1, S4.2, and S4.3**

Table 4-6 summarizes the characteristics of the subcatchments used to model the filter strips in this example. The areas of the filter strips are calculated from the lengths of the strips measured in the model and the flow path length (4 ft) used in this example. This

means the areas calculated by *Auto-Length* for these subcatchments will be replaced with those calculated in Table 4-6.

**Table 4-6. Properties of the filter strip subcatchments**

Subcatchment	Upstream Subcatchment	Outlet	Area <sup>1</sup> (ft <sup>2</sup> )	Area (ac)	Width <sup>2</sup> (ft)	Slope (%)	Depression <sup>3</sup> Storage (in)
S_FS_1	S3.2	J15	1640	0.038	410	2	0.3
S_FS_2	S3.3	J17	1852	0.043	463	2	0.3
S_FS_3	S4.2	J16	1068	0.025	267	2	0.3
S_FS_4	S4.3	J16	3576	0.082	894	2	0.3

Once the filter strip properties shown in Tables 4-5 and 4-6 are added to the model, all filter strips are assigned a **zero** percent imperviousness, an impervious roughness of **0.015** and an impervious depression storage of **0.06** in (although neither of the latter properties are used due to 0% imperviousness); a pervious roughness of **0.24**, and a pervious depression storage of **0.3** in. These last two values are the same as those for the rest of the pervious areas within the watershed. Finally, both the maximum and minimum infiltration rates for Horton infiltration are set to the same value, **0.2** in/hr, which is the minimum infiltration rate of the soil in the study area. This is a conservative approach that takes into account possible losses to infiltration capacity as well as the fact that soil may be saturated at the beginning of a new storm.

All subcatchments must have a rain gage assigned to them in order for the SWMM model to run. However, no rain should fall on the subcatchments representing the filter strips because their areas are considered part of the subcatchments within which they were placed. Thus, a new Time Series called “*Null*” is defined with a single time and rain value equal to zero. A new rain gage also called “*Null*” is created and linked to the time series “*Null*”. This gage is defined to be the rain gage of all the filter strip subcatchments, while the subcatchments that generate runoff are connected to the original rain gage “*RainGage*”, as in the previous examples.

## 4.4 Model Setup – Infiltration Trenches

The infiltration trenches (IT) included in this example are located in the north part of the study area (see Figure 4-1). To model these devices subcatchments *S1* and *S2* are first divided into six smaller subcatchments as shown in Figure 4-7. Their areas are determined automatically by using the SWMM’s *Auto-Length* tool. Their widths are based on an assumed urban runoff length of 125 ft (measured from the back of the lot to the street), as discussed in Example 1. The imperviousness of each newly sub-divided subcatchment is based on the type of land use (*M*, *L* or *DL*) as listed in Table 1-6 of Example 1. Table 4-7 summarizes the properties of the newly sub-divided subcatchments. The remaining properties not shown in the table (Horton infiltration, pervious depression storage, etc.) are the same as those given to the original subcatchments.



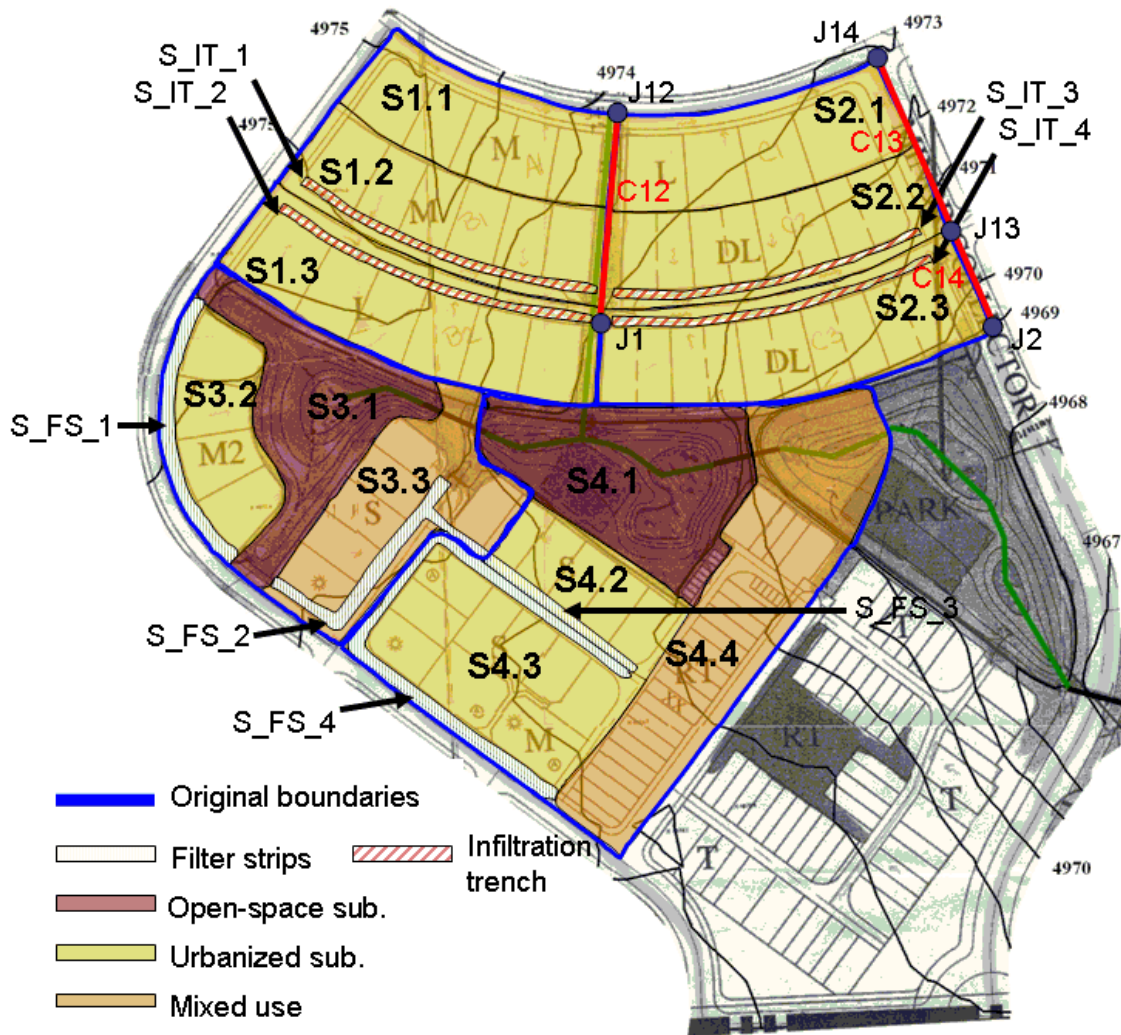


Figure 4-7. Re-discretization of subcatchments S1 and S2 for infiltration trenches

Table 4-7. Properties of the subcatchments derived from S1 and S2

New Subcatchment	Outlet	Area (ac)	Width (ft)	Slope (%)	Imperviousness (%)
S1.1	J12	1.21	422	2	65
S1.2	S_IT_1	1.46	509	2	65
S1.3	S_IT_2	1.88	655	2	45
S2.1	J14	1.30	453	2	45
S2.2	S_IT_3	1.50	523	2	70
S2.3	S_IT_4	1.88	655	2	70

The infiltration trenches added to the model are identified in Table 4-8. Their names begin with the letters **IT** followed by a number. The location of these infiltration trenches is shown in Figure 4-7. The four infiltration trenches are added into the model as

subcatchments and have an **S\_** prefix added to their name. They serve as the outlets of subcatchments *S1.2*, *S1.3*, *S2.2* and *S2.3* as shown in Table 4-9. The outlets of the other new subcatchments, *S1.1* and *S2.1*, are junctions *J12* and *J14*, respectively.

When sizing the infiltration trenches, note that the definition of length and width is opposite of the manner in which length and width were defined for filter strips. The length of each infiltration trench is listed in Table 4-8. The trench area can be computed using this length and assuming a width of 3 ft. The widths and areas of each infiltration trench are listed in Table 4-9. Each trench is modeled as having a depression storage depth of 1.2 ft (below this depth water is infiltrated, while above this depth there is flow along the trench). The actual storage depth of the infiltration trenches is 3 ft, but when the 40% porosity of the 1-1/2 in. diameter gravel filling the trenches is considered, the depth of the trenches available to store the water to be infiltrated is reduced to 40% of the actual trench depth. Table 4-9 summarizes the properties assigned to the infiltration trench subcatchments.

**Table 4-8. Subcatchments containing infiltration trenches**

Subcatchment Controlled	Infiltration Trench	Trench Length (ft)
S1	IT 1	450
S1	IT 2	474
S2	IT 3	450
S2	IT 4	470

**Table 4-9. Properties of the infiltration trench subcatchments**

Subcatchment	Upstream Subcatchment	Outlet	Area <sup>1</sup> (ft <sup>2</sup> )	Area <sup>1</sup> (ac)	Width (ft)	Slope <sup>2</sup> (%)	Depression <sup>3</sup> Storage (in)
S_IT_1	S1.2	J1	1350	0.031	3	0.422	14.4
S_IT_2	S1.3	J1	1422	0.033	3	0.422	14.4
S_IT_3	S2.2	J13	1350	0.031	3	0.444	14.4
S_IT_4	S2.3	J13	1410	0.032	3	0.468	14.4

<sup>1</sup> Area = length of the trench × 3 ft width perpendicular to the flow direction.

<sup>2</sup> Slope as computed from the site map.

<sup>3</sup> This corresponds to the effective depth of the trench, 0.4 x 36 in.

The slopes of the infiltration trenches are calculated directly from the site map, their imperviousness is **0%**, their Manning’s roughness coefficient is **0.24**, and their storage depths are accounted for by setting their depression storage to the effective storage depth of **14.4** in (1.2 ft). As was done for the filter strips, a constant (minimum) soil infiltration capacity of **0.2** in/hr will be used for the infiltration trenches. This ignores any horizontal infiltration that might occur through the sides of the trench. Also the rain gage assigned to each trench subcatchment is the newly created “*Null*” rain gage so that no rainfall occurs directly over the trench’s area.

This trench design assumes there is no barrier affecting the flow of water into the trench. A layer of grass and soil above the 1-1/2 in. diameter gravel, for example, might slow the rate of flow into trench; causing a smaller quantity of flow to be treated by the trench. This will be touched on again in the results section of this example.

To complete the model it is necessary to define the additional channels and junctions that connect the newly added subcatchments to the drainage system. These new elements are illustrated in Figure 4-7 (conduits in red and junctions in blue). Table 4-10 lists the properties of the new junctions and Table 4-11 does the same for the new conduits.

**Table 4-10. Properties of the new junctions**

New Junction	Invert Elevation (ft)
J12	4973.8
J13	4970.7
J14	4972.9

**Table 4-11. Properties of the new conduits**

New Conduit	Inlet Node	Outlet Node	Length (ft)	Type of Section <sup>1</sup>	Manning Coefficient	Maximum Depth (ft)	Bottom Width (ft)	Left Slope	Right Slope
C12	J12	J1	281.7	Swale	0.05	3	5	5	5
C13	J14	J13	275.5	Gutter	0.016	1.5	0	0.0001	25
C14	J13	J2	157.48	Gutter	0.016	1.5	0	0.0001	25

<sup>1</sup> Type of section based on sections defined in Example 2

## 4.5 Model Results

The final model with all LIDs included can be found in the file **Example4.inp**. It was run under *Dynamic Wave* flow routing using a wet runoff time step of **1** minute, a reporting time step of **1** minute, and a routing time step of **15** s for each of the three design storms. Figure 4-8 compares the resulting influent and effluent runoff hydrographs for filter strip *S\_FS\_1* for each of the design storms. Figure 4-9 does the same for infiltration trench *S\_IT\_1*. Results for the other LIDs look similar to these. Tables 4-12 and 4-13 list runoff coefficients for each filter strip and infiltration trench, respectively. As used here, the runoff coefficient is the ratio of the effluent-runoff volume flowing out of the LID to the influent-runoff volume flowing into the LID. The fractional reduction in runoff volume provided by the LID is simply 1 minus the runoff coefficient.



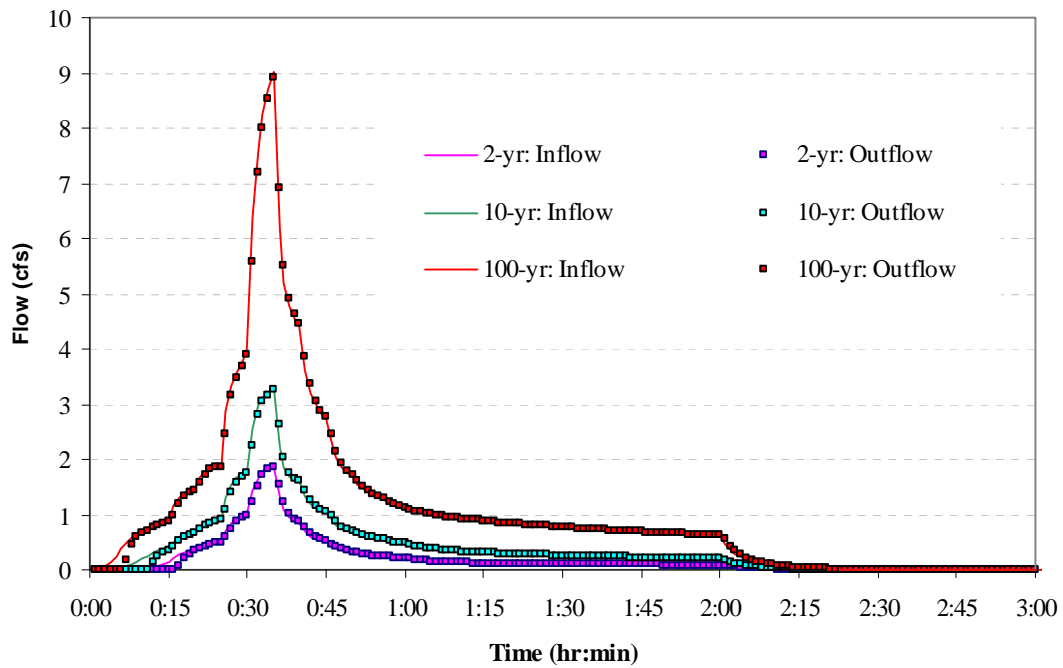


Figure 4-8. Influent and effluent hydrographs for filter strip *S\_FS\_1*

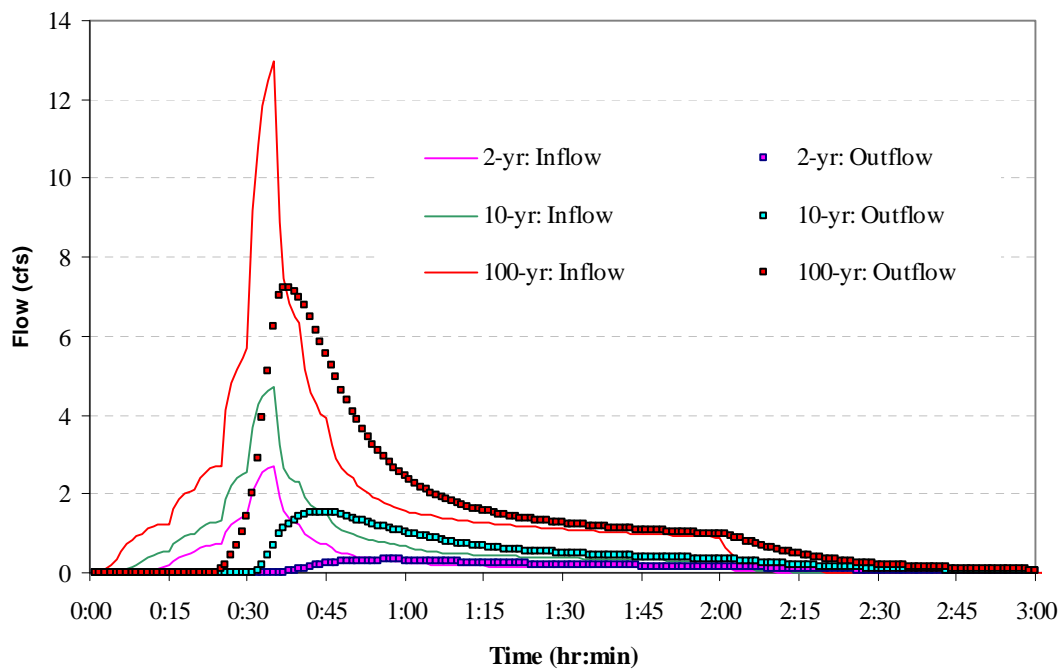


Figure 4-9. Influent and effluent hydrographs for infiltration trench *S\_IT\_1*

**Table 4-12. Runoff coefficients for filter strips**

Filter Strip	Runoff Coefficient		
	2-year Storm	10-year Storm	100-year Storm
S_FS_1	0.95	0.98	0.99
S_FS_2	0.95	0.98	0.99
S_FS_3	0.96	0.98	0.99
S_FS_4	0.94	0.97	0.99

**Table 4-13. Runoff coefficients for infiltration trenches**

Infiltration Trench	Runoff Coefficient		
	2-year Storm	10-year Storm	100-year Storm
S_IT_1	0.45	0.73	0.89
S_IT_2	0.35	0.71	0.90
S_IT_3	0.51	0.75	0.90
S_IT_4	0.59	0.79	0.92

It is apparent that filter strips provide negligible runoff control, with the outflow rate very nearly equaling the inflow rate for all storm magnitudes. This confirms what was mentioned in section 4.2 regarding the utility of filter strips, i.e. they are primarily pollutant removal devices and provide no benefit in controlling runoff flow rates or volumes. In contrast, all of the infiltration trenches provide significant reductions in runoff volume, particularly for the smaller rainfall events. It should be remembered, however, that the trenches in this example do not have grasses planted above the gravel backfill. Adding such a vegetative layer to the trench may reduce its effectiveness, depending on how it is designed.

Figure 4-10 compares the discharges simulated at the outlet of the study area for each design storm (2-, 10- and 100-yr return period) both with and without LIDs. For each design storm LIDs reduce both runoff volumes and peak discharges. As the storm event becomes larger, LIDs become less effective and the attenuation of their volumes and peak-discharges is reduced. These percent reductions in outlet volumes and peaks are compared in Figure 4-11. It shows how the benefit of LID controls decrease with increasing size of storm.

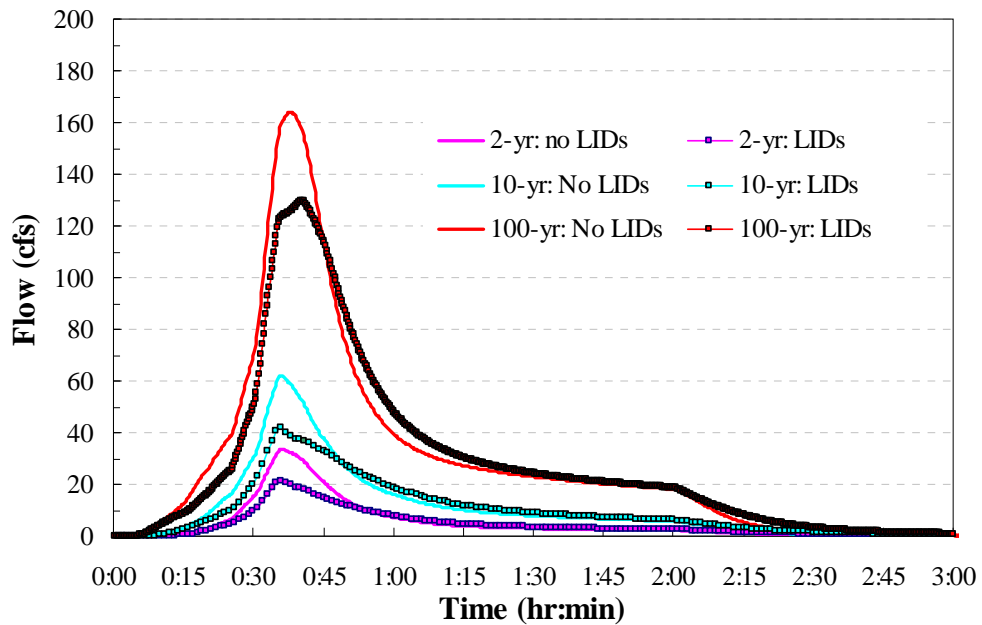


Figure 4-10. Comparison of outlet discharges with and without LID controls

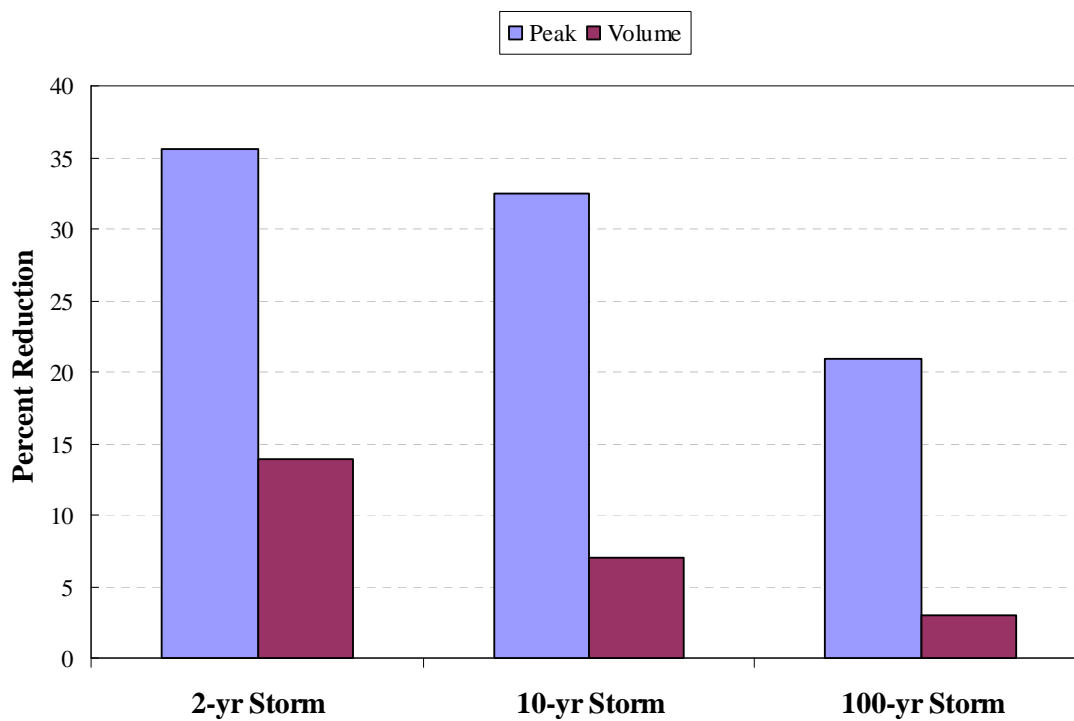


Figure 4-11. Percent reduction in outlet peak flows and runoff volumes with LIDs

## 4.6 Summary

This example illustrated how SWMM can be used to evaluate two types of Low Impact Development (LID) alternatives, filter strips and infiltration trenches. The key points illustrated in this example were:

1. A filter strip can be modeled as a rectangular, 100% pervious subcatchment with a constant infiltration rate.
2. An infiltration trench can be modeled as a rectangular, 100% pervious subcatchment with a constant infiltration rate whose depression storage is the effective pore volume depth of the trench.
3. Modeling these types of LIDs can require a finer level of subcatchment discretization to properly account for their localized placement.
4. Infiltration trenches (without a top soil layer) are more effective than filter strips in reducing runoff volumes and peaks.
5. The effectiveness of LIDs at reducing runoff volumes and peaks decreases with increasing size of storm event.

Although this example used a series of design storms to evaluate LID performance, a more accurate estimate of their stormwater control capabilities would require that a continuous long-term simulation be run for the site. Using several years' worth of actual rainfall inputs would allow the model to properly account for the variation in antecedent soil conditions between storm events. This factor becomes a critical concern when infiltration-based controls are being considered. Example 9 in this manual illustrates how to perform such a continuous simulation.

## Example 5. Runoff Water Quality

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This example demonstrates how to simulate pollutant buildup and washoff in an urban catchment. The influence of different land uses on pollutant buildup is considered and both Event Mean Concentrations (EMCs) and exponential functions are used to represent the washoff process.

Surface runoff quality is an extremely important, but very complex, issue in the study of wet-weather flows and their environmental impacts. It is difficult to accurately represent water quality within watershed simulation models because of a lack of understanding of the fundamental processes involved as well as a lack of sufficient data needed for model calibration and validation. SWMM has the ability to empirically simulate nonpoint source runoff quality as well as water quality treatment (an example of which will be shown in Example 6). It provides a flexible set of mathematical functions that can be calibrated to estimate both the accumulation of pollutants on the land surface during dry weather periods and their release into runoff during storm events. The same study area used in Examples 1 through 4 will be used to illustrate how these functions can be applied to a typical urban catchment.

### 5.1 Problem Statement

The 29 acre urban catchment and drainage system presented in Example 2 will be extended to include water quality modeling. Pollutant buildup, washoff and routing will be simulated in order to estimate the quality of the water released at the catchment outlet under post-development conditions with no runoff controls applied (meaning no BMPs or flow detention in the system). The study area site is shown in Figure 5-1 and the input file that will be modified to include water quality is named **Example2-Post.inp**.

Examination of long precipitation records reveals that most storms are quite small. For instance, in Example 3 the water quality capture volume (WQCV) of a detention pond located in the Colorado high-plains near the foothills was estimated to be only 0.23 inches. (See Example 3 for a methodology to calculate the WQCV in other areas in the country.) This volume corresponds to a depth that is exceeded by only 1 in 4 storms and is only 25% of the 2-yr design storm that was used in the previous examples (1.0 in). Therefore, small-sized, frequently occurring storms account for the predominant number of recorded events. It is these storms that result in significant portions of stormwater runoff and pollutant loads from urban catchments (UDFCD, 2001).

To explore the effect of storm volume on pollutant loading, this example will compute runoff loads produced by two smaller-sized 2-hour storms with volumes of 0.1 in. and 0.23 in. respectively. These loadings will be compared against those generated from the 2-year design event storm used in the previous examples whose volume is 1.0 in. The time series of intensities at five minute intervals for each of the two smaller storms are shown in Table 5-1.

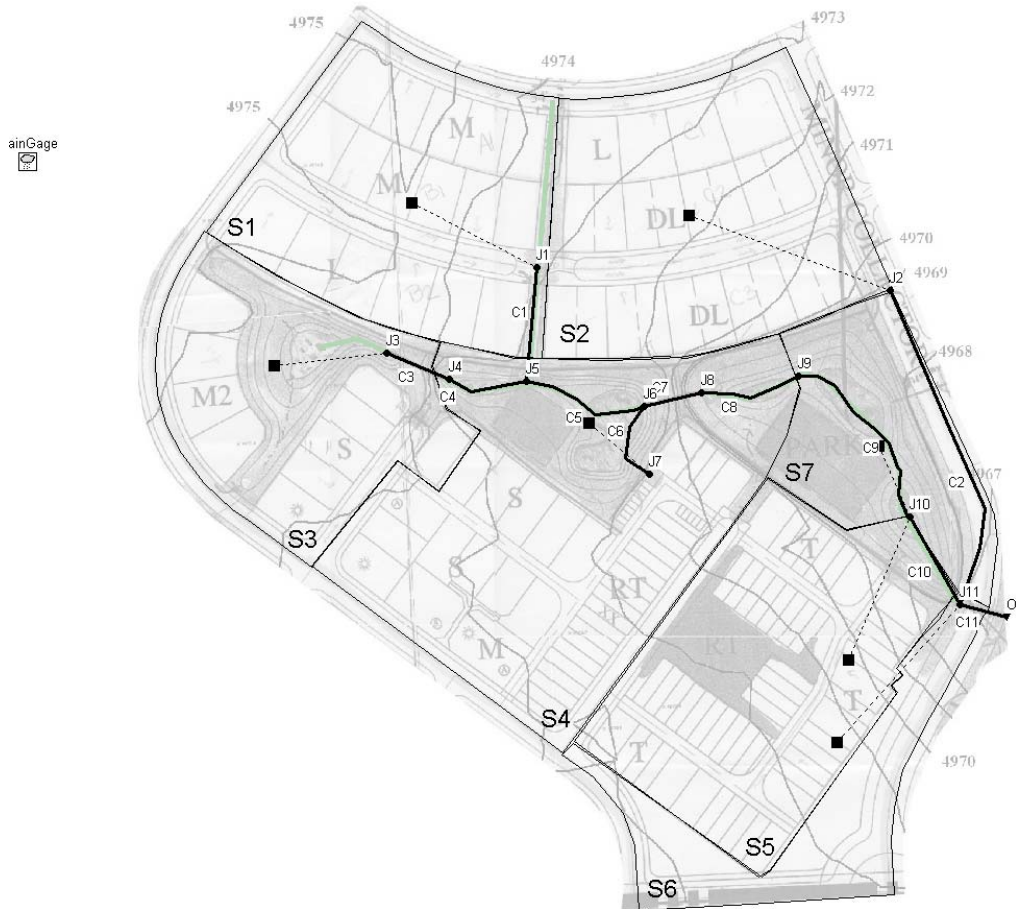


Figure 5-1. Post-development site with no runoff controls

Table 5-1. Rainfall time series for the 0.1 and 0.23 inch events

Time (min)	0.1 in Storm (in./h)	0.23 in Storm <sup>1</sup> (in./h)	Time (min)	0.1 in Storm (in./h)	0.23 in Storm <sup>1</sup> (in./h)
0:00	0.030	0.068	1:00	0.020	0.047
0:05	0.034	0.078	1:05	0.019	0.045
0:10	0.039	0.089	1:10	0.018	0.042
0:15	0.065	0.150	1:15	0.017	0.040
0:20	0.083	0.190	1:20	0.017	0.040
0:25	0.160	0.369	1:25	0.016	0.038
0:30	0.291	0.670	1:30	0.015	0.035
0:35	0.121	0.277	1:35	0.015	0.035
0:40	0.073	0.167	1:40	0.014	0.033
0:45	0.043	0.099	1:45	0.014	0.033
0:50	0.036	0.082	1:50	0.013	0.031
0:55	0.031	0.071	1:55	0.013	0.031

<sup>1</sup> 0.23 in. corresponds to the WQCV.

These new hyetographs are defined in SWMM using its *Time Series Editor*. The names of the new rainfall series will be *0.1-in* and *0.23-in*, respectively. They will be used by the model's single rain gage in addition to the 2-yr, 10-yr and 100-yr storms used in previous examples. This example will only employ the 2-yr storm along with the 0.1 in. and 0.23 in. events.

## 5.2 System Representation

SWMM employs several specialized objects and methods to represent water quality in urban runoff. These tools are very flexible and can model a variety of buildup and washoff processes, but they must be supported by calibration data to generate realistic results. The following is a brief description of the objects and methods used by SWMM to model water quality.

- **Pollutants**

Pollutants are user-defined contaminants that build up on the catchment surface and are washed off and transported downstream during runoff events. SWMM can simulate the generation, washoff and transport of any number of user-defined pollutants. Each defined pollutant is identified by its name and concentration units. Pollutant concentrations in externally applied water sources can be added directly to the model (e.g. concentrations in rain, groundwater and inflow/infiltration sources). Concentrations generated by runoff are computed internally by SWMM. It is also possible to define a dependency between concentrations of two pollutants using the co-pollutant and co-fraction options (e.g., lead can be a constant fraction of the suspended solids concentration).

- **Land Uses**

Land uses characterize the activities (e.g. residential, commercial, industrial, etc.) within a subcatchment that affect pollutant generation differently. They are used to represent the spatial variation in pollutant buildup/washoff rates as well as the effect of street cleaning (if used) within a subcatchment. A subcatchment can be divided into one or more land uses. This division is done independently of that used for pervious and impervious sub-areas, and all land uses in the subcatchment are assumed to contain the same split of pervious and impervious area. The percentages of named land uses assigned to a subcatchment do not necessarily have to add up to 100. Any remaining area not assigned a land use is assumed to not contribute to the pollutant load.

- **Buildup**

The buildup function for a given land use specifies the rate at which a pollutant is added onto the land surface during dry weather periods which will become available for washoff during a runoff event. Total buildup within a subcatchment is expressed as either mass per unit of area (e.g., lb/acre) or as mass per unit of curb length (e.g., lb/mile). Separate buildup rates can be defined for each pollutant and land use. Three options are provided in SWMM to simulate buildup: the power function, the

exponential function and the saturation function. The mathematical representation of each function is described in the *SWMM 5 Users Manual* (Rossman, 2008). These formulations can be adapted, by using the proper parameters, to achieve various kinds of buildup behavior, such as a linear rate buildup or a declining rate buildup.

Defining an initial pollutant loading over the subcatchment is an alternative to using a buildup function for single event simulations. Initial loading is the amount of a pollutant over the subcatchment, in units of mass per unit area, at the beginning of a simulation. This alternative is more easily adapted to single-event simulations and overrides any initial buildup computed during the antecedent dry days.

- **Washoff**

Washoff is the process of erosion, mobilization, and/or dissolution of pollutants from a subcatchment surface during wet-weather events. Three choices are available in SWMM to represent the washoff process for each pollutant and land use: event mean concentrations (EMCs), rating curves and exponential functions (see the *SWMM 5 Users Manual* for mathematical representations). The main differences between these three functions are summarized below.

- EMC assumes each pollutant has a constant runoff concentration throughout the simulation.
- Rating curves produce washoff loads that are functions of the runoff rate only, which means that they simulate the same washoff under the same discharge, regardless of the time in the storm that the discharge occurs.
- Exponential curves differ from rating curves in that the washoff load is a function not only of the runoff rate but also the amount of pollutant remaining on the watershed.
- Buildup functions are not required when EMCs or rating curves are used to represent the pollutant concentrations. If buildup functions are used, regardless of washoff function, buildup is continuously depleted as washoff proceeds, and washoff ceases when there is no more buildup remaining.
- Because rating curves do not use the amount of buildup remaining as a limiting factor, they tend to produce higher pollutant loads at the end of a storm event than do exponential curves which do take into account the amount of buildup remaining on the surface. This difference can be particularly important for large storm events where much of the buildup may be washed off in early stages.

After pollutants are washed off the subcatchment surface, they enter the conveyance system and are transported through the conduits as determined by the flow routing results. Here they may experience first-order decay or be subjected to reduction at specific nodes where treatment functions have been defined.

- **Pollutant Reduction from Land Surfaces**

Two procedures for reducing surface pollutant loads within subcatchments are available in SWMM. They are:



- **BMP Treatment:** This mechanism assumes that some type of BMP has been utilized in the subcatchment that reduces its normal washoff load by a constant removal fraction. BMP treatment will not be used in this example but will instead be illustrated in Example 6.
- **Street sweeping:** Street sweeping can be defined for each land use and is simulated in parallel with buildup prior to the beginning of the first storm event and in-between the next events. Street sweeping is defined by four parameters used to compute the pollutant load remaining on the surface at the start of a storm: (1) days between street sweeping, (2) fraction of the buildup that is available for removal by sweeping, (3) number of days since last sweeping at the start of the simulation and (4) street sweeping removal efficiency (in percent). These parameters are defined for each land use while the fourth one is defined for each pollutant as well.

### 5.3 Model Setup

Total Suspended Solids (TSS) will be the lone water quality constituent considered in this example. TSS is one of the most common pollutants in urban stormwater and its concentration is typically high. The U.S. EPA (1983) reported TSS EMCs in the range of 180 - 548 mg/L while the UDFCD (2001) reports values between 225 mg/L and 400 mg/L depending on the land use. Some of the receiving water impacts associated with this pollutant are habitat change, stream turbidity, and loss of recreation and aesthetics. The solids associated with TSS can also contain toxic compounds, such as heavy metals and adsorbed organics. The following paragraphs discuss how to modify the model built in Example 2 (file **Example2-Post.inp**) to consider the buildup, washoff, and transport of TSS within the post-development site.

#### Define the Pollutant

The first step is to define *TSS* as a new pollutant under the *Quality* category in SWMM's *Data Browser*. Its concentration units will be **mg/L**, and a small amount (**10** mg/L) is assumed to be present in rainwater. Concentrations in groundwater as well as a first order decay are not considered in this example, nor will any co-pollutant be defined for TSS.

#### Define Land Uses

Three different land uses will be considered in this example: *Residential\_1*, *Residential\_2* and *Commercial*. The *Residential\_1* land use will be used in residential areas with low and medium densities (lot types "L", "M" and "M2") while the *Residential\_2* land use will be used with high density apartments and duplexes (lot types "DL" and "S"). The *Commercial* land use will be used with lot types "T" and "RT".

Land uses are defined in SWMM under the *Quality* category in the *Data Browser*. Street sweeping is not considered in this example so sweeping parameters are not defined. A mixture of land uses will be assigned to each subcatchment area. This is done by opening the *Property Editor* for a given subcatchment, selecting the *Land Use* property and clicking the ellipsis button. A *Land Use Assignment* dialog will appear where one enters the percentage of surface area that is assigned to each land use. Percentages are estimated visually from the study area map. Table 5-4, shown later in this example, summarizes the assignment of land uses in each of the subcatchments.

### **Defining Pollutants and Land Uses**

**Pollutants:** Pollutants are defined in the Pollutant Editor of SWMM under the *Quality* category of the *Data Browser*. The minimum amount of data needed to define a new pollutant is a name and concentration units. Other characteristics include the pollutant's concentration in various external (non-buildup) sources (rainwater, groundwater, and RDII), its first-order decay coefficient ( $\text{day}^{-1}$ ) and name of a co-pollutant that its buildup is dependant upon.

Property	Value
Name	TSS
Units	MG/L
Rain Concn.	10
GW Concn.	0.0
I&I Concn.	0.0
Decay Coeff.	0.0
Snow Only	NO
Co-Pollutant	
Co-Fraction	

User-assigned name of the pollutant.

OK Cancel Help

**Land Uses:** Different land uses will generate pollutants at different rates. Land uses are defined in SWMM under the *Quality* category of the *Data Browser*. Their properties are edited using the *Land Use Editor* which is divided into three categories: *General*, *Buildup* and *Washoff*. The *General* tab contains the land use name and details on street sweeping for that particular land use. The *Buildup* tab is used to select a buildup function, and its parameters, for each pollutant generated by the land use. The choice of normalizer variable (total curb length or area) is also defined here. Finally, the *Washoff* tab is used to define the washoff function and its parameters, for each pollutant generated by the land use, as well as removal efficiencies for street cleaning and BMPs.

**General Tab:**

Property	Value
Land Use Name	Residential_1
Description	STREET SWEEPING
Interval	0
Availability	0
Last Swept	0

User assigned name of land use.

OK Cancel Help

**Buildup Tab:**

Property	Value
Pollutant	TSS
Function	EXP
Max. Buildup	0.11
Rate Constant	0.5
Power/Sat. Constant	0.0
Normalizer	CURB

Buildup function: POW = power, EXP = exponential, SAT = saturation.

OK Cancel Help

**Washoff Tab:**

Property	Value
Pollutant	TSS
Function	EMC
Coefficient	160
Exponent	0.0
Cleaning Effic.	0.0
BMP Effic.	0.0

Washoff function: EXP = exponential, RC = rating curve, EMC = event mean concentration.

OK Cancel Help

## Specify a Buildup Function

One of SWMM's buildup equations will be selected to characterize the accumulation of TSS during dry weather periods. Unfortunately, the choice of the best functional form is never obvious, even if data are available. Even though most buildup data in the literature imply a linear buildup with time, it has been observed that this linear assumption is not always true (Sartor and Boyd, 1972), and that the buildup rate tends to decrease with time. Thus, this example will use an exponential curve with parameters  $C_1$  (maximum buildup possible) and  $C_2$  (buildup rate constant) to represent the buildup rate  $B$  as a function of time  $t$ :

$$B = C_1(1 - e^{-C_2 t}) \quad (5-1)$$

Buildup data for TSS reveal that commercial and residential areas tend to generate similar amounts of the dust and dirt that comprise the TSS (again, there is a large variation for different cases). Similarly, high-density residential areas tend to produce more of this pollutant than low-density residential areas. Typical values of dust-and-dirt buildup rates based on a nationwide study by Manning et al. (1977) are shown in Table 5-2.

**Table 5-2. Typical dust and dirt buildup rates after Manning et al. (1977)**

Land Use	Mean (lb/curb-mi/day)	Range (lb/curb-mi/day)
Commercial	116	3 – 365
Multiple family residential	113	8 – 770
Single family residential	62	3 – 950

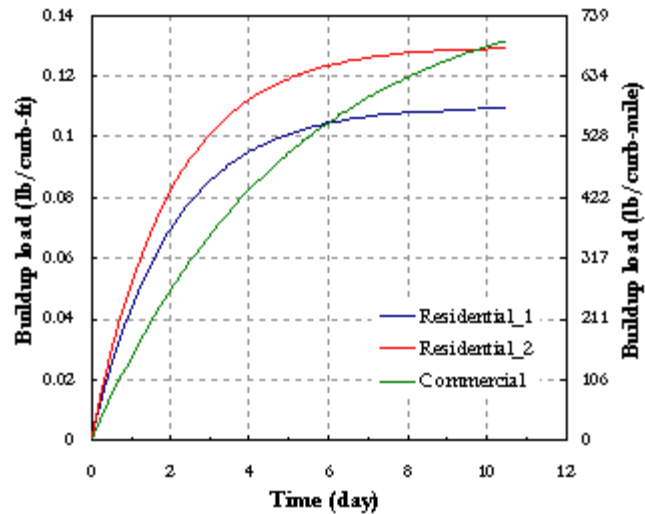
Table 5-3 shows the parameters  $C_1$  and  $C_2$  used in equation 5-1 for each land use defined earlier. A graphical representation of the exponential buildup model with these parameters is shown in Figure 5-2. In SWMM the buildup function and its parameters are defined for each land use on the “*Buildup*” page of the *Land Use Editor*. The *Buildup Function* used here is **Exp**, the constant  $C_1$  is entered in the field *Max. Buildup* and constant  $C_2$  is entered in the field *Rate Constant*. The field *Power/Sat. Constant* is not defined when the Exponential model is used.

The values of the parameters used in this TSS buildup function were obtained from the literature. No other justification supports their use and it is strongly recommended that modelers define them based on site data specific to their project.

Buildup in all the subcatchments will be normalized in this example by the curb length (typically there is more literature data per unit length of street/gutter than per unit area). This choice is specified for each land use in the *Land Use Editor*. Curb lengths can be estimated by using SWMM's *Ruler* tool to trace over the streets within the study area map (see the sidebar “*Measuring Tools Available in SWMM*” in Example 1). They should be similar to those listed in Table 5-4. These values are assigned to each subcatchment by using the *Property Editor*. The curb length units (e.g. feet or meters) must be consistent with those used for the buildup rate (e.g. lbs/curb-ft or kg/curb-m) in the *Land Use Editor*; do not mix units between the two systems.

**Table 5-3. Parameters for TSS buildup**

Land Use	C <sub>1</sub> (lb/curb-ft)	C <sub>2</sub> (1/day)
Residential_1	0.11	0.5
Residential_2	0.13	0.5
Commercial	0.15	0.2

**Figure 5-2. TSS buildup curve****Table 5-4. Curb length and land uses for each subcatchment**

Subcatchment	Curb Length (ft)	Residential_1 (%)	Residential_2 (%)	Commercial (%)
1	1680	100	0	0
2	1680	27	73	0
3	930	27	32	0
4	2250	9	30	26
5	2480	0	0	98
6	1100	0	0	100
7	565	0	0	0

Finally, in order to start the simulation with some initial buildup already present, it is assumed that there were **5** days of dry antecedent conditions before the start of the simulation. The program will apply this time interval to the TSS buildup functions to compute an initial loading of TSS over each subcatchment. The *Antecedent Dry Days* parameter is specified on the *General* page of the *Simulation Options* dialog in SWMM.

### Specify a Washoff Function

Two methods are used in this example to simulate washoff: EMCs and an exponential washoff equation. The following sections explain how these are added to the model.

### EMCs

An estimation of the EMCs can be obtained from the Nationwide Urban Runoff Program (NURP) conducted by EPA (U.S. EPA, 1983). According to this study, the *median TSS EMC* observed in urban sites is 100 mg/L. Based on the general observation that residential and commercial areas produce similar pollutant loads, and taking into account the differences among land uses, this example uses the EMCs shown in Table 5-5 at the end of this section. These EMCs are entered into the model using the *Land Use Editor's Washoff* page for each defined land use. The entry for the *Function* field is **EMC**, the concentration from Table 5-5 is entered in the *Coefficient* field and the remaining fields can be set to **0**. The resulting SWMM input file is saved as **Example5-EMC.inp**.

### Exponential Washoff

The exponential washoff function used in SWMM is:

$$W = C_1 \cdot q^{C_2} \cdot B \quad (5-2)$$

where:

- $W$  = rate of pollutant load washed off at time  $t$  in lbs/hr
- $C_1$  = washoff coefficient in units of  $(\text{in/hr})^{-C_2}(\text{hr})^{-1}$
- $C_2$  = washoff exponent
- $q$  = runoff rate per unit area at time  $t$ , in/hr
- $B$  = pollutant buildup remaining on the surface at time  $t$ , lbs.

According to sediment transport theory, values of the exponent  $C_2$  should range between 1.1 and 2.6, with most values near 2 (Vanoni, 1975). One can assume that commercial and high-density residential areas (land uses *Commercial* and *Residential\_2*), because of their higher imperviousness, tend to release pollutants faster than areas with individual lots (*Residential\_1*). Thus a value of **2.2** is used for  $C_2$  in the *Residential\_2* and *Commercial* land uses and **1.8** is used for the *Residential\_1* land use.

Values of the washoff coefficient ( $C_1$ ) are much more difficult to infer because they can vary in nature by 3 or 4 orders of magnitude. This variation may be less extreme in urban areas, but is still significant. Monitoring data should be used to help estimate a value for this constant. The current example assumes a  $C_1$  equal to **40** for *Residential\_2* and *Commercial* and a  $C_1$  equal to **20** for the land use *Residential\_1*.

Table 5-5 summarizes the  $C_1$  and  $C_2$  coefficients used for each land use under exponential washoff. These are entered into the model using the *Land Use Editor Washoff* page for each defined land use. The entry for the *Function* field is **EXP**, the  $C_1$  value from Table 5.5 is entered in the *Coefficient* field, and the  $C_2$  value from the table is entered into the *Exponent* field. The remaining fields can be set to **0**. The resulting SWMM input file is saved as **Example5-EXP.inp**.

**Table 5-5. Washoff characteristics for each land use**

Land Use	EMC (mg/L)	$C_1$ [(in/hr) <sup>-C<sub>2</sub></sup> sec <sup>-1</sup> ]	$C_2$
Residential_1	160	20	1.8
Residential_2	200	40	2.2
Commercial	180	40	2.2

## 5.4 Model Results

Both the EMC washoff model (**Example5-EMC.inp**) and the exponential washoff model (**Example5-EXP.inp**) were run for the 0.1 in., 0.23 in. and 2-yr rainfall events under the following set of analysis options:

*Simulation Period:* 12 hours  
*Antecedent Dry Days:* 5  
*Routing Method:* Dynamic Wave  
*Routing Time Step:* 15 seconds  
*Wet-weather Time Step:* 1 minute  
*Dry-weather Time Step:* 1 hour  
*Reporting Time Step:* 1 minute

A discussion of the results obtained from each model is next presented.

### EMC Washoff Results

Figures 5-3 and 5-4 show the runoff concentrations simulated at different subcatchments both for the 0.1 in. (Figure 5-3) and the 0.23 in. (Figure 5-4) storms. The concentrations are constant and correspond to the summation of the constant concentration in the rain (10 mg/L) and the EMCs assigned to the land uses within each subcatchment. Once the surface runoff ceases, the TSS concentration goes to zero. That is why no concentration is displayed for subcatchment S7, since it generates no runoff (all rainfall is infiltrated). Note that with EMC washoff, the size of the storm has no effect on a subcatchment's runoff concentration.

Figure 5-5 shows the TSS concentration over time (pollutograph) simulated at the study area outlet for each of the three storm events (0.1, 0.23, and 1.0 in.). The outlet concentration reflects the combined effect of the TSS washoff produced from each subcatchment and routing through the conveyance network. The peak-concentrations and shapes for the pollutographs are very similar. Compared with the washoff concentrations generated by the individual subcatchments (Figures 5-3 and 5-4), the outlet concentrations are not constant but attenuate over time. This attenuation is caused primarily by the longer time it takes runoff from the lower EMC subcatchments (such as S3 and S4) to reach the outlet. Some of it is also a result of the numerical dispersion in the model resulting from the assumption of complete mixing within each conveyance conduit during the pollutant routing process.

## How to Read the SWMM Status Report in Terms of Water Quality

The simulation of water quality generates added information in SWMM's *Status Report*. This information can be broken into four general sections (A, B, C and D). These additions to the status report are discussed below using **Example5-EMC.inp** with the 0.1 inch precipitation event as an example.

**Part A** shows the runoff-quality continuity balance over the entire study area. The "input" loads include (a) *Initial Buildup* before the start of the simulation, (b) *Surface Buildup* during all dry weather periods, and (c) *Wet Deposition* (from pollutant in the rainfall). The "output" loads include (1) *Sweeping Removal* (not simulated), (2) *Infiltration Loss* for any direct rainfall or runoff from other subcatchments (simulated automatically), (3) removal associated with *BMP Removal* (not simulated in this example), and (4) pollutant load in the *Surface Runoff* (which includes the portion of buildup that is washed off as well as any loads produced by direct deposition and runoff). Finally, the continuity report indicates the *Remaining Buildup*.

**Part B** shows the quality-routing continuity balance. In this example, only runoff loads are routed through the conveyance system. No dry weather, groundwater, RDII, or user-supplied external inflows, nor is any treatment or decay considered. Therefore, the only three variables represented in this summary are the *Wet Weather Inflow*, the *External Outflow* and the *Final Stored Mass*. Note that the *Wet Weather Inflow* in Part B is equal to the *Surface Runoff* in Part A.

**Part C** provides a summary of the load of pollutant washed off from each subcatchment. Subcatchments S7 and S3

EPA STORM WATER MANAGEMENT MODEL - VERSION 5.0 (Build 5.0.014- beta)

```
*****
Runoff Quality Continuity
*****
Initial Buildup ..... 913.850
Surface Buildup ..... 29.400
Wet Deposition ..... 6.533
Sweeping Removal ..... 0.000
Infiltration Loss ..... 2.272
BMP Removal ..... 0.000
Surface Runoff ..... 33.855
Remaining Buildup ..... 913.082
Continuity Error (%) ..... 0.060
```

**A**

```
*****
Quality Routing Continuity
*****
Dry Weather Inflow ..... 0.000
Wet Weather Inflow ..... 33.855
Groundwater Inflow ..... 0.000
RDII Inflow ..... 0.000
External Inflow ..... 0.000
Internal Flooding ..... 0.000
External Outflow ..... 33.355
Mass Reacted ..... 0.000
Initial Stored Mass ..... 0.000
Final Stored Mass ..... 0.071
Continuity Error (%) ..... 1.266
```

**B**

\*\*\*\*\*  
Subcatchment Washoff Summary  
\*\*\*\*\*

```
*****
Subcatchment      TSS
                  lbs
-----
S1                5.327
S2                7.213
S3                2.099
S4                5.389
S5                9.494
S6                4.333
S7                0.000
System            33.855
```

**C**

\*\*\*\*\*  
Outfall Loading Summary  
\*\*\*\*\*

Outfall Node	Flow Freq. Pcnt.	Avg. Flow CFS	Max. Flow CFS	Total Volume Mgal	Total TSS lbs
01	80.63	0.09	0.70	0.024	33.307
System	80.63	0.09	0.70	0.024	33.307

**D**

generate the lowest loads of TSS. S7 does not generate any load because it does not produce any runoff while S3 produces a smaller load due to a large amount of pervious surface in the subcatchment.

**Part D** shows the total loads leaving the system through its outfalls.

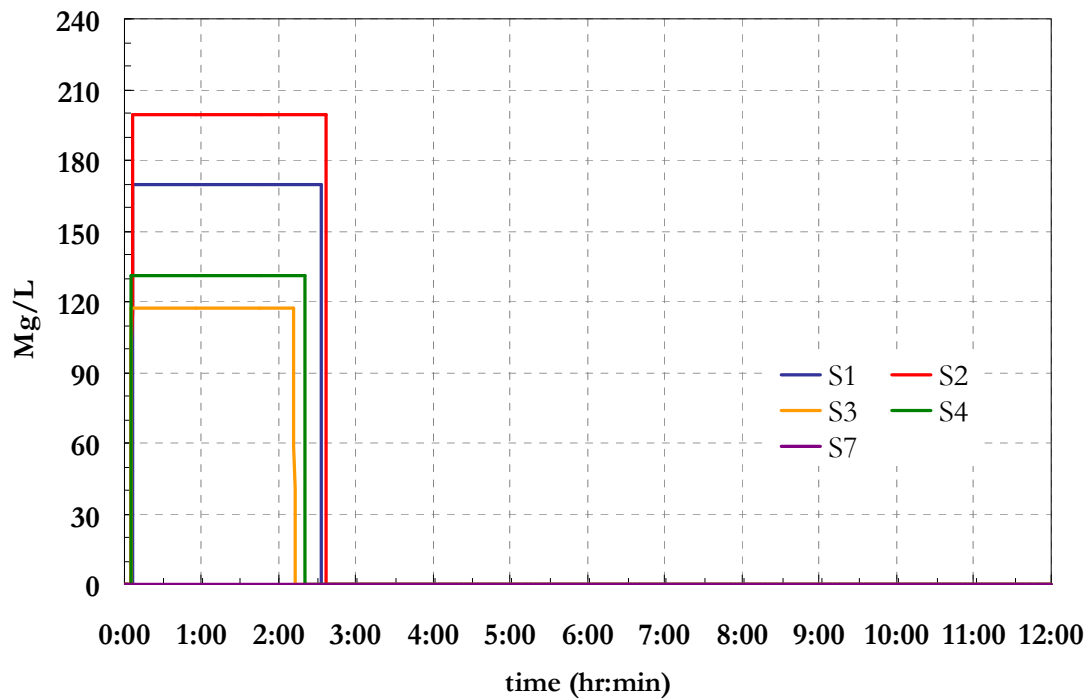


Figure 5-3. TSS concentrations for the 0.1 in. storm with EMC washoff

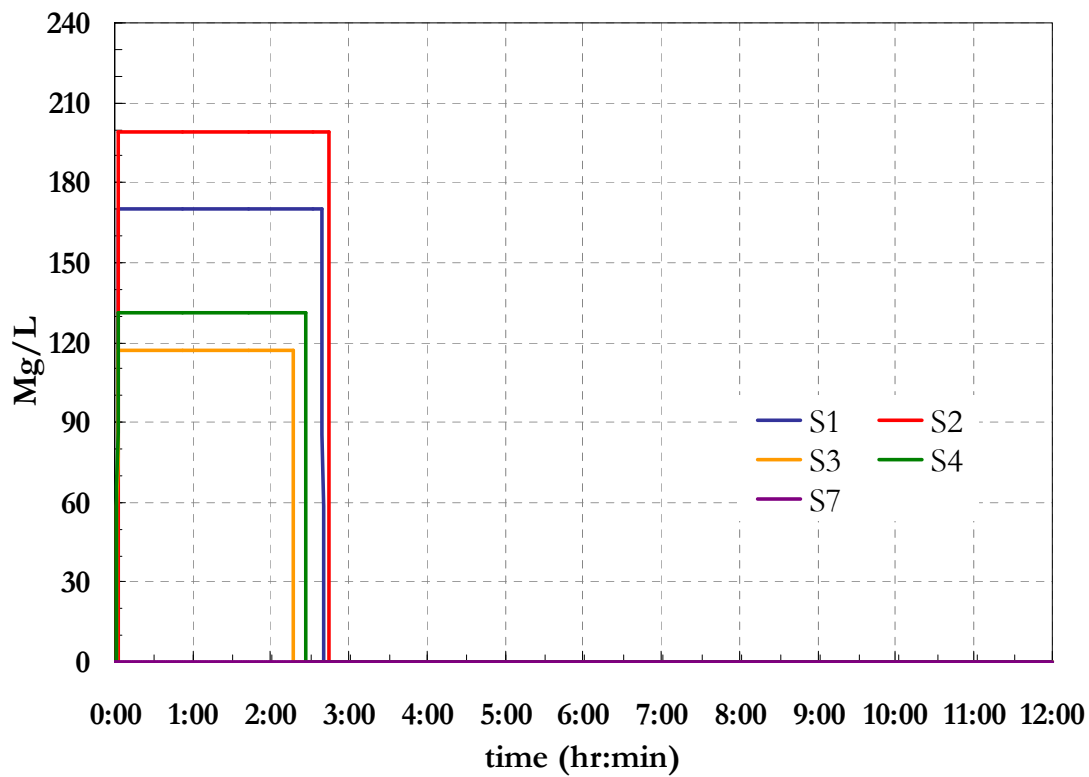
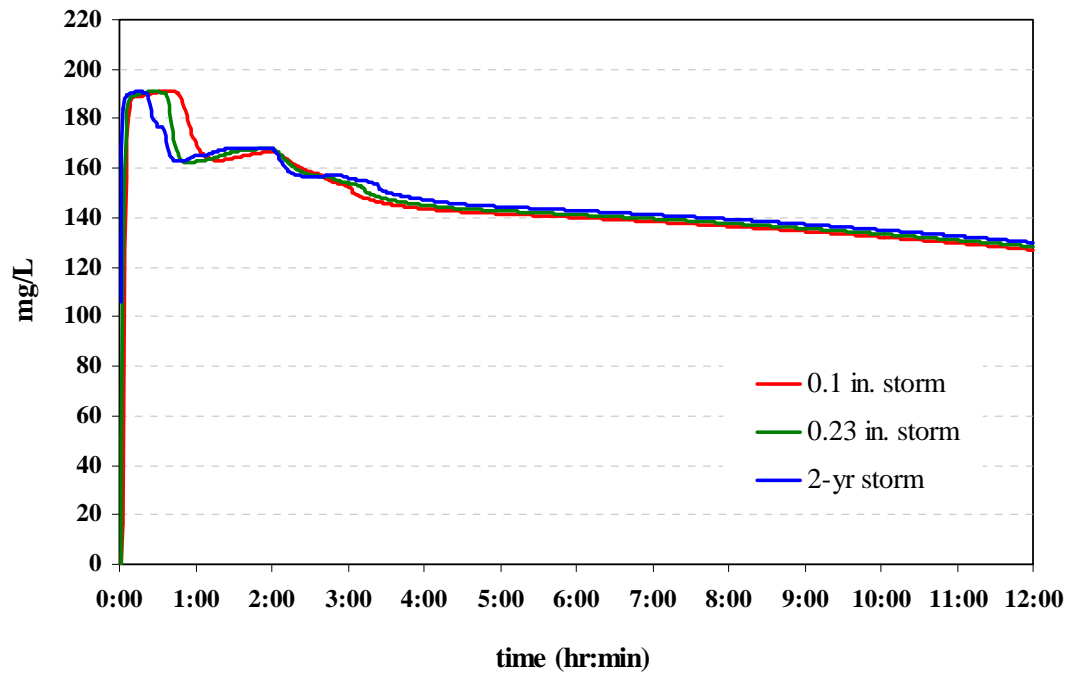


Figure 5-4. TSS concentrations for the 0.23 in. storm with EMC washoff





**Figure 5-5. TSS concentration at site outlet with EMC washoff**

Figure 5-5 also shows that TSS concentrations continue to appear at the outlet for an extended period of time after the end of the storm event. This is an artifact of the flow routing procedure wherein the conduits continue carry a very small volume of water whose concentration still reflects the high EMC levels. Thus although the concentrations appear high, the mass loads carried by these small discharges are negligible. This is evident when the outlet hydrograph is plotted alongside the outlet loadograph for a given storm. A loadograph is a plot of concentration times flow rate versus time. An example for the 0.1 in. event is shown in Figure 5-6. This plot was generated by exporting the time series table for Total Inflow and TSS concentration at the outfall node *O1* into a spreadsheet, using the spreadsheet to multiply flow and concentration together (and converting the result to lbs/hr), and then plotting both flow and load versus time. Note how the TSS load discharged from the catchment declines in the same manner as does the total runoff discharge.

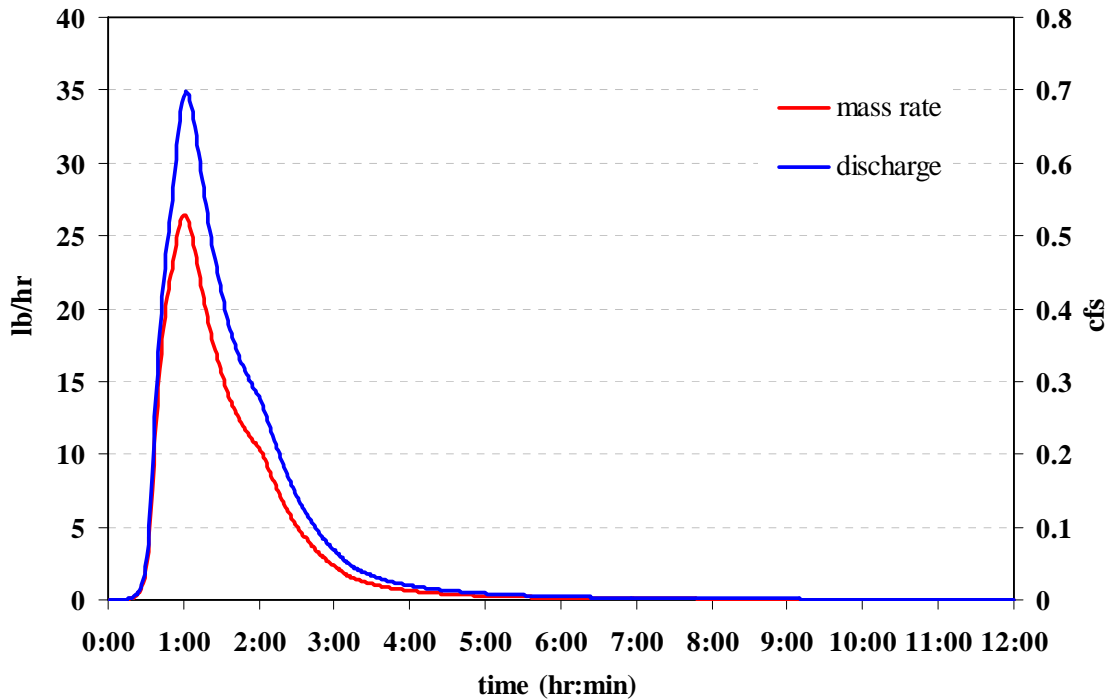


Figure 5-6. Runoff flow and TSS load at site outlet for the 0.1 in. storm with EMC washoff

### Exponential Washoff Results

Figure 5-7 shows the simulated TSS concentration in the runoff from different subcatchments using the 0.1 in. storm and the Exponential washoff equation. Unlike the EMC results, these concentrations vary throughout the runoff event and depend on both the runoff rate and the pollutant mass remaining on the subcatchment surface. Figure 5-8 shows the same plots but for the 0.23 in storm. Note two significant differences with respect to the results obtained for the 0.1 in storm. The maximum TSS concentrations are much larger (around 10 times) and the generation of TSS is much faster, as seen by the sharper-peaked pollutographs in Figure 5-8. Finally, Figure 5-9 shows the same graphs for the larger 1-in., 2-yr storm. The TSS concentrations are slightly larger than those for the 0.23 in. storm but the difference is much smaller than the difference between the 0.1 in and 0.23 in storms. Similar results hold for the pollutographs generated for the watershed's outlet as seen in Figure 5-10.

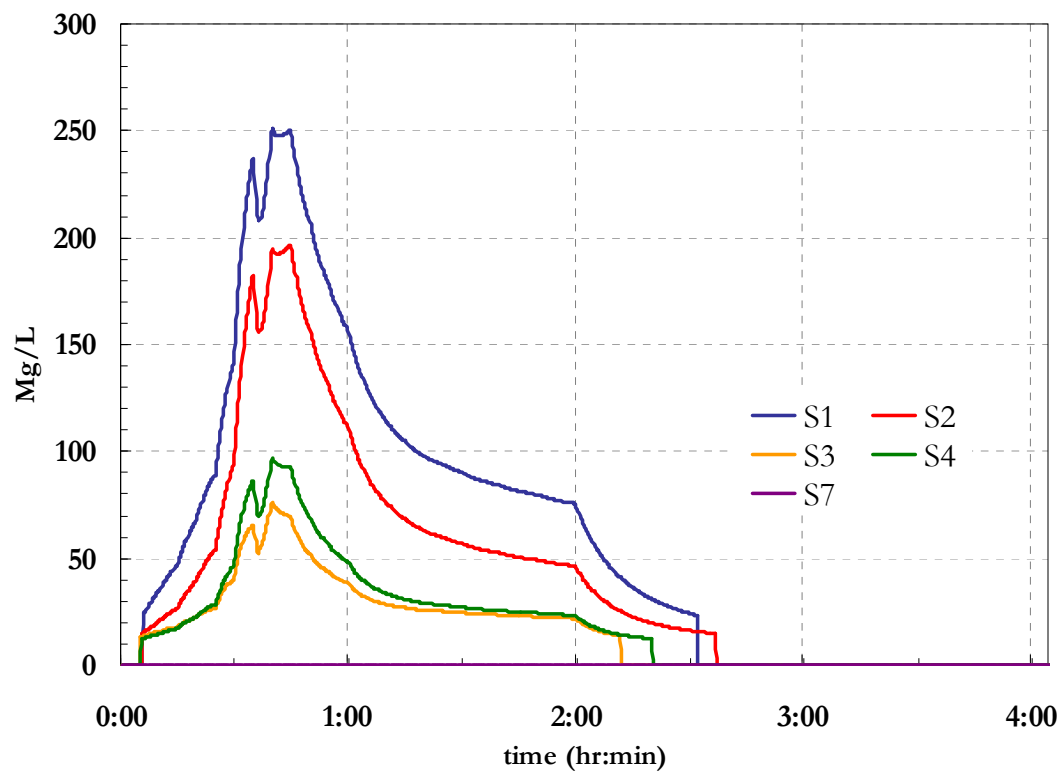


Figure 5-7. TSS concentrations for the 0.1 in. storm with Exponential washoff

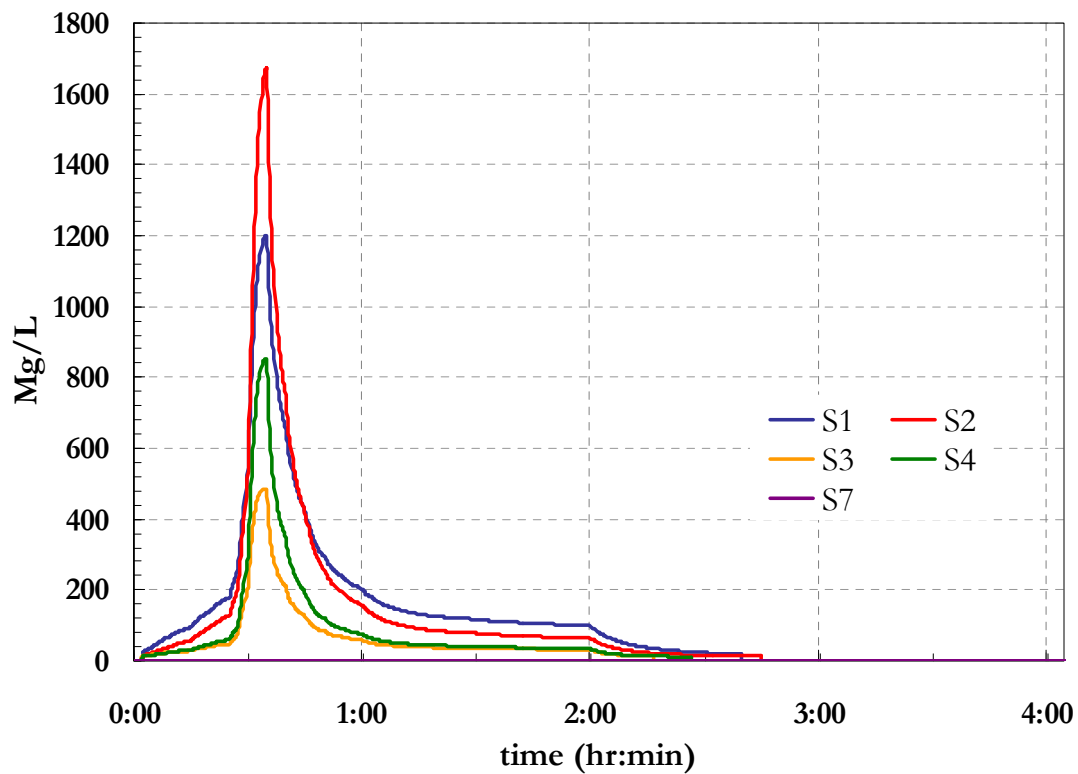


Figure 5-8. TSS concentrations for the 0.23 storm with Exponential washoff

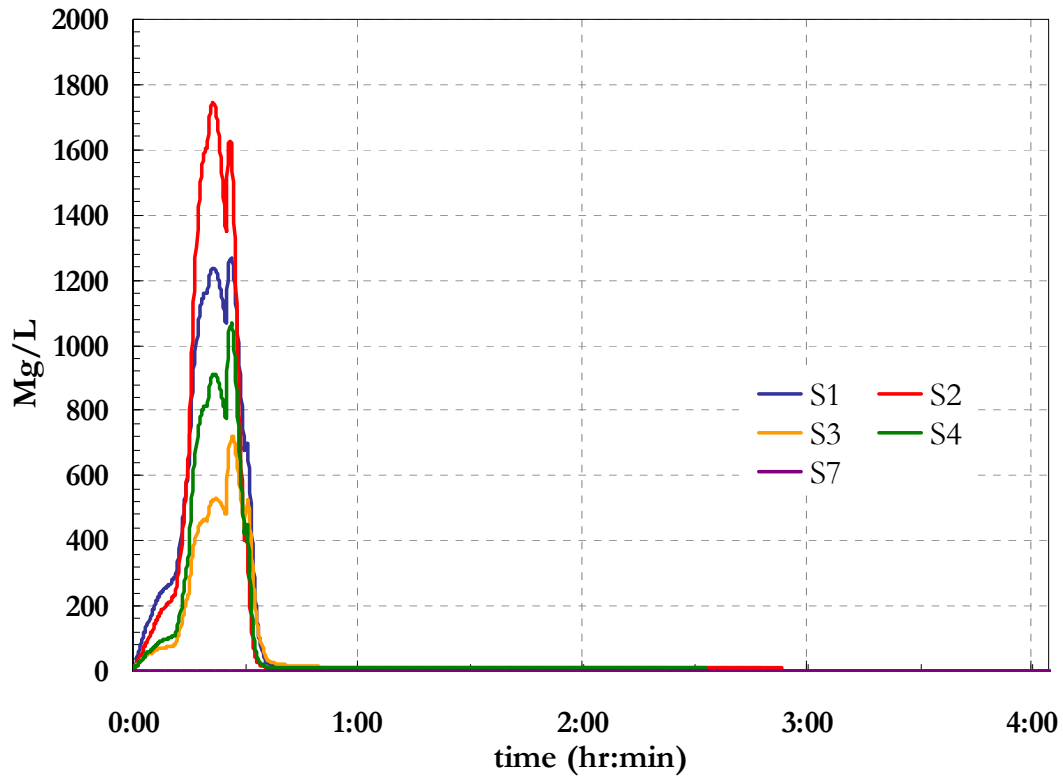


Figure 5-9. TSS concentrations for the 2-yr (1 in.) storm with Exponential washoff

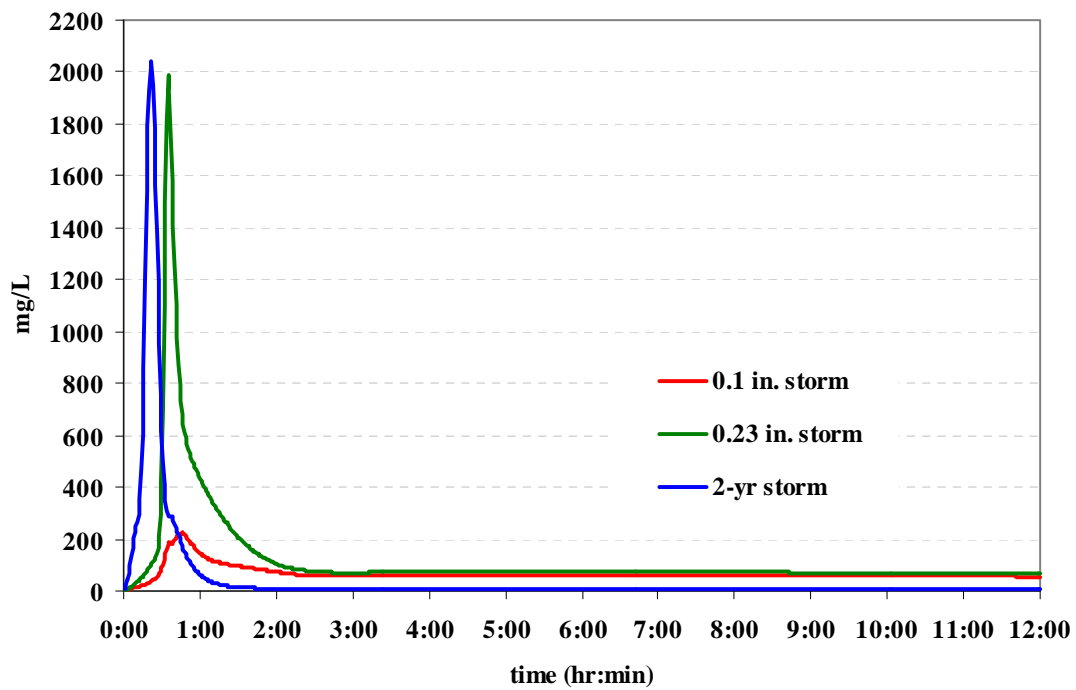


Figure 5-10. TSS concentration at the site outlet for Exponential washoff

Even though the EMC and Exponential washoff models utilize different coefficients that are not directly comparable, it is interesting to compute what the average event concentration in the runoff from each subcatchment was under the two models. The resulting averages are shown in Table 5-6 for the case of the 0.23 in. storm. The point being made here is that even though the pollutographs produced by the two models can look very different, with the proper choice of coefficients it is possible to get event average concentrations that look similar. Although the results of the Exponential model are more pleasing to one's sense of how pollutants are washed off the watershed, in the absence of field measurements one cannot claim that they are necessarily more accurate. Most SWMM modelers tend to use the EMC method unless data are available to estimate and calibrate the coefficients required of a more sophisticated buildup and washoff model.

**Table 5-6. Average TSS concentration for the 0.23 in. event**

Subcatchment	EMC Model (mg/L)	Exponential Model (mg/L)
S1	170	180.4
S2	199.2	163.6
S3	117.2	67.7
S4	131.2	91.4
S7	0	0

## 5.5 Summary

This example illustrated how SWMM is used to model the quality of stormwater runoff within an urban catchment without any source or regional BMP controls. One pollutant, TSS, was simulated with one buildup method (exponential) and two different washoff methods (EMC and exponential). The key points illustrated in this example were:

1. SWMM models runoff water quality through the definition of pollutants, land uses, pollutant buildup, and pollutant washoff. Any number of user-defined pollutants and land uses can be modeled. Pollutant buildup and washoff parameters are defined for each land use and more than one land use can be assigned to each subcatchment.
2. There are several options available to simulate both pollutant buildup and washoff. Buildup expressions are defined by a buildup rate and a maximum buildup possible per unit of area or curb length. Pollutant washoff can be defined through an event mean concentration (EMC), a rating curve, or an exponential function. The exponential method is the only one that directly depends on the amount of buildup remaining on the surface. Rating-curve calculations are dependant only on the runoff across the subcatchment, while EMCs have constant concentrations throughout the simulation.

3. Exponential washoff produces a runoff pollutograph with rising and falling limbs, similar to that of a runoff hydrograph. The EMC pollutograph is flat throughout the duration of the event.
4. Small storms can have a high impact on receiving waters because they are more frequent and can still generate significant washoff concentrations.

There are many uncertainties associated with both the process representation and the data required to properly estimate, calibrate and validate a runoff water quality model. It is strongly recommended that modelers use site specific data whenever possible when building a runoff water quality model with SWMM.

## Example 6. Runoff Treatment

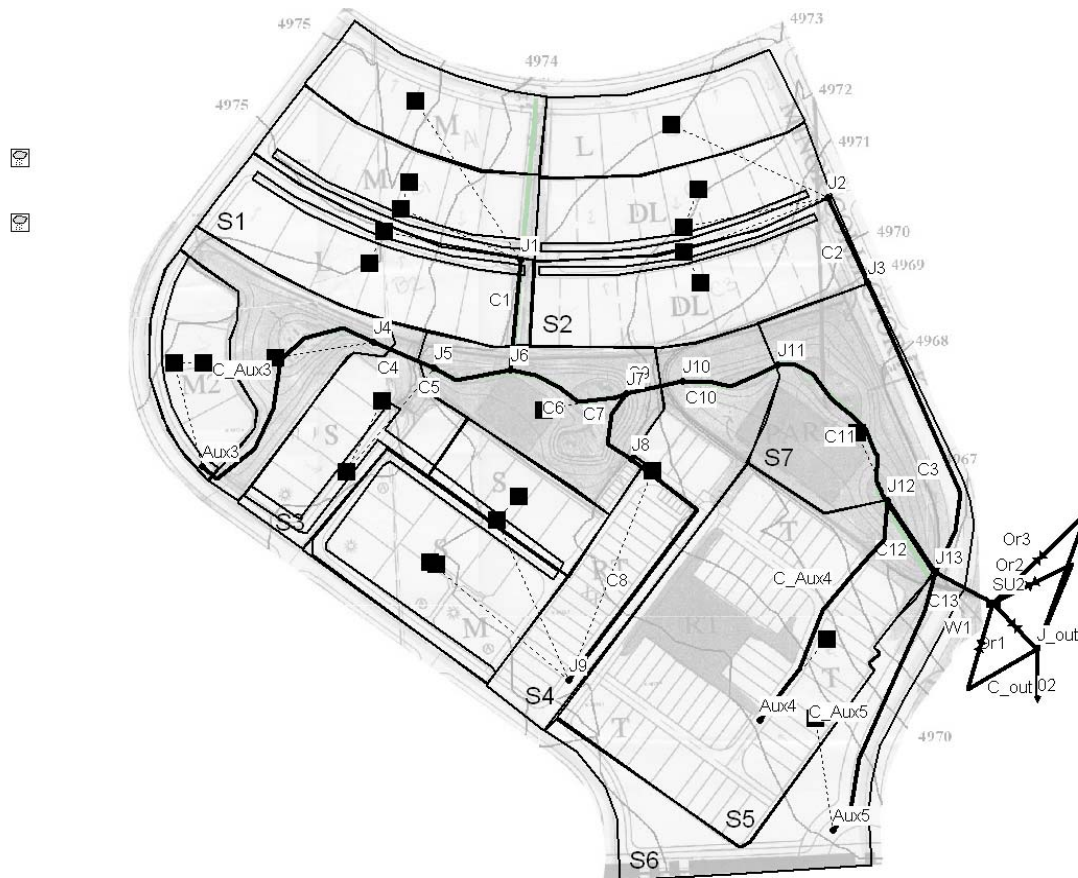
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This example illustrates how to model water quality treatment in the BMPs that were used in two earlier examples to control runoff from a new residential development on a 29 acre site. Treatment of total suspended solids (TSS) is applied at both the detention pond introduced in Example 3 and the filter strips and infiltration trenches added in Example 4. The pond from Example 3 is re-designed to a smaller volume for this example to account for the runoff reduction associated with the upstream infiltration trenches. TSS removal in the detention pond is modeled as an exponential function of time and water depth. TSS removal in the filter strips and infiltration trenches is a fixed percent reduction in loading.

### 6.1 Problem Statement

In Example 3 a regional detention pond was designed for a 29 acre residential site to detain a water quality capture volume (WQCV) for a specific period of time and to reduce peak runoff flows to their pre-development levels. Example 4 added two different types of distributed Low Impact Development (LID) source controls throughout the site to help reduce the volume of runoff generated. Example 5 illustrated how to model total suspended solids buildup and washoff within the site without considering any TSS removal that might occur within the LIDs or the pond. These previous models will be extended to explicitly account for the removal of TSS that occurs in both the LIDs and the pond. A comparison will be made of the TSS concentrations and loads in the runoff produced by the site both with and without considering treatment.

Figure 6-1 shows the example study area with the LIDs and detention pond included. The SWMM storage unit that represents the detention pond as designed in Example 3 was renamed to *SU2* and resized for this example to incorporate a smaller Water Quality Capture Volume (WQCV) due to the runoff reduction produced by the LIDs. It was concluded from Example 4 that of the two types of LIDs modeled, infiltration trenches have the greatest impact on reducing the volume of water that needs to be treated in the pond's WQCV. The total volume of water that can be captured by the infiltration trenches was determined to be 6,638 ft<sup>3</sup>. Because the WQCV is a requirement for the watershed as a whole, the volume captured in the infiltration trenches can be subtracted from the volume required in the regional pond, reducing it from the 24,162 ft<sup>3</sup> required in Example 3 to **17,524 ft<sup>3</sup>** (24,162 - 6,638).



**Figure 6-1. Developed site with LIDs and detention pond**

The storage unit's shape and outlet structures were redesigned to control this new WQCV and meet the general design criteria introduced in Example 3 (e.g. 40 hr drawdown time for the WQCV and peak shaving of the 2-, 10- and 100-yr storms). Table 6-1 compares the storage curve of the pond designed in Example 3 without LIDs and the pond designed for this example with LIDs. Table 6-2 does the same for the dimensions and inverts of the orifices and weirs that comprise the pond's outlet structure. Figure 6-2 shows the general placement of the various outlets.

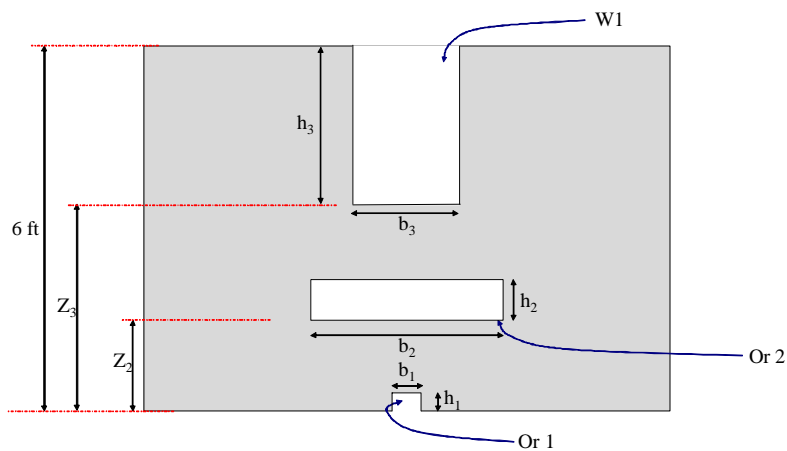
**Table 6-1. Storage curve for the re-designed pond**

This Example				
Depth (ft)	0	2.2	2.3	6
Area (ft <sup>2</sup> )	10368	14512	32000	50000
Example 3				
Depth (ft)	0	2.22	2.3	6
Area (ft <sup>2</sup> )	14706	19659	39317	52644



**Table 6-2. Properties of the pond's re-designed outlet structure**

ID	Type of Element	Event Controlled	Shape	Height h (ft)	Width, b (ft)	Invert Offset z (ft)	Discharge Coefficient	Orifice or Weir Area (ft <sup>2</sup> )
<b>Example 3</b>								
Or1	Orifice	WQCV	Side Rectangular	0.3	0.25	0	0.65	0.08
Or2	Orifice	2-yr	Side Rectangular	0.5	2	1.5	0.65	1
Or3	Orifice	10-yr	Side Rectangular	0.25	0.35	2.22	0.65	0.09
W1	Weir	100-yr	Rectangular	2.83	1.75	3.17	3.3	4.95
<b>This Example</b>								
Or1	Orifice	WQCV	Side Rectangular	0.16	0.25	0	0.65	0.04
Or2	Orifice	2- and 10-yr	Side Rectangular	0.5	2.25	1.5	0.65	1.13
W1	Weir	100-yr	Rectangular	2.72	1.6	3.28	3.3	4.35

**Figure 6-2. Schematic of the re-designed pond outlet structure**

## 6.2 System Representation

### Water Quality Treatment in LIDs

As defined in Example 4, the filter strip and infiltration trench LIDs are modeled as subcatchments in order to represent the combined effects of infiltration and storage on storm runoff. This example will also consider their ability to reduce pollutant loads in the surface runoff they handle. There are no widely accepted mechanistic models of pollutant removal through these types of LIDs. The best one can do is to apply average removal efficiencies for specific contaminants based on field observations reported in the literature.

SWMM can apply a constant *BMP Removal Efficiency* for any pollutant in the washoff generated from a particular land use. At each time step the pollutant load generated by a given land use is reduced by this user-supplied value. This reduction also applies to any upstream runoff that runs onto the subcatchment. The LIDs added in Example 4 receive runoff from upstream subcatchments and do not generate any pollutant load themselves. Thus it will be convenient to define a new land use, named “*LID*”, that is used exclusively for the LID subcatchments and has a specific *BMP Removal Efficiency* for TSS associated with it.

## Water Quality Treatment in Detention Ponds

Detention ponds are modeled as storage unit nodes within SWMM. By adding *Treatment Functions* to the storage node’s properties, SWMM can reduce the pollutant concentrations in the pond’s outflow. This example uses an empirical exponential decay function to model solids removal through gravity settling within a pond. For controlling the WQCV event, the pond fills relatively quickly over a period of 2 hours and then drains slowly over an extended period of 40 hours during which solids removal occurs. At some interval  $\Delta t$  during this drain time, and assuming homogenous concentration, the fraction of particles with a settling velocity  $u_i$  that are removed would be  $u_i \Delta t / d$  where  $d$  is the water depth. Summing over all particle settling velocities leads to the following expression for the change in TSS concentration  $\Delta C$  during a time step  $\Delta t$ :

$$\Delta C = C_t \times \sum_i f_i u_i \times (\Delta t / d) \quad (6-1)$$

where  $C_t$  is the total concentration of TSS particles at time  $t$  and  $f_i$  is the fraction of particles with settling velocity  $u_i$ . Because  $\sum f_i u_i$  is generally not known, it can be replaced with a fitting parameter  $k$  and in the limit Equation (6-1) becomes:

$$\frac{\partial C_t}{\partial t} = -\frac{k}{d} C_t \quad (6-2)$$

Note that  $k$  has units of velocity (length/time) and can be thought of as a representative settling velocity for the particles that make up the total suspended solids in solution.

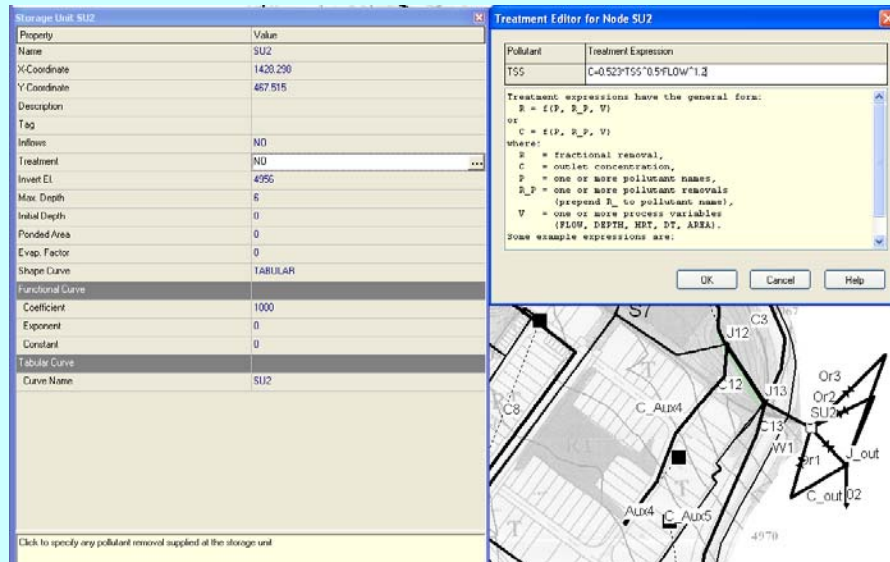
Integrating Equation (6-2) between times  $t$  and  $t + \Delta t$ , and assuming there is some residual amount of suspended solids  $C^*$  that is non-settleable leads to the following treatment function for TSS in the pond:

$$C_{t+\Delta t} = C^* + (C_t - C^*) e^{-(k/d)\Delta t} \quad (6-3)$$

Equation (6-3) is applied at each time step of the simulation to update the pond’s TSS concentration based on the current concentration and water depth.

## Simulating Treatment within a Conveyance Network

SWMM can apply water quality treatment at any node of a drainage system's conveyance network. Treatment for a node is defined by opening its *Property Editor* and clicking the ellipsis button next to the *Treatment* property. This brings up a *Treatment Expression* dialog box in which the user can define a treatment function for each pollutant that passes through the node.



The treatment function for a given pollutant can have one of the following forms:

$$R = f(P, R\_P, V)$$

$$C = f(P, R\_P, V)$$

where  $R$  is the fractional removal,  $C$  is the outlet concentration,  $P$  is one or more concentrations given by the pollutant names (e.g.,  $TSS$ ),  $R\_P$  is one or more pollutant removals (e.g.,  $R\_TSS$ ), and  $V$  is one or more of the following process variables:  $FLOW$  (flow rate into the node),  $DEPTH$  (water depth above node invert),  $HRT$  (hydraulic residence time),  $DT$  (routing time step) and  $AREA$  (node surface area). Some examples of treatment expressions are:

$$C = BOD * \exp(-0.05 * HRT)$$

$$R = 1 - (1 + (0.001 / (2 * FLOW / AREA)))^{(-2)}$$

With a fractional removal expression, the new concentration at the node,  $C$ , is defined as  $C_{in}(1-R)$  where  $C_{in}$  is the inflow concentration to the node. Also, when a concentration  $P$  appears in an expression applied to a non-storage node, it is the same as  $C_{in}$  for the node whereas for a storage node it is the current concentration  $C$  in the storage unit.

### 6.3 Model Setup

The starting point for adding treatment to the runoff controls placed on the study site is the file **Example6-Initial.inp**. This input file already contains the local LIDs defined in Example 4 and the redesigned storage unit and outlet structures described in section 6.1. The land uses in this file were re-calculated and re-assigned to each subcatchment based on the discretization employed in Example 4. The washoff function for TSS is the EMC washoff function used in Example 5. The curb lengths and land uses assigned to each subcatchment are listed in Table 6-3. The subcatchments treated by infiltration trenches or filter strips are marked in grey in this table.

**Table 6-3. Curb lengths and land uses for LID subcatchments**

Subcatchment	Area (ac)	Curb Length (ft)	Residential_1 (%)	Residential_2 (%)	Commercial (%)
S1.1	1.21	450	100	0	0
S1.2	1.46	600	100	0	0
S1.3	1.88	630	100	0	0
S2.1	1.3	450	100	0	0
S2.2	1.5	600	0	100	0
S2.3	1.88	630	0	100	0
S3.1	1.29	0	0	0	0
S3.2	1.02	430	100	0	0
S3.3	1.38	500	0	85	0
S4.1	1.65	0	0	0	0
S4.2	0.79	400	0	100	0
S4.3	1.91	1150	36	64	0
S4.4	2.4	700	0	0	71
S5	4.79	2480	0	0	98
S6	1.98	1100	0	0	100
S7	2.33	565	0	0	0

#### LID Treatment

It is assumed that each filter strip and infiltration trench can provide 70% TSS removal for the runoff that passes over it. This is a typical removal observed for infiltration-based LIDs (Sansalone and Hird, 2003). A new land use, named “*LID*” is created with no TSS buildup function, an EMC TSS washoff function with 0 mg/L of TSS, and a TSS “BMP efficiency” of 70%. The land use assignment for each of the LID subcatchments (*S\_FS\_1* through *S\_FS\_4* and *S\_IT\_1* through *S\_IT\_4*) is set to 100% *LID*. As a result, all of the runoff generated from upstream subcatchments that flow over these LID subcatchments will receive 70% TSS removal.

## Detention Pond Treatment

The removal of TSS in the detention pond is simulated using the exponential model given by Equation 6-3. One can roughly estimate what the removal constant  $k$  in this expression must be so that a targeted level of pollutant removal is achieved within a 40 hour detention time for the 0.23 in. WQCV design storm. If Equation 6-3 were applied over a 40 hour period to achieve a target TSS reduction of 95%, then an estimate of  $k$  would be:

$$k = -\bar{d} \times \ln(0.05) / 40 \quad (6-4)$$

where  $\bar{d}$  is some representative value of the pond depth during the 40 hour release period. As can be verified later on, the average depth in the pond for the 0.23 in design storm over a 40 hour duration is 0.15 ft. Using this value in the expression for  $k$  yields an estimate of **0.01** ft/hr. This value is of the same order as the 0.03 ft/hr figure quoted in US EPA (1986) that represents the 20-th percentile of settling velocity distributions measured from 50 different runoff samples from seven urban sites in EPA's Nationwide Urban Runoff Program (NURP).

With this value of  $k$  and assuming a minimum residual TSS concentration  $C^*$  of **20** mg/L, the following expression is entered into SWMM's Treatment Editor for the storage unit *SU2*:  $C = 20 + (TSS - 20) * EXP(-0.01 / 3600 / DEPTH * DT)$

Note the meaning of the individual terms in this expression with respect to those in Equation (6-3): **20** is the value assumed for  $C^*$ , **TSS** is the identifier given to the TSS concentration  $C$  for this model, **0.01/3600** is the value of  $k$  expressed in units of ft/s, **DEPTH** is the reserved word that SWMM uses for the water depth  $d$  in feet, and **DT** is the reserved word that SWMM uses for the routing time step  $\Delta t$  in seconds. When SWMM sees reserved words like *DEPTH* and *DT* within a treatment expression it knows to automatically insert their current values into the expression at each time step.

The following analysis options should be used for all of the simulations made with this treatment-augmented input file:

*Flow Routing Method:*     *Dynamic Wave*  
*Wet Weather Time Step:*   1 minute  
*Flow Routing Time Step:*   15 sec  
*Reporting Time Step:*       1 minute  
*Total Duration:*             2 days (48 hr)

The 48 hour duration was chosen so that the full effect of the drawdown of the WQCV in the detention pond could be observed. Finally, it is suggested to increase the number of significant figures (from 2, the default value, to 4) for the subcatchment parameter "*Runoff*" and the node parameters "*Quality*" and "*Total Inflow*" by selecting **Tools | Program Preferences | Number Formats** from SWMM's main menu bar. This will help when tabulated results are copied from SWMM to a spreadsheet program to compare results between different runs in cases where variations are small and would not be visible if only two significant figures were used. The resulting input file is named **Example6-Final.inp**.

## 6.4 Model Results

Results for several sets of comparison runs will be discussed. First, the effect of TSS treatment at the LIDs is considered. Figure 6-3 compares the TSS concentration in the treated runoff from filter strip *S\_FS\_1* to that of the upstream runoff from subcatchment *S3.2* for the 0.1 in. storm. Also shown for reference are the runoff flows from each area. The reduction from 170 mg/L to 51 mg/L through the filter strip matches the 70 % TSS removal efficiency specified for the LID. Similar results are obtained for the remaining filter strips under all design storms.

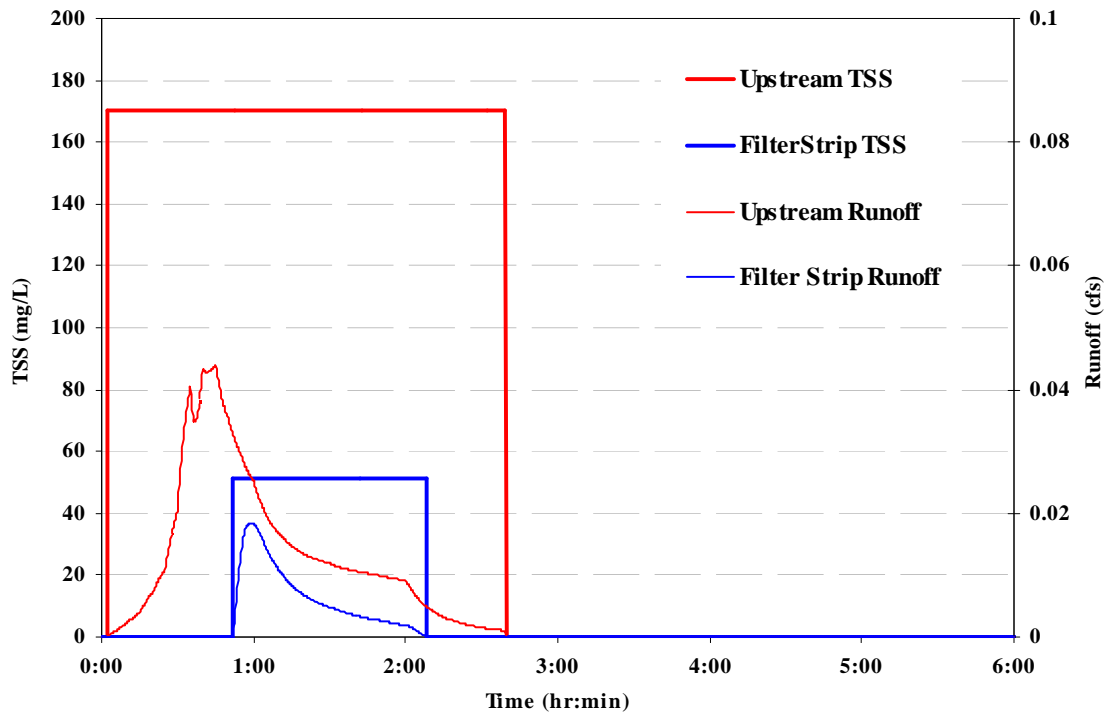
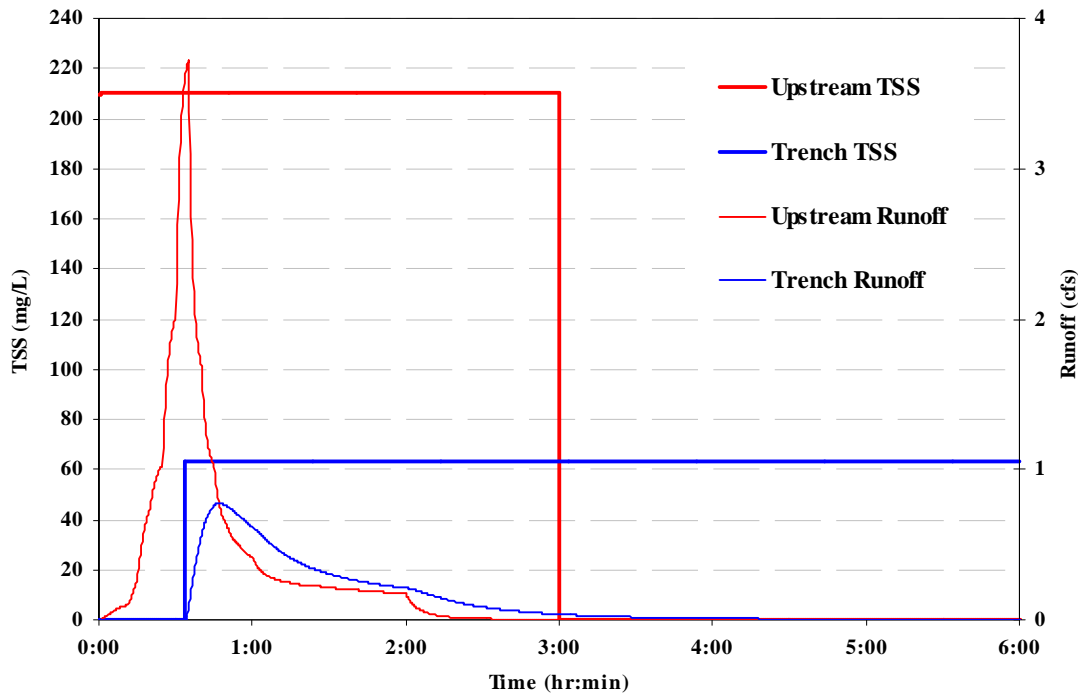


Figure 6-3. TSS and runoff reduction through filter strip *S\_FS\_1* for the 0.1 in. storm

Figure 6-4 presents a similar comparison for the infiltration trench *S\_IT\_4* that treats runoff from subcatchment *S2.3*. These results are for the 2-yr (1 in.) event since for the smaller storms all rainfall infiltrates through the trenches. Once again note how the 70% removal causes a drop in TSS concentration from 210 mg/L down to 63 mg/L. As was the case with the filter strips, the remaining infiltration trenches show a similar behavior to that of *S\_IT\_4*.



**Figure 6-4. TSS and runoff reduction through infiltration trench *S\_IT\_4* for the 2-yr storm**

The next comparison is between the levels of treatment provided by the detention pond for the various design storms. Comparing the time series of pond influent TSS concentrations with the treated effluent concentrations is not particularly useful since the flow rates of these two streams are so different. Instead the pond effluent concentration, both with and without treatment will be compared for each design storm. The result is shown in Figure 6-5. These plots were made by running the model for each design storm (0.1 in., 0.23 in., and 2-year) both with the treatment function for the storage unit node *SU2* and without it. After each run a time series table of the TSS concentration at node *SU2* was generated and exported to a spreadsheet program from which Figure 6-5 was generated.

The following observations can be drawn from Figure 6-5:

- With the  $k$ -value of 0.01 ft/hr, the pond behaves as designed for the WQCV storm (0.23 in) by removing essentially all of the settleable solids over a period of 40 hours.
- The larger the 2-hour design storm, the less effective is treatment in reducing TSS levels over time because of the deeper water depths experienced in the pond.
- For all size storms, it takes a considerable amount of time for any significant removal of TSS to occur; a 50% reduction in settleable TSS requires 10, 19, and 35 hours for the 0.1 in, 0.23 in, and 2-year storms, respectively.

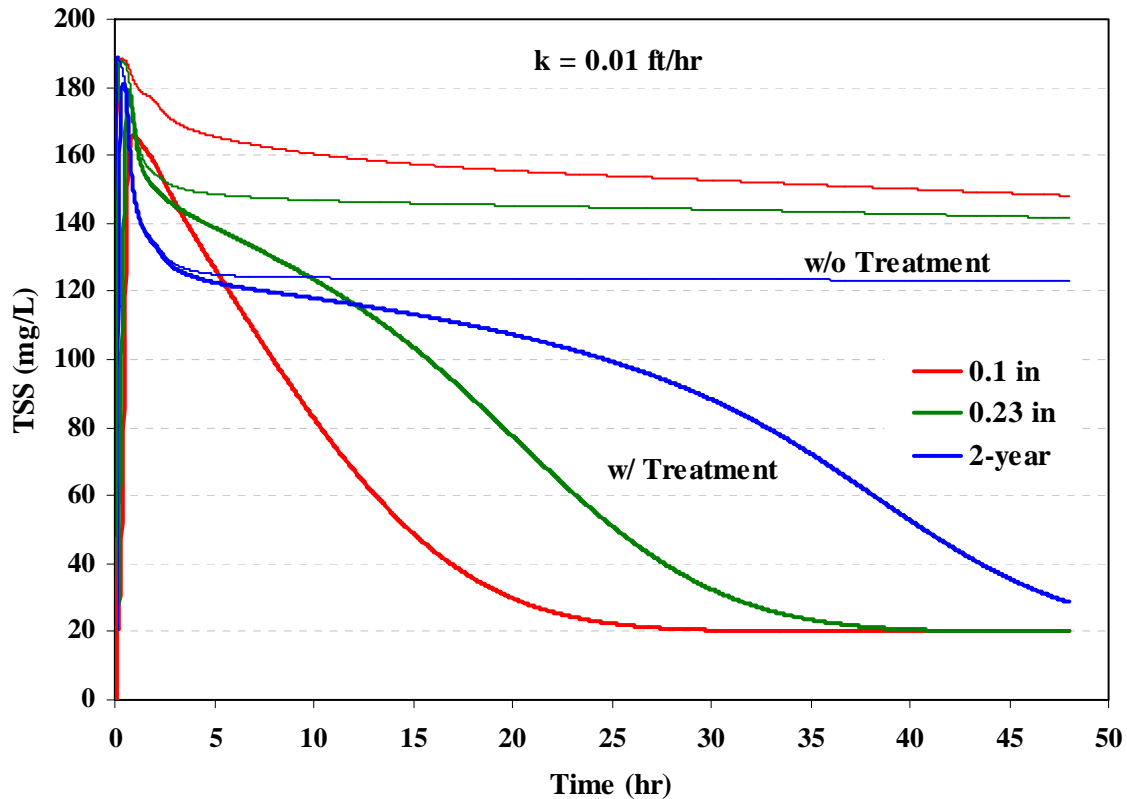


Figure 6-5. TSS concentrations in pond SU2 with and without treatment ( $k = 0.01 \text{ ft/hr}$ )

Another way to evaluate treatment in the pond is to compare the TSS mass loading that it releases both with and without treatment. This is done in Figure 6-6 for the 0.23 WQCV storm. Plots for the other design storms look similar to this one. The effect of treatment on reducing the mass of TSS released is not as significant as it was for concentration. In fact, the *Status Report* for the run with treatment shows that of 72.5 lbs of TSS washed off for this storm event, only 15.7 lbs were removed in the pond. This yields an overall mass removal of only 21.7 %. The percent mass removals for the other design storms were 44.4 % for the 0.1 in. storm and 6 % for the 2-year (1 in.) storm. These low to moderate mass removals are a consequence of the extended time required for solids to settle in the pond during which it still is releasing an outflow.



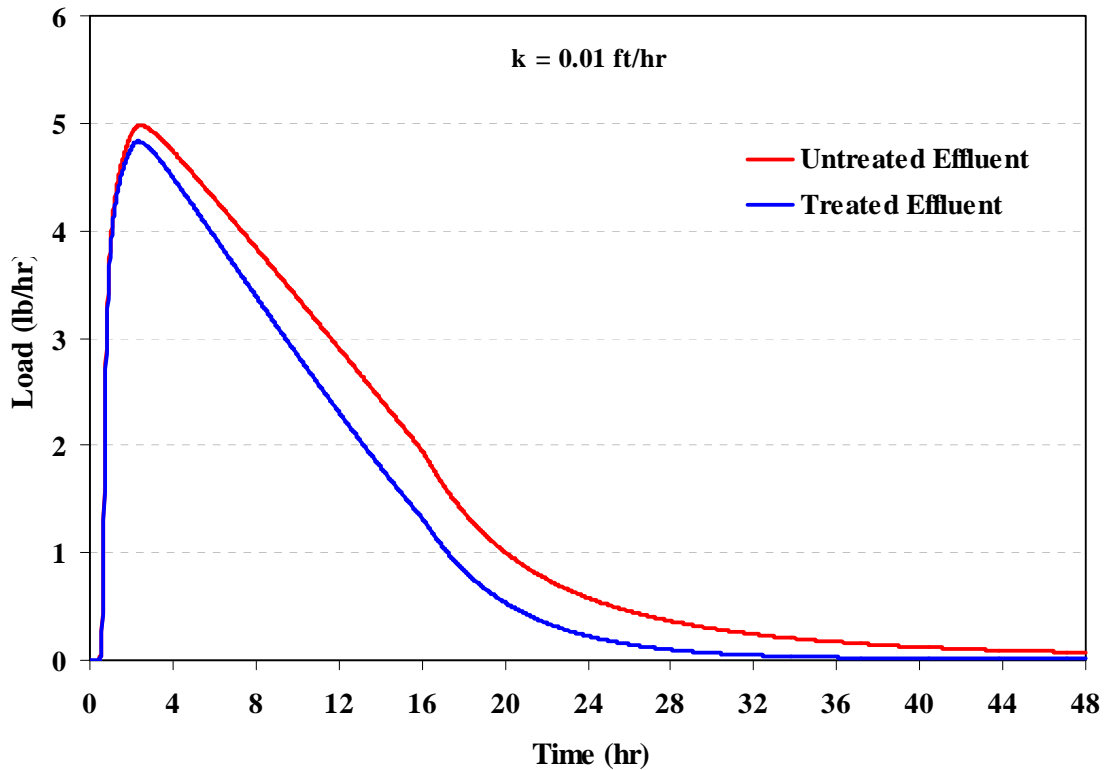


Figure 6-6. TSS mass load released by pond *SU2* for the 0.23 in storm ( $k = 0.01$  ft/hr)

These rather modest levels of detention pond performance were computed using a removal constant  $k$  that represents particles with a very low settling velocity, below that of the lowest 20% determined from a nationwide survey. Suppose the size distribution of particles comprising the TSS washoff were larger, as reflected in a  $k$ -value of **0.3** ft/hr. This represents the 40-th percentile of the settling velocities found from the NURP study (US EPA, 1986). Figure 6-7 shows the resulting TSS concentrations in the pond discharge with this higher  $k$ -value. Figure 6-8 does the same for the TSS discharge loading for the 0.23 in. event. Table 6-4 summarizes the pond's treatment performance for the two different  $k$ -values. These results show that uncertainty in the removal constant will significantly impact predictions of TSS removal within the detention pond. Unfortunately, as noted in US EPA (1986), there can be high variability in solids settling velocity distributions from site to site and from storm to storm within a given site. This variability makes it very difficult to make reliable estimates of detention pond treatment effectiveness.

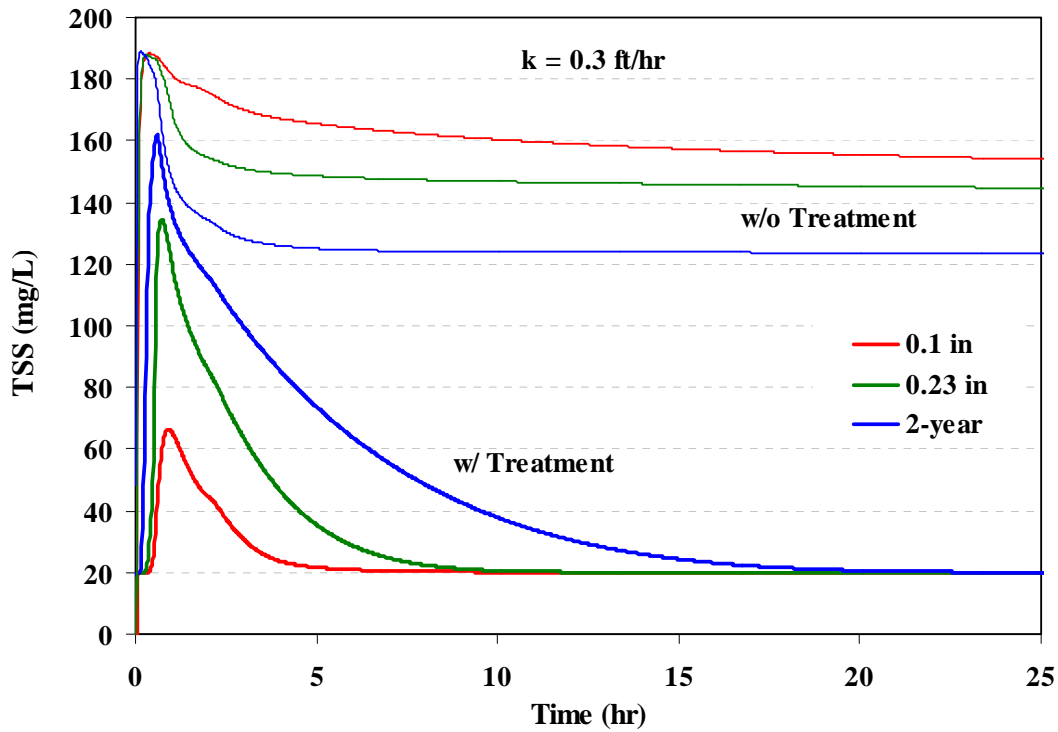


Figure 6-7. TSS concentrations in pond *SU2* with and without treatment ( $k = 0.3$  ft/hr)

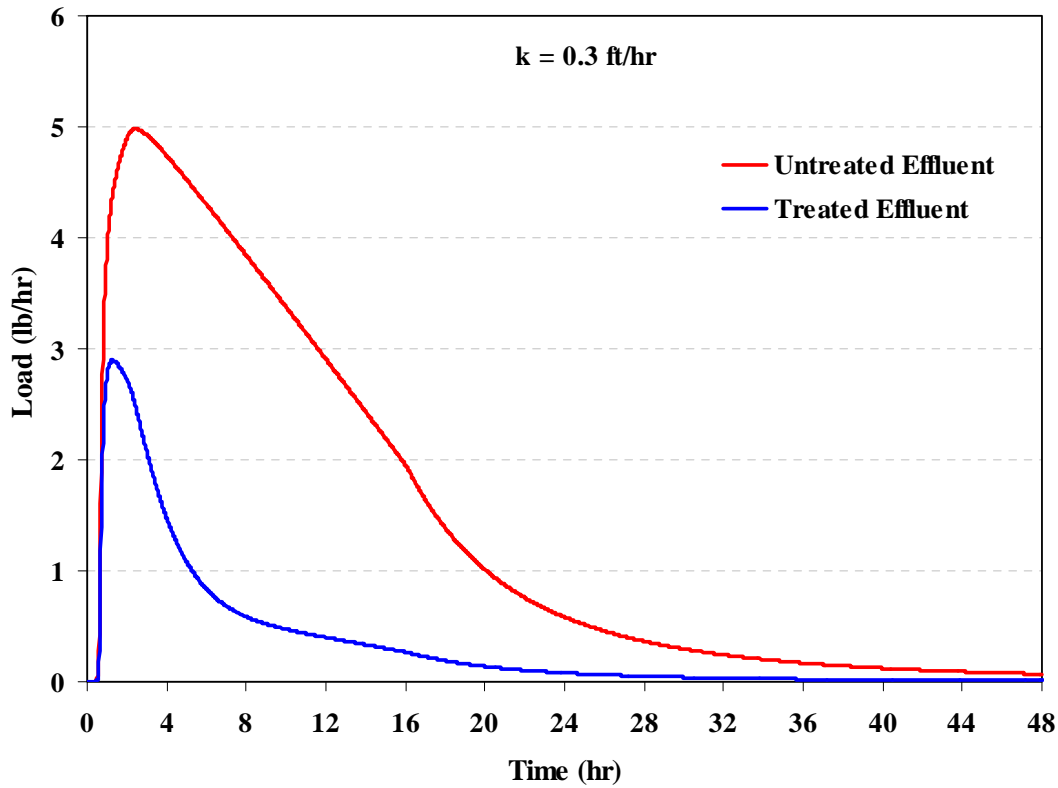


Figure 6-8. TSS mass load released by pond *SU2* for the 0.23 in. storm ( $k = 0.3$  ft/hr)

**Table 6-4. Detention pond TSS treatment performance summary**

	0.1 in. Storm		0.23 in. Storm		1.0 in. Storm	
	$k = 0.01$	$k = 0.3$	$k = 0.01$	$k = 0.3$	$k = 0.01$	$k = 0.3$
Time to achieve 50% reduction, hr	10	1	19	3	35	6
Time to achieve full reduction, hr	30	7	40	10	> 48	20
Overall mass removal, %	44.4	81.8	21.7	75.0	6.0	35.7

Finally, Figure 6-9 compares the total pounds of TSS discharged from the study area site for each design storm with no treatment, with just LIDs, and with both LIDs and the detention pond. These loadings can be read from the *Status Reports* generated by running an analysis of each storm with (a) both types of treatment, (b) with the treatment function for the pond removed (LID treatment only), and (c) with the input file developed for EMC washoff in Example 5 (no treatment). The  $k$  value used in this comparison is the 0.01 ft/hr value. Note the consistent pattern of load reduction as more treatment is applied. Also note that the pond provides a lower increment of overall load reduction than do the LIDs even though the pond is a regional BMP that treats all of the catchment's runoff while the LIDs are local BMPs that receive runoff from only 41 % of the catchment's area. This is a result of the conservative  $k$  value used in the pond's treatment expression. Using a higher value, which would reflect a larger particle size distribution in the runoff, would result in lower loadings from the pond.

## 6.5 Summary

This example showed how water quality treatment could be modeled within SWMM. Total Suspended Solids (TSS) removal was considered in both local LID source controls as well as in a regional detention basin. The key points illustrated in this example were:

1. LID controls can be modeled as distinct subcatchments with a single landuse that has a constant removal efficiency assigned to it. SWMM applies this removal efficiency to the runoff that the control receives from upstream subcatchments.
2. Treatment within a detention pond is modeled with a user-supplied *Treatment Function* that expresses either the fractional removal or outlet concentration of a pollutant as a function of inlet concentration and such operational variables as flow rate, depth, and surface area.
3. An exponential treatment function was used to predict TSS removal within this example's detention pond as a function of a removal constant and the pond's water depth, where the removal constant reflects the settling velocity of the particles to be removed.
4. SWMM's use of constant removal efficiencies for LID controls makes its LID treatment performance insensitive to size of storm.
5. Treatment performance for detention ponds decreases with increasing size of storm due to an increase in pond depth. It also decreases with decreasing size distribution of the sediments that constitute the TSS in the runoff.

6. For the treatment function used in this example, the pond provided less incremental TSS load reduction than did the LIDs. This result, however, is completely dependent on the value of the removal constant used within the pond's treatment function.
7. The large variability reported for particle settling velocities in urban runoff makes it extremely difficult to estimate a removal constant for a detention pond's treatment function that can consistently provide reliable estimates of the pond's treatment performance.

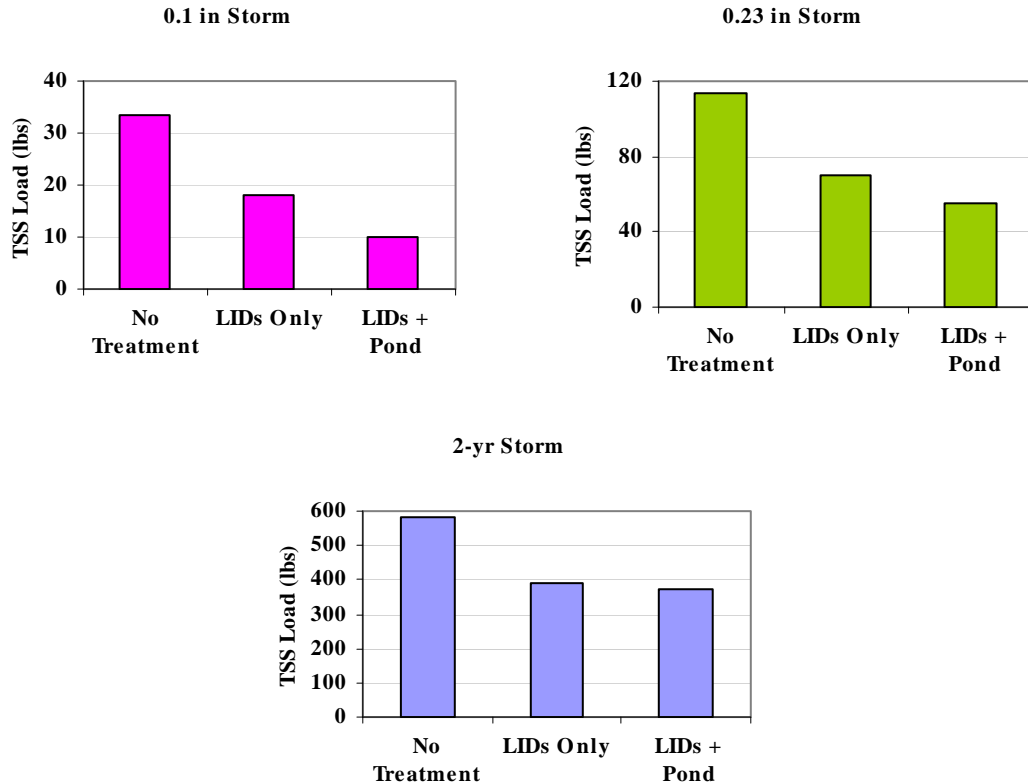


Figure 6-9. Total TSS load discharged at site outlet under different treatment scenarios

# Example 7. Dual Drainage Systems

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The post-development model in Example 2 simulated simple hydraulic routing within a surface drainage system that employed open channels in the form of gutters and swales. The three hydraulic routing methods (*Steady Flow*, *Kinematic Wave* and *Dynamic Wave*) were introduced and their effects on the drainage system behavior shown. Example 7 will convert some of the open channels in Example 2 to parallel pipe and gutter systems. A series of storm sewer pipes placed below the existing swales will also be added to help drain the downstream section of the site's park area. Both the 2-yr and 100-yr design storms will be used to size and analyze the performance of this expanded dual drainage system. Particular attention will be paid to the interaction between the below-ground storm sewer flows and the above-ground street flows that occurs during high rainfall events.

## 7.1 Problem Statement

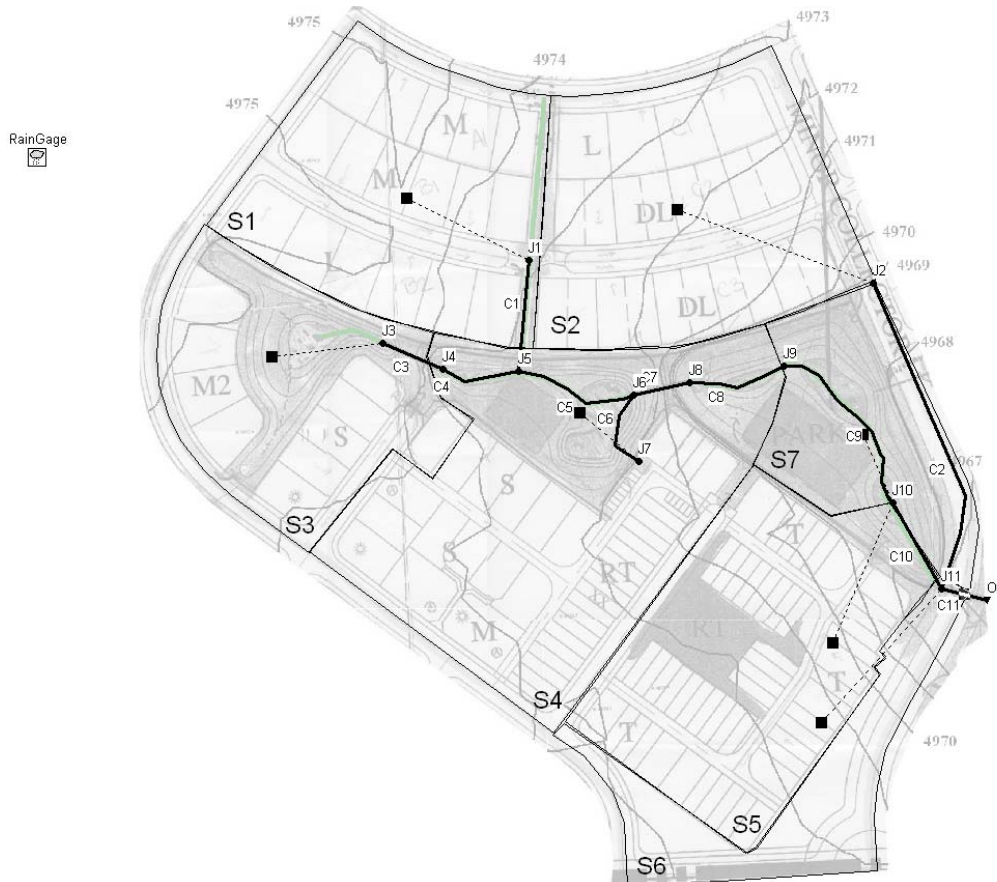
The objective of this example is to simulate the interaction between the minor and major drainage systems through the interconnection of part of their underground and surface sections. For frequent events the minor or "initial" system operates (Grigg, 1996); overland flows are conveyed by gutters and enter into the pipe system. For large events these pipes surcharge and flood, and the major system handles the flows (Grigg, 1996). In particular, the entire street (not only the gutters) becomes a conveyance element.

Figure 7-1 shows the post-development layout of the site analyzed in Example 2. Example 2 modeled the drainage system with open channels and culverts whose invert elevations were the same as the ground surface elevations found on the site contour map. A series of below ground pipes will be added to the site that share inlets with the open (surface) channel running through the park. Gutters will also be added to the upstream section of the study area. The cross sections of these gutters will be that of a typical street, representing the surface "channel" through which water would flow if the pipe system surcharged and flooded the street. Thus, in this example the pipes and flow in gutters represent the minor system, and the channel in the park and the flow in streets represent the major system. The pipe system will be sized for the 2-yr event and its behavior observed during the major storm (100-yr event).

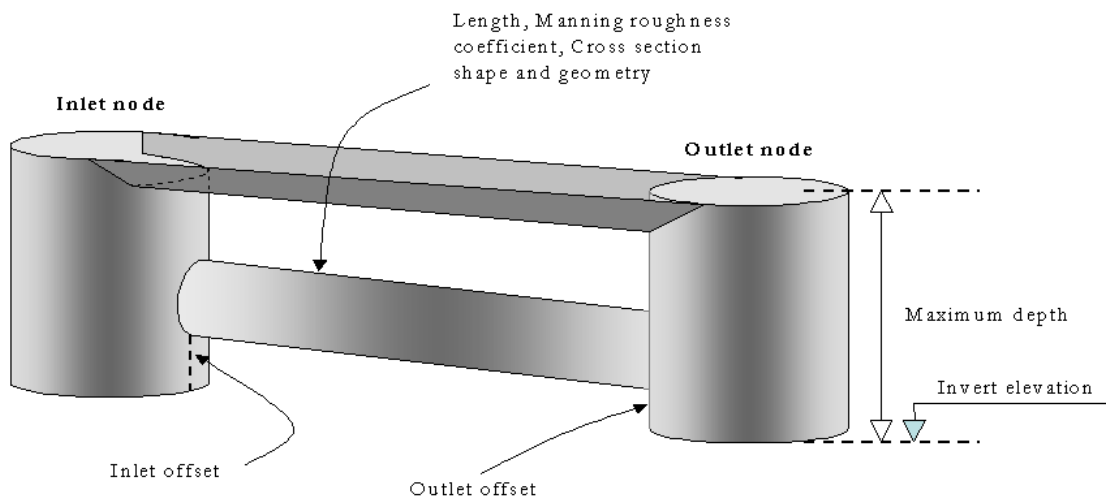
## 7.2 System Representation

Example 2 introduced junction nodes and conduit links as the basic elements of a drainage network. An example of the parallel pipe and gutter conveyance arrangement that will be used in this example is shown in Figure 7-2. It consists of a below-grade circular pipe connected to manhole junctions on either end, plus an above grade street and gutter channel also connected to the same two manhole junctions. The details of the gutter inlet and drop structures that make the actual connection with the manholes are not

important for our purpose. The parameters needed to characterize this type of node-link arrangement are as follows:



**Figure 7-1. Post-development site with simple drainage system**



**Figure 7-2. Parallel pipe and gutter conveyance**

- **Manhole Invert Elevation**

The invert elevation of the manhole junction is the elevation of the bottom of the manhole relative to the model's datum (such as mean sea level). The invert elevation establishes the junction's vertical placement in the SWMM model.

- **Manhole Maximum Depth**

The maximum depth of the manhole junction is the distance from its invert to the ground surface elevation where street flooding would begin to occur. If this maximum depth is left set at zero, SWMM automatically uses the distance from the junction's invert to the top of the highest connecting link, which would be the top of the gutter/street channel in this particular example.

- **Surcharge Depth**

The surcharge depth of a junction is the additional depth of water beyond the maximum depth that is allowed before the junction floods. SWMM uses this depth to simulate pressurized conditions at bolted manhole covers or force main connections but it will not be used in this example. Different types of surcharge will be discussed later in this example.

- **Conduit Offsets**

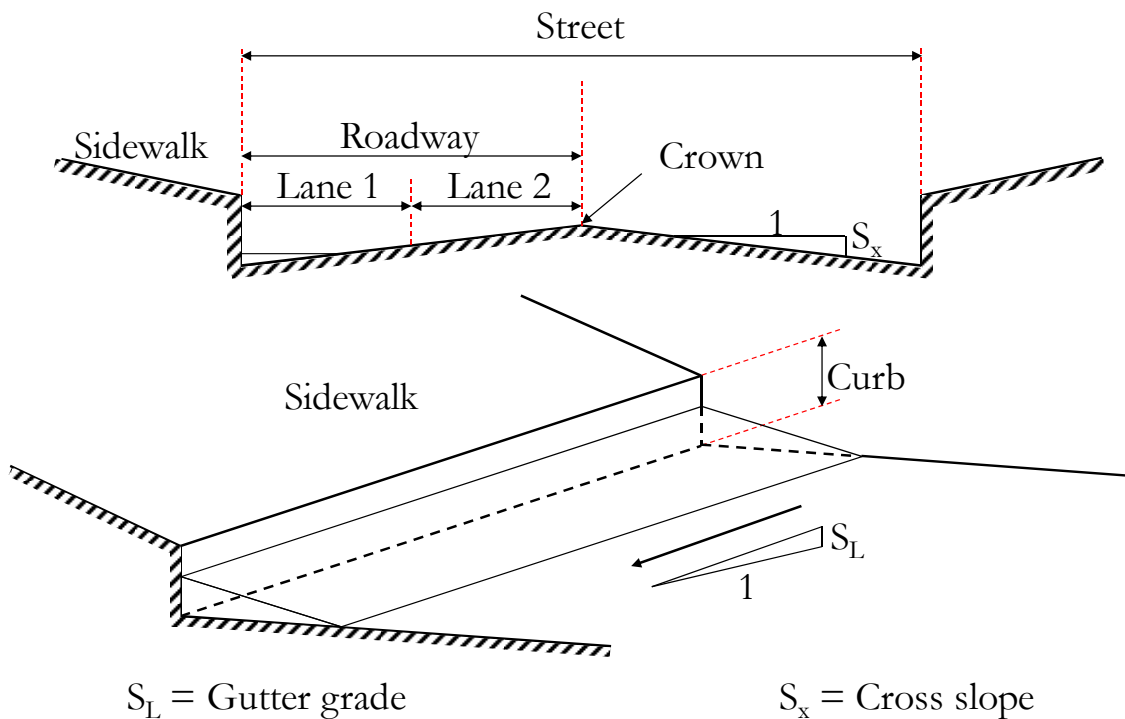
The inlet offset for a conduit is the distance that its inlet end lies above the invert of the junction that it connects to. A similar definition applies to the offset for the outlet end of a conduit. In the parallel pipe and gutter system, the elevations of the gutters are set above the pipes using their inlet and outlet offsets (Figure 7-2).

Note that because this representation of a dual drainage system has more than one conduit exiting a junction node, *Dynamic Wave* flow routing must be used to analyze its hydraulic behavior. An alternative way to represent these systems is to use the *Overflow* variety of Flow Divider nodes to connect pairs of pipes and gutter channels together. This scheme can be analyzed using the simpler *Kinematic Wave* method, where any flow in excess of the sewer pipe capacity would be automatically diverted to the gutter channel. The disadvantage of this approach is that it cannot model a two-way connection between the pipes and gutters/streets, nor can it represent the pressurized flow, reverse flow and backwater conditions that can exist in these systems during major storm events.

## **Drainage System Criteria**

The general drainage system criteria that will be used in this example are listed below. These criteria are based on those defined for the city of Fort Collins (City of Fort Collins, 1984 and 1997). Figure 7-3 shows the different elements of the street considered in these criteria. Two storms will be used to design the drainage system: a minor or initial storm (2-yr) and a major storm (100-yr). The initial storm is one that occurs at fairly regular intervals while the major storm is an infrequent event. In this example, the streets are classified as "collectors". Note that the general drainage criteria presented here apply only to this example and will change depending on the location of the system being designed. The criteria are:

- The minimum gutter grade ( $S_L$  in Figure 7-3) shall be 0.4%, and the maximum shall be such that the average flow velocity does not exceed 10 ft/sec.
- The cross-slope ( $S_x$  in Figure 7-3) of all streets will be between 2% and 4%.
- The encroachment of gutter flows onto the streets for the initial storm runoff will not exceed the specification presented in the second column from the left in Table 7-1. The encroachment of gutter flows on the streets for the major storm runoff will not exceed the specification presented in the third column from the left in Table 7-1.
- The pipe system should carry the 2-yr storm and work as an open channel system.



**Figure 7-3. Elements of streets defined in the drainage criteria**



**Table 7-1. General drainage system criteria for Fort Collins (City of Fort Collins, 1984 and 1997)**

<b>Street classification</b>	<b>Initial storm</b>	<b>Major storm</b>
Local (includes places, alleys and marginal access)	No curb-topping. Flow may spread to crown of street	Residential dwelling and other dwelling cannot be inundated at the ground line. The depth of water over the crown cannot exceed 6 inches.
Collector	No curb-topping. Flow spread must leave at least one lane width free of water	Residential dwelling and other dwelling cannot be inundated at the ground line. The depth of water over the crown cannot exceed 6 inches. The depth of water over the gutter flowline cannot exceed 18 inches. (The most restrictive of the last two conditions governs)
Major Arterial	No curb-topping. Flow spread must leave at least one-half of roadway width free of water in each direction	Residential dwelling and other dwelling cannot be inundated at the ground line. The street flow cannot overtop the crown. The depth of water over the gutter flowline cannot exceed 18 inches. (The most restrictive of the last two conditions governs)

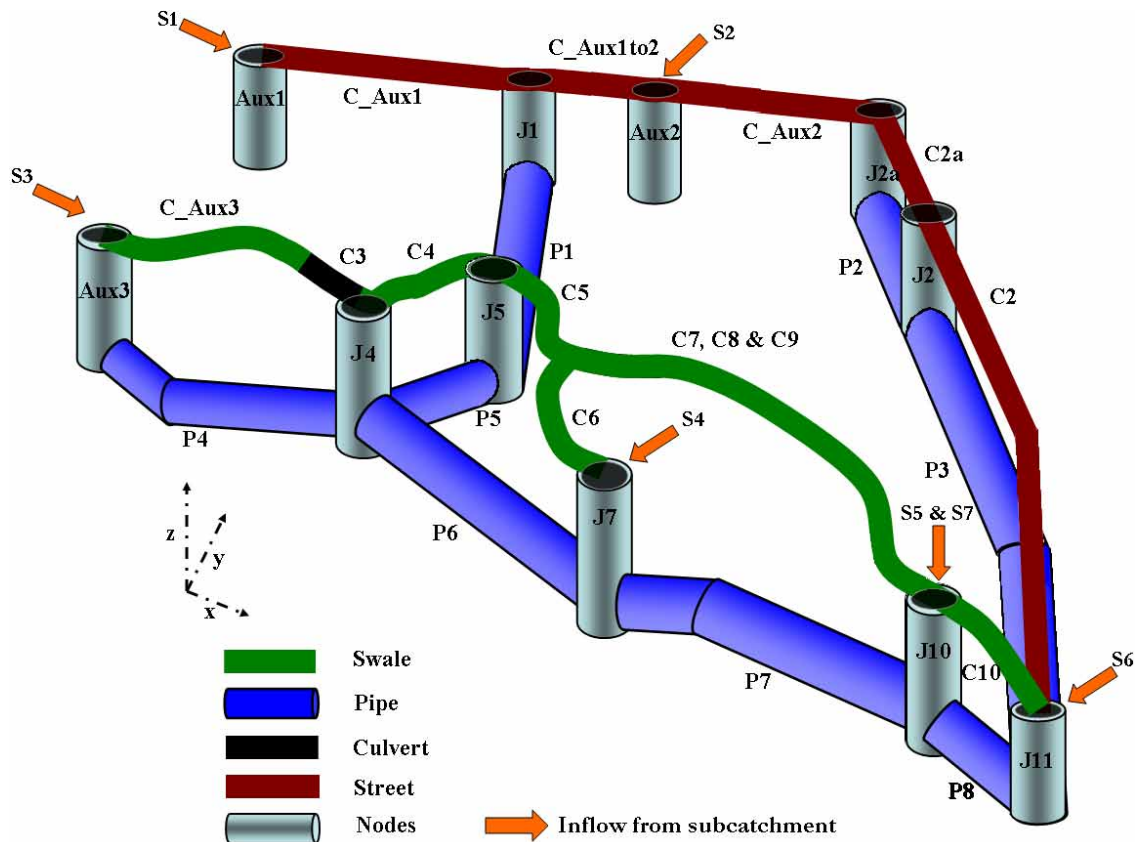
### 7.3 Model Setup

Figure 7-4 shows the layout of the dual drainage system with the pipes, streets and swales that will be included in the model. Note that runoff from subcatchments *S1* and *S2* is introduced first to the street system (gutters) through nodes *Aux1* and *Aux2*, and then enters the storm sewers through inlet grates represented by nodes *J1* and *J2a*. For the rest of the system the runoff is assumed to enter directly into the pipe system. The following steps are used to build the complete dual drainage model starting from the layout defined in the input file **Example2\_Post.inp**.

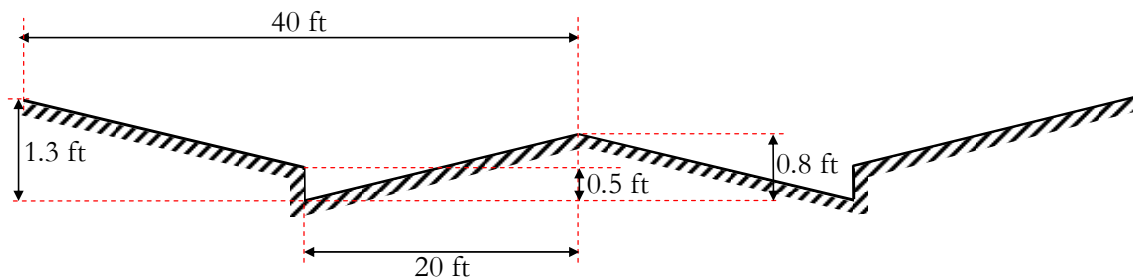
#### Surface Elements

1. The first step is to add the additional junction nodes *Aux1*, *Aux2* and *J2a*, shown in Figure 7-4. The invert elevations of these nodes for now are simply their surface elevations (*Aux1* = **4975** ft, *Aux2* = **4971.8** ft, and *J2a* = **4970.7** ft).
2. Next the additional gutters *C\_Aux1*, *C\_Aux1to2*, *C\_Aux2*, and *C2a* are added into the model and their lengths are determined using the Auto-Length option. These gutters have a roughness coefficient of **0.016**.
3. Instead of the trapezoidal cross section used in Example 2, irregular shapes will be used to represent the cross sections of the gutter conduits. The transects that define these shapes are created using SWMM's *Transect Editor* (see the sidebar "Defining Channel Cross Sections as Irregular Channel Transects"). The transect *Full\_Street* shown in Figure 7-5 is used to represent the entire section of all streets within the study area. A cross-slope  $S_x = 4\%$  is used to define this section. The transect *Half\_Street* depicts only half of the *Full\_Street* section, from the sidewalk to the crown of the street. It is used to represent the north-south street that runs down the

4. The two types of transect cross-sections just created are then assigned to the corresponding street conduits shown in Figure 7-4. Conduits *C\_Aux1*, *C\_Aux1to2*, and *C\_Aux2* represent streets that have the *Full\_Street* section. *C2a* and *C2* represent streets that have the *Half\_Street* section.



**Figure 7-4. Three-dimensional layout of the site's dual drainage system**

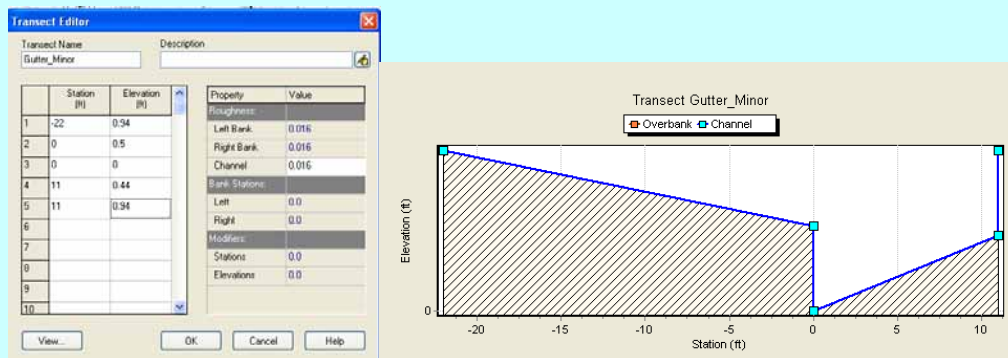


**Figure 7-5. *Full\_Street* cross section. The *Half\_Street* section is half of this section.**

## Defining Channel Cross Sections as Irregular Channel Transects

Channel cross sections can be assigned to each channel using the *Cross Section Editor* as was shown in Example 2 or by defining a general cross section Transect object. The main advantages of using a Transect are: 1) the shape of a channel cross section can be entered into the model once and then assigned to multiple channels that share the same shape, 2) changes to channels with the same cross section (e.g. in Mannings roughness and depth) can be made more efficiently through the *Transect Editor* and 3) any channel cross section shape can be entered. Transects are generally used to model natural irregular channel shapes in SWMM but they can also be used to model gutter cross sections as well. The process used to add a new Transect object to a project is:

1. Open the *Transect Editor* by selecting **Hydraulics | Transects | “+”** in the *Data Browser*.
2. Enter a name for the new Transect.
3. Define the shape of the Transect’s cross section by entering Stations (distance from the left edge of the Transect) and their corresponding Elevations into the editor’s data grid.



4. The location of the left bank, the right bank and the channel section is defined by entering the corresponding station locations in the *Bank Stations* entry fields. In this example, there is no left or right bank so the Left and Right bank stations are set to zero.
5. The Mannings roughness values are defined for the cross section’s left bank, right bank and channel sections. No values are required for the bank roughness if they are not modeled.

To assign a specific Transect object to a conduit’s cross-section:

1. Open the *Property Editor* for the conduit.
2. Click the ellipsis button next to the *Shape* property to bring up the *Cross-Section Editor*.
3. Select **Irregular** for the choice of cross-sectional shape and then select the Transect of choice from the *Transect Name* combo box.

**Table 7-2. Cross section data for street transects**

<b>Full</b>	<i>Station</i>	-40	-20	-20	0	20	20	40
<b>Street</b>	<i>Elevation</i>	1.3	0.5	0	0.8	0	0.5	1.3
<b>Half-</b>	<i>Station</i>	-40	-20	-20	0	0	-	-
<b>Street</b>	<i>Elevation</i>	1.3	0.5	0	0.8	1.3	-	-

### Below-Ground Elements

The changes made so far have been to surface elements of the watershed. Now the pipes below the streets will be added to the drainage system and the invert elevations of the surface conduits will be re-defined.

1. Conduit *C1* is replaced with a pipe (***P1***) and two pipes are added between nodes *J2a* and *J2* (pipe ***P2***) and *J2* and *J11* (pipe ***P3***). The roughness of these three pipes is **0.016** and their diameter is **1** ft. As usual, the lengths of the pipes are calculated automatically with the *Auto-Length* tool.
2. At this point there are pairs of parallel links connected to the same input and output nodes (for example *C2a* and *P2*; *C2* and *P3*). Right now, they are located at the same elevation (the surface elevation). These elevations must be adjusted so that the sub-surface pipes lie below the surface gutters. This is done by decreasing the invert elevation of *J1* and *J5* by 4 ft (*J1* = **4969** ft and *J5* = **4965.8** ft), and raising the outlet offset of *C\_Aux1* to **4** ft and the inlet offset of *C\_Aux1to2* to **4** ft. Then, the invert elevations of *J2a* and *J2* are decreased by 4 ft (*J2a* = **4966.7** ft and *J2* = **4965** ft), an outlet offset of **4** ft is given to *C\_Aux2*, an inlet and outlet offset of **4** ft is given to *C2a*, and an inlet offset of **4** ft and outlet offset of **6** ft to *C2*.
3. The properties of the different elements of the model should be those listed in Tables 7-3 and 7-4. Table 7-3 summarizes the junction inverts; Table 7-4 shows the conduit shapes and their inlet and outlet offsets.

### Park Area Elements

The next phase is to add the series of pipes that drain the park area containing the natural channel swales, shown in Figure 7-4. This process consists of the following steps.

1. A new node, ***Aux3***, is added to the model with an invert elevation of **4974.5** ft. A new swale, ***C\_Aux3***, is defined connecting nodes *Aux3* and *J3*. Its roughness is **0.05** and its cross section is the same as the cross section used in Example 2 for swales (for instance, the cross section of conduit *C4*).
2. Next the series of pipes that drain the park area is created as shown in Figure 7-4. These pipes (***P4***, ***P5***, ***P6***, ***P7*** and ***P8***) connect with the surface system at junctions *Aux3*, *J4*, *J5*, *J7*, *J10*, and *J11*. The latter represent inlet grate structures in the surface swales. The roughness coefficient of these pipes is **0.016**.
3. The pipes underneath the park have an initial diameter of **3** ft and lie **6** ft below the invert elevation of the natural channel through the park. These pipes are 2 ft deeper

than the pipes buried below the streets to avoid giving *P5* an adverse slope. The junctions where the swales and the pipe system connect together are assigned invert elevations that are 6 ft below the ground surface. The invert elevations of these junctions (in feet) become: *Aux3* = **4968.5**, *J4* = **4965**, *J7* = **4963.5**, *J10* = **4957.8**, *J11* = **4957** and *O1* = **4956**.

4. As currently defined, one pipe in the model, *P6*, has an adverse slope. This negative slope is corrected by lowering the invert elevation of *J7* from 6 ft below the surface to 8 ft (elevation *J7* = **4963.5** ft).
5. The next step is to set the inlet and outlet offsets of the surface conduits where they share nodes with the series of pipes to maintain their surface elevations. The inlet and outlet offset elevations of *C\_Aux3*, *C4* and *C10* are **6** ft and **0** ft, **6** ft and **4** ft, and **6** ft and **6** ft, respectively. Additionally, the inlet offset of *C5* and *C6* are increased to **4** ft and **8** ft respectively, and the outlet offset of *C3* and *C9* are increased both to **6** ft. These new offsets are defined to keep the swale at the surface. Note that the depths of the gutters are assumed negligible in this example. The depths of the swales (3 ft) are not considered negligible but were accounted for in Example 2 when their invert elevations were modeled.
6. The invert elevation of junctions *J3*, *J6*, *J8* and *J9* defined in Example 2 remain the same for this example. The final layout should look like the one shown in Figure 7-6. The final invert elevations of the junctions as well as the size, inlet and outlet offsets of the conduits are summarized in Tables 7-3 and 7-4.

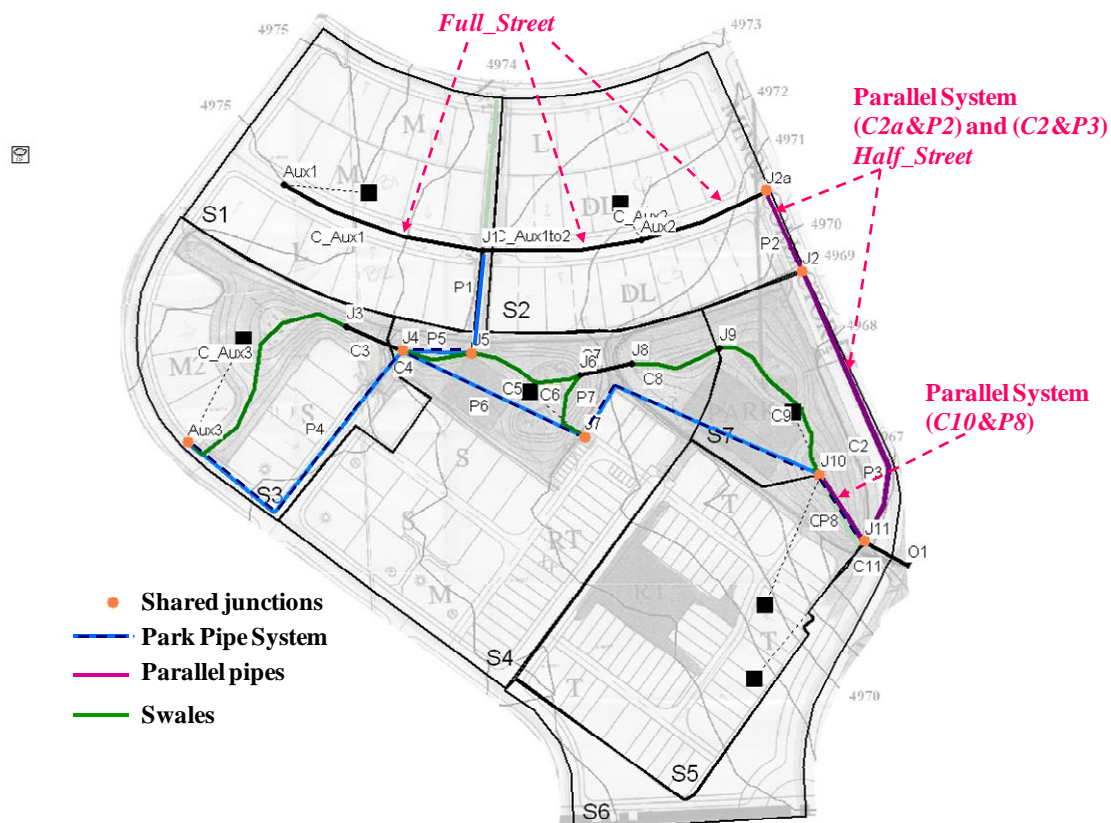


Figure 7-6. Post-development site with dual drainage system

**Table 7-3. Junction invert elevations for dual drainage system**

<b>Junction ID</b>	<b>Surface Elevation (ft)</b>	<b>Elev. Change for Parallel Pipe (ft)</b>	<b>Final Invert Elevation (ft)</b>
J1	4973.0	4	<b>4969.0</b>
J2a	4970.7	4	<b>4966.7</b>
J2	4969.0	4	<b>4965.0</b>
J3	4973.0	0	<b>4973.0</b>
J4	4971.0	6	<b>4965.0</b>
J5	4969.8	4	<b>4965.8</b>
J6	4969.0	0	<b>4969.0</b>
J7	4971.5	8	<b>4963.5</b>
J8	4966.5	0	<b>4966.5</b>
J9	4964.8	0	<b>4964.8</b>
J10	4963.8	6	<b>4957.8</b>
J11	4963.0	6	<b>4957.0</b>
Aux1	4975.0	0	<b>4975.0</b>
Aux2	4971.8	0	<b>4971.8</b>
Aux3	4974.5	6	<b>4968.5</b>
O1	4962.0	6	<b>4956.0</b>

**Table 7-4. Conduit shapes and offsets for dual drainage system**

<b>Conduit ID</b>	<b>Type</b>	<b>Initial Depth or Diameter (ft)</b>	<b>Length (ft)</b>	<b>Inlet Offset (ft)</b>	<b>Outlet Offset (ft)</b>
C2a	Half_Street	1.3	157.48	4	4
C2	Half_Street	1.3	526.00	4	6
C3	Culvert	2.25	109.00	0	6
C4	Swale	3	133.00	6	4
C5	Swale	3	207.00	4	0
C6	Swale	3	140.00	8	0
C7	Culvert	3.5	95.00	0	0
C8	Swale	3	166.00	0	0
C9	Swale	3	320.00	0	6
C10	Swale	3	145.00	6	6
C11	Culvert	4.75	89.00	0	0
C_Aux1	Full_Street	1.3	377.31	0	4
C_Aux2	Full_Street	1.3	239.41	0	4
C_Aux1to2	Full_Street	1.3	286.06	4	0
C_Aux3	Swale	3	444.75	6	0
P1	Pipe	1	185.39	0	0
P2	Parallel Pipe	1	157.48	0	0
P3	Parallel Pipe	1	529.22	0	0
P4	Pipe	3	567.19	0	0
P5	Pipe	3	125.98	0	0
P6	Pipe	3	360.39	0	0
P7	Pipe	3	507.76	0	0
P8	Parallel Pipe	3	144.50	0	0



## 7.4 Model Results

### General System Behavior

The first step in analyzing this dual drainage system is to run the model in its preliminary sized state for the 2-yr storm with *Dynamic Wave* routing to check its general performance. The hyetographs for both the 2-yr and 100-yr storms are the same as those used in Example 1. For this run the reporting and wet-weather time steps were set to **1** minute, the routing time step to **15** seconds and the dry-weather runoff time step to **1** hour. In addition, the “*Report Input Summary*” check box on the *General* page of the *Simulation Options* dialog was checked so that the slopes computed for all of the system’s conduits would be displayed in the run’s *Status Report*. The input data file for this initial system design is named **Example7-Initial.inp**.

After running this model, the resulting *Status Report* shows that no nodes are either surcharged or flooded (see the *Node Surcharge Summary* and *Node Flooding Summary* sections of the report). However, conduits *P1*, *P2*, and *P3* were all surcharged (see the *Conduit Surcharge Summary* section) indicating that the system was undersized.

The differences in these surcharge reports illustrate that node surcharge and conduit surcharge are two different behaviors. Figure 7-7 shows several examples of surcharge and flooding. Figure 7-7a shows conduit surcharge; Figure 7-7b shows conduit surcharge, node surcharge and node flooding; Figure 7-7c shows conduit surcharge and node surcharge; and Figure 7-7d shows conduit surcharge, node surcharge and node flooding. Note that in the case of the open channel (Figures 7-7a and 7-7b) node surcharge occurs simultaneously with node flooding.

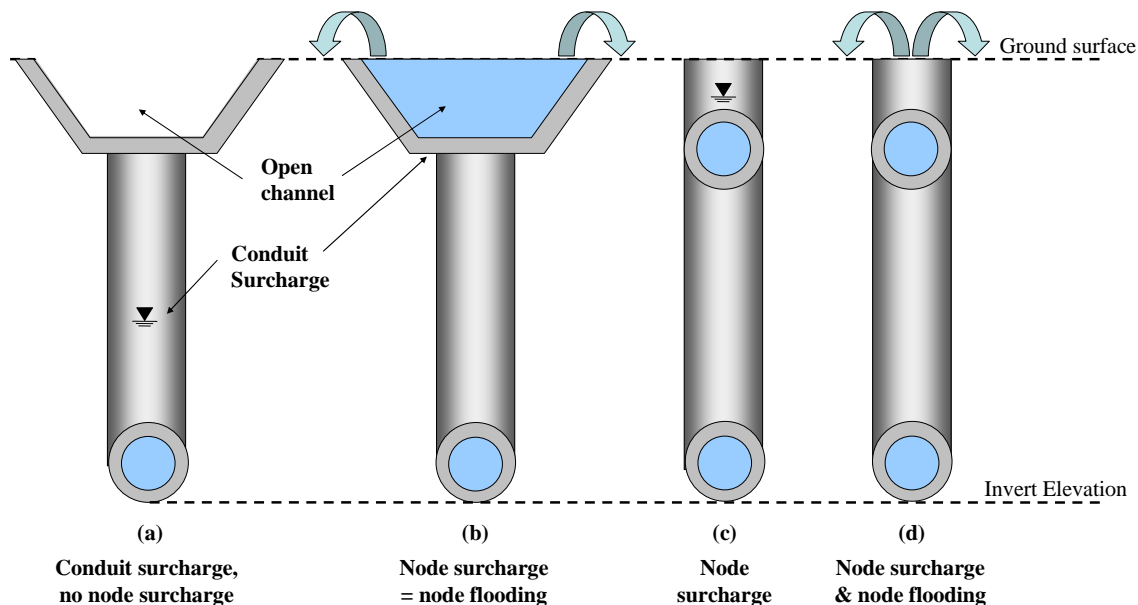


Figure 7-7. Examples of surcharge and flooding

The drainage criteria in Section 7.3 also require a minimum gutter grade of 0.4% and maximum average flow velocity of 10 ft/sec. Therefore, it is required to check that the minimum slope of the conduits representing gutters is larger than 0.4%. Table 7-5, taken from the *Link Summary* table of the *Status Report*, shows the slopes of the conduits representing the gutters. All the gutters have a longitudinal grade of 0.4% or greater and thus meet the design standards considered for this example. The velocities in the gutters and all the conduits will be checked once the pipes of the system are properly sized.

**Table 7-5. Calculated gutter grades**

Conduit ID	Length (ft)	Slope (%)
C2a	157	1.1
C2	526	1.1
C_Aux1	377	0.5
C_Aux2	239	0.5
C_Aux1to2	286	0.4

### Design for the 2-yr Storm

The next step of the analysis is to size the pipes so that they can carry the 2-yr storm without surcharging. In addition, the design criteria listed in Section 7.3 require the collector streets to be sized so there is no curb topping and at least one lane remains free of water. This phase of the analysis is carried out as follows:

1. The sizes of the pipes are iteratively modified, and the model run, until the *Status Report's Conduit Surge Summary* shows that no conduits are surcharged and that the values of the *Max/Full Depth* ratio in the *Link Flow Summary* are close to 0.85 for all the pipes (a safety factor of 15% is considered to reduce the risk of surcharging). Table 7-6 lists the standard pipe sizes that were used in this process. Table 7-7 shows the results from three iterations, including the final one. Note how increasing the size of the upstream pipes allows some of the downstream pipes to be reduced in size (but made no smaller than the upstream ones).
2. Using the final pipe sizing from the previous step (Trial 3 in Table 7-6), the peak velocities in the conduits representing the streets are checked in the *Link Flow Summary* section of the *Status Report*. The peak velocity for all these conduits is less than 10 ft/sec, the maximum allowed by the drainage criteria.
3. According to the drainage criteria, no curb-topping should occur for the minor storm in any street. Figure 7.5 shows that curb-topping implies a *Max/Full Depth* ratio of  $0.5/1.3 = 0.38$ . The *Link Flow Summary* in the *Status Report* for the final pipe sizing shows that the highest value of the *Max/Full Depth* ratio for the street conduits is 0.29 for conduit C\_Aux2. Thus, the final design of the system is appropriate in terms of the flows in the streets.

The input file with the final pipe sizes for this example is named **Example7-Final.inp**.



**Table 7-6. Available drainage pipe sizes**

in.	ft	in.	ft	in.	ft	in.	ft
6	0.5	18	1.5	24	2	38	3.17
12	1	20	1.67	28	2.33	42	3.5
16	1.33	22	1.83	36	3	48	4

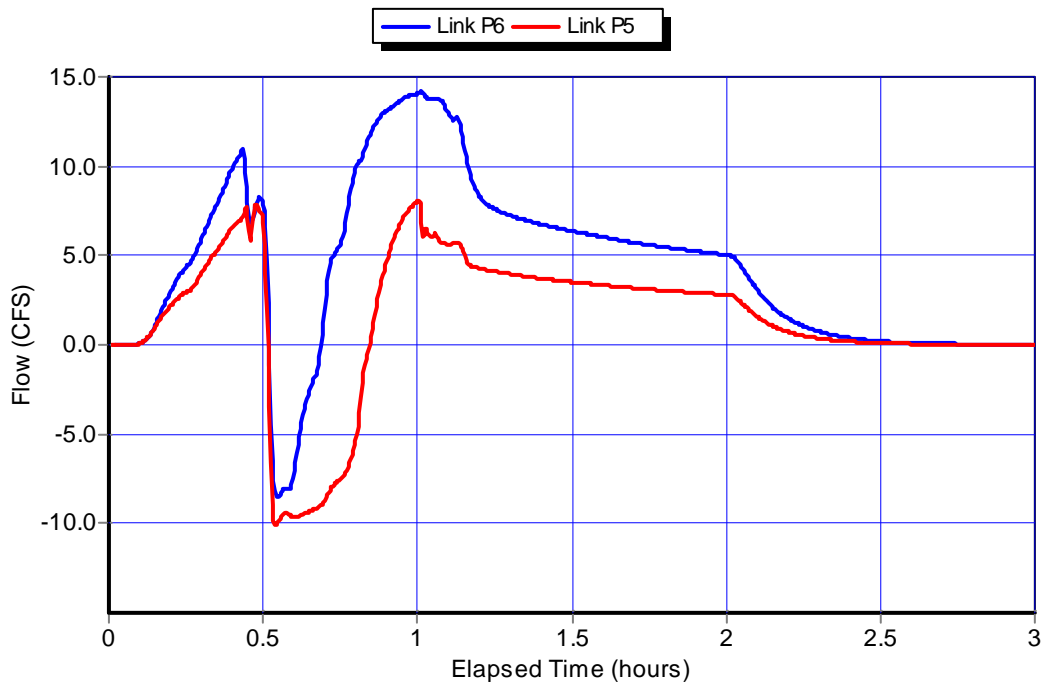
**Table 7-7. Iterations for 2-yr storm pipe sizing**

Pipe	Trial 1			Trial 2			Trial 3, final		
	Size (ft)	Surcharged	Max/Full Depth	Size (ft)	Surcharged	Max/Full Depth	Size (ft)	Surcharged	Max/Full Depth
P1	1	yes	0.85	<b>1.33</b>	no	0.69	<b>1.33</b>	no	0.71
P2	1	yes	1	1.33	yes	0.90	<b>1.5</b>	no	0.67
P3	1	yes	1	1.33	no	0.90	<b>1.5</b>	no	0.80
P4	3	no	0.27	2	no	0.52	<b>1.67</b>	no	0.64
P5	3	no	0.29	2	no	0.58	<b>1.83</b>	no	0.65
P6	3	no	0.36	2	no	0.71	<b>2</b>	no	0.72
P7	3	no	0.48	2	no	0.81	<b>2</b>	no	0.81
P8	3	no	0.53	3.17	no	0.51	<b>3.17</b>	no	0.51

## Major Storm Performance

Finally, the model is run for the 100-yr storm by changing the rainfall time series used by its rain gage. The *Status Report's Node Surge Summary* and *Node Flooding Summary* tables show that no nodes are surcharged or flooded during the 100-yr storm. Additionally, the *Conduit Surge Summary* shows that, as expected, all of the pipes are surcharged. According to the *Link Flow Summary* table, the highest velocities occur in conduits *C7*, *C11* and *P8* (11.25, 11.93 and 10.76 ft/sec respectively), all of them over 10 ft/sec. Standard drainage criteria define a maximum allowable velocity in pipes and culverts of 15 to 18 ft/sec (CCRFCD, 1999; Douglas County, 2008). Thus, the maximum velocities predicted for pipe *P8* and culverts *C7* and *C11* are acceptable, and no re-sizing of the conveyance system is required. Finally, the *Link Flow Summary* table shows that the *Max/Full Depth* value of all street conduits is less than one. Therefore the water depth in the streets never reaches the crown or the highest point of the sidewalks, and the requirements for the 100-yr storm are successfully satisfied.

To illustrate the complex flow conditions that can occur in these systems, Figure 7-8 shows the hydrographs in pipes *P5* and *P6* under the 100-yr storm. Negative flows caused by backwater effects occur in both pipes after 30 minutes of simulation. These special flow conditions can be only simulated when *Dynamic Wave* routing is used, which demonstrates the capabilities of this method. These negative discharges do not occur for the 2-yr storm. This shows that the behavior of a system under a minor and a major event can be quite different. In this case, the runoff generated by subcatchment *S4* into junction *J7* for the 100-yr storm is large enough to cause a backwater effect that generates these negative flows.

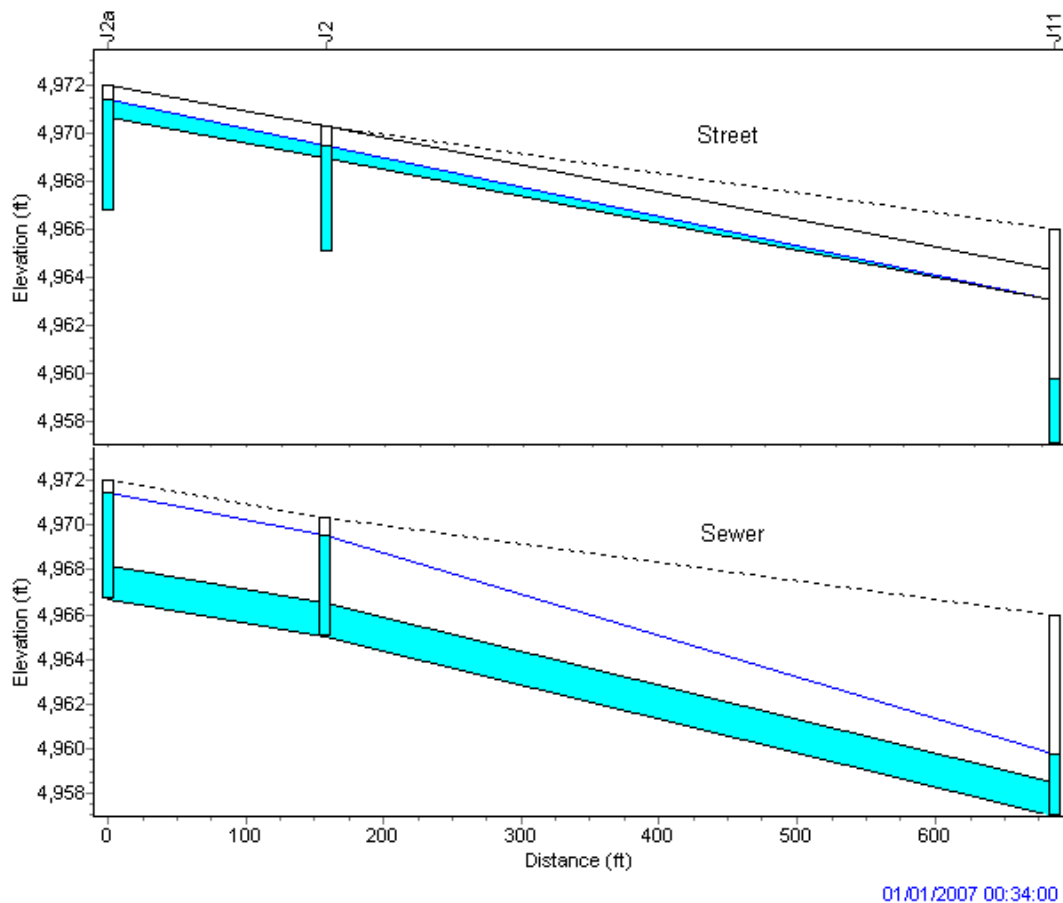


**Figure 7-8. Flows in pipes P5 and P6 during the 100-yr storm**

Another way to visualize the behavior of the dual drainage system is with *Profile Plots*. Figure 7-9 contains two such plots stacked on top of one another. They depict the surcharge condition that occurs between nodes *J2a*, *J2*, and *J11* at 34 minutes into the 100-yr storm. The lower plot applies to the below-ground sewer pipes *P2* and *P3* and shows that both pipes are surcharged. The upper plot is for the streets *C2a* and *C2* that lie above *P2* and *P3*. The water levels at junctions *J2a* and *J2* are high enough to cause water to flow out of the sewer pipe *P2* and onto the street, but not high enough to flood the street. On the other hand, the invert elevation of junction *J11* is low enough so that the surcharged pipe *P3* does not create street flooding and instead, street flow re-enters the sewer system there. (The plot makes it appear that the flow at the downstream end of street *C2* is zero, but this is just an artifact of the way that SWMM draws the water surface profile within a conduit (by connecting the water elevations between its end nodes without allowing the profile to cross either its bottom or top surface)).

## 7.5 Summary

This example built on the simple drainage system modeled in Example 2. It converted some of the open surface channels used in that example to parallel pipe and gutter systems and added a new series of pipes to drain the downstream section of the park. The system was sized for a minor (2-yr) storm event and its behavior was also analyzed under a major (100-yr) storm event. The key points illustrated in this example were:



**Figure 7-9. Surcharge behavior along the eastern boundary of the site**

1. The three-dimensional structure of dual drainage systems can be modeled by using manhole junctions set below ground that connect parallel pairs of sewer pipes and gutter/street channels, where the latter are offset to ground elevation.
2. These systems are designed so that the below-ground sewer system will flow only partly full during the more frequent minor storm events and will surcharge into the street channels during the larger, less frequent major events without causing any overtopping of the street.
3. An iterative process can be used to properly size the sewer pipe elements to meet both the small storm and large storm design criteria.
4. Most of the results needed to evaluate the performance of a dual drainage system can be found in the various tables produced by SWMM's *Status Report*.

As this example shows, dual drainage system models require a significant amount of additional effort to set up. The need to represent both the minor and major portions of a drainage system will depend on the objectives set forth for the analysis and on the level of detail required from the model.

## Example 8. Combined Sewer Systems

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This example demonstrates how to model systems that convey both sanitary wastewater and stormwater through the same pipes. Systems like these are known as combined sewer systems and are still quite common in older communities and cities. During periods of moderate to heavy rainfall the capacity of these systems to convey and properly treat the combined flow can be exceeded, resulting in what are known as Combined Sewer Overflows (CSOs). CSO discharges can cause serious pollution problems in receiving waters. Contaminants from these discharges can include conventional pollutants, pathogens, toxic chemicals and debris.

This example will use SWMM to analyze the occurrence of overflows in a combined sewer system. Particular attention is paid to properly representing the flow regulators that divert flow between collection sewers, treatment plant interceptors and CSO outfalls. In addition, the example shows how to model a pump station that conveys the intercepted flow through a force main pipeline to the headworks of a treatment facility. Although the focus of this example is on combined systems, many of the same modeling elements it employs (wastewater inflows, pump stations, and force mains) can also be used to model separate sanitary sewer systems.

### 8.1 Problem Statement

Rather than being a new development, the 29 acre urban catchment studied in Example 2 is now assumed to be an older area that is served by an existing combined sewer system. The hydraulic behavior of this combined system, including the magnitude of any overflows, will be analyzed for several different size storms. These include the 0.23 in. water quality storm defined in Example 3 as well as the 1.0 in., 2-yr and 1.7 in., 10-yr storms used throughout the previous examples.

Combined sewer pipes conveying both wastewater and stormwater flows generated within different sewersheds (i.e., areas that contribute wastewater flows to a single point) will be added to the model. Constant wastewater flows (also known as dry-weather flows) will be based on average generation rates per capita. An interceptor pipe will be sized to convey both the base dry weather flow and a portion of the combined stormwater flow to a pump station that pumps to the headworks of a wastewater treatment plant (WWTP) through a force main. Various combinations of orifices, weirs and pipes will be used to represent different types of flow diversion structures located within the interceptor. The CSOs that cannot be diverted by these devices will discharge directly into the stream running through the site's park area.

The schematic representation of the combined sewer system modeled in this example is shown in Figure 8-1. It includes the combined sewer pipes (in green) that drain the subcatchments (or sewersheds) *S1*, *S2*, *S3*, *S4* and *S5*, the stream (in blue), the interceptor (in brown), the flow regulators (red boxes), and the pump station.

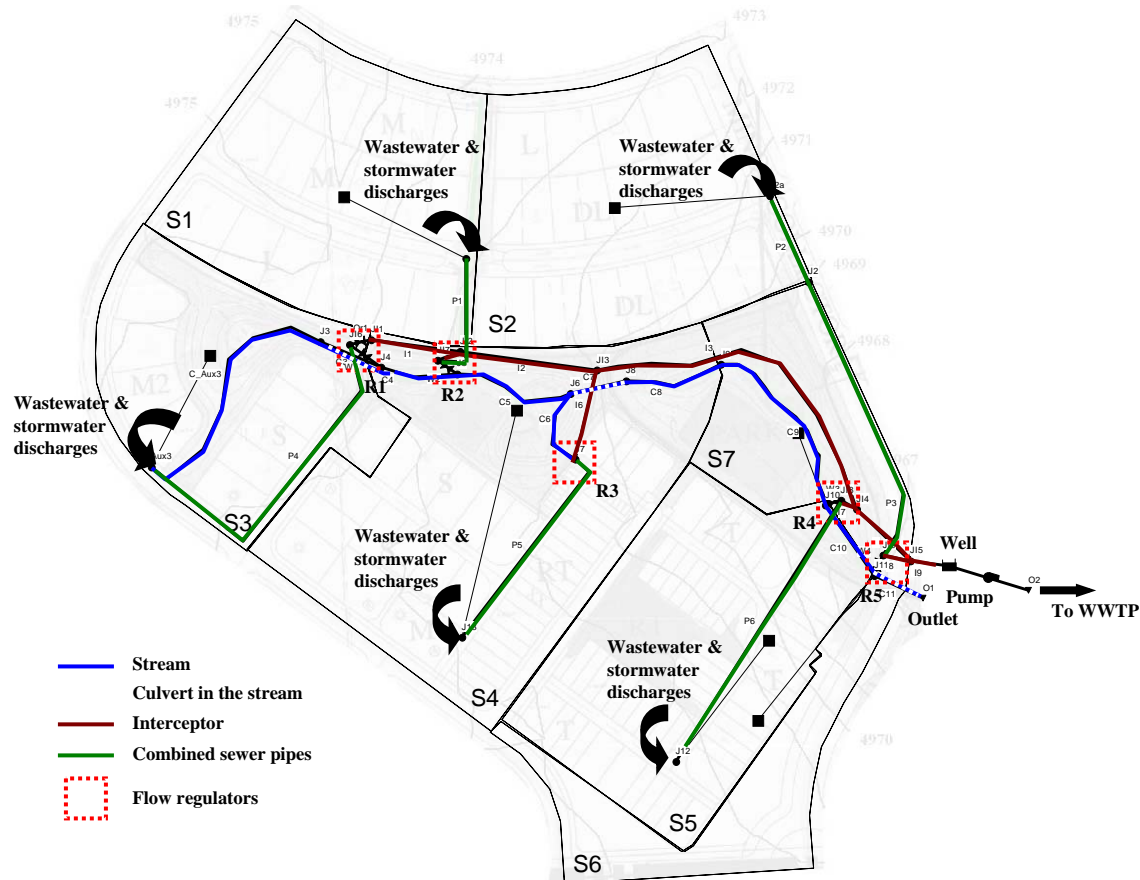


Figure 8-1. Combined sewer system study area

## 8.2 System Representation

Combined sewer systems are systems that convey both sanitary sewerage and stormwater through the same pipes. Interceptors are pipes designed to capture 100% of the sanitary flows during dry weather periods and convey them to a WWTP. During periods of moderate or heavy rainfall, however, the wastewater volume in the combined sewer system can exceed the capacity of the interceptor or the WWTP. For this reason, combined sewer systems are designed to discharge the excess wastewater directly to a nearby stream or water body through diversion regulators. Figure 8-2 shows a schematic representation of a combined sewer system and CSO occurring in the system. The figure shows how for wet-weather flows the interceptor at the bottom is able to convey only part of the flow into the WWTP and CSOs occur.

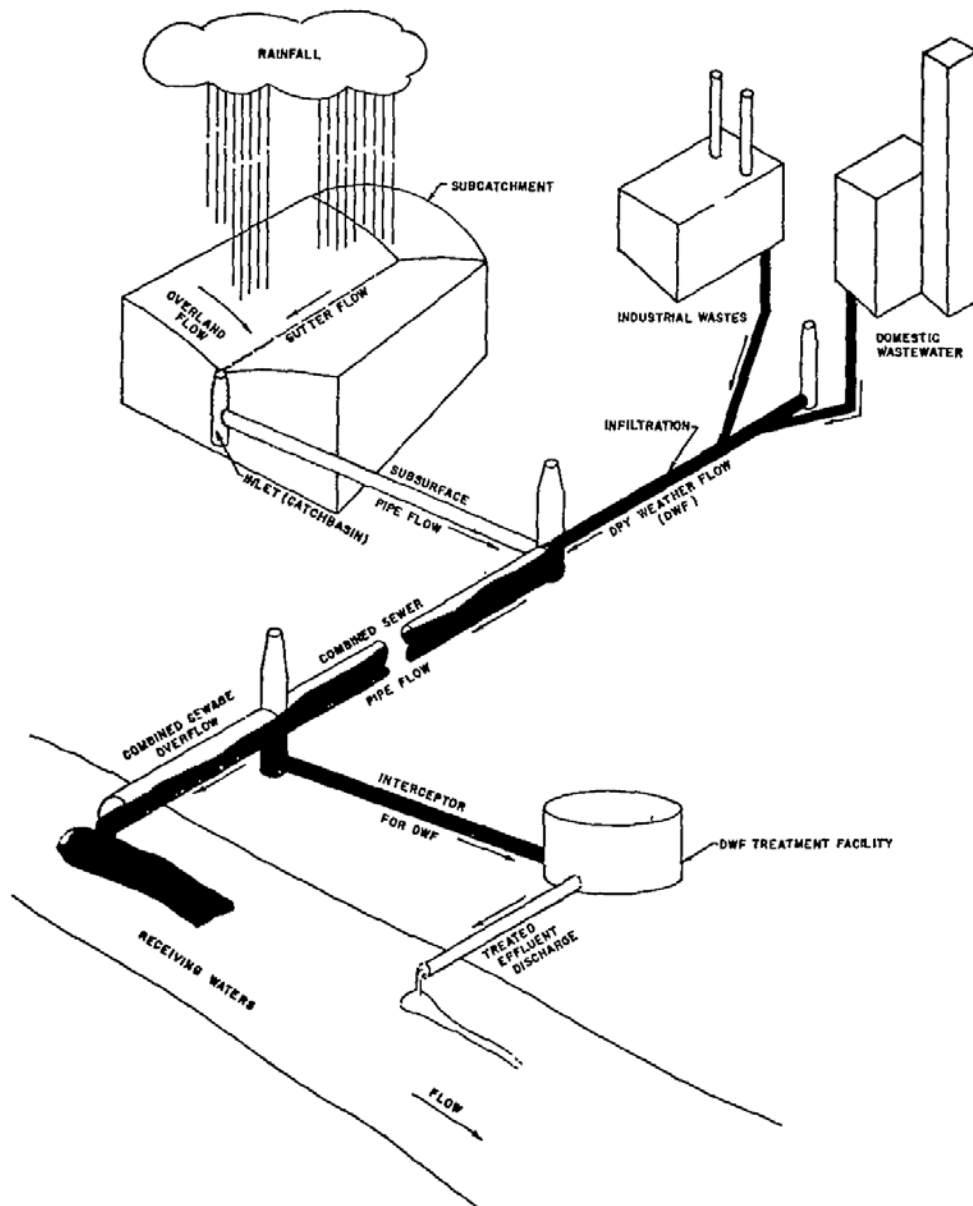


Figure 8-2. Conceptual representation of overflows in a combined system (Field and Tafuri, 1973)

## Dry Weather Flows

To create a combined sewer system in SWMM one adds dry weather wastewater flows into the appropriate nodes of a previously created stormwater conveyance system. These nodes typically represent locations where collector sewers discharge into trunk sewers. Their number and location will depend on the level of aggregation used to combine individual wastewater sources (homes, businesses, etc.) together. The sidebar below explains how to use a node's *Inflow Editor* to specify the time series of dry weather flow entering the node.

### Adding Dry Weather Flows into SWMM

This example requires that dry weather flows representing the wastewater discharges be added into the combined sewer system model. The *Dry Weather* page of the *Inflow Editor* is used to specify a continuous source of dry weather flow or any pollutant entering a specific node of the drainage system. The *Inflow Editor* is accessed through the *Inflows* property of the node for which the flow is being defined. Dry weather flows in SWMM are characterized by an average (or baseline) value and up to four optional time patterns (TP) that can represent monthly, daily and hourly (for both weekday and weekend) variations. Dry weather flows are then computed as shown below:

**Dry weather flow at time  $t$  = (average value)\*(TP 1 <sub>$t$</sub> )\*(TP 2 <sub>$t$</sub> )\*...**

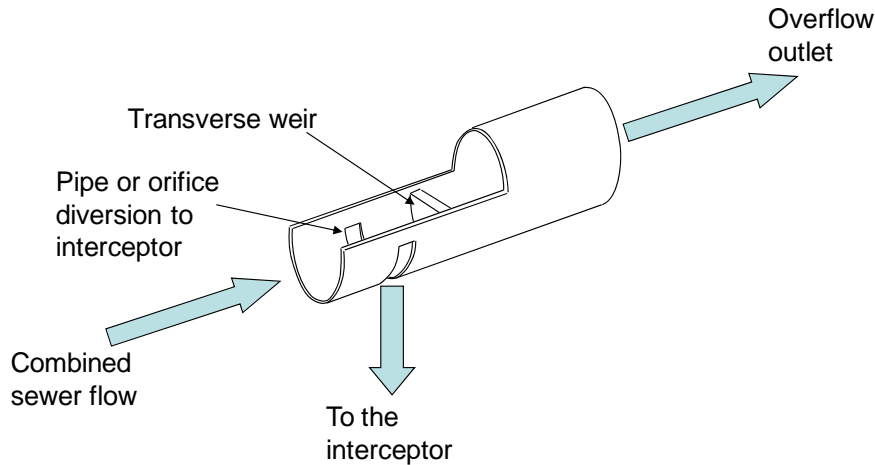
where **TP1 <sub>$t$</sub>**  is the multiplier for time pattern 1 at time  $t$ , **TP2 <sub>$t$</sub>**  is the multiplier for time pattern 2 and so on.

If no time patterns are defined then the dry weather flow entering the node is simply the average value. The *Inflow Editor* for the dry weather flow at a particular node is shown below.

The screenshot shows the 'Inflows for Node 31' dialog box. The 'Dry Weather' tab is active. The 'Constituent' dropdown is set to 'FLOW'. The 'Average Value (CFS)' is entered as 0.008. There are four empty 'Time Patterns' slots, each with a dropdown arrow, a help icon, and a delete icon. A note at the bottom states: 'NOTE: Leave Average Value field blank to remove any dry weather inflow for a given constituent at this node.' Buttons for 'OK', 'Cancel', and 'Help' are at the bottom.

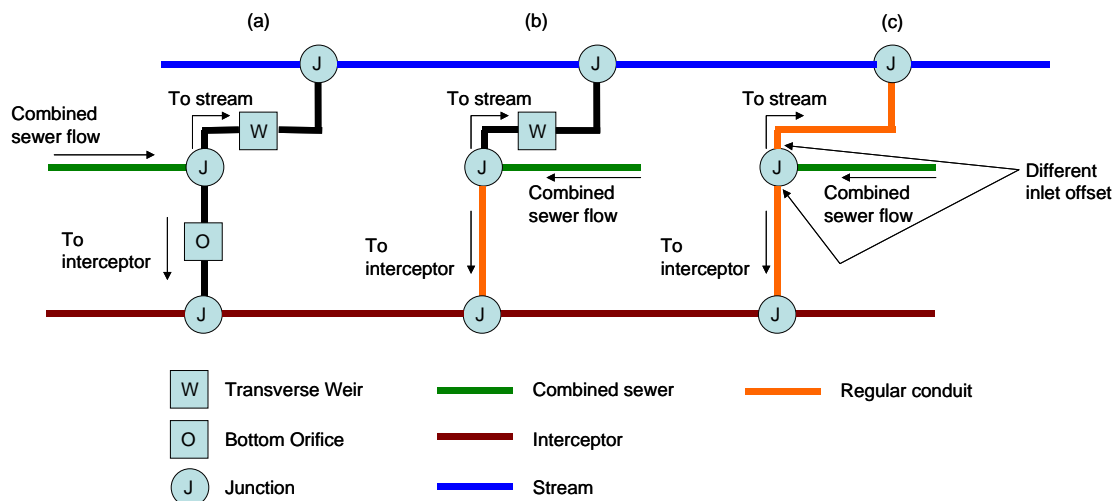
### **Flow Regulator Structures**

*Flow regulators* (or *diversion structures*) are used to control the flow between collection sewers and the interceptor. These regulators allow the conveyance of wastewater to treatment facilities during dry weather conditions. During wet weather conditions the regulators divert flows away from the interceptor and discharge directly into a water course to avoid surcharge and flooding of the combined sewer system. Flow regulator devices include side weirs, leaping weirs, transverse weirs, orifices and relief siphons. Metcalf & Eddy, Inc. (1991) presents a detailed description of these different devices. This particular example will use the *transverse weir with orifice* type of regulator illustrated in Figure 8-3. In this regulator there is a weir or a small plate placed directly across the sewer perpendicular to the line of flow. Low flows are diverted to the interceptor through an orifice located upstream of the weir. During periods of high flow, the weir is overtopped and some flow is discharged through the overflow outlet, eventually reaching a CSO outfall.



**Figure 8-3. Transverse weir flow regulator**

A transverse flow regulator can be represented in SWMM by using weir and orifice elements. Because these elements correspond to hydraulic links, additional junction nodes must be added into the model. A schematic representation of three possible definitions of a transverse flow regulator in SWMM is shown in Figure 8-4. Each of these three configurations will be used in this example. Configurations (a) and (b) both contain the weir shown in Figure 8-3, but use different elements to divert to the interceptor: (a) uses a bottom orifice while (b) uses a pipe. Finally, the third configuration (Figure 8-4c) uses neither a weir nor an orifice. Instead, it simply diverts flow by using different inlet offsets for the pipes that convey flows to the interceptor and to the stream. The first pipe has an inlet offset of zero while the pipe linked to the stream has a larger invert elevation.



**Figure 8-4. Alternative ways to represent transverse weir flow regulators in SWMM**



This example uses each of these regulator configurations for purely illustrative purposes. The choice of a particular configuration to use in a real application will depend on the specific conditions encountered in the field and on how numerically stable the resulting model will be. Some unwanted hydraulic phenomena that can be artificially introduced into the model by these different representations include surcharged weirs, instabilities caused by short pipes and excessive storage associated with large pipes.

## Pump Stations

Pumps are devices used to lift water to higher elevations. They are defined in the model as a link between two nodes and can be in-line or off-line. The principal input parameters for a pump include the identification of the inlet and outlet nodes, its pump curve, initial on/off status and startup and shutoff depths. A pump's operation is defined through its characteristic curve that relates the flow rate pumped to either the water depth or volume at its inlet node or to the lift (i.e., hydraulic head) provided. Its on/off status can be controlled dynamically by defining startup and shutoff water depths at the inlet node or through user-defined control rules. A pump is defined in the model in the same fashion as any other link, while the pump curve is created using the *Pump Curve Editor* and is linked to the pump by the latter's *Pump Curve* property as shown in Figure 8-5.

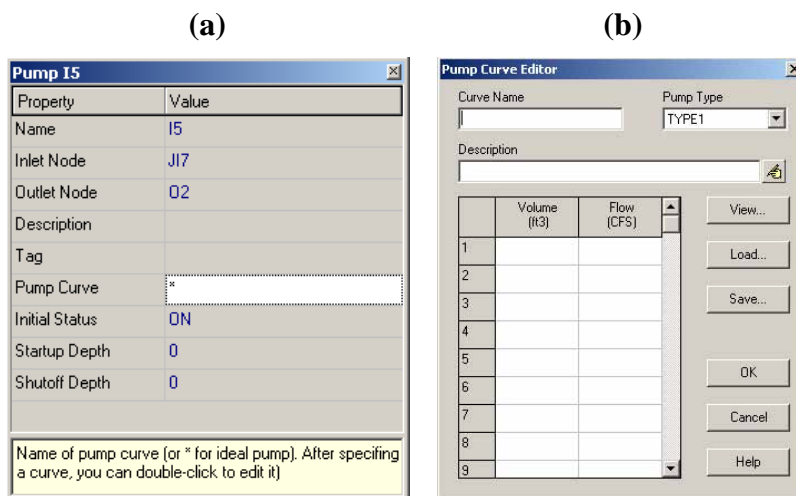


Figure 8-5. (a) Pump Property Editor and (b) Pump Curve Editor

## 8.3 Model Setup

### Preliminaries

Figure 8-6 shows the system to be modeled in this example. Example 7 (**Example7-Final.inp**) is the starting point for the model setup, although major changes are required. Because combined sewer systems are no longer used for new developments, this example assumes that the combined sewer system being modeled has been in place for many years.



S5. The upstream nodes (or inlet nodes) for these pipes, **J13** and **J12**, are added to the model as well. The properties of these junctions are shown in Table 8-1. The properties of the combined sewer pipes are shown in Table 8-2.

**Table 8-1. Properties of the combined sewer system nodes<sup>1</sup>**

Node ID	Invert Elevation (ft)	Maximum Depth (ft)	Node ID	Invert Elevation (ft)	Maximum Depth (ft)
J1	4969.0	4.2	J13	4968.0	4.8
J2a	4966.7	4.0	Aux3	4968.5	0.0
J2	4965.0	4.0	J11	4958.0	16.0
J3	4968.0	0.0	J12	4957.0	15.8
J4	4966.0	0.0	J13	4955.0	16.0
J5	4964.8	0.0	J14	4952.0	14.0
J6	4964.0	0.0	J15	4950.0	16.0
J7	4960.0	12.0	J16	4967.0	7.2
J8	4961.5	0.0	J17	4967.0	6.0
J9	4959.8	0.0	J18	4962.0	6.2
J10	4958.8	0.0	J19	4960.0	5.2
J11	4958.0	0.0	O1	4957.0	-
J12	4968.0	4.2	Well	4945.0	14.0

<sup>1</sup> Colors indicate whether the nodes belong to the stream (blue), the sewer pipes (green), the interceptor (brown) or they are the diversion junction (grey)

## Subcatchment Outlets

With the sewer pipes entered in the model, it is necessary now to redefine the outlet nodes of the different subcatchment. These nodes will receive the stormwater runoff generated by the subcatchments as well as the flows corresponding to the wastewater flows, defined later in this section. In other words, the subcatchments used for drainage are also defined as sewersheds in this example. No other changes to the properties of the subcatchments are required. Table 8-3 shows the new outlets for the subcatchments.

## Interceptor Pipe

An interceptor line is now added to the model that runs along the north side of the stream and conveys all wastewater flows to the pump station located on the east side of the study area. Its pipes are identified with the letter **I** and brown color, as shown in Figure 8-6. Conduits **I1**, **I2**, **I3**, **I4** and **I9** are the main pipes of the interceptor. The new nodes belonging to the interceptor are identified with the letters **J1** with the last node being the pump's wet well which is a storage unit named **Well**. Properties of the nodes and pipes of the interceptor are summarized in Tables 8-1 and 8-2. Figure 8-7 displays how the combined sewer system looks before the interceptor is connected to the rest of the system with flow regulators and the pump station is defined.

**Table 8-2. Properties of the combined sewer system conduits<sup>1</sup>**

Pipe ID	Shape	Inlet Node	Outlet Node	Length (ft)	h or d (ft) <sup>2</sup>	Rough. Coeff.	b (ft) <sup>3</sup>	Z1 <sup>4</sup>	Z2 <sup>5</sup>	Inlet Offset (ft)	Outlet Offset (ft)
C3	Circular	J3	J4	109.00	2.25	0.016	0	0	0	0	0
C4	Trapezoidal	J4	J5	133.00	3	0.05	5	5	5	0	0
C5	Trapezoidal	J5	J6	207.00	3	0.05	5	5	5	0	0
C6	Trapezoidal	J7	J6	140.00	3	0.05	5	5	5	5	0
C7	Circular	J6	J8	95.00	3.5	0.016	0	0	0	0	0
C8	Trapezoidal	J8	J9	166.00	3	0.05	5	5	5	0	0
C9	Trapezoidal	J9	J10	320.00	3	0.05	5	5	5	0	0
C10	Trapezoidal	J10	J11	145.00	3	0.05	5	5	5	0	0
C11	Circular	J11	O1	89.00	4.75	0.016	0	0	0	0	0
C_Aux3	Trapezoidal	Aux3	J3	444.75	3	0.05	5	5	5	6	0
P1	Circular	J1	J17	185.39	1.33	0.016	0	0	0	0	0
P2	Circular	J2a	J2	157.48	1.5	0.016	0	0	0	0	0
P3	Circular	J2	J19	529.22	1.5	0.016	0	0	0	0	0
P4	Circular	Aux3	J16	567.19	1.67	0.016	0	0	0	0	0
P5	Circular	J13	J7	377.76	1.67	0.016	0	0	0	0	0
P6	Circular	J12	J18	498.42	1.67	0.016	0	0	0	0	0
I1	Circular	J11	J12	150.36	1	0.016	0	0	0	0	0
I2	Circular	J12	J13	230.38	1	0.016	0	0	0	0	0
I3	Circular	J13	J14	578.27	1.5	0.016	0	0	0	0	0
I4	Circular	J14	J15	124.45	1.5	0.016	0	0	0	0	0
I5	Circular	J17	J12	10.65	0.33	0.016	0	0	0	0	0
I6	Circular	J7	J13	153.02	0.66	0.016	0	0	0	0	0
I7	Circular	J18	J14	32.88	0.5	0.016	0	0	0	0	0
I8	Circular	J19	J15	47.72	0.5	0.016	0	0	0	0	0
I9	Circular	J15	Well	100	2	0.016	0	0	0	0	4

<sup>1</sup> Colors indicate whether the pipes belong to the stream (blue), the sewer pipes (green) or the interceptor (brown)

<sup>2</sup> h or d corresponds to the depth (Trapezoidal shape) or diameter (Circular shape)

<sup>3</sup> b corresponds to the bottom width (Trapezoidal shape)

<sup>4,5</sup> Z1 and Z2 correspond to the left and right slope (Trapezoidal shape)

**Table 8-3. Subcatchment outlets**

Subcatchment	Outlet Node
S1	J1
S2	J2a
S3	Aux3
S4	J13
S5	J12
S6	J11
S7	J10

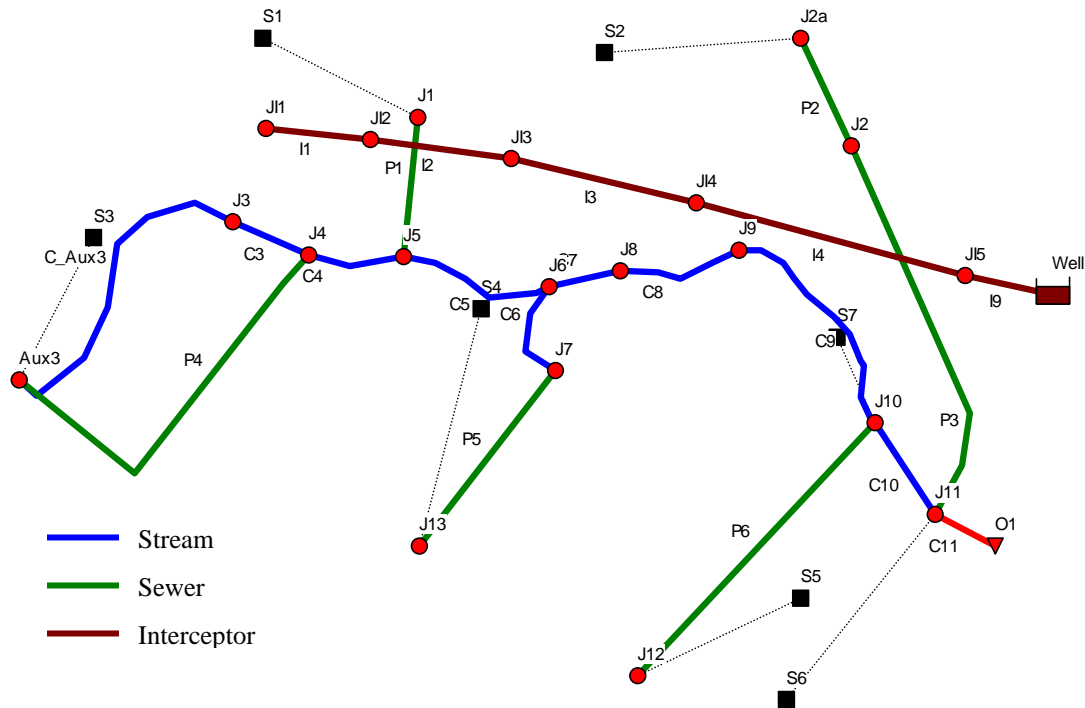


Figure 8-7. Layout of the combined sewer system before adding regulators and pump

## Flow Regulators

The flow regulator structures are the next elements to be represented in the model. Five regulators identified with the letter **R** will be used to control the flows from the five combined sewers (*P1*, *P3*, *P4*, *P5* and *P6*) into the interceptor. These identifiers (*R1*, ..., *R5*) are given only for reference purposes in the text; in the actual model regulators are not defined as elements directly, but rather through a combination of orifices, weirs and pipes (i.e., in the model there is no element named *R1*), as seen in Figure 8-6. The steps needed to create each of these regulators are as follows: (**Note: it is recommended that SWMM's *Auto-Length* feature be turned off so that conduit lengths are not altered as connections are made to the flow regulators.**)

### Regulator R1:

1. Place a new junction named **J16** somewhere between *J4* and *J11*.
2. Change the downstream node of pipe *P4* to *J16* (using the pipe's *Property Editor*).
3. Add a weir link **W1** connecting *J16* to *J4*.
4. Add a bottom orifice link **O1** connecting *J16* to *J11*.

Regulator R2:

1. Place a new junction **J17** somewhere between *J5* and *J12*.
2. Change the downstream node of pipe *P1* to *J17*.
3. Add weir link **W2** connecting *J17* to *J5*.
4. Add a new pipe **I5** connecting *J17* to *J12*.

Regulator R3:

1. Add a new pipe **I6** between *J7* and *J13*.

Regulator R4:

1. Place a new junction **J18** somewhere between *J10* and *J14*.
2. Change the downstream end of pipe *P6* to *J18*.
3. Add a weir link **W3** that connects *J18* to *J10*.
4. Add a pipe **I7** connecting *J18* to *J14*.

Regulator 5:

1. Place a new junction **J19** somewhere between *J11* and *J15*.
2. Change the downstream node of pipe *P3* to *J19*.
3. Add a weir link **W4** that connects *J19* to *J11*.
4. Add a pipe **I8** that connects *J19* to *J15*.

The resulting layout of the combined system with the regulators added is shown in Figure 8-6. Note that adding the regulator junctions *J16*, *J17*, *J18*, and *J19* required changing the discharge nodes of combined sewer pipes *P4*, *P1*, *P6*, and *P3*, respectively, from their original stream junctions to their respective regulator junctions.

To complete the process of adding in the regulators, the properties of the new junctions (*J16*, *J17*, *J18*, and *J19*) must be set using the data from Table 8-1. The same holds true for the new interceptor pipes (*I5*, *I6*, *I7*, and *I8*) whose properties are listed in Table 8-2. The dimensions and offsets for the newly added weirs and orifices (*W1*, *W2*, *W3*, *W4*, and *O1*) are listed in Table 8-4. Note that for each weir, the sum of its inlet offset and its opening height is smaller than the depth of the corresponding inlet node, defined in Table 8-1. The discharge coefficient for each of the weirs is **3.3** and is **0.65** for the orifice. In addition to these newly added elements, it is necessary to set the inlet offset for stream channel *C6* to **5** ft higher than the invert of junction *J7* so that it can serve as an overflow diversion while the new pipe *I6*, with no offset, connects *J7* to the interceptor (see Table 8-2)

**Table 8-4. Properties of the regulator structures**

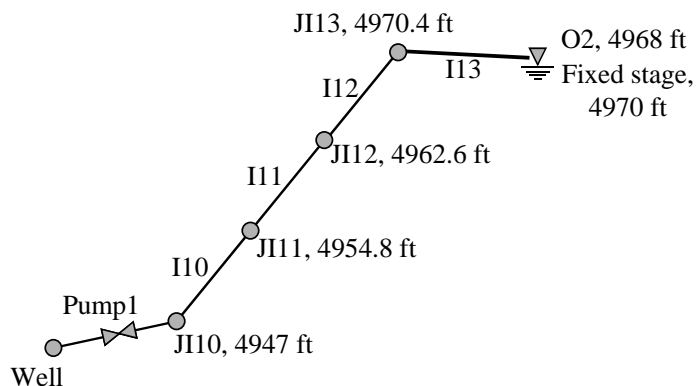
Flow regulator	Sewer pipe controlled	Inlet node	Discharge into stream					Discharge into interceptor			
			ID	Height (ft)	Length (ft)	Inlet offset	Outlet node	Type	ID	Diameter (ft)	Outlet node
R1	P4	J16	W1	6.5	3	0.4	J4	Orifice	Or1	0.5	J11
R2	P1	J17	W2	5.5	5	0.33	J5	Pipe	I5	See Table 8-2	
R3	P5	J7	C6		Pipe, see Table 8-3			Pipe	I6	See Table 8-2	
R4	P6	J18	W3	5	4	0.4	J10	Pipe	I7	See Table 8-2	
R5	P3	J19	W4	4.5	4	0.35	J11	Pipe	I8	See Table 8-2	

## Pump Station and Force Main

To complete the combined system model the pump station and force main must be added at the downstream end of the interceptor. As described earlier, node *Well* will serve as the wet well for the pump station. It is represented by a storage node with an invert elevation of **4945** ft, a maximum depth of **14** ft, an initial depth of **3** ft, and has a surface area of 300 ft<sup>2</sup> that remains constant over its entire height. To specify the latter, the *Storage Curve* entry in the node's *Property Editor* is set to **Functional**, the *Coefficient* and *Exponent* entries are **zero** and the entry for the *Constant* field is **300**.

To make room for the force main on the study area map, the map's dimensions must be expanded. Select **View | Dimensions** from the main menu bar. In the *Map Dimensions* dialog that appears enter **-235.8, -70.2** for the *Lower Left* coordinate and **1960.5, 1514.2** for the *Upper Right* coordinate. Do not select the “*Re-compute all lengths and areas*” option which will appear if the *Auto-Length* option is currently on.

There should now be room to add a series of four junctions downstream of the wet well node that defines the path of the force main (**J110**, **J111**, **J112**, and **J113**) along with the final outfall node (**O2**) that represents the WWTP (see Figure 8-8). The invert elevations of these nodes are **4947.0**, **4954.8**, **4962.6**, **4970.4**, and **4968.0** ft, respectively. As explained in the sidebar “*Defining a Force Main Pipeline*” all of the junction nodes in the main are assigned **zero** maximum depth and a surcharge depth of **265** ft so that they can pressurize (to at least 115 psi) without flooding. The outfall **O2** is of type **Fixed**, with a fixed stage elevation of **4970.0** ft.



**Figure 8-8. Force main line**

## Defining a Force Main Pipeline

The junctions used to connect sections of a closed force main together are typically welded or bolted fittings that do not allow water to escape the junction when it becomes surcharged. This type of junction can be modeled in SWMM by using a junction node whose *maximum depth* is zero and whose *surcharge depth* is set to an arbitrarily high number (such as the burst pressure of the main). Using a maximum depth of zero means that the actual depth will equal the distance from the node's invert to the top of the highest connecting conduit. A high surcharge depth is required to allow the node to remain in a pressurized state without causing any flooding.

Property	Value
Name	J110
X-Coordinate	1573.004
Y-Coordinate	618.390
Description	
Tag	
Inflows	NO
Treatment	NO
Invert EL.	4947
Max. Depth	0
Initial Depth	0
Surcharge Depth	50
Ponded Area	0

Depth in excess of maximum depth before flooding occurs (ft)

Any of SWMM's closed conduit shapes can be used for the cross-section of a force main, although circular pipes are most commonly used. SWMM also provides a special circular shape named **Force Main** which uses either the Hazen-Williams or the Darcy-Weisbach formulas to compute friction losses during pressurized flow instead of the Manning equation as would otherwise be the case. Some engineers prefer to use one of the former two head loss formulas instead of the Manning equation for pipes that are known to have pressurized flow almost all of the time. In the current example, the force main consists of simple circular pipes and the Manning equation is used throughout to compute losses.

After these nodes are created, a set of force main pipes **I10**, **I11**, **I12** and **I13** are added between nodes **J110**, **J111**, **J112**, **J113**, and **O2**, respectively (refer to Figure 8-8). The properties of these pipes are listed in Table 8-5.

**Table 8-5. Properties of the force main line**

Pipe ID	Shape	Inlet Node <sup>1</sup>	Outlet Node <sup>1</sup>	Length (ft)	Diameter (ft)	Rough. Coeff.	Inlet Offset (ft)	Outlet Offset (ft)
I10	Circular	J110 (4947 ft)	J111	500	2	0.016	0	0
I11	Circular	J111 (4954.8 ft)	J112	500	2	0.016	0	0
I12	Circular	J112 (4962.6 ft)	J113	500	2	0.016	0	0
I13	Circular	J113 (4970.4 ft)	O2 (4968)	500	4	0.016	0	0

<sup>1</sup> Number in parenthesis indicates the invert elevation of the node.

The final step is to add a pump link named **Pump1** between the nodes **Well** and **J110**. The pump curve (yet to be defined) associated with this pump is also named **Pump1**, the initial status is **OFF**, and the startup depth is **5** ft while the shutoff depth is **2** ft. This means that the pump turns on when the water depth in the wet well reaches 5 ft and it shuts down when the depth drops to 2 ft.



Each pump added to a SWMM model is required to have a curve that defines how the pump's discharge flow rate depends on the volume, depth or head added at its inlet (suction) side. This example will use a *Type 3* pump curve where discharge varies inversely with the head (or lift) added by the pump between its inlet and outlet nodes. This type of curve provides the most realistic representation of how pumps actually behave and is usually available from the pump's manufacturer. The head-discharge data that define the pump curve used here are listed in Table 8-6. To add this curve to the model:

1. In the *Data Browser*, select the *Pump Curve* sub-category.
2. Click the + button to bring up the *Pump Curve Editor* dialog.
3. Enter **Pump1** as the name of the curve and select **Type3** as the type.
4. Enter the Head-Discharge data from Table 8-6 into the grid on the form.

**Table 8-6. Pump curve data**

Head (ft)	11	12	13	14	15	16	17	18	19	20	21	22	23	24
Discharge (cfs)	7.2	6.9	6.5	6.1	5.7	5.2	4.7	4.1	3.6	2.9	2.3	1.5	0.8	0

## Dry Weather Flows

Wastewater flow from the sewersheds is added into the model using SWMM's *Dry Weather Inflow* tool (see the sidebar "*Adding Dry Weather Flows into SWMM*"). This example assumes that the individual subcatchments in the model also represent the individual sewersheds. Only average daily dry weather inflows will be used to keep things simple. In reality, peak daily flows can be two to four times greater than the average. Therefore, if a more accurate analysis is required, a diurnal time pattern should be used to capture the full range of dry weather inflows. This, together with a continuous rainfall record, would allow one to simulate the dynamic performance of the system under all types of events.

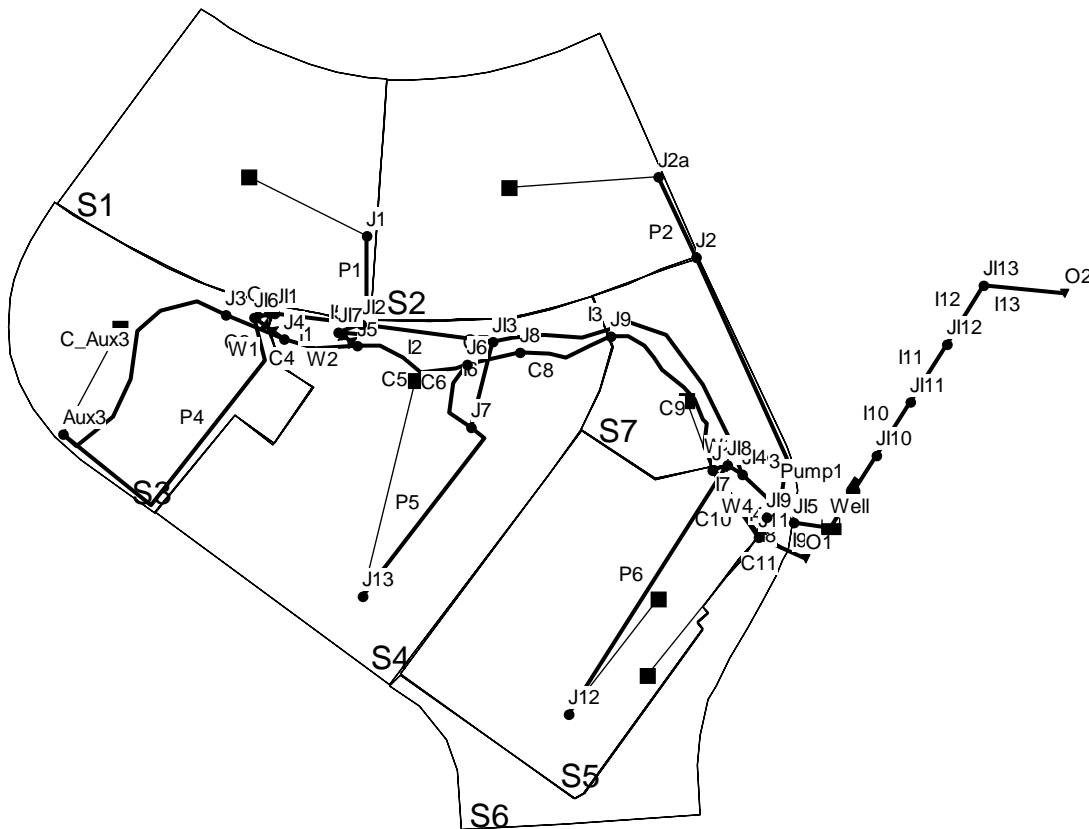
Typical per capita domestic wastewater generation rates vary between 40 gpd (gallons per day) for apartments and 150 gpd for luxury residences and estates (Nicklow et al., 2004). ASCE (1992) defines a range of average per capita domestic loading rates between 50 gpd and 265 gpd. Based on these ranges and an estimate of 3 to 5 inhabitants per lot, an average domestic loading rate per lot of 300 gpd is assumed. In addition to the domestic rates, the discharge rates from the commercial areas in subcatchments *S5* and *S6* also need to be included. These are estimated to be 2850 gpd in subcatchment *S5* and 8100 gpd in subcatchment *S5*. The dry-weather flows for the subcatchments are then computed as the sum of the domestic loading rate and the commercial rate. Table 8-7 summarizes this computation, showing the number of residential lots in each subcatchment (sewershed), the corresponding dry weather flows (in gpd and cfs) and the nodes that receive these inflows. Note that no dry weather flows are used in subcatchments *S6* and *S7* because they contain no residential or commercial lots.

**Table 8-7. Summary of dry weather flows**

Subcatchment	Node	Number of residential lots	Commercial dry weather inflow	Dry weather Inflow	
				gpd	cfs
S1	J1	17	-	5100	0.008
S2	J2a	22	-	6600	0.010
S3	Aux3	10	-	3000	0.004
S4	J13	17	2850	5100+2850	0.0123
S5	J12	-	8100	8100	0.0125
S6	J11	-	-	0	0
S7	J10	-	-	0	0

## Precipitation Data and Simulation Options

The 0.23 inches storm defined previously in Table 5-1, Example 5, and both the 2-year and 10-year storms will be used to evaluate the performance of the combined sewer system. The 0.23 in. storm is added to the model by creating a new time series called **0.23-in.** with the corresponding values of time and intensity. *Dynamic Wave* flow routing with a time step of **15 s**, a **1 minute** wet weather runoff time step, a **1 hr** dry weather runoff time step, a **1 minute** reporting time step and a total duration of **12 hours** will be used in all the simulations. All the information for the model has been summarized in Tables 8-1 to 8-7. The complete model input data can be found in the file named **Example8.inp** and the study area map should resemble that shown in Figure 8-9.



**Figure 8-9. Final combined sewer system model layout**

## 8.4 Model Results

### 0.23 in. Storm

The model is first run for the 0.23 in. storm event. Viewing the *Link Flow Summary* of the resulting *Status Report* shows that there is no flow in any of the links that divert water from the combined sewers into the stream (*W1*, *W2*, *W3*, *W4* and *C6*). Thus no CSOs occur for this size storm. Figure 8-10 shows the flow through each channel of the stream. The only channel with any flow is *C11* which receives only stormwater runoff (and no wastewater flow) from subcatchment *S7*. Flows in any other of the stream's conduits would imply that CSOs are occurring somewhere in the system.

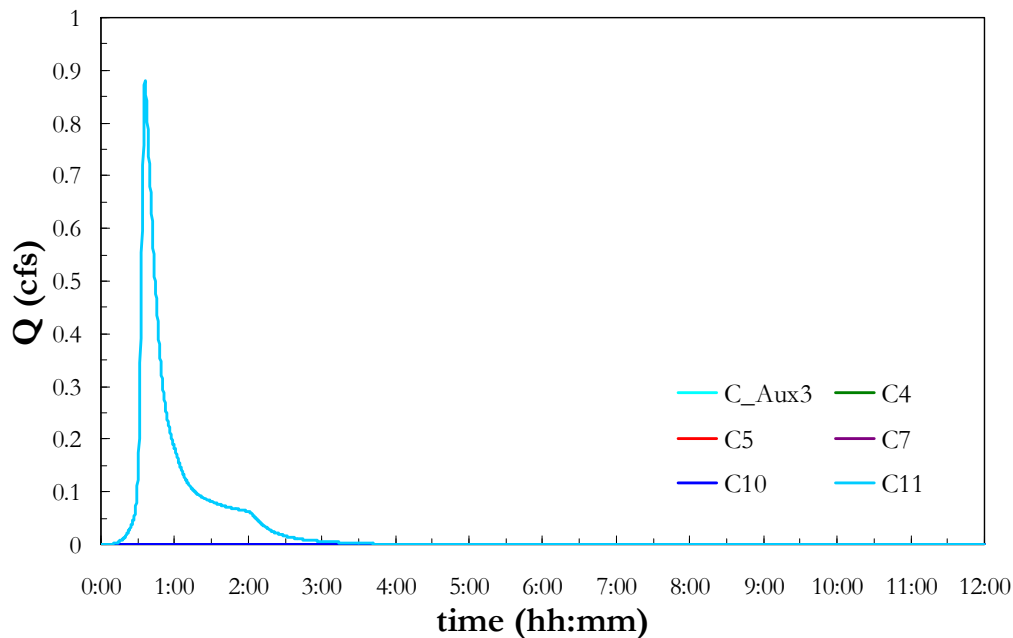


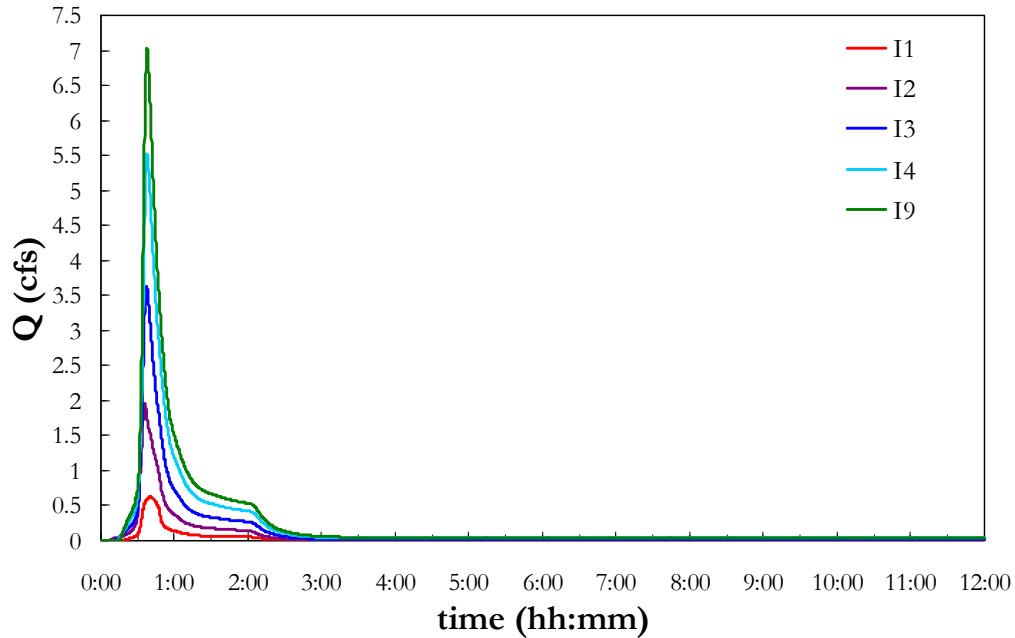
Figure 8-10. Flow (Q) along sections of the stream for the 0.23 in. storm

Figure 8-11 shows the flows through each section of the interceptor for the 0.23 in. storm. Note how the peak discharges and volumes increase in the downstream direction (from *I1* to *I9*) as the different combined sewer pipes discharge into the interceptor through the flow regulators, as illustrated in Figure 8-12. The latter figure shows that all the orifices and pipes that connect the combined sewers to the interceptor are contributing flows to the interceptor. In this figure, the first ID identifies the link of the regulator that carries the diverted flow; the second ID corresponds to the flow regulator to which it belongs.

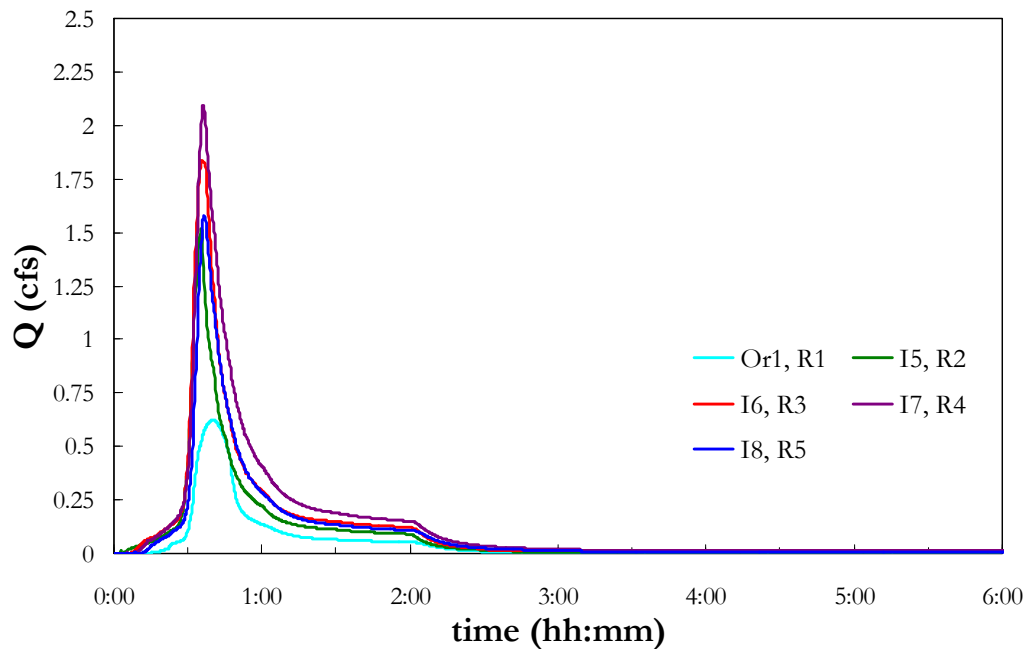
### 2-yr Storm

Results obtained for the 2-yr storm (with a volume of 1.0 in.) are now presented. Figures 8-13 and 8-14 show the flows through the various stream and interceptor sections, respectively. Note that for this larger storm, flow occurs in all sections of the stream. The *Link Flow Summary* of the *Status Report* shows that CSOs occur and all the

flow regulators start discharging flow into the stream once the capacity of diversion into the interceptor has been reached. The irregular fluctuations in flow through the interceptor seen in Figure 8-14 are caused by fluctuations in the flow pumped into the force main at the pump station.



**Figure 8-11. Flow (Q) along sections of the interceptor for the 0.23 in. storm**



**Figure 8-12. Flow (Q) diverted from each regulator to the interceptor for the 0.23 in. storm**

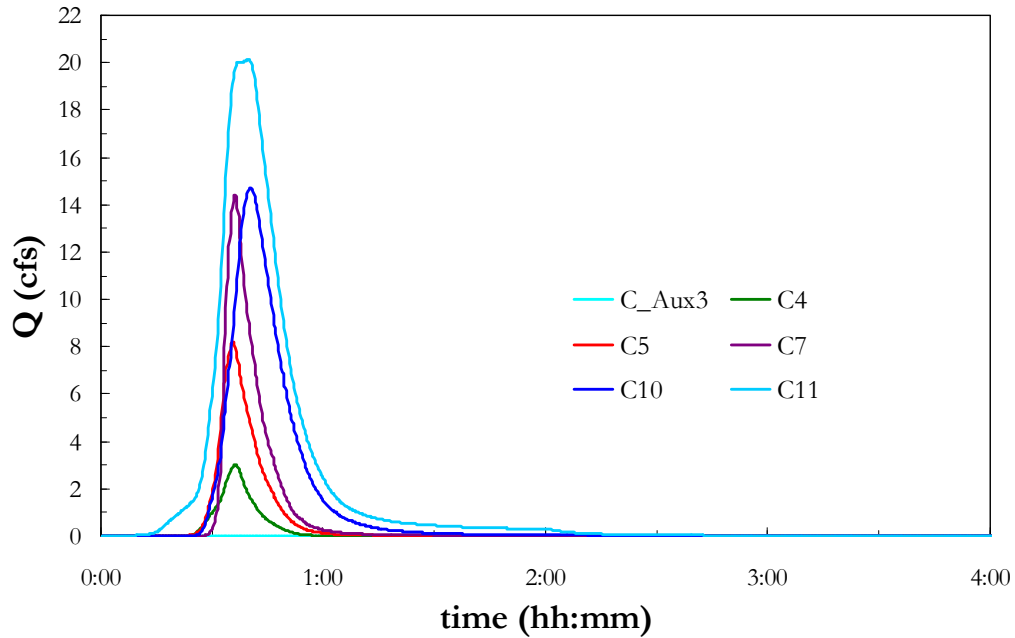


Figure 8-13. Flow ( $Q$ ) along sections of the stream for the 2-yr storm

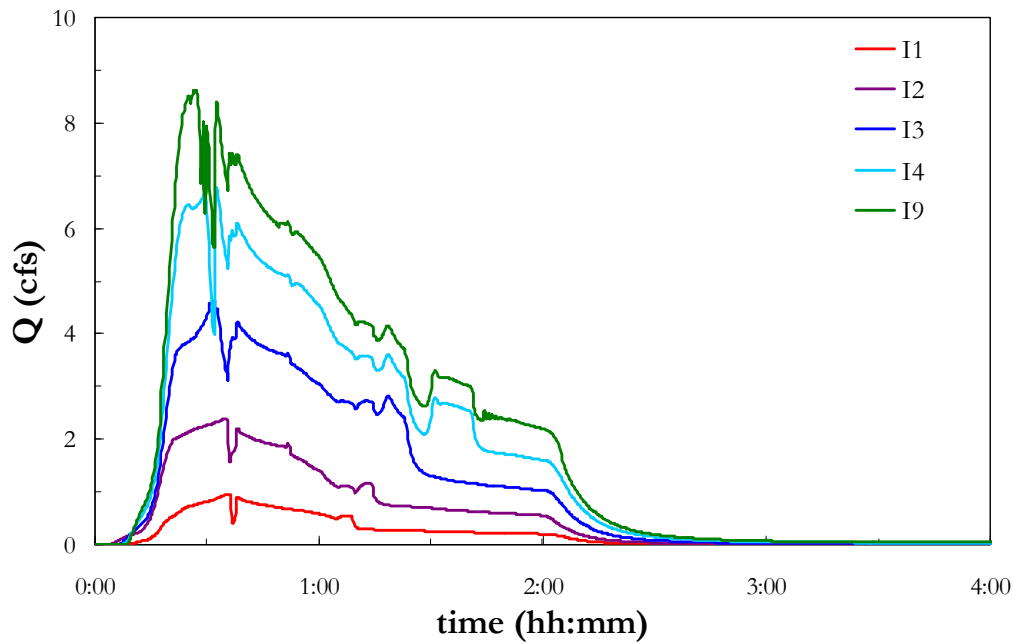
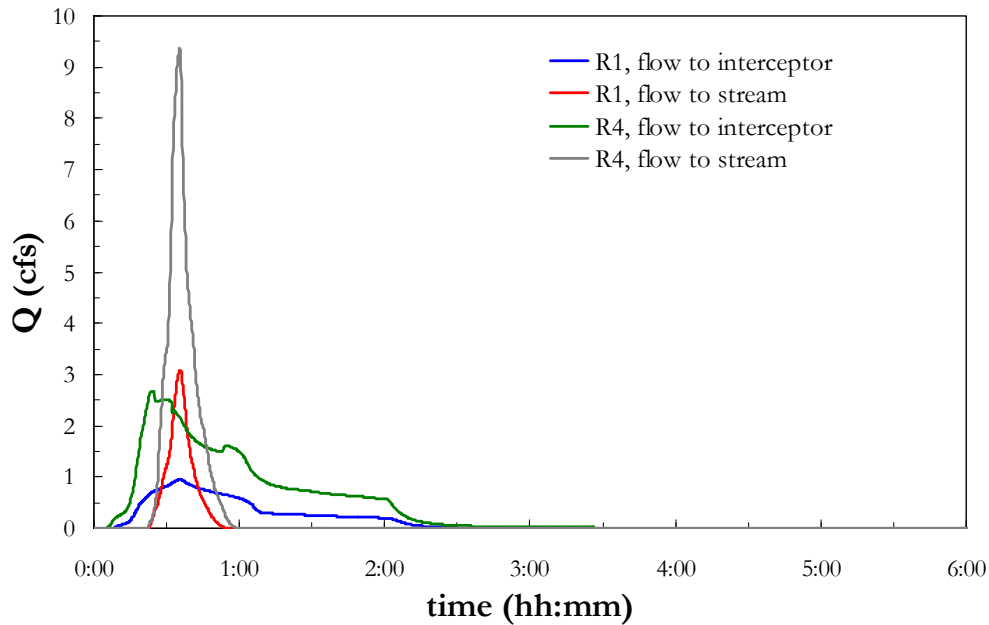


Figure 8-14. Flow ( $Q$ ) along the sections of the interceptor for the 2-yr storm

Figure 8-15 compares the flows that are diverted into the interceptor and to the stream CSO for flow regulators *R1* and *R4*. The interceptor diversions are from orifice *Or1* and conduit *I7*, while the CSO discharges are from weirs *W1* and *W3*. The regulators are able to convey the discharges into the interceptor for a while, but at a certain point the maximum capacity is reached and a CSO occurs. Note that the durations of CSOs are

smaller than those of the discharges into the interceptor; however, the peak discharge is larger.



**Figure 8-15. Flows (Q) in regulators *R1* and *R4* for the 2-yr storm**

### Pump Station Behavior

Figure 8-16 shows how the pump in this combined system behaves under the 0.23 in. event. Part (a) of the figure plots the water depth in the pump's wet well and part (b) shows the pump's discharge flow rate. The startup and shutoff depths are shown with dashed lines. At the start of the simulation the water depth is 3 ft and the pump is initially off (Point 1). The pump turns on once the startup depth of 5 ft is reached (Point 2). Inflow to the pump station is large enough so that the pump continues working and the wet well water depth stays above the shutoff level. The water depth eventually reaches a maximum (Point 3) and then starts decreasing until reaching the 2 ft shutoff depth at which time the pump stops operating (Point 4). After the runoff flow ceases some 2+ hours into the simulation, only wastewater flows are received at the wet well, and the water depth increases slowly again until reaching the startup depth (Point 5); the pump turns on but rapidly stops when the shutoff depth is reached (Point 6). From hereafter pumped discharges fluctuate between the startup and shutoff limits

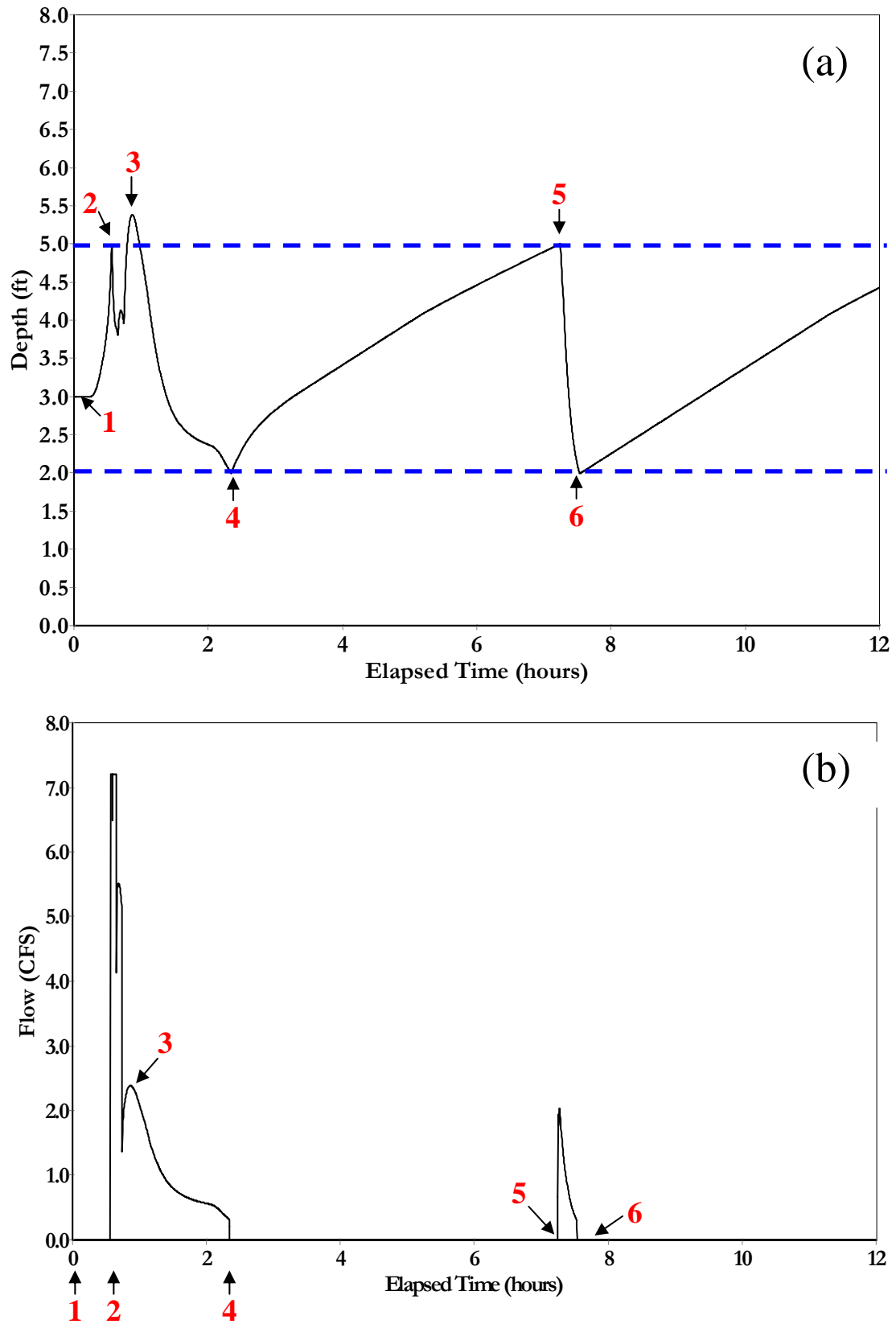


Figure 8-16. Pump behavior for the 0.23 in. storm: (a) wet well water depth, (b) pump flow

## Overall Performance

Table 8-8 compares the main results for the 0.23 in., 2-yr, and 10-yr storms. No CSOs occur for the 0.23 in. storm and all the wastewater flow is diverted into the interceptor. For the 2- and 10-yr storms, all the regulators are releasing discharges into the stream. Note how the occurrence of CSOs is reflected in the large increase in peak discharge at the receiving stream outfall *O1*. The peak discharge changes from 0.88 cfs for the 0.23 in. storm to 20.02 cfs for the 2-yr storm and 45.65 cfs for the 10-yr storm. Table 8-8 also shows that the maximum discharge conveyed by the interceptor barely changes with the magnitude of the storm once all the regulators are discharging CSOs to the stream. The maximum water depth in the wet well is practically the same for the 2- and 10- year storm. This result clearly shows that flow regulators work in a way such that all the flows above the diversion capacity are directly discharged into the water body.

**Table 8-8. Summary of system performance for different storm events**

<b>Node ID</b>	<b>0.23 in.</b>	<b>2-yr (1.0 in.)</b>	<b>10-yr (1.7 in.)</b>
Max. wet well water depth (ft)	5.35	13.49	13.86
Max. interceptor pump flow (cfs)	7.2	7.2	7.2
Max. flow at stream outfall <i>O1</i> (cfs)	0.88	20.02	45.65
Max. overflow at regulator R1 (cfs)	0	3.11	5.92
Max. overflow at regulator R2 (cfs)	0	5.88	8.76
Max. overflow at regulator R3 (cfs)	0	7.13	13.44
Max. overflow at regulator R4 (cfs)	0	9.35	13.44
Max. overflow at regulator R5 (cfs)	0	6.60	8.72

## 8.5 Summary

This example showed how to model a combined sewer system, composed of combined sewer pipes, flow regulators, a pump station and a force main line, within SWMM. The resulting model was used to determine the occurrence of Combined Sewer Overflows (CSOs) under different size storm events. The key points illustrated in this example were:

1. The main components of a combined sewer system model are pipes that carry both dry weather sanitary and wet weather runoff flows, flow regulators that divide flow between an interceptor pipe and a CSO outfall, and, if required, pump stations that carry interceptor flows to a treatment facility through a force main.
2. Continuous wastewater flows, which can vary periodically by time of day, day of week, and month of the year, are directly added into nodes associated with collector sewers that service individual sewersheds.
3. Flow regulators can be represented using a combination of pipes, orifices and weirs. The best way to model these regulators will depend on local conditions in the combined sewer system under analysis.
4. A pump station can be modeled using a storage unit node to represent the wet well that connects a pump link to the inlet node of a force main. The operation of the pump



is defined through a pump curve and a set of wet well water levels that determine when the pump starts up and shuts down.

5. A force main line can be defined by using a set of pipes between junction nodes that are assigned a high surcharge depth so that flooding does not occur when the main pressurizes.
6. By examining a number of design storms (or by running a continuous simulation as described in the next example) one can determine the frequency at which combined sewer overflows will occur. In this particular example, the 0.23 in. storm produced no overflows while the 2-yr and 10-yr storms did. As shown from an analysis of the rainfall record for this site made in Example 9, roughly 1 in 4 storms is larger than 0.23 in. and would thus have the capability to result in a CSO.

## Example 9. Continuous Simulation

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This example shows how to run a continuous simulation with SWMM using a long-term rainfall record. It will analyze the performance of the drainage system and BMP detention pond for the 29 acre residential site designed in Example 3. The multi-purpose pond was designed to detain a water quality capture volume (WQCV) and control peak post-development release rates to their pre-development levels for the 2-yr, 10-yr and 100-yr design storms. This model will be re-run using a set of monthly average evaporation rates and a continuous precipitation record, so that its behavior over a 10-year period can be studied. The use of SWMM's *Statistics* tool for analyzing the results of a continuous simulation will also be demonstrated. Because it only takes a few steps to set up Example 3 for a continuous simulation, the bulk of this chapter will focus on analyzing the results produced by the simulation.

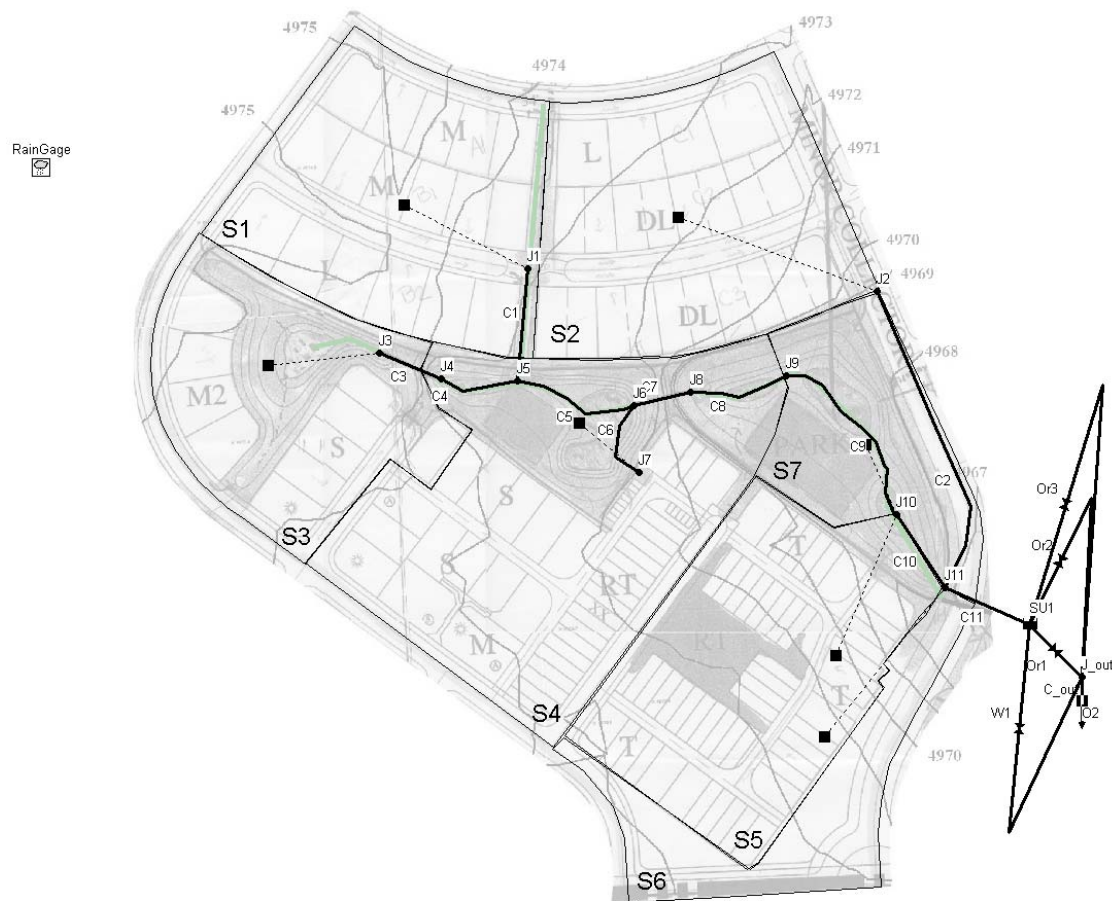
Continuous simulation is important because it allows actual historic data to be used to analyze the performance of drainage systems and their components. Drainage systems are typically designed using synthetic design storms. These single event design storms do not take into account varying patterns of rainfall duration and intensity, variation of time between storms, changing antecedent soil and storage conditions within the watershed, and the effect of evaporation. Continuous simulation considers all of these factors and allows for a more accurate and robust comparison of the long-term water balance and hydrologic performance of alternative stormwater management scenarios.

### 9.1 Problem Statement

Figure 9-1 shows the drainage system model developed in Example 3 for a new residential development on a 29 acre site. The system includes a BMP detention pond (*SUI*) that was designed to provide a water quality capture volume for the more frequently occurring small storms and also provide peak runoff control for the 2-, 10- and 100-yr design storms of 2-hr duration. In this example the site will be analyzed using continuous rainfall records from the city of Fort Collins, CO that were downloaded from the **National Climatic Data Center** (NCDC) along with a set of monthly average evaporation rates (in/day). The performance of the conveyance system and the detention pond will be studied over the ten year period of 1968 to 1978. The results of the simulation will be analyzed with regard to the following questions:

- Was the flow target used to design the detention pond adequate?
- How significant is evaporation within the overall system water balance?
- How significant are antecedent conditions in affecting system behavior?
- How effective is the detention pond in reducing peak discharges?
- How effective is the pond in capturing the majority of runoff events within its water quality control volume?

- What statistical properties of the rainfall record set it apart from the design storms used previously?



**Figure 9-1. Drainage system and detention pond (SUI) designed in Example 3**

## 9.2 System Representation

Continuous or long term simulation involves the simulation of multiple events over a continuous period of months to years. In single-event simulation the rainfall record is first analyzed separately to create an Intensity-Duration-Frequency (IDF) curve from which design storms can be chosen; these design storms have a specific duration and return period (e.g. the 2-hr 10-yr design storm) and are used within SWMM to generate a single runoff hydrograph for design. In comparison, continuous simulation applies a long term record of rainfall directly within SWMM to generate a long-term record of simulated runoff; statistical analysis is then run on this generated record to characterize the long-term performance and achieve a final design. It is advantageous to use continuous simulation because it accounts for antecedent soil conditions and other initial values of variables, such as the initial water level in storage units, which affect the response of the drainage system to individual storm events. It also allows the

representation of actual storm events of varying magnitudes, durations, and occurrence intervals. The main disadvantages of continuous simulation are the additional computation time required and the lack of high quality, long-term rainfall records for many locations.

## **Rainfall**

SWMM can utilize long-term rainfall data stored in external files. The program recognizes several different file formats for these data: (1) the National Climatic Data Center (NCDC) DSI-3240 format for hourly rainfall and the DSI-3260 format for 15 min rainfall (both available from [www.ncdc.noaa.gov/oa/ncdc.html](http://www.ncdc.noaa.gov/oa/ncdc.html)), (2) HLY03 and HLY21 formats for hourly rainfall and the FIF21 format for 15 min rainfall at Canadian stations, available from Environment Canada (EC) at [www.climate.weatheroffice.ec.gc.ca](http://www.climate.weatheroffice.ec.gc.ca), and (3) a standard user-prepared format as described in the *SWMM Users Manual*. The quality and quantity of the record will vary from station to station, but it is unusual to find long precipitation records with no missing or incorrect data. It is very important to check the quality of records before using them.

## **Evaporation**

Single event simulations are usually insensitive to the evaporation rate. Thus, evaporation is typically neglected when a single rainfall event or a synthetic storm is simulated. However, this process is more significant when a continuous simulation is performed because it is through evaporation that depression storage is recovered and water levels in extended detention and wet ponds are reduced; thus it becomes an important component of the overall water budget. Several options are available for representing evaporation data in SWMM, including: (1) a single constant value, (2) historical daily average values stored in an external file, (3) a time series when high temporal resolution is available, and (4) monthly averages. Evaporation data are supplied on the *Evaporation* page of SWMM's *Climatology Editor*.

Although conceptually evaporation should also affect the recovery of infiltration capacity within the pervious areas of the watershed, SWMM's infiltration models do not explicitly take it into account. Instead, they employ simple empirical functions for this purpose. The Horton infiltration model, used in the examples throughout this manual, employs an exponential function to restore infiltration capacity during dry periods. The rate coefficient in this function is inversely proportional to the soil's drying time, i.e., the number of days it takes a saturated soil to drain completely. A drying time of 7 days is used for the site being analyzed, which is a typical value for the silt loam soil that is assumed to cover the site (see Example 1).

## How to Load a Continuous Rainfall File into SWMM

A continuous precipitation record can be supplied to SWMM by using an external file as the data source. What follows are the steps used to identify such a file to SWMM.

1. Open the *Property Editor* of the Rain Gage that will use the rainfall data and select **FILE** as the *Data Source*.
2. Click the ellipsis button in the *File Name* field. Navigate through your files and select the file containing the continuous rainfall data.
3. If the file is in the **User-Prepared** format then the *Rain Format* and the *Rain Interval* of the data in the file must be specified in the *Property Editor*. For data from **NCDC** or **EC** files, SWMM automatically determines the appropriate rain format and rain interval to use, and will override any values specified for these properties in the editor.
4. Provide an entry for the *Station ID* field in the *Property Editor*. For **NCDC** and **EC** files any identifier can be used – it need not be the station ID used in the file. However for **User-Prepared** files it must correspond to an ID that appears in the file since these files can contain data for more than one station.
5. If a **User-Prepared** rainfall file is being used then the correct *Rain Units* for the rain depths in the file (inches or millimeters) must be specified. For **NCDC** and **EC** data files this property is determined automatically.
6. Finally, specify the desired start and end analysis dates in the *Simulation Options Editor* based on the rainfall record. These dates must fall within the period of record. Time periods that fall outside those contained in the rainfall file will have no rainfall associated with them.

The Rain Gage *Property Editor* for Example 9 is shown in the figure below. Note that the entry in the *File Name* property will be different depending on where you installed the rainfall file for this example on your computer's file system.

Property	Value
Name	RainGage
X-Coordinate	-148.485
Y-Coordinate	1207.602
Description	
Tag	
Rain Format	VOLUME
Rain Interval	1:00
Snow Catch Factor	1.0
Data Source	FILE
TIME SERIES:	
- Series Name	*
DATA FILE:	
- File Name	T:\projects\HHSP\Projects\Examples SWMM5, EPA\Jennifer\4 ->
- Station ID	053005
- Rain Units	IN
Units of rainfall data	

## Statistical Tools

The large amount of output produced by a continuous simulation requires statistical tools to analyze it in a concise and meaningful manner. SWMM provides an interactive statistical query tool that can be applied to any output variable associated with any specific object (subcatchment, node, or link) or the system as a whole. The tool performs the following steps when analyzing the statistics for a specific output variable:

- It first segregates the simulation period into a sequence of non-overlapping events either by day, month, year or cluster of consecutive reporting periods. The occurrence of these events is defined by the following minimum threshold values:
  - *Analysis Variable Threshold* - the minimum value of the variable under analysis that must be exceeded for a time period to be included in an event.
  - *Event Volume Threshold* - the minimum flow volume (or rainfall volume) that must be exceeded for a result to be counted as part of an event.
  - *Separation Time* - the minimum amount of time that must occur between the end of one event and the start of the next event. Events with fewer hours are combined together. This value applies only to events formed from consecutive reporting periods, not to daily, monthly or annual event periods.
- It then computes a user-specified event statistic for the analysis variable over all reporting time periods that fall within each event period. This could be the event mean value, the peak value, the total volume, etc. Thus each event is characterized by a single value for whatever variable is being analyzed.
- Summary statistics for the event values over the entire set of events are then computed. These statistics include the maximum, minimum, mean, standard deviation and skewness.
- Finally a frequency analysis for the collection of event values is performed. The events are rank ordered by value in a table that lists the event's date, its duration, its event value, its cumulative exceedance frequency, and estimated return period. A histogram of event values and a cumulative frequency plot of these values are also produced.

As an example, a typical query might ask SWMM to segregate the output record of flows discharged from a particular outfall into periods where the flow is above 0.05 cfs and there are at least 6 hours between events, and compute the summary statistics and frequency distribution of the peak flows within these events.

## 9.3 Model Setup

Only three steps are required to convert the SWMM input file for Example 3 (**Example3.inp**) into one that can be used for continuous simulation. These are:

### 1. Add evaporation data:

In this example evaporation is supplied as monthly averages in units of in/day. Table 9-1 shows the monthly averages for the city of Fort Collins that are used in this example; the values were obtained from the **Western Regional Climate Center** ([www.wrcc.dri.edu/htmlfiles/westevap.final.html#colorado](http://www.wrcc.dri.edu/htmlfiles/westevap.final.html#colorado)). The first row shows the total monthly pan evaporation in inches. These amounts are adjusted in the second row by multiplying them by 0.70 to more closely estimate the evaporation from naturally existing surfaces, as proposed by Bedient and Huber (2002). Finally, the third row shows the daily rates obtained by dividing the monthly totals by the number of days in each month. Note that total values from December to February are 0.00. Typically this means that the station does not measure pan evaporation during these months. The values in the last row are entered on the *Evaporation* page of SWMM's *Climatology Editor*.

Evaporation from the surface of the site's detention pond should also be accounted for. To do this, open up the *Property Editor* for the storage unit node *SU1* and enter a value of **1.0** for the *Evap. Factor* property.

**Table 9-1. Monthly evaporation for Fort Collins, CO (Source: Western Regional Climate Center)**

	J	F	M	A	M	J	J	A	S	O	N	D
Monthly average pan evaporation (in)	0.00	0.00	2.5	4.52	5.42	6.32	6.92	6.07	4.74	3.07	1.48	0.00
Monthly average corrected (in)	0.00	0.00	1.75	3.16	3.79	4.42	4.84	4.25	3.32	2.15	1.04	0.00
Monthly average rate (in/day)	0.00	0.00	0.06	0.11	0.12	0.15	0.16	0.14	0.11	0.07	0.03	0.00

### 2. Specify a rainfall file:

The 30-year hourly rainfall record provided for Fort Collins is linked into the model as explained in the sidebar "[How to Load a Continuous Rainfall File into SWMM](#)". The name of this file is **Record.dat**. Because this file is from the NCDC and adheres to the DSI-3240 format there is no need to alter the *Rain Format*, *Rain Interval*, or *Rain Units* properties of the model's single rain gage. The *Station ID* property can be set to the ID associated with the data's recording station which is **053005**. The period of record for these rain data extends from 1949 to 1979, but only the 10 year period from 1968 to 1978 will be simulated. Figure 9-2 shows the precipitation record for this period of time. There are many factors to consider when selecting a sub-period of a long-term record, such as the quality of the data it contains and how representative it is of both the overall record and of current rainfall patterns.

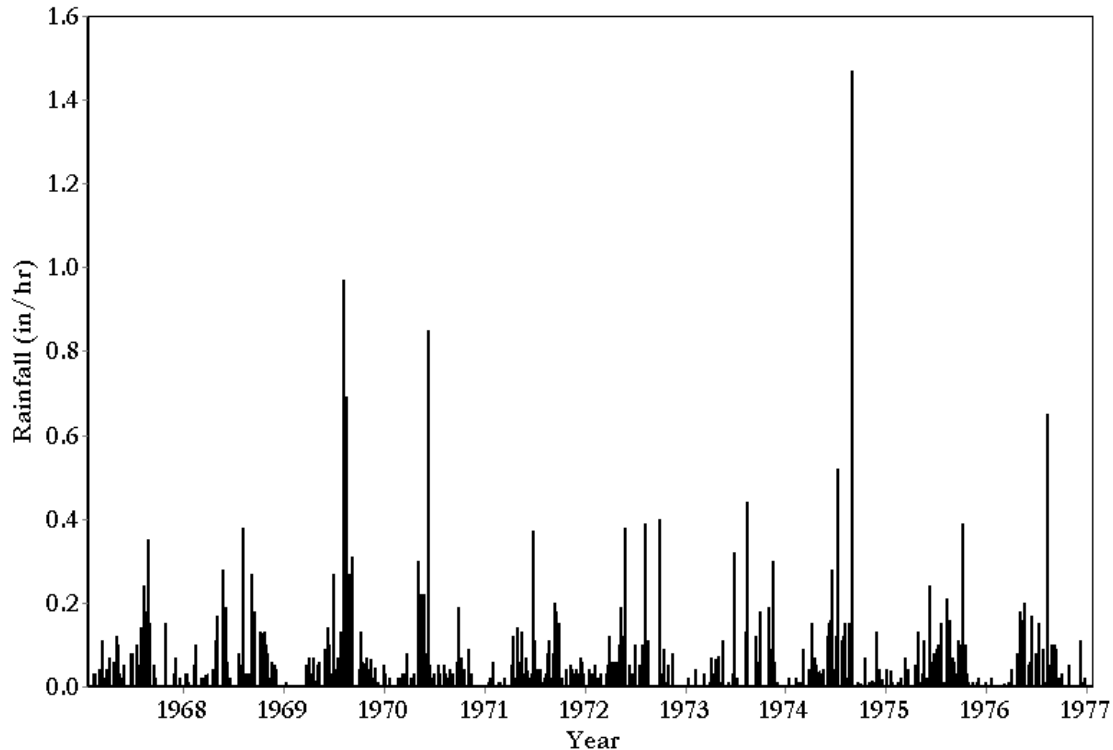


Figure 9-2. Ten year rainfall record for Fort Collins, CO (Source: National Climatic Data Center)

### 3. Revise the simulation options:

Finally it is necessary to change some of the original simulation options so that the correct period of rainfall record is simulated and a manageable amount of output is generated. Using the *Dates* page of the *Simulation Options* dialog, both the simulation start date and reporting start date are set to **01/01/1968**. The simulation ending date is set to **01/01/1978**. Next the following changes are made on the *Time Steps* page of the dialog: reporting step = **15** minutes, wet weather runoff step = **15** minutes, dry weather runoff step = **6** hours, and routing step = **60** sec. Note that these time steps are longer than those used in most of the previous examples. This is done to cut down on the amount of time needed to run the simulation and on the amount of output results that are produced.

After making these changes the modified input file should be named **Example9.inp**. Running this model will take about three to five minutes on a 2.41 GHz computer. Most of this time is consumed by the flow routing computations. In general, the run time will depend on the complexity of the watershed being modeled, the routing method employed and the size of the routing time step used. The larger the time steps, the faster the simulation but the less detailed the results. If accurate simulation of peak flows on small watersheds is desired, then smaller time steps must be used. One way to determine the proper time step is to simulate a single event at different time steps, then choose the longest time step that produces the desired resolution in the hydrograph but still produces a stable solution with acceptable continuity error.



The run time for this example could be cut in half if *Kinematic Wave* routing were used instead of *Dynamic Wave* routing. This is a viable alternative for long-term simulations if one is willing to ignore possible backwater effects and pressurized flows, and if the drainage network is not bifurcated. SWMM also has another feature that can possibly reduce run times. This is the *Skip Steady Periods* option that appears on the *General* page of the *Simulation Options* dialog. When this option is used the simulation skips over periods of time when there is no runoff and no changes in flow. The criteria used to determine when such periods exist are quite stringent, and for this example it resulted in a negligible savings in run time.

## 9.4 Model Results

### General Results

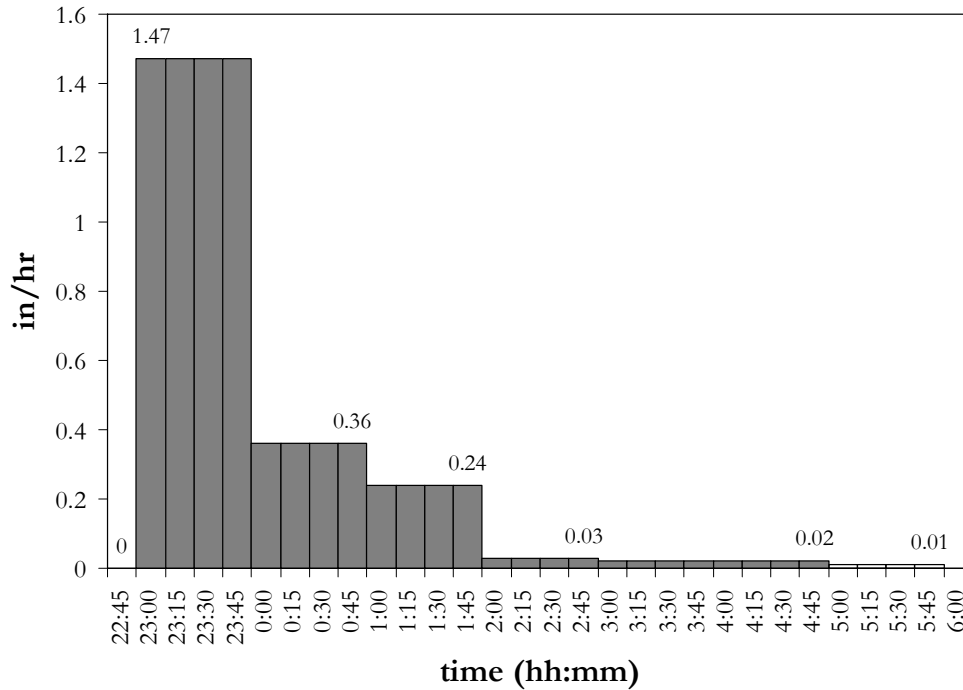
SWMM's *Status Report* summarizes overall results for the 10-yr simulation. The runoff continuity error is -0.412 %, the flow routing continuity error is -1.557 %, and no flooding occurred within the conveyance network (listed as *Internal Outflow* in the *Flow Routing Continuity* section of the report). This is expected because the conveyance system in Example 3 was designed for the 100-yr storm. The *Runoff Quantity Continuity* table shows the significance of evaporation in the total water budget. Almost 20 inches of water were evaporated from the depression storage surfaces of the catchment with another 1.2 inches evaporated from the pond; this corresponds to 19.2 % of the total rain over the 10 years (110.35 inches).

### Detention Pond

The *Status Report* section “*Node Inflow Summary*” shows that the maximum flow rate into the detention pond (*SU1*) was 31.7 cfs. The outflow from the pond never exceeded 8.75 cfs (as determined by the maximum inflow for junction *J\_out* in the same table). This control occurs over a period of 37 minutes, given that the maximum inflow to the pond was at 12:02 AM of day 2781, while the maximum discharge was at 12395 AM of the same day. Note that the maximum flow entering the pond is much lower than the initial estimate for the 10-yr storm computed in Example 3, 62.1 cfs. Interestingly, however, the maximum discharge released by the pond in the continuous simulation (8.48 cfs) is somewhat larger than 7.34 cfs, the 10-yr peak discharge target used in Example 3. In other words, even though the peak flow entering the pond during the 10 year record is lower than the design value, the peak outflow is actually larger than its original design value.

To explain this result, consider Figure 9-3 which shows the rainfall hyetograph for the storm event that produced the maximum inflow and outflow from the pond. The volume of rain that falls in the first two hours is 1.83 in. This is larger than the 1.71 in associated with the 10-yr 2-hr design storm used in Example 1, yet the inflow rate into the pond is smaller (31.7 cfs versus 62.1 cfs). The main reason for this discrepancy is the different rainfall interval used for the two storm events. The 10-yr design storm used a 5

minute interval and its maximum intensity was 4.87 in/hr (see Figure 1-6 in Example 1); in contrast, the rain interval for the storm from the continuous record is only 1 hr and its maximum intensity was 1.47 in, 30 % of the maximum intensity of the design storm.



**Figure 9-3. Rainfall event producing the largest flow rates to the detention pond**

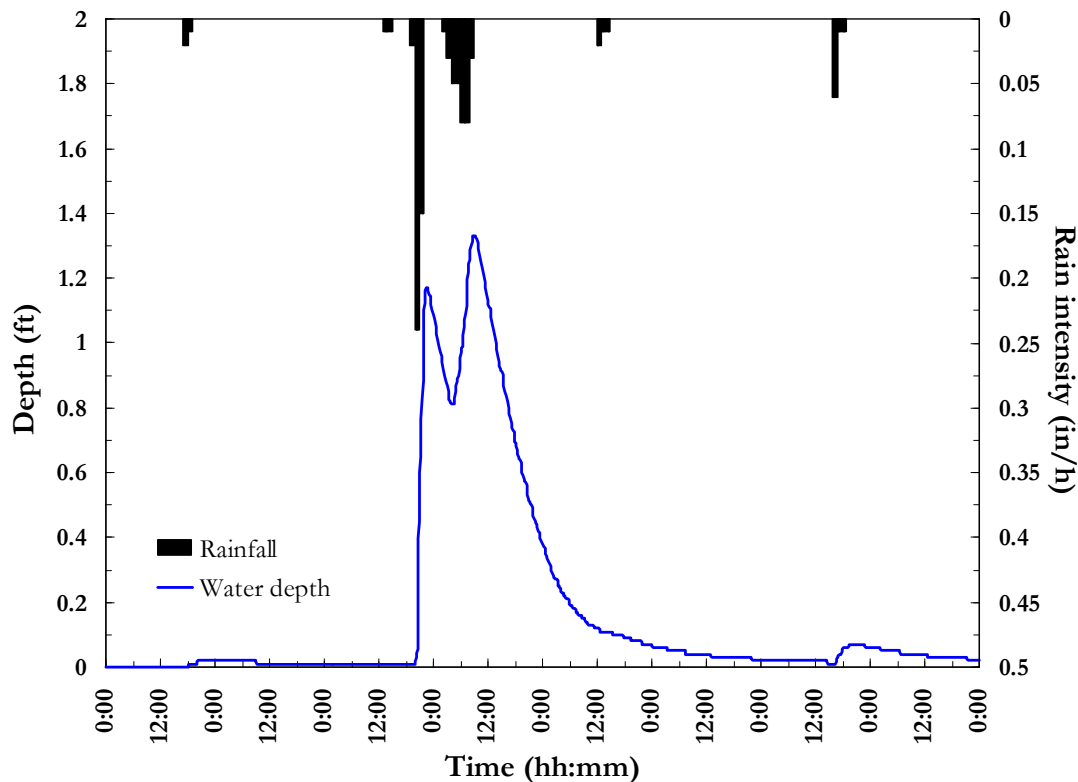
This difference explains the considerably smaller peak runoff discharged into the pond for the continuous simulation. The aggregation associated with larger rainfall intervals is a critical issue in continuous simulation that affects the performance of the model significantly. Peak discharges are very sensitive to high rainfall intensities, which typically occur in short periods of time. These high intensities are lost when data are aggregated in larger time steps, and the peak discharges are not well simulated. It is strongly recommended to use rainfall records with fifteen-minute or less time resolution, if available.

The difference in maximum intensities does not impact the peak discharges released by the detention pond. As previously mentioned, the highest peak flow discharged by the pond in the continuous simulation was 8.75 cfs, slightly higher than the 7.34 cfs peak produced from the 10-yr design storm. This is consistent with the fact that the storm associated with the continuous simulation peak is larger than the 10-yr design storm. The attenuation effect of the pond through time is such that the rainfall intensity is not as significant as the volume.

## Antecedent Conditions

One of the benefits of continuous simulation is that the model accounts for the initial state of the catchment and its conveyance network at the beginning of each new storm event. For instance, with continuous simulation the model simulates the initial water depth in the detention pond at the beginning of a new rainfall event. A high water depth will reduce the volume available for controlling the next storm, so that the discharges released will be larger than for the case where the pond is empty at the start of the storm.

Figure 9-4 shows the water depth in the pond over a period of 8 days, from day 3061 to day 3069 (from 5/19/1976 to 5/26/1976). Figure 9-5 shows the inflow and outflow rates for the pond over the same period of time. Rainfall is also included in both plots. Two storm events separated by 4 dry hours are shown: Event 1 has a maximum intensity of 0.24 in/h and a total volume of 0.41 in.; Event 2 has a maximum intensity of 0.08 in/h and a total volume of 0.33 in. Despite the smaller volume and intensity of the second storm, the water depth in the pond reaches 1.33 ft, which is larger than the water depth reached with the first event, 1.17 ft. Because of the short dry time in between the two events, the water level in the storage unit has only descended to a depth of 0.80 ft when the next event starts. The storage available to control the second event is such that the smaller peak inflow produced by the second event, 1.25 cfs is controlled to a peak outflow of 0.43 cfs, while the larger peak inflow produced by the first event, 3.91 cfs, is controlled to an outflow of only 0.39 cfs.



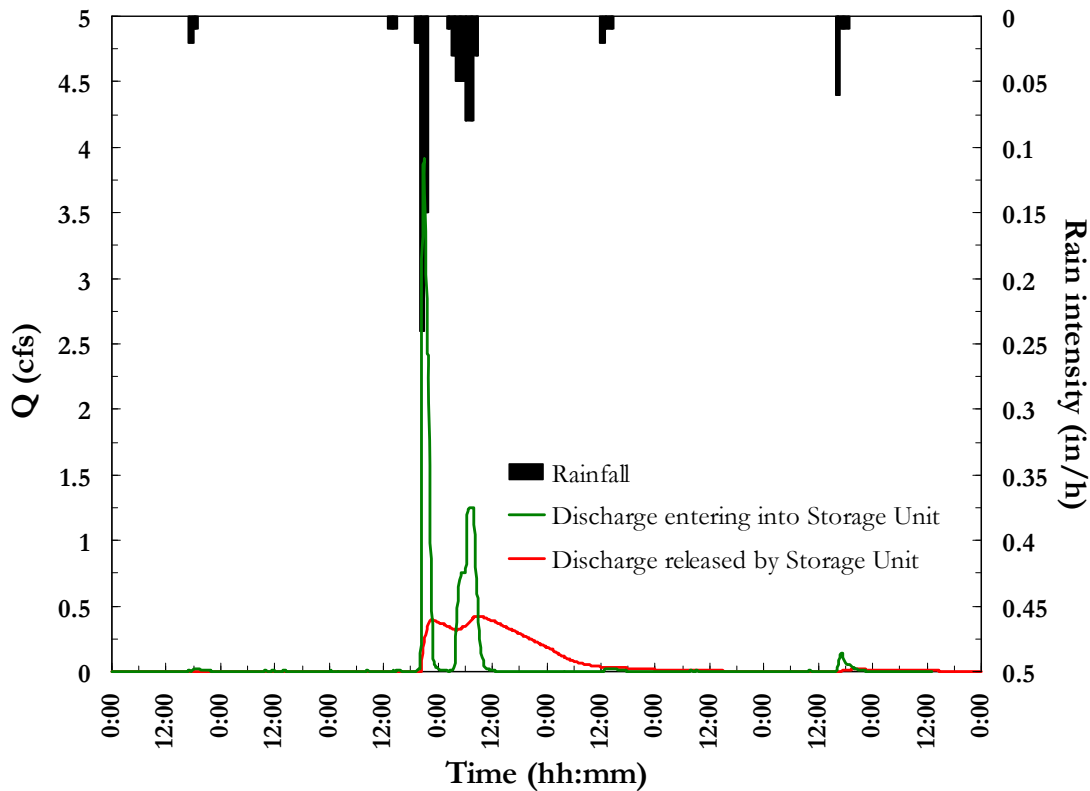


Figure 9-5. Inflow and outflow (Q) for the detention pond between days 3061 and 3069

## Evaporation

Finally, another advantage of continuous simulation is the inclusion of evaporation in the overall system water balance. Evaporation from both the pervious and impervious depression storage areas takes place between rain events. Thus, the amount of depression storage available to capture the initial portion of the next storm depends on the interval between storms. As an example, Figure 9-6 shows the rainfall and losses (infiltration plus evaporation) simulated in subcatchment *S1* over the period of time from 5/19/1976 to 5/26/1976. It is seen that after large events, and once infiltration has stopped, the losses stabilize at 0.0021 in/h. These losses are caused by evaporation acting over the water stored on the impervious area. To confirm this, consider that the average evaporation rate in May is 0.12 in/day or 0.005 in/h; the percent imperviousness of subcatchment *S1* is 56.8%, and 25% of that area does not have depression storage. Therefore, evaporation acts over  $0.568 \cdot (1-0.25) = 42.6\%$  of the total area, and so the loss rate over the entire subcatchment *S1* equals  $42.6\% \cdot 0.12 \text{ in/day} = 0.0021 \text{ in/hr}$ , the same as the loss rate shown in Figure 9-6.

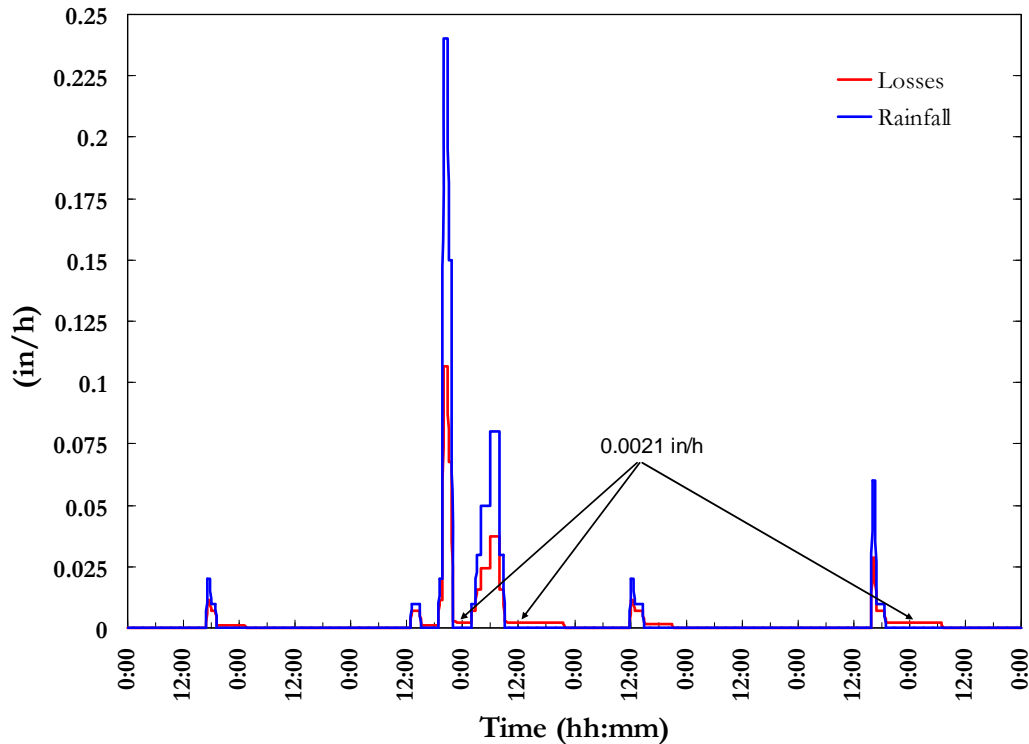
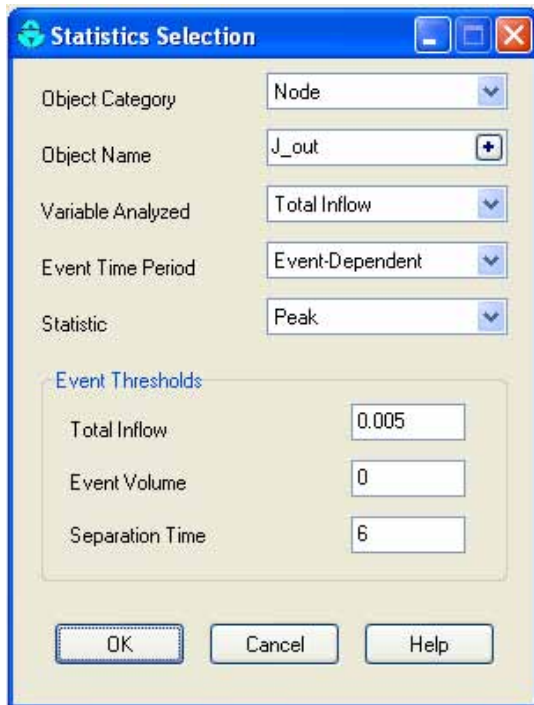


Figure 9-6. Losses for subcatchment *SI* between days 3061 and 3069

### Detention Pond Outflows

To illustrate the use of SWMM's *Statistics* tool, a frequency analysis of the peak outflows from the site's detention pond will be made. The analysis will show how often and for what periods of time the pond has a certain discharge and how the magnitudes of the peak discharges during these periods are distributed. To begin, one first opens the *Statistics Selection* dialogue by clicking the *Statistics* icon ( $\Sigma$ ) on SWMM's main toolbar or by selecting **Report | Statistics** from the main menu bar.

Figure 9-7 shows how the dialog should be filled in for this particular query. The object to be analyzed is node **J\_out** since it directly receives the outflow that exits the pond from each of its outlet devices. The variable to be analyzed for this node is **Total Inflow**. The time period used to define events is **Event-Dependent**, meaning that separate events will be defined by consecutive reporting periods where certain event threshold conditions are met. Within each such event period, the statistic to be analyzed is the **Peak** value of total inflow to the node (which is equivalent to the peak pond discharge through all of its outlet structures). And finally, the threshold criteria state that a new event begins whenever an inflow of at least **0.005** cfs occurs at least **6** hours after the last inflow of at least this amount was recorded. The selection of these values will affect how many events are counted. In general, larger values of the minimum flow or larger values of the time of separation will produce a fewer number of events.



**Figure 9-7. Statistics Selection dialog for analyzing peak pond outflows**

The report generated by running this statistical query contains four sections as shown in Figures 9-8 through 9-11. The first section is the *Summary Statistics*, shown in Figure 9-8. According to this summary, 295 events were identified in the record based on the thresholds values for flow and inter-event time defined for the frequency analysis; these events comprise 13.7 % of the total simulation time. Note the high positive skewness coefficient (4.93) which implies a large number of low discharges and few large ones. This is confirmed by the small value of the mean peak flow, 0.4 cfs.

Figure 9-9 shows a portion of the second section of the report, the *Event Listing* for the variable under study. The events are listed in order of decreasing value of the event statistic (the peak value) for the variable being analyzed (total inflow rate at node *J\_out*). Five fields are included: Start date of the event, duration, value of the variable under study (in this case peak flow), the exceedance frequency and an estimation of the corresponding return period. Note the length of the events listed in this table; the majority of them last longer than 24 hours. This shows that discharges are released quite slowly from the pond, as expected based on its design criteria. Moreover, a similar frequency analysis for duration instead of the peak discharge would show that the mean event duration is 40.7 hours.

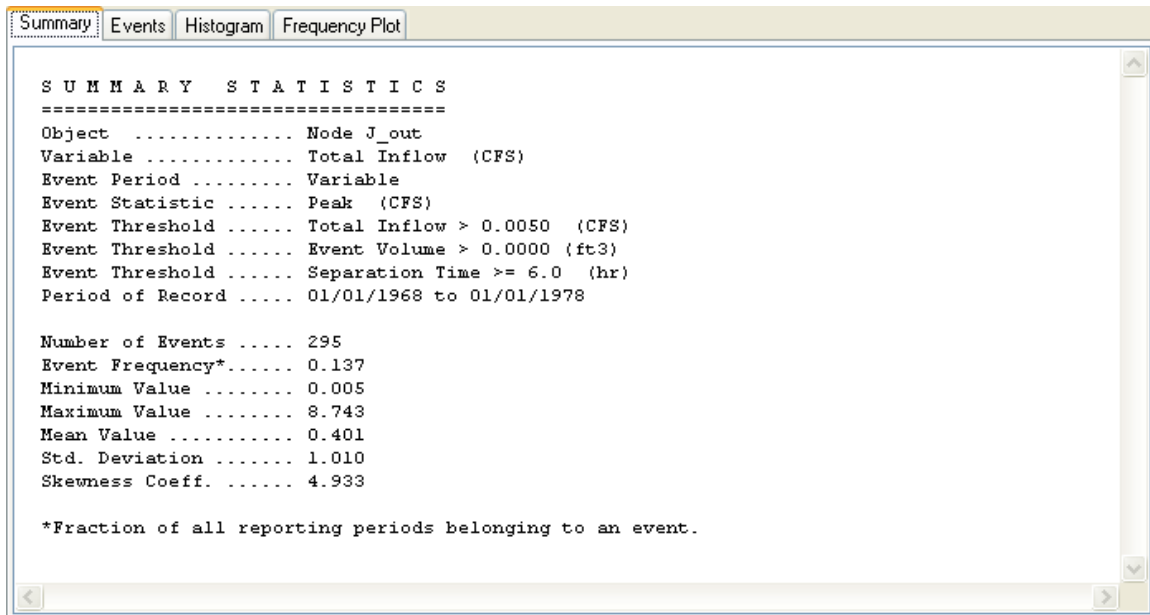


Figure 9-8. Summary statistics for peak inflow to node *J\_out* (same as peak pond outflow)

Rank	Start Date	Event Duration (hours)	Event Peak (CFS)	Exceedance Frequency (percent)	Return Period (years)
1	08/12/1975	90.8	8.743	0.34	11.00
2	07/23/1977	89.3	7.723	0.68	5.50
3	07/19/1970	53.3	5.163	1.01	3.67
4	05/23/1971	54.8	4.988	1.35	2.75
5	06/07/1974	69.3	4.266	1.69	2.20
6	07/27/1970	54.3	4.131	2.03	1.83
7	06/10/1970	80.3	3.909	2.36	1.57
8	05/05/1973	64.5	3.906	2.70	1.38
9	05/27/1975	144.3	3.686	3.04	1.22
10	04/19/1971	226.3	3.148	3.38	1.10
11	05/04/1969	120.5	3.016	3.72	1.00
12	06/05/1972	56.5	2.400	4.05	0.92

Figure 9-9. Event listing of peak inflow to node *J\_out* (same as peak pond outflow)

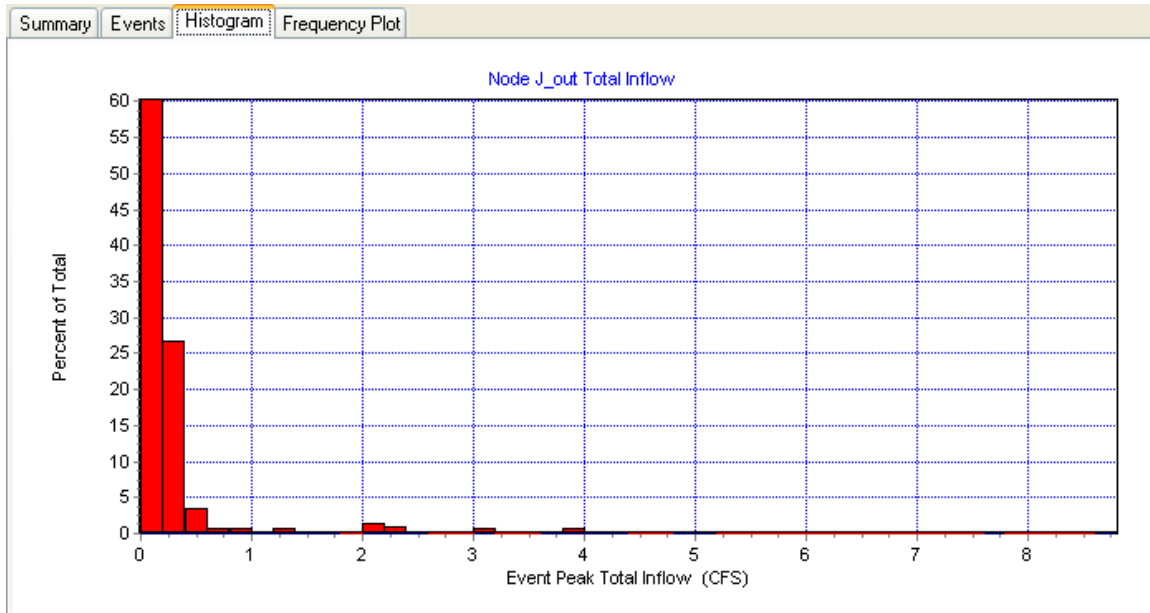


Figure 9-10. Histogram of peak inflow to node J\_out (same as peak pond outflow)

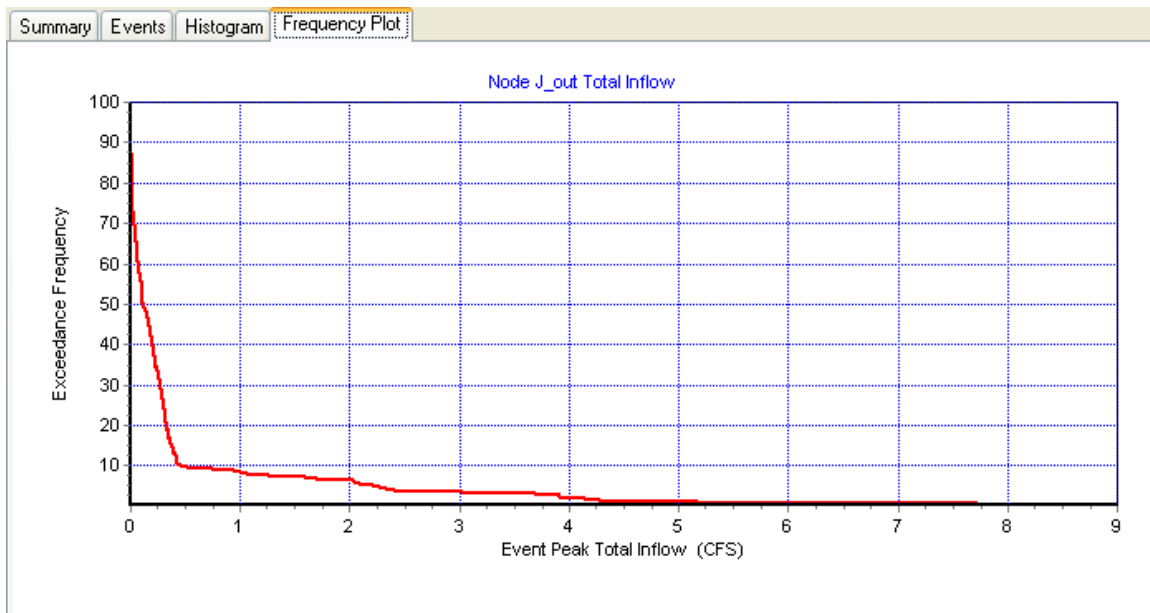


Figure 9-11. Cumulative frequency of peak inflow to node J\_out (same as peak pond outflow)

The exceedance frequency and return period displayed in the Event Listing are both computed using the Weibull formula for plotting position. Therefore, for a specific event the exceedance frequency  $F$  and the return period in years  $T$  are calculated using the following equations:

$$F = \frac{m}{n_R + 1} \quad (9-1)$$



$$T = \frac{n + 1}{m} \quad (9-2)$$

where  $m$  is the event's rank,  $n_R$  is the total number of events and  $n$  is the number of years under analysis. For example, for the 4<sup>th</sup> event occurring on 05/23/1971, the exceedance frequency is equal to  $F_4 = 4 / (295 + 1) = 0.0135 = 1.35 \%$ , and the return period is equal to  $T_4 = (10 + 1)/4 = 2.75$  years.

The event listing also shows that during the 10 years of simulation there were 2 events where the peak discharge was greater than the 7.34 cfs value that was computed in Example 3 when the model was run with the 10-yr design storm. This small discrepancy likely reflects the fact that the 10 years of record used here is only one of 29 different consecutive 10-year sequences that could have been selected from the 30 years worth of rainfall data available.

The third section of the Statistics report contains a histogram of the event statistic being analyzed, as shown in Figure 9-10. For this particular example it shows what fraction of all the events had a peak flow of a given size. Note how the figure confirms what was said earlier, that the distribution of peak inflows is highly skewed towards the low end of the flow scale. Finally, the fourth section of the report, as shown in Figure 9-11, presents a cumulative frequency plot of the event statistic under study. In this particular example, only 10 % of the peak discharges over the 10-yr period exceed 0.5 cfs, and only 6 % exceed 2 cfs. Note that both the histogram and cumulative frequency plots are just graphical representations of the same information as provided in the event listing.

### Detention Pond Water Depth

A second application of the SWMM *Statistics* tool to this example will analyze the maximum depth in the detention pond. This will help verify if the Water Quality Capture Volume (WQCV) of the pond was effective in capturing the more frequently occurring storms. Recall from Example 3 that the first 1.5 feet of storage volume was designated for this purpose. The statistical query used to answer this question is shown in Figure 9-12. Note that this time events are defined to consist of all days where the water depth in the pond was at least 0.05 ft deep. The “Daily” option is chosen for the *Event Period* because the drawdown time for the WQCV is 40 hours; thus a daily analysis of the depth in this portion of the pond is sufficient. Figure 9-13 shows the frequency plot that results from this query. On only 7 % of the days when the pond is wet does its depth exceed the WQCV. Thus, one can conclude that the large majority of all storms are captured within the WQCV and the pond will function as an effective BMP control.

**Statistics Selection**

Object Category: Node

Object Name: SU1

Variable Analyzed: Depth

Event Time Period: Daily

Statistic: Peak

**Event Thresholds**

Depth: 0.05

Event Volume: 0

Separation Time: 6

OK Cancel Help

Figure 9-12. Statistical query for daily peak water depth in the detention pond

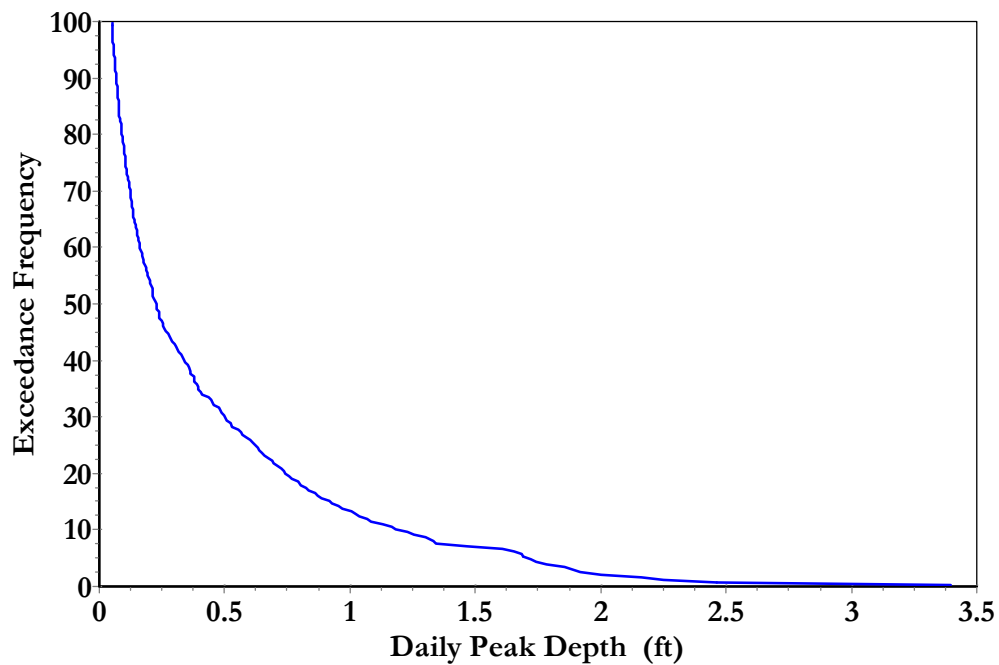


Figure 9-13. Frequency plot of daily peak water depth in the detention pond

## Rainfall Statistics

Another variable typically analyzed using statistics is rainfall. Several quantities that characterize storm events for the rainfall record used in this example will be examined and compared. These are the event duration, mean intensity, total volume, and peak intensity. As before, the record will be separated into a sequence of independent storm events of varying magnitude and duration using SWMM's *Statistic* tool. Note that a rainfall analysis can be made without having to run a complete runoff/routing simulation for the entire SWMM model. See the sidebar "[\*Analyzing Stand-Alone Rainfall Files with SWMM\*](#)" for details on how to do this.

### **Analyzing Stand-Alone Rainfall Files with SWMM**

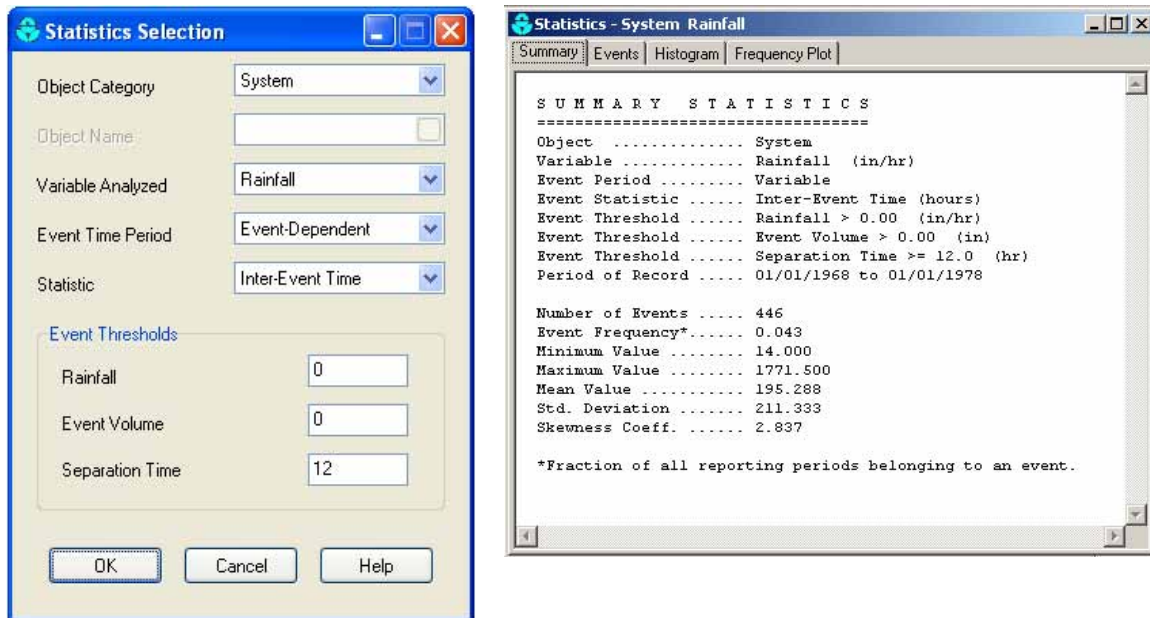
Modelers often have a need to statistically analyze a long-term rainfall record independently of using it within a particular SWMM model. For example, one might want to identify a sub-period of the record that is representative of the entire record or identify particular events, extreme or otherwise, that could be used for single event analyses. It is quite simple to set up a general SWMM project for just this purpose and run it independently for any rainfall file. The steps involved are as follows:

1. Create a new project with SWMM that consists of just a single subcatchment and associated rain gage and set the outlet of the subcatchment to itself.
2. Save the project with a meaningful name, such as **RainStats.inp**.
3. Whenever a rainfall file needs to be analyzed, launch SWMM and open this special project. (Note that a new SWMM session can be launched while other SWMM sessions are still active).
4. Edit the Rain Gage to use the file of interest (remembering to enter appropriate values for the *Rain Format*, *Rain Interval*, *Station ID*, and *Rain Units* if a file with the User-Supplied format is being used).
5. Edit the **Dates** in the **Simulation Options** dialog to cover the period of interest within the rainfall file. Also set the *Reporting* and *Wet Runoff* time steps to be the same as the *Rain Interval* used for the rainfall data.
6. Run a simulation and then use the Time Series Graph, Tabular Report, and Statistics tools to analyze the System Rainfall variable.

Regarding the final step of this process, time series plots are useful for providing an overall picture of how rainfall varies within a period of several years or months, tabular reports can retrieve rainfall hyetographs for specific events that can be pasted directly into a SWMM Time Series created in another SWMM session, and examples of using the *Statistics* tool to analyze the rainfall record are shown elsewhere in this chapter.

In rainfall analysis, the separation time used to decide when one event ends and another begins is referred to as the *Minimum Inter-Event Time (MIT)*. This is the smallest number of consecutive dry periods that must occur before the next wet period is considered as a separate event. There is no established “correct” value for the *MIT*, although 3 to 30 hours are often used for rainfall data (Hydroscience, 1979). See Adams and Papa (2000) for a detailed discussion on this subject. When storm events are characterized as a Poisson process, the time between events follows an exponential distribution for which the mean equals the standard deviation (i.e., the coefficient of variation (*CV*) is 1). Thus one suggested approach to choosing a *MIT* is to find a value that produces a *CV* of 1 for the resulting collection of inter-event times.

Figure 9-14 shows the statistical query used to test how well a *MIT* (separation time) of 12 hours produces a sequence of events whose inter-event times have a *CV* of 1 for the rainfall record used in this chapter. The resulting *CV* (standard deviation divided by the mean) is  $211.33 / 195.28 = 1.08$ , indicating that 12 hours is a reasonable *MIT* to use for this rainfall record.



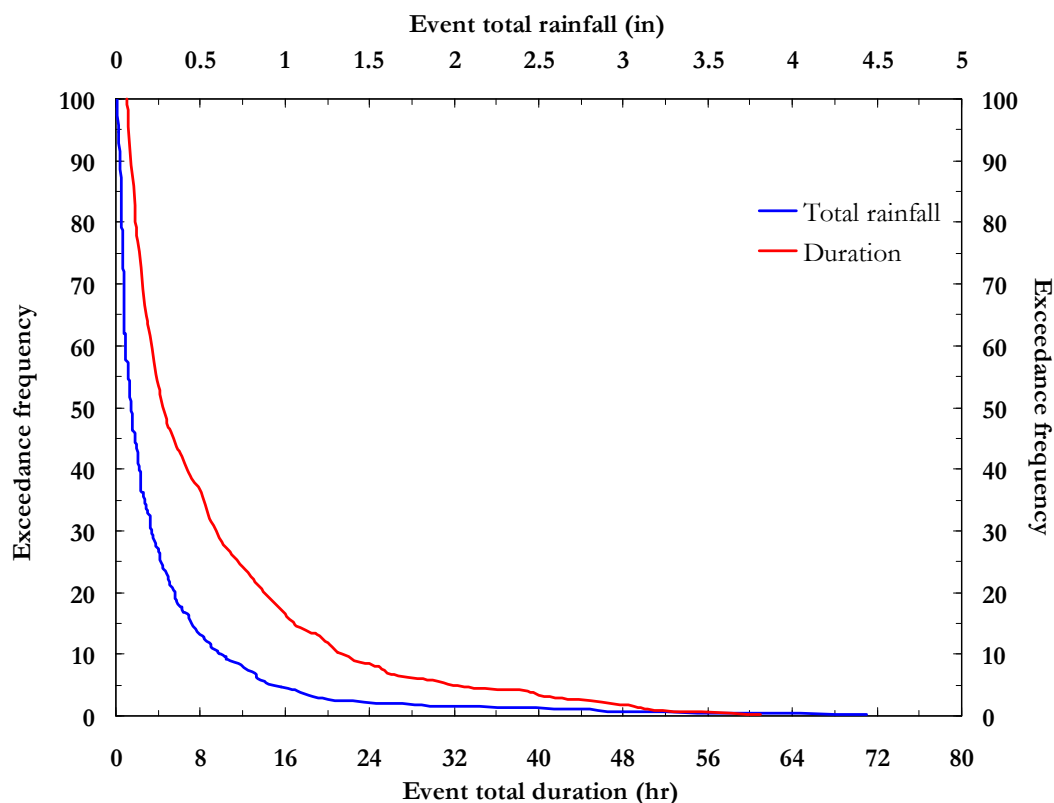
**Figure 9-14. Selection of an *MIT* for analyzing a rainfall record**

Next a frequency analysis is made for each of the following rainfall quantities: duration, mean intensity, total volume, and peak intensity. Each analysis uses a *Statistics Selection* dialog that looks the same as that in Figure 9-14, except that a different choice of event statistic is used for each. Table 10-2 lists the summary statistics found for each frequency analysis. The first two properties (number of events and event frequency, or percentage of total time in which rainfall is registered) are the same for each rainfall property since all of the frequency analyses used the same thresholds to define an event.

**Table 9-2. Summary statistics for various rainfall event properties**

Property	Event statistic			
	Duration (h)	Mean Intensity (in/hr)	Total Volume (in)	Peak Intensity (in/hr)
Number of Events	446	446	446	446
Event Frequency	0.043	0.043	0.043	0.043
Minimum Value	1.000	0.006	0.01	0.006
Maximum Value	61.000	0.6900	4.44	1.470
Mean Value	8.466	0.042	0.247	0.078
Std. Deviation	10.557	0.058	0.445	0.125
Skewness Coeff.	2.278	5.642	4.733	5.550

Figure 9-15 shows the frequency plot produced by this analysis for event duration and total rainfall. (This figure was generated by using SWMM's **Edit | Copy To** menu option to copy the data associated with each frequency plot to the Windows Clipboard from which it was pasted into a spreadsheet program and combined together on a single graph.) This plot can also be used as a less direct indicator than the pond depth frequency analysis performed earlier to see if the Water Quality Capture Volume (WQCV) of 0.23 inches is sufficient to capture the majority of runoff events.

**Figure 9-15. Frequency plots for event duration and depth**

It is seen (blue line) that about 70% of the rainfall events have a total volume smaller than 0.23 inches. Even if the watershed were totally impervious with no surface retention capacity, the pond would control around 70% of the events according to the model. In reality there are infiltration and storage losses so that a larger percentage of the rainfall events are controlled by the pond's volume. Finally, note also that long storms are not very frequent, with 50% of them being shorter than 5 hours, and 80% shorter than 14 hours. Storms longer than one day are very uncommon at this recording station located in the Colorado foothills.

Another use of this type of multi-variate statistical analysis is to study in more detail the correspondence between the frequencies of the events based on different rainfall characteristics. The largest event is not necessarily the longest one or the most intense. It is useful to determine the degree of dependence among storm event characteristics in order to see if these characteristics are correlated or not, and to identify a subset of the most critical events that could be used for design purposes, depending on the objective of the analysis. The intensity of the events is significant in determining peak discharge, but the magnitude may be more important when storage control structures are designed or evaluated.

Table 9-3 shows the ten most extreme events according to the four characteristics under analysis: duration, mean intensity, volume and peak intensity. The table reveals that the peak and mean intensities are closely correlated; seven of the ten events with the largest peak intensity are also in the groups of events with a large mean intensity (dates of these common events are underlined). There is also a correspondence between duration and total volume; four of the longest events are also in the group that contains the largest ones (the dates of these common events are in blue). A weaker correlation is observed between the mean intensity and volume (one common event in a red box), and between volume and peak intensity (two common events in green).

**Table 9-3. Ten most severe events based on duration, depth and intensity**

Rank	T <sub>r</sub> (year)	Duration (hr)		Mean (in/hr)		Volume (in)		Peak (in/hr)	
		Start Date	Value	Start Date	Value	Start Date	Value	Start Date	Value
1	11	5/5/1969	61	7/27/1970	0.69	7/24/1977	4.44	8/12/1975	1.47
2	5.5	10/23/1975	56	7/20/1973	0.39	5/27/1975	3.29	7/19/1970	0.97
3	3.67	10/10/1969	51	7/16/1969	0.38	5/5/1969	2.98	5/23/1971	0.85
4	2.75	10/3/1969	51	7/19/1970	0.36	6/10/1970	2.78	7/27/1970	0.69
5	2.2	3/29/1970	50	5/23/1971	0.36	6/7/1974	2.75	7/24/1977	0.65
6	1.83	8/2/1976	49	7/28/1974	0.33	8/12/1975	2.25	8/14/1975	0.55
7	1.57	5/27/1975	49	8/8/1970	0.27	10/3/1969	1.97	6/17/1975	0.52
8	1.38	5/20/1975	48	5/8/1971	0.22	10/10/1969	1.66	7/28/1974	0.44
9	1.22	3/9/1968	46	7/24/1977	0.20	4/24/1971	1.62	9/11/1973	0.40
10	1	6/7/1968	46	8/14/1975	0.19	4/24/1973	1.57	7/20/1973	0.39

Table 9-4 shows the ranks of the common events identified in Table 9-3. Numbers before and after the “&” symbol indicate the rank according to the variables defined in the first row of the table. For example, from the second column, the longest event (61 hrs) is also the third largest event (2.98 inches), the third longest event (51 hrs) is the eighth largest event (1.66 inches), and so on. Note that the two common events identified among the most severe in terms of the volume and peak intensity are the same two events associated with the largest peak discharges released by the storage unit discussed earlier; these are the events of 07/47/1977 and 8/12/1975. Finally, there are no common events among the ten most severe in terms of both duration and intensities (both mean and peak). This analysis corresponds to a preliminary step in evaluating the correlation between storm event characteristics. Other methodologies including the use of scatter plots between variables and correlation coefficients can be used. These methods are applied to all the events and not only the most severe ones.

**Table 9-4. Correspondence among the most severe events**

<b>Duration &amp; Mean</b>	<b>Duration &amp; Volume</b>	<b>Duration &amp; Peak</b>	<b>Mean &amp; Volume</b>	<b>Mean &amp; Peak</b>	<b>Volume &amp; Peak</b>
-	1 & 3	-	9 & 1	1 & 4	1 & 5
-	3 & 8	-	-	2 & 10	6 & 1
-	4 & 7	-	-	4 & 2	-
-	7 & 2	-	-	5 & 3	-
-	-	-	-	6 & 8	-
-	-	-	-	9 & 5	-
-	-	-	-	10 & 6	-

## 9.5 Summary

This example demonstrated how to use continuous simulation along with statistical analysis to evaluate the long-term behavior of a drainage system. Ten years of continuous rainfall data (1-hr resolution) and monthly average evaporation rates were used to analyze the performance of the conveyance system and detention pond designed in Example 3. The key points illustrated in this example were:

1. Continuous simulation allows modelers to more faithfully represent the behavior of drainage systems because it subjects them to a long sequence of actual rainfall events of varying magnitudes and durations and also accounts for the variability of antecedent conditions that exist from one event to the next.
2. Evaporation is an important component of the long-term water budget that should be considered in continuous simulation. In this example it accounted for 19 % of the total rainfall input to the catchment.
3. Model setup for continuous simulation is quite straightforward in SWMM, providing that reliable long-term rainfall records and evaporation data are available.

4. High time resolution on rainfall data is required if accurate predictions of peak runoff flows are needed. The hourly time interval of the rainfall data used in this example may cause peak discharges to be underestimated.
5. SWMM's *Statistics* tool is a valuable aid in interpreting the large amount of output data that can be generated from a long-term continuous simulation. It was used here to determine how effective the detention pond was in reducing peak discharges and in capturing the majority of runoff events within its water quality control volume. The tool was also used to characterize the properties of the most severe rainfall events occurring over the 10-year simulation period.



# References

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