MGS Flood - Proprietary Version Users Manual

A Continuous Hydrological Simulation Model for Stormwater Facility Analysis for Western Washington

MGS Software LLC

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Version 4
July 27, 2009

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PART I – PROGRAM BACKGROUND INFORMATION

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1 Introduction

MGSFlood is a general, continuous, rainfall-runoff computer model developed for the Washington State Department of Transportation specifically for stormwater facility design in Western Washington. The program uses the Hydrological Simulation Program-Fortran (HSPF)[26] routine for computing runoff from rainfall. The public domain version of the program includes a routing routine that uses a stage-storage-discharge rating table to define a stormwater retention/detention facility or reservoir, routines for computing streamflow magnitude-frequency and duration statistics, and graphics routines for plotting hydrographs and streamflow frequency and duration characteristics. The program meets the requirements of the 2005 Washington State Department of Ecology Stormwater Management Manual for Western Washington[9].

2 MGS Flood Model Applicability, Limitations and Program Configuration

2.1 Model Applicability and Limitations

MGSFlood is intended for the analysis of stormwater detention facilities in the lowlands of western Washington. The program utilizes the HSPF routines for computing runoff from rainfall for pervious and impervious land areas. The program does not include routines for simulating the accumulation and melt of snow and its use should be limited to lowland areas where snowmelt is typically not a major contributor to floods or to the annual runoff volume. In general, these conditions correspond to an elevation below approximately 1500 feet.

The program is applicable for the analysis of stormwater facilities for small sites (several thousand square feet) to watersheds (10's of square miles). The program includes precipitation timeseries with a 5-minute time step for much of western Washington. For sites outside of the 5-minute time series coverage, precipitation time series with a 1-hour time step are included. Peak discharge rates computed using the 1-hour time series should not be used for conveyance design unless the conveyance system is downstream of a stormwater detention pond, where regulation of runoff renders a 1-hour time-step sufficiently accurate for conveyance design.

2.2 Disclaimer

MGSFlood is a complex program that requires engineering expertise to use correctly. MGS Software LLC assumes absolutely no responsibility for the correct use of this program. All results obtained should be carefully examined by an experienced professional engineer to determine if they are reasonable and accurate.

Although MGS Software LLC has endeavored to make this program error free, the program is not and cannot be certified as infallible. Therefore, MGS Software LLC makes no warranty, either implicit or explicit, as to the correct performance or accuracy of this software.

In no event shall MGS Software LLC be liable to anyone for special, collateral, incidental, or consequential damages in connection with or arising out of use of this program.

2.3 Program Configuration

Figure 2.1 shows a schematic of the MGSFlood modeling package. The main program module, MGSFlood.exe, controls the user interface, HSPF, statistics, routing, and pond optimization routines. When the program starts, the location of the program and project subdirectories on the computer system is read from the Windows Registry. Project data files have a user specified name and a *.fld* extension. These files are Microsoft Access database files and are stored in the project subdirectory.

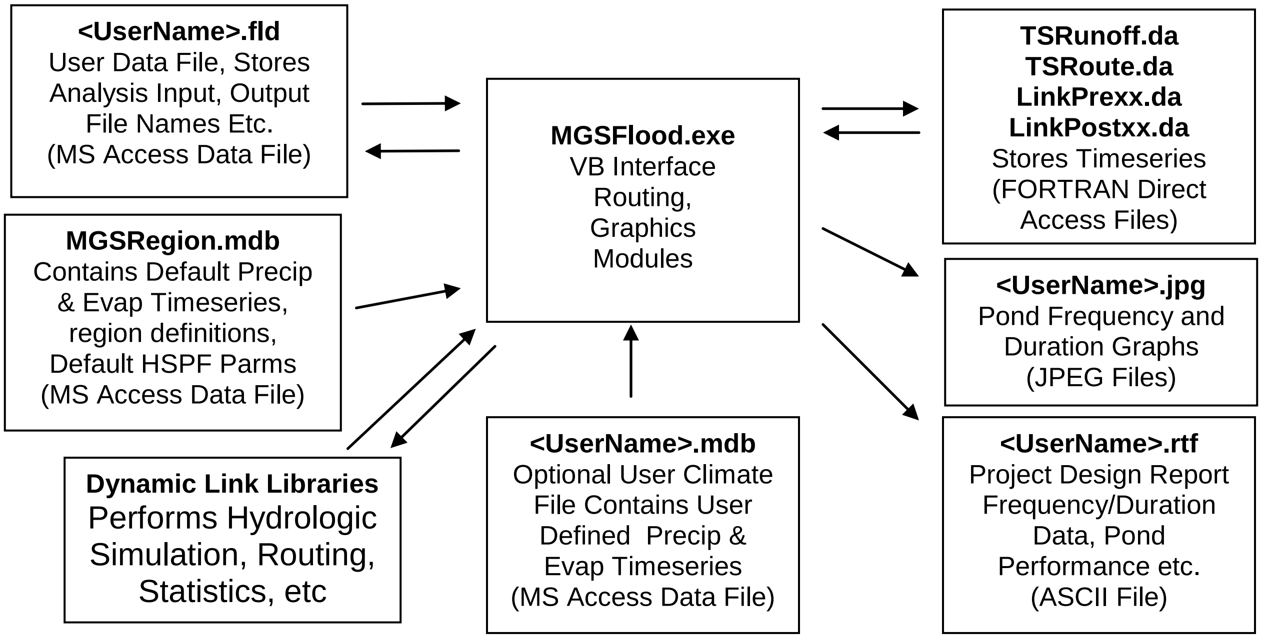


Figure 2.1 – MGSFlood Model Components

2.4 Precipitation and Evaporation Input

MGSRegions.mdb is an Access database file that contains the precipitation and evaporation timeseries for each region, and the default HSPF parameters.

2.5 Runoff Data File

Runoff is computed by MGSFlood using the HSPF[26] library routine. Precipitation and evaporation are read from the MGSRegion.mdb file, runoff is computed for predevelopment and postdevelopment conditions, and saved to FORTRAN, binary, direct access files called TSRunoff.da, TSRoute.da, LinkxxPre, and LinkxxPost. The same FORTRAN direct access files are overwritten for each project analyzed by the flood model, i.e. the computed runoff timeseries are not saved for each project. Thus, the project runoff must be recomputed to ensure that the files are up-to-date and contains runoff for the project currently under consideration.

2.6 Project Documentation and Graphics Files

Project documentation is stored in a Windows Rich Text File format in the project subdirectory with an *.rtf* extension. These files may be read using Microsoft Word or WordPad. This file is created/overwritten each time the report is written by the program.

Files containing images of graphs plotted on the screen may also be saved by the user by clicking the save button on the Graphs tab. These files are *JPEG* format and contain the images of hydrograph, flood frequency, and flow duration plots generated by the program. The JPEG graphic images can be imported into any software that accepts the JPEG format. This feature is intended to support importing graphics into word processing programs for preparation of reports and other documents.

3 HSPF Runoff Routine and Runoff Parameters

MGS Flood uses the rainfall-runoff routines from Version 12 of the Hydrological Simulation Program-Fortran (HSPF)[26]. HSPF uses multi-year inputs of hourly precipitation and evaporation, keeps a running accounting of the moisture within the soil column and in groundwater storage, and simulates a multi-year timeseries of hourly runoff.

3.1 Pervious Land Parameters

Default HSPF model parameters that define interception, infiltration, and movement of moisture through the soil, are based on work by the USGS[7,8] and King County[17]. Pervious areas have been grouped into three land cover categories; forest, pasture, and lawn, and three soil/geologic categories; till, outwash, and saturated/wetland soil for a total of seven cover/soil type combinations as shown in Table 3.1. The combinations of soil type and land cover are called *pervious land segments* or *PERLNDS*. Default runoff parameters for each PERLND are summarized in Table 3.2. These values are loaded automatically by the program for each project. If these values are changed by the user, the changed values are noted in the project documentation report (See Section 20).

Green Roof model parameters were developed using monitoring data from the Hamilton Building in Portland Oregon. The parameters were developed by Clear Creek Solutions for Seattle Public Utilities[34]. The parameters included in MGSFlood were developed using the 5 inch green roof monitoring site.

Table 3.1 - Pervious Land Soil Type/Cover Combinations used with HSPF Model Parameters

Pervious Land Soil Type/Cover Combinations
1. Till/Forest
2. Till/Pasture
3. Till/Lawn
4. Outwash/Forest
5. Outwash/Pasture
6. Outwash/Lawn
7. Saturated Soil/All Cover Groups
8. Green Roof

Table 3.2 – Default Runoff Parameters for Each Pervious Land Segment (PERLND)

	Pervious Land Segment (PERLND)							
	Till Soil			Outwash Soil			Saturated Soil	
Parameter	**Forest**	**Pasture**	**Lawn**	**Forest**	**Pasture**	**Lawn**	**Forest/Pasture/ or Lawn**	**Green Roof**
LZSN	4.5	4.5	4.5	5.0	5.0	5.0	4.0	1.25
INFILT	0.08	0.06	0.03	2.0	1.6	0.8	2.0	0.05
LSUR	400	400	400	400	400	400	100	50
SLSUR	0.1	0.1	0.1	0.05	0.05	0.05	0.001	0.001
KVARY	0.5	0.5	0.5	0.3	0.3	0.3	0.5	0.5
AGWRC	0.996	0.996	0.996	0.996	0.996	0.996	0.996	0.10
INFEXP	2.0	2.0	2.0	2.0	2.0	2.0	10.0	2.0
INFILD	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
BASETP	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AGWETP	0.0	0.0	0.0	0.0	0.0	0.0	0.7	0.8
CEPSC	0.2	0.15	0.1	0.2	0.15	0.1	0.1	0.1
UZSN	0.5	0.4	0.25	0.5	0.5	0.5	3.0	0.13
NSUR	0.35	0.3	0.25	0.35	0.3	0.25	0.5	0.55
INTFW	6.0	6.0	6.0	0.0	0.0	0.0	1.0	1.0
IRC	0.5	0.5	0.5	0.7	0.7	0.7	0.7	0.1
LZETP	0.7	0.4	0.25	0.7	0.4	0.25	0.8	0.8

PERLND parameter definitions:
LZSN =lower zone storage nominal (inches)
INFILT =infiltration capacity (inches/hour)
LSUR =length of surface overland flow plane (feet)
SLSUR =slope of surface overland flow plane (feet/feet)
KVARY =groundwater exponent variable (inch $^{-1}$)
AGWRC =active groundwater recession constant (day $^{-1}$)
INFEXP =infiltration exponent
INFILD =ratio of maximum to mean infiltration
BASETP =base flow evapotranspiration (fraction)
AGWETP =active groundwater evapotranspiration (fraction)
CEPSC =interception storage (inches)
UZSN =upper zone storage nominal (inches)
NSUR =roughness of surface overland flow plane (Manning 's n)
INTFW =interflow index
IRC =interflow recession constant (day $^{-1}$)
LZETP =lower zone evapotranspiration (fraction)

A complete description of the PERLND parameters can be found in the HSPF User Manual[26]. Default PERLND Parameters used in the model were developed for the Puget Sound Lowlands by the US Geological Survey[7,8]

3.2 User Defined Pervious Land Segments

An additional Pervious Land Segments (PERLNDs) may be specified by the user by opening the HSPF Parameter sheet and clicking the *User* button at the bottom of the page. A window will appear with parameter fields for up to two additional PERLNDs. The user can specify the name of these as well as the HSPF parameters. This feature allows the user to define land cover/soil type combinations not included in the default parameters.

3.3 Impervious Land Parameters

Default runoff parameters for impervious surface, called *IMPLNDs* are summarized in Table 3.3.

Table 3.3 – Impervious Cover (IMPLND) Parameters

Parameter	Value
LSUR	500
SLSUR	0.01
NSUR	0.1
RETSC	0.1

IMPLND Parameter Definitions:
LSUR = length of surface overland flow plane (feet)
SLSUR = slope of surface overland flow plane (feet/feet)
NSUR = roughness of surface overland flow plane (Manning ’s n)
RETSC = retention storage (inches)

A complete description of the IMPLND parameters can be found in the HSPF User Manual[26].
IMPLND Parameters were developed for the Puget Sound Lowlands by the US Geological Survey[7,8]

3.4 Determining PERLND Soil Type from SCS Soil Mapping

The soils at the project site must be classified into one of the three default categories for use in the MGSFlood model. These soils categories are: till, outwash, or saturated soil, as defined by the USGS[7,8].

Soils formed in areas with glacial till are underlain at shallow depths by relatively impermeable glacial till (also known as “hard-pan”). Glacial till deposits contain large percentages of silt or clay and have low percolation rates. Only a small fraction of infiltrated precipitation reaches the groundwater table through the till. The rest moves laterally through the thin surface soil above the till deposit as interflow. Shallow soils over bedrock should also be classified as till soils because the hydrologic response from these areas is similar to till.

Soils formed in areas with glacial outwash deposits consist of sand and gravels that have high infiltration rates. The majority of rainfall is infiltrated and percolates to the groundwater table in these areas. Creeks draining outwash deposits often intersect the groundwater table and receive most of their flow from groundwater discharge. Site developments in outwash areas are typically located higher in the watershed and groundwater discharge is not present. Thus, groundwater is typically not included in runoff calculations in outwash (or till) areas.

Wetland soils remain saturated throughout much of the year. The hydrologic response from wetlands is variable depending on the underlying geology, the proximity of the wetland to the regional groundwater table, and the bathymetry of the wetland. Generally, wetlands provide some baseflow to streams in the summer months and attenuate storm flows via temporary storage and slow release in the winter.

Mapping of soil types by the Soil Conservation Service (SCS, now the National Resource Conservation Service (NRCS)) is the most common source of soil/geologic information used in hydrologic analyses for stormwater facility design. Each soil type defined by the SCS has been classified into one of four hydrologic soil groups; A, B, C, and D. As is common practice in hydrologic modeling in western Washington, the soil groups used in the MGSFlood model generally correspond to the SCS hydrologic soil groups as shown in Table 3.4.

Table 3.4 – Relationship Between SCS Hydrologic Soil Group and MGS Flood Soil Group

SCS	MGS Group
A	Outwash
B	Till or Outwash
C	Till
D	Wetland

SCS Type B soils can be classified as either glacial till or outwash depending on the type of soil under consideration. Type B soils underlain by glacial till or bedrock, or have a seasonally high water table would be classified at till. Conversely, well-drained B type soils would be classified as outwash.

The Ecology Stormwater Management Manual for Western Washington[9] relates SCS hydrologic soil groups to HSPF soil/geologic groups as shown in Table 3.5

Table 3.5 – Relationship between SCS and HSPF Soil Groups

SCS Hydrologic Soil Group	MGSFlood/HSPF Soil/Geologic Group
A/B	Outwash
C	Till
D	Wetland

4 Precipitation Input

MGSFlood uses multi-year inputs of precipitation and evaporation to compute a multi-year timeseries of runoff from the site. Using precipitation input that is representative of the site under consideration is critical for the accurate computation of runoff and the design of stormwater facilities. Precipitation and evaporation timeseries have been assembled for most areas of western Washington and are stored in a database file accessed by the program. These timeseries should be used for stormwater facility design.

4.1 Selection and Scaling of Precipitation for Stormwater Facility Design

Accurate assessment of streamflow characteristics at a particular site is dependent upon numerous watershed and hydrometeorological factors. Among those factors, it is critically important to have a precipitation timeseries representative of the climatic and storm characteristics at the site of interest. However, it is rare that a long precipitation timeseries is available at the site of interest. This problem is commonly addressed by transposing the timeseries record from a "nearby" gage to the site of interest using some type of scaling routine to account for the differences in storm characteristics at the source and target sites.

Proper transposition is a very complex problem as storm characteristics vary by both duration and physical topographic setting across western Washington. For example, a site with a mean annual precipitation of 50-inches to the west of central Puget Sound has different precipitation magnitude-frequency characteristics than a site with 50-inches mean annual precipitation located to the east of central Puget Sound. Ideally, a dense network of hourly precipitation gages would be available and only minor amounts of scaling would be needed. Unfortunately, only a limited number of long-term, high quality hourly precipitation recording stations are available in western Washington. Therefore, the transposition of timeseries by scaling is a critical aspect of obtaining a representative timeseries for most sites.

Two methods of transposing precipitation timeseries are available in the MGSFlood model. The first method utilizes a family of pre-scaled precipitation and evaporation timeseries. These precipitation timeseries were developed using statistical scaling functions to scale hourly precipitation amounts for eight selected inter-durations within the timeseries. This method was used to produce "extended precipitation timeseries" with record lengths in excess of 100-years by combining and scaling precipitation records from widely separated stations[21,22]. Extended hourly precipitation and evaporation timeseries have been developed using this method for most of the lowland areas of western Washington where stormwater projects will be constructed. The extended hourly time series were disaggregated to 5-minutes utilizing storm temporal patterns from the Seattle Public Utilities rain gage network[35]. These timeseries should be used for facility design for projects located in the region shown in Figure 4.1.

For projects sites located outside of the extended timeseries region, a second precipitation scaling method is used. This method uses a simple scaling procedure that scales all hourly precipitation amounts in the source timeseries by a common scaling factor. These precipitation time series have a time step of 1-hour and should not be used for conveyance design purposes for structures located upstream of detention. Use of these two methods is described in the following sections.

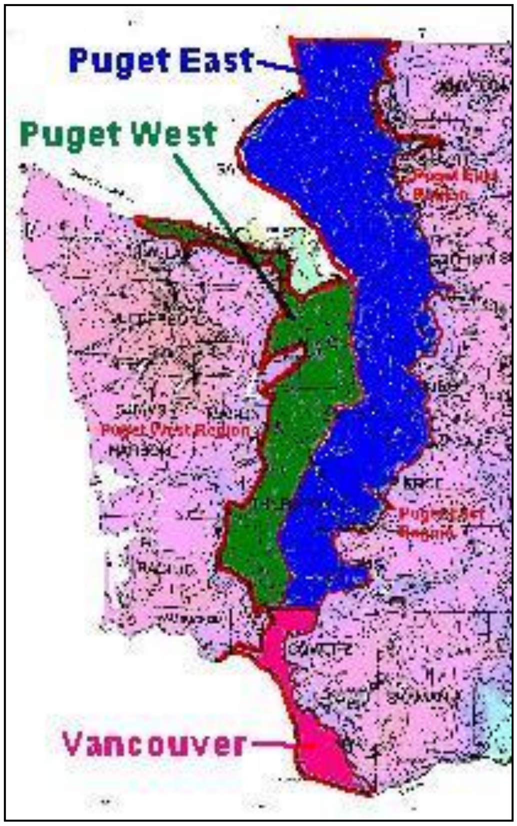


Figure 4.1 – Extended Precipitation Timeseries Regions

4.1.1 Extended Precipitation Timeseries

Extended, 5-minute precipitation and evaporation timeseries have been developed for most of the lowland areas of western Washington where stormwater projects will be constructed. This collection of 27 timeseries is applicable to sites with mean annual precipitation ranging from 24-inches to 60-inches in the lowlands from the Canadian border to the Oregon border. The timeseries are grouped according to region; Puget West, Puget East, and Vancouver. An additional 5-minute extended precipitation was developed for projects within the City of Seattle. Timeseries applicable to sites with mean annual precipitation ranging from 38 to 52 inches in Pierce County were also included. The Pierce County time series have a time step of 1-hour.

The extended precipitation timeseries were developed by combining and scaling records from distant precipitation stations. The precipitation scaling was performed such that the scaled precipitation record would possess the regional statistics at durations of 2-hour, 6-hours, 24-hours, 3-days, 10-days, 30-days, 90-days, 6-months and annual. The evaporation timeseries were developed using a stochastic evaporation generating approach whereby daily evaporation was generated in a manner to preserve the daily and seasonal variability and accounting for differences observed on rainy versus rain-free days. The evaporation timeseries were developed from data collected at the Puyallup 2 West Experimental Station (station number 45-6803). Details on the development of the precipitation and evaporation timeseries can be found in the report; *Extended Precipitation Time-Series for Continuous Hydrological Model in Western Washington*, MGS Engineering Consultants, Inc., 2002[20]. Information on the procedure used disaggregate the extended time series may be found in the report Development of 5-Minute Extended Precipitation Time Series for the Puget Sound Lowlands, MGS Engineering Consultants, Inc., 2008[35].

Recommended Applicability of Extended Precipitation Time-Series
Extended precipitation time-series are preferred to precipitation time-series obtained from simple scaling procedures for all locations in the lowlands of western Washington where extended time-series are available. Extended time-series are preferred for a number of reasons as discussed in the following sections.

Multiple Scaling Functions
Extended precipitation time-series are developed using a series of statistical scaling functions rather than a single scaling factor. These scaling functions provide for scaling of precipitation maxima at the 2-hour, 6-hour, 24-hour, 72-hour, 10-day, 30-day, 90-day and annual durations. This scaling is done in a manner to match the storm statistics (magnitude-frequency characteristics) expected for a given climatological setting based on regional analyses of the time-series records at over 50 precipitation gages located in the lowlands of western Washington. Thus, the storm characteristics are based on a very large sample-set of storms and stations rather than the record from a single station.

Scaling Difficulties Due to Complex Nature of Storm Characteristics
Storm characteristics vary by duration, storm type, and season in western Washington. This complex behavior includes: short-duration, high-intensity storm events in the warm season; intermediate-duration, moderate intensity events in the early-fall through early-winter season; and long-duration, low intensity storm events in the late-fall through winter season. Multi-day through weekly periods of heavy precipitation are important events for rainfall-runoff modeling of forested conditions where the runoff response is primarily interflow and subsurface flow. Monthly and multi-month precipitation can also be important because they affect soil moisture conditions antecedent to storm events. Each of these durations and associated storm types has distinctive magnitude-frequency relationships that must be preserved in the scaling operation. Therefore, proper scaling of precipitation time-series must be accomplished at a wide range of durations to preserve the storm characteristics that are important for continuous rainfall-runoff modeling. Preservation of storm characteristics at numerous durations is not possible with a single scaling factor.

Storm Characteristics Vary by Topographic/Climatological Setting

Storm characteristics also vary by topographic and climatological setting in western Washington. For example, storm characteristics and statistics are different for sites to the west of central Puget Sound in areas where mean annual precipitation is decreasing from west to east relative to sites east of central Puget Sound where mean annual precipitation is increasing from west to east. These two regions may be considered as being leeward of the Olympic Mountains and windward of the Cascade Mountains, respectively. This situation was specifically addressed in development of the extended precipitation time-series by providing separate time-series for west and east of central Puget Sound. This complex situation is more difficult to address when using a single scaling factor approach for stations that are randomly spaced throughout western Washington.

Diversity of Storm Temporal Patterns

The long 121-year and 158-year records provide a rich diversity of the storm temporal patterns, multi-day sequences of storms, and seasonality of occurrence of storm events that are possible in western Washington. These long records represent three to five times the number of combinations of storm magnitudes and storm patterns that are typically available in the record from a single station. Long records with a diversity of storm temporal patterns provide for a robust examination of the performance of detention and water-quality facilities over a very wide range of flow conditions.

Estimation of Moderate to Rare Floods

Estimation of rare flood events is always of interest in hydrologic modeling. Use of the extended record allows for interpolation rather than extrapolation in estimating the characteristics of 25-year, 50-year, and 100-year floods. This is particularly important for estimation of the flood magnitude-frequency characteristics of streamflows downstream of detention facilities, as these streamflows are not amenable to standard statistical frequency analysis.

Quality and Resolution of Precipitation Records

The quality of precipitation records at hourly recording gages varies widely. The operators for most hourly gages are volunteers, who are part of the national cooperative network for precipitation measurement. High quality precipitation records are dependent upon both proper mechanical operation and diligent monitoring by the operator. Quality can be compromised by mechanical problems, poor maintenance, inattention to malfunctions, misoperation, and poor record keeping. Unfortunately, it is all too common to have numerous episodes of both short and prolonged periods of missing data in the records from many of the hourly gages in western Washington.

Hourly precipitation records from 1948 to the late 1960’s were recorded by weighing bucket gages with paper strip charts. These gages provided for a resolution to 0.01-inches. In the late 1960’s, these gages were replaced by tipping bucket gages. The vast majority of the replacement gages have a bucket volume that provides resolution to 0.10-inches, which gives poor temporal resolution for the low to moderate-intensity winter storms common in western Washington. Tipping bucket gages with 0.10-inch buckets are also susceptible to evaporation loses from partially full buckets between storm events. This can lead to significant underestimation of monthly and annual precipitation totals.

The extended precipitation-time-series records were developed from gages with very high quality records, operated by weather service personnel, and that measured to a resolution of 0.01-inches or higher.

Extended Precipitation Timeseries Selection Example

A project site is located in Thurston County as shown in Figure 4.2. The Project Site is located in the Puget Sound West region with a mean annual precipitation of 51 inches. From the Climatic Region drop down box, select the extended precipitation timeseries for the western Puget Sound Region with mean annual precipitation closest to the project site. In this case, select Puget Sound West Region, 52 inches MAP. The mean annual precipitation may also be determined by entering the project latitude and longitude in the Mean Annual Precipitation Calculator (in decimal degrees) and clicking the *Compute MAP* button.

Figure 4.2 – Extended Precipitation Timeseries Selection Example

4.1.2 Single Scaling Factor Approach

For projects sites located outside of the extended timeseries region, a *source* gage is selected and a single scaling factor is applied to transpose the hourly record to the site of interest (target site). The current approach for single scaling, as recommended in the *Stormwater Management Manual for Western Washington*[9], is to compute the scaling factor as the ratio of the 25-year 24-hour precipitation[21] for the target and source sites:

$$Scale\ Factor = P_{25\ TargetSite}/P_{25\ SourceGage} \qquad (4.1)$$

where: $P_{25\ TargetSite}$= 25-year 24-hour precipitation at the project site of interest (entered by user)

$P_{25\ SourceGage}$ = 25-year 24-hour precipitation at the source gage (provided by program)

The values of the 25-year 24-hour precipitation may be obtained from NOAA Atlas #2[18] or from the recently released WSDOT update of precipitation-frequency information for western Washington[21]. To utilize the recently updated precipitation-frequency information for western Washington, open the *Precipitation Map* from the *Project Location Tab*. Regions of influence for each gage are identified on the map along with the 25-year 24-hour precipitation. Choose the precipitation region where the project site is located. Read the project site 25-year 24-hour precipitation from the map and enter it in the appropriate field on the Project Location Tab. The computed scale factor will be displayed in the Scale Factor field. Alternatively, the project 25-year 24-hour precipitation may be computed by entering the project latitude and longitude in the Precip Calculator (in decimal degrees) and clicking the *Compute 25-Yr. 24-Hr* button.

Precipitation Input Selection Example

A project site is located in Grays Harbor County as shown in Figure 4.3. The project is located outside of the region where extended precipitation timeseries are available and the simple scaling approach must be used. The project site is located in the Clearwater precipitation region. The 25-year 24-hour precipitation at the Project Site is 6.0 inches. The Clearwater gage should be selected as the source for this project, and a project site 25-year, 24-hour precipitation of 6.0 inches should be entered in the appropriate field on the Project Location tab. The Scale factor would be computed by the program as the ratio of the project site to station 25-year, 24-hour precipitation, or 6.0 inches divided by 7.9 inches equals 0.759. This value would be displayed in the *Scale Factor* field and all precipitation values subsequently read by the program would be multiplied by this value.

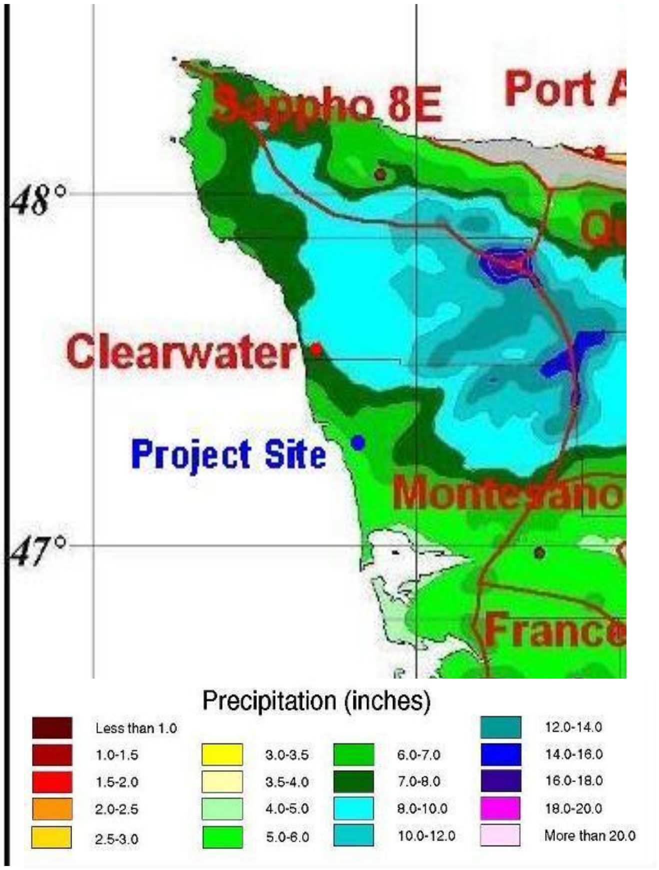


Figure 4.3 – Precipitation Input Selection Example for Project Sites Located Outside of Extended Precipitation Timeseries Region

5 Watershed Definition

MGSFlood utilizes a graphical interface for defining the layout of a watershed. Two scenarios may be defined; Predeveloped and Postdeveloped (Figure 5.1). Icons representing subbasins and other watershed features can be dragged onto the screen to define the watershed layout. Right clicking on an icon brings up a properties menu that can be used to define parameters, connect to a downstream link, compute statistics, etc.. The number of subbasins and links is limited by the memory and speed of the computer. As a practical limit, approximately 200 icons (subbasins plus links) can be simulated by the model. Earlier versions of MGSFlood utilized nodes to connect subbasins and links. These are now handled internally by the program and the user does not need to specify them explicitly.

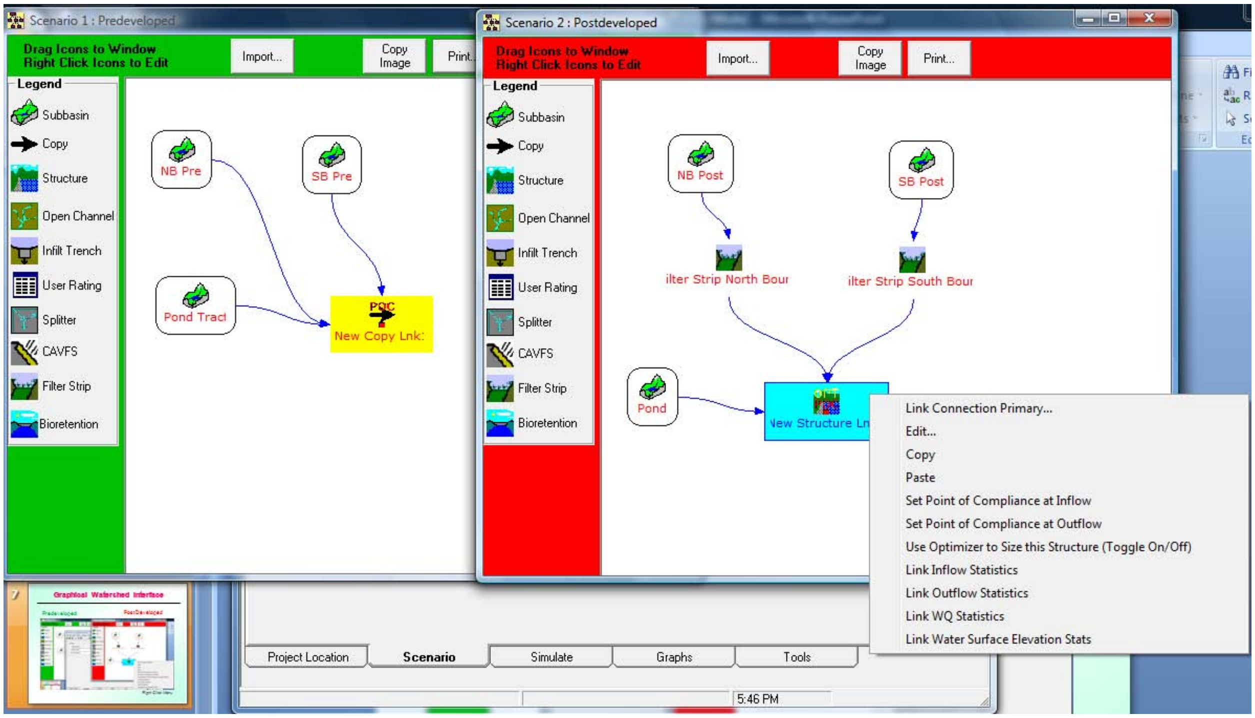


Figure 5.1 – Predeveloped and Post Developed Scenario Input Screens

To facilitate rainfall-runoff modeling, the project watershed must be defined in terms of *subbasins* and *links* (Figure 5.2). Land cover and soil type can vary within a subbasin and the program conducts rainfall-runoff modeling for each land cover/soil type combination separately. Links are used to route subbasin flows and may be defined as channels, ponds, wetlands, infiltration trenches, etc. Link definitions are discussed in Section 6, Network Connections.

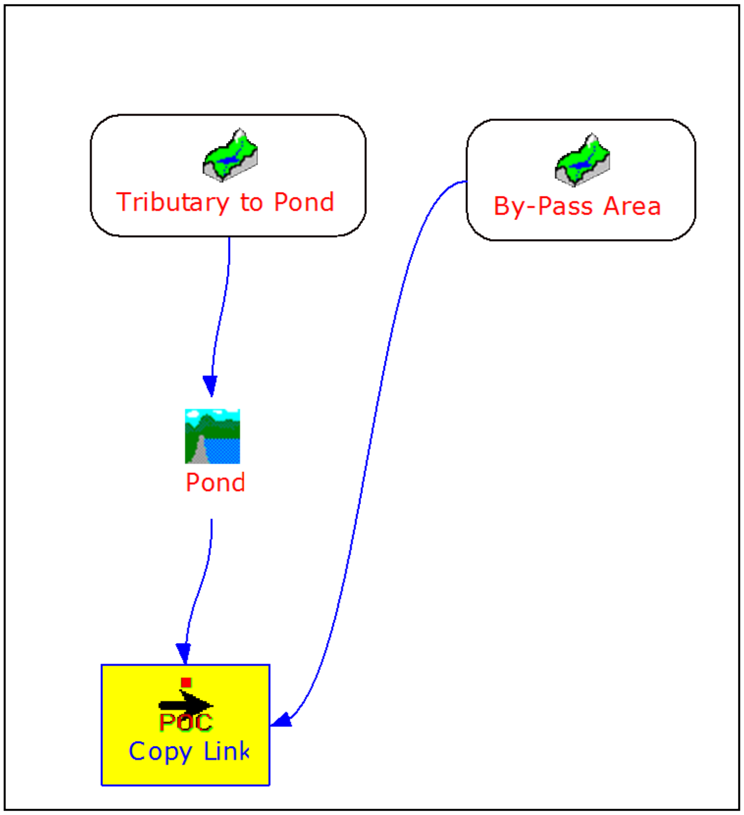


Figure 5.2 – Project Delineation, Single Subbasin with Bypass, and Stormwater Pond

5.1 Subbasin Land Use Input

The subbasin land cover is defined by dragging a subbasin icon onto the scenario input screen. Right clicking the icon and selecting Edit allows for the total acreage of each land cover/soil type combination to be specified.

Consult the stormwater management manual for the local regulatory jurisdiction and the Washington State Stormwater Management Manual for Western Washington (SWMMWW) regarding possible regulatory restrictions for:

- Predeveloped Forest Cover,
- Post Developed Forest Or Pasture Cover,
- Off-Site Run-On To Project,

- On-Site Stormwater Bypass.

Mapping of soil types by the Soil Conservation Service (SCS) is the most common source of soil/geologic information used in hydrologic analyses for stormwater facility design. Each soil type defined by the SCS has been classified into one of four hydrologic soil groups; A, B, C, and D. The *Stormwater Management Manual for Western Washington*[9] relates SCS hydrologic soil groups to HSPF soil/geologic groups as shown in Table 5.1

Table 5.1 – Relationship between SCS and HSPF Soil Groups

SCS Hydrologic Soil Group	MGSFlood/HSPF Soil/Geologic Group
A/B	Outwash
C	Till
D	Wetland

Note: The surface area of the pond must be included under the land use for the subbasin because precipitation is not applied to the pond surface by the program. This can be accomplished by adding impervious surface equal to the maximum pond surface area under the Subbasin Definitions window.

5.2 By-pass Areas

Local topographic constraints often make it impractical to direct all runoff from developed areas to a detention facility. If a portion of the developed watershed bypasses the facility, then a second subbasin that includes the by-pass area can be specified in the post developed scenario (Bypass area in Figure 5.2). This feature is useful for allowing a portion of the developed site to bypass the stormwater detention pond and the link inflow downstream of the pond is used as the point of compliance.

5.3 Connecting Subbasins and Links

Subbasins and link can be connected to downstream links by right clicking the upstream icon and selecting Link Connection Primary from the popup menu. A drop down menu appears listing the links in the scenario. Click the link of interest and flows will be routed to the downstream link. During the simulation, the outflow of each subbasin is saved for subsequent statistical analyses. For links, the inflow and outflow is saved. All inflows are combined and added before routing through the link.

6 Link Type Definitions

Links are used to connect one part of the watershed to another; subbasins to links and links to other links. The following summarizes the type of links currently in the model:

1. Copy – The Copy Link may be thought of as a dummy reach because is copies discharge from the inflow point to the outflow point without routing or lagging,
2. Structure – Includes detention and infiltration ponds, and sand filters,
3. Channel – Performs routing in open channels,
4. Infiltration Trench – Performs routing through infiltration trenches,
5. Rating Table – User defined stage storage discharge table,
6. Flow Splitter – Splits a fraction of the discharge from one link to another.
7. CAVFS – Compost amended vegetated filter strip,
8. Filter Strip – Similar to CAVFS, but doesn't include compost amendment,
9. Bioretention – Simulates bioretention facility with surface detention storage, infiltration, and underdrain return flow.

Information for each type of Link is discussed in the following Sections.

7 Copy Link

The copy link copies timeseries from the upstream subbasin or link and adds it to the inflow at the downstream link. Hydrographs are transferred to the outflow without attenuation or lagging. The copy link is appropriate for small watersheds where there is little attenuation of the flood hydrograph due to routing. If the conveyance channel is long with large overbank storage, then the link should be defined as an open channel. As a general rule, channel routing may be neglected for watersheds smaller than about ½ square mile (320 acres) and the link may be defined using the copy routine.

8 Structure Link

Structure links are used to define stormwater ponds, infiltration ponds, and sand filters. Pond optimization information for post-development condition ponds is also input on the structure link input screens.

A variety of hydraulic devices can be included in the design of stormwater treatment facilities. Devices attached to the riser structure include; circular orifices, circular orifices under backwater influence, rectangular orifices, rectangular weirs, V-notch weirs, and proportional weirs. In addition, the riser structure can also be defined with an open top to function as an overflow weir, or the top may be capped. Any combination of up to six devices plus the riser structure and a sand filter can be included for each structure. A trapezoidal broad crested weir may also be specified to function as an emergency overflow.

The following sections describe the input for Structure Links.

8.1 Pond/Vault Geometry Input

Two options are available for specifying pond or vault geometry. The first assumes a prismatic geometry with pond length, width, depth, and side slopes as shown in Figure 8.1.

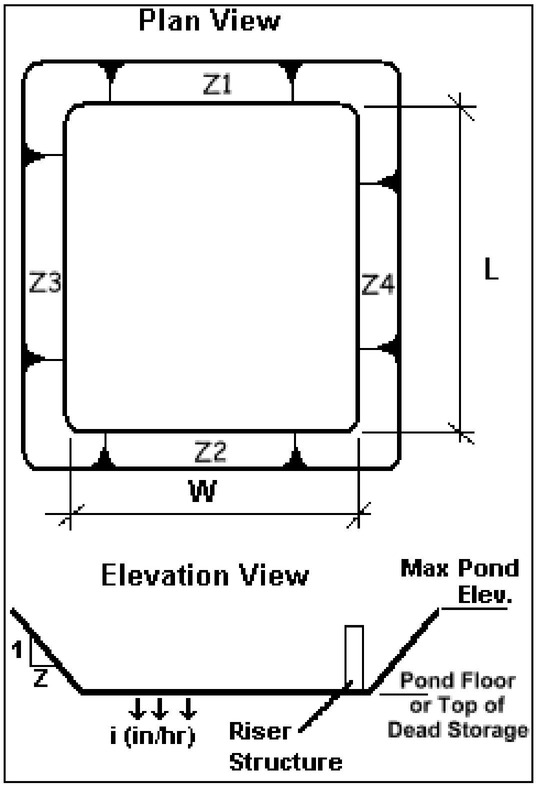


Figure 8.1 – Pond Geometry Definition for Prismatic Ponds

where:

L – is the pond length in feet,

W – is the pond width in feet,

Z1, Z2, Z3, Z4 - are side slopes for each side of the pond where Z is the number of feet in the horizontal plane for every foot of rise,

Pond Floor Elevation – Represents the bottom of the *live* pond storage. Live storage is defined as the storage used to detain stormwater runoff and eventually flows through the outlet structure. Dead storage is retained in the pond below the elevation of the outlet structure. The pond floor elevation should be input if the pond is <u>not</u> a combined wet pond. If the pond is a combined wet pond, then enter the elevation of the top of the dead storage, i.e. the elevation where water begins to discharge from the pond

Riser Crest Elevation – The elevation at which water begins to flow into the overflow riser. The maximum flood recurrence interval detained by the pond generally corresponds with this elevation (or slightly above this elevation). For example, the Ecology flow duration standard requires control of the flow duration between ½ of the 2-year and the 50-year recurrence interval. Water will begin to spill into the riser structure near the 50-year recurrence interval. It is acceptable for water to spill into the riser structure for floods smaller than the 50-year provided that the flow duration standard is met.

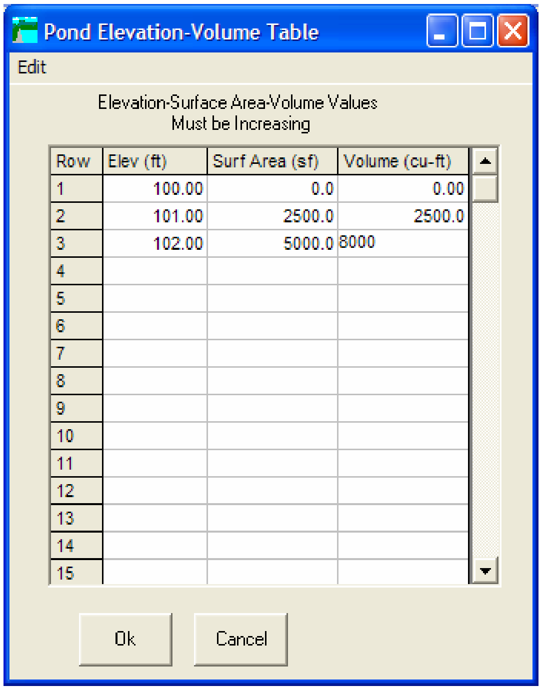
Max Pond Elevation – Is the maximum elevation used in pond routing calculations and typically extends above the riser crest elevation a sufficient distance to accommodate large floods or to allow for flood passage if one or more of the lower level outlets become blocked. The required maximum pond elevation depends on the design standards of the local jurisdiction.

The automatic pond sizing routine (optimizer) in MGSFlood determines the riser diameter and maximum pond elevation so that the 100-year peak inflow will pass through the riser structure assuming the lower level outlets are blocked. The user is advised to check the maximum pond elevation returned by the optimizer with the design standards of the local jurisdiction including any freeboard requirements.

If a vault is to be analyzed, then side slopes (*Z1, Z2, Z3, Z4*) of zero are input denoting vertical sides. The pond volume for elevations ranging from the floor to one foot above the maximum pond elevation is computed according to this geometry.

The second method for specifying pond geometry is with a user defined elevation-volume table as shown in Figure 8.2. This is useful for specifying the geometry of irregularly shaped ponds. The elevation-volume relationship can be computed using a spreadsheet program and pasted into the form using the Windows Clipboard utility.

Note: Precipitation falling on the surface of the detention pond is not automatically computed by MGSFlood. This approach was taken to allow use of both ponds and vaults. The difference being ponds are open to collection of precipitation, and vaults are closed to precipitation input. To include precipitation on the pond surface in the computations, the surface area of the pond must be included under the land use for the subbasin where the pond resides. This can be accomplished by adding impervious surface equal to the maximum pond surface area under the *Subbasin Definitions* window for the sub-basin where the pond resides. A simple approach to get an initial estimate of the pond surface area would be to run the *Quick Optimization routine* after the tributary subbasins have been defined.



Elevation-Surface Area-Volume Values
Must be Increasing

Row	Elev (ft)	Surf Area (sf)	Volume (cu-ft)
1	100.00	0.0	0.00
2	101.00	2500.0	2500.0
3	102.00	5000.0	8000
4			
5			
6			
7			
8			
9			
10			
11			
12			
13			
14			
15			

Figure 8.2 – Pond Elevation-Volume Entry Screen
(Useful for Irregularly Shaped Ponds)

8.2 Pond Infiltration

MGSFlood includes two options for simulating infiltration; Massmann[30] equations and fixed infiltration. The Massmann equations are based on field observations of infiltration ponds in western Washington (See Section 16). This infiltration approach accounts for the side slope geometry of the pond, pond aspect (length to width ratio), the proximity of the pond to the regional groundwater table, and the potential for soil clogging and fouling. Inputs include; Soil Hydraulic Conductivity (inches/hour), Depth to the Regional Water Table (ft) (Figure 8.3), whether bio-fouling potential is low, and whether average or better maintenance is performed. Infiltrated moisture is lost from the system and does not contribute to the discharge rate through the riser or orifices.

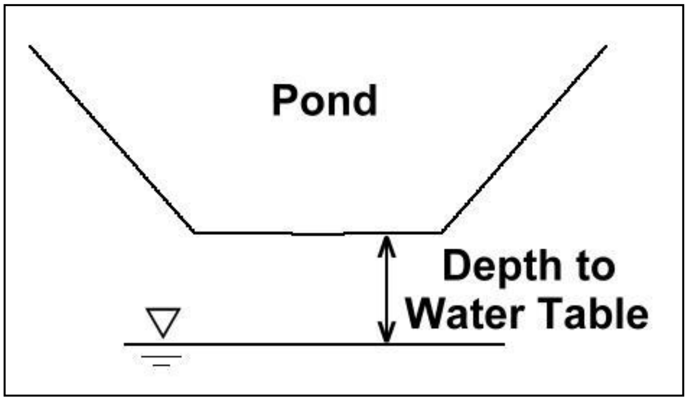


Figure 8.3 – Infiltration Pond Depth to Water Table (Accounts for Groundwater Mounding Beneath Pond)

The fixed infiltration option uses a constant user defined infiltration rate.

8.3 Circular Orifice

Orifices[3,6] can be defined as oriented in either the vertical or the horizontal plane. The discharge for orifices oriented horizontally, or fully submerged orifices in the vertical plane (Figure 8.4) are computed using Equation 8.1.

$$Q = C_d A\sqrt{2gH} \qquad 8.1$$

Where: Q is the discharge at a given pond water surface elevation,
C_d is a coefficient of discharge (0.61 without elbow, 0.58 with elbow),
A is the orifice area,
g is the acceleration due to gravity, and
H is the head, as measured between the pond water surface elevation and the water surface elevation at the orifice outlet.

For orifices mounted in the vertical plane and not subject to backwater from the outlet conduit (Figure 8.4), head (*Hm*) is measured as the difference between the water surface elevation in the pond and the elevation of the centroid of the orifice.

For orifices mounted in the horizontal plane and not subject to backwater from the outlet conduit (Figure 8.4), head (*Ht*) is measured as the difference between the water surface elevation in the pond and the water surface elevation at the outlet from the orifice.

Note when specifying the elevation of the lowest orifice (such as the bottom orifice in Figure 8.4) the controlling elevation that governs the head on the orifice must be identified. The controlling elevation may be the invert elevation of the orifice, the centerline elevation of the orifice, the invert elevation of the outlet conduit, or the hydraulic grade line in the outlet pipe.

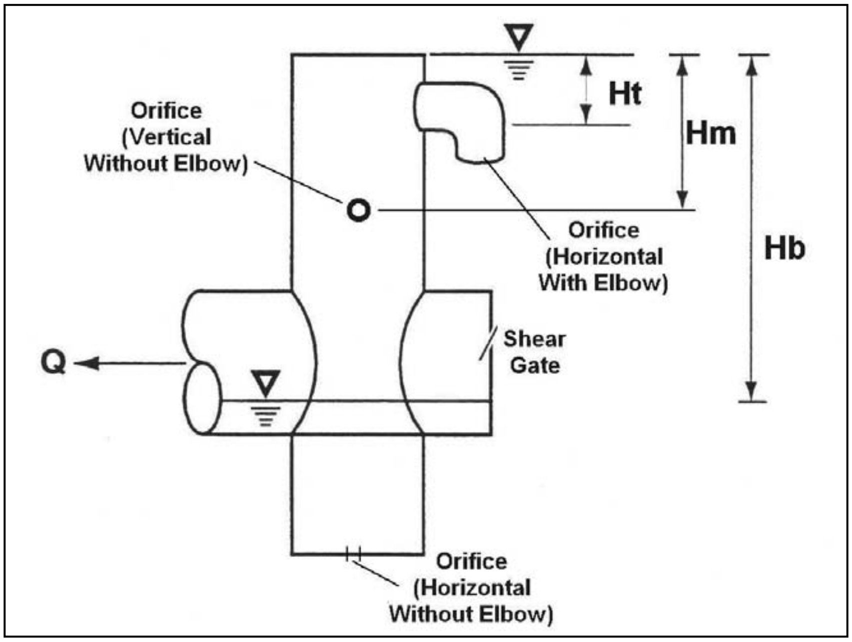


Figure 8.4 – Riser Structure Schematic for Three Possible Orifice Configurations

For orifices oriented in the vertical plane, where the water surface in the pond results in the orifice flowing partly full, discharge is computed based on critical depth occurring at the orifice face. The transition from flowing partly full to orifice flow occurs when the head (H_i) is near 110% of the orifice diameter. The governing discharge relationships for this situation (Equations 8.2a,b,c) are based on critical depth occurring in a circular section at the orifice face[4].

$$Q = A_c\sqrt{\frac{gD_c}{\alpha}} \qquad 8.2a$$

and $D_c = A_c/T$ 8.2b

and $H_i = Y_c + D_c/2$ 8.2c

Where: Q is the discharge at a given pond water surface elevation,
Y_c is critical depth at the face of the circular orifice,
A_c is the cross-sectional area of flow at critical depth,
T is the top width of flow at the orifice opening for critical depth,
D_c is the hydraulic depth,
H_i is the head on the orifice, as measured from the water surface elevation of the pond to the invert elevation of the orifice,
g is the acceleration due to gravity, and
α is a type of discharge coefficient (1.00)

8.4 Circular Orifice with Tailwater

Discharge for the lowest circular orifice can be computed with or without tailwater conditions due to downstream controls such as pipe networks. If tailwater conditions are present, an elevation-discharge rating table must be included that describes the tailwater condition downstream. A minimum of four elevation-discharge pairs are needed to define the tailwater rating table. The program uses an iterative procedure whereby the discharge computed using Equation 8.1 is based on the difference between the pond water surface elevation and the water surface elevation in the outlet conduit/riser section (*Hb,* Figure 8.4), and the computed discharge matches the discharge and tailwater elevation obtained from the rating table defining the downstream conditions.

Some possible applications for the tailwater routine might include:

a) Tailwater from a lake;
b) Tailwater from another stormwater pond;
c) Tailwater from high groundwater level that causes backwater against the outfall of the outlet conduit;
d) Tailwater from high tide or other tidal influence;
e) Tailwater from floodwaters from a receiving stream or overbank area of a floodplain;
f) Tailwater from concurrent discharges where the pond outlet connects into a closed stormwater system.

Cases a) and c): – tailwater may be essentially fixed with a very small change in tailwater elevation for various discharges from stormwater pond. The user would enter a constant tail water elevation for each entry in the elevation-discharge table. Discharge values would then be entered that covered the full range of possible discharges for the pond.

Cases e) and f): – if the receiving systems are sufficiently complex and difficult to analyze, an analysis approach would be to assume the frequency of floods discharging from the pond are similar to the receiving system. That is, the pond discharges at a 10-year recurrence interval at the time the receiving system is experiencing a 10-year flood. This assumption would allow determination of a tailwater level for the receiving system (floodplain analysis) and obtaining the corresponding 10-year flood discharge from MGSFlood for the stormwater pond.

Design Steps For Tailwater Situations

1. Design the outlet structure for the stormwater pond for the case of no tailwater to provide an initial estimate of the configuration of the outlet structure and pond. Note the maximum discharge from the pond. This provides the range of possible discharges from the pond (0 cfs to maximum discharge);

2. Review the flood-frequency curve for pond discharges (MGSFlood) to provide information on the frequency of occurrence of various discharges throughout the range of possible discharges. This information may be helpful if the

tailwater conditions vary based on the magnitude of the concurrent flood event in the receiving system. This may be true for cases a), b), e) and f) above.

3. Determine the range of reasonable tailwater elevations through analysis, judgment, and/or policy for the range of possible discharges from the stormwater pond. Tailwater conditions may be independent of discharge magnitude from the stormwater pond (Case d), or they may be related through seasonality (Cases a, c), or they may be related by concurrent flood events (Cases a, b, e, f). Provide a minimum of four data pairs for tailwater elevation and corresponding discharge (Steps 1 and 2) that reflect the operation of the "system" that is causing the tailwater condition. The tailwater elevations must be distinct values, even if only slightly different from one-another for the range of possible discharges.

4. Rerun the problem with MGSFlood using the tailwater elevation-discharge rating curve obtained from Step 3 and note how the range of possible discharges has changed from the no tailwater case (Step 1).

5. If the revised tailwater elevation-discharge relationship is significantly different (based on solution from Step 4), then use the revised tailwater elevation-discharge relationship and rerun the problem again. Continue iterating until the proposed tailwater elevation-discharge relationship is consistent with that obtained for the solution of the pond configuration and the range of possible discharges from the pond.

8.5 Rectangular Orifice/Slot

A rectangular orifice[3,6] functions as an orifice when submerged at the orifice entrance, or as a rectangular sharp crested weir when partially submerged at the orifice entrance. This approach is also used for rectangular orifices (vertical slots) cut in the side of the riser to the riser crest.

It is assumed that a rectangular orifice would be mounted near mid-height on the outlet structure and would not be subject to tailwater conditions. Equation 8.1 is used in the calculation of discharge from a rectangular orifice if the orifice is submerged at the orifice entrance. If the orifice is constructed in the vertical plane, then it is treated as a rectangular sharp crested weir according to Equations 8.5a and 8.5b for low heads. At higher heads, the rectangular opening behaves as an orifice and Equation 8.1 is used. The transition from weir flow to orifice flow occurs when the depth above the slot invert reaches the value as defined in Equation 8.3. Note that the height of the orifice can be any size up to the top of the riser.

$$H_t = 1.60\,(1.0 - B/L) + 1.08 \qquad 8.3$$

Where: H_t is the depth above the orifice bottom where the discharge changes from weir flow to orifice flow

B is the effective weir length as defined in Equation 8.5b

L is the weir length.

8.6 V-Notch Sharp Crested Weir

The V-Notch weir is assumed to be constructed in the side of the riser structure with the top of the notch intersecting the riser crest (Figure 8.5a) and would not be subject to tailwater conditions. Discharge from the V-Notch weir[3,6] is computed according to Equation 8.4, where the weir coefficient (*Cw*) can be obtained from Figure 8.5b as described by Daugherty and Franzini[6]. Limited test data are available for V-notch weirs with narrow openings (small θ, less than 10°), users should be aware of greater uncertainty in the discharge coefficients for narrow openings.

$$Q = CwTan\left(\frac{\theta}{2}\right)H^{2.5} \qquad 8.4$$

Where: Q is the discharge at a given pond water surface elevation,
C_w is a weir coefficient of discharge (Figure 8.5b),
θ is interior angle of the V notch in degrees, and
H is the head above the weir invert.

When the water surface elevation in the pond exceeds the riser lip elevation and results in a 10 percent or more increase in the head (H) on the weir over the head associated with the riser lip elevation, the entrance to the V-notch weir becomes submerged and the orifice equation (Equation 6.1) is used to compute the discharge through the V-Notch. In this situation, the head on the orifice is measured from the pond water surface elevation to the centroid of the V-notch opening.

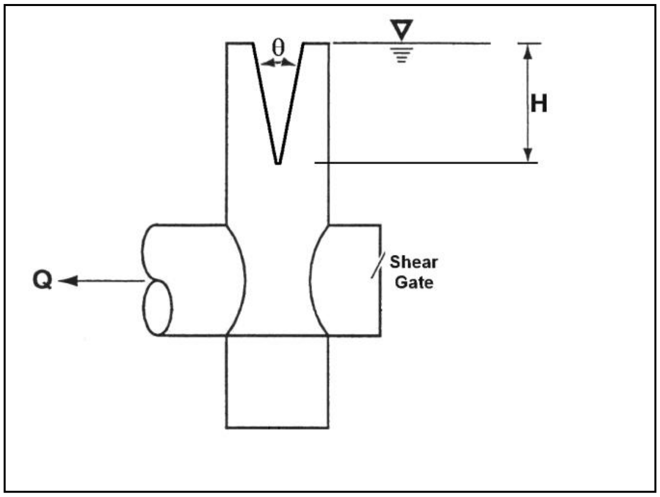


Figure 8.5a – V-Notch Weir

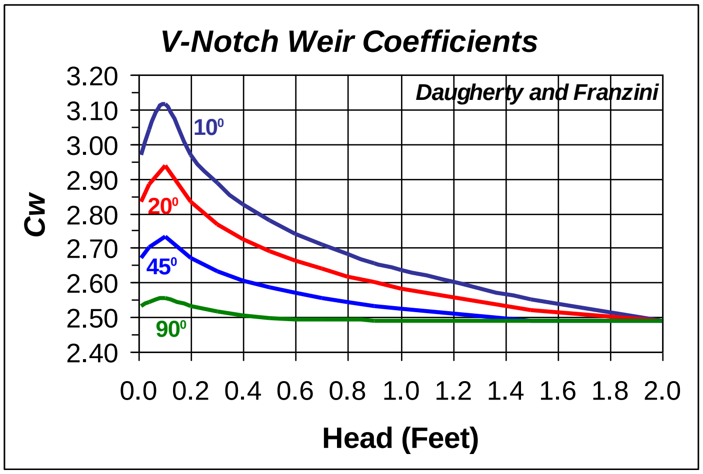


Figure 8.5b – Weir Coefficients for Various Opening Angles of V-Notch Weirs

8.7 Rectangular Sharp Crested Weir

The rectangular sharp crested weir[3,6] is assumed to be constructed in the side of the riser structure with the top of weir intersecting the riser crest (Figure 8.6) and would not be subject to tailwater conditions. Discharge from the weir is computed according to Equations 8.5a,b where *L* is the weir length and the weir coefficient (*Cw)* is 3.33. For narrow sharp crested weirs that function as a slot, contraction of the flow at the sides of the weir yields an effective weir length (*B*) equal to 80% of the constructed weir length (*L*).

$$Q = CwBH^{1.5} \qquad 8.5a$$

$$B = L(1.0 - 0.2H) \quad and \quad B \geq 0.8L \qquad 8.5b$$

Where: Q is the discharge at a given pond water surface elevation,
C_w is a weir coefficient (3.33),
B is the effective weir length, which is a function of the weir length (L) and the head on the weir (Equation 8.5b), and
H is the head as measured above the weir invert.

If the weir opening is narrow relative to the height, then it will behave as a weir at low heads and an orifice at higher heads. The transition from rectangular sharp crested weir (Equations 8.5a and 8.5b) to orifice (Equation 8.1) occurs when the depth above the weir invert reaches the value as defined in Equation 8.3. In this situation, the head on the orifice is measured from the pond water surface elevation to the centroid of the weir opening.

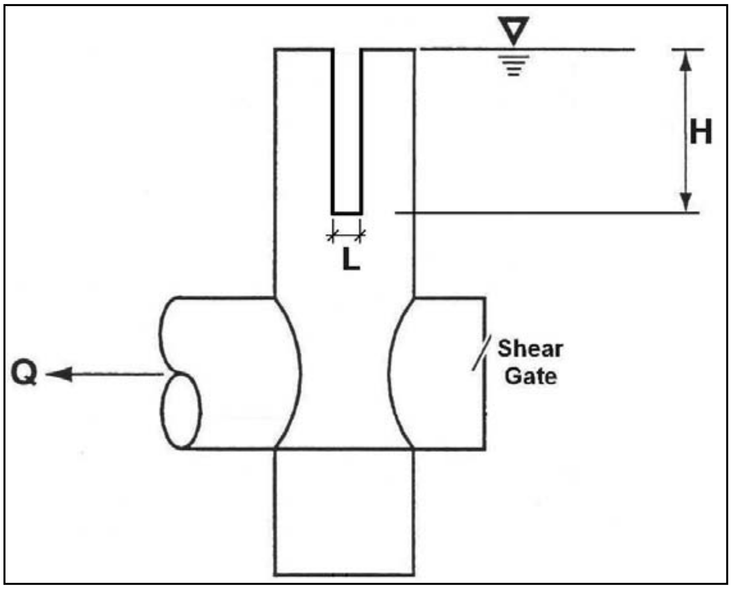


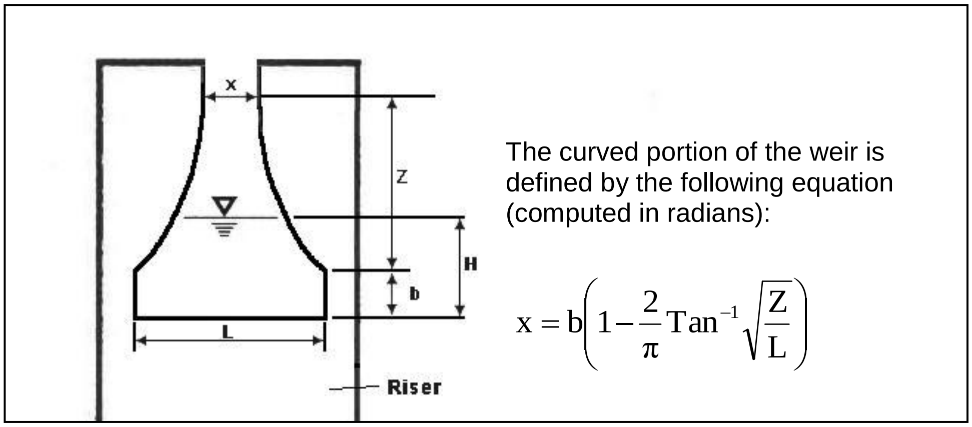
Figure 8.6 – Rectangular Sharp Crested (Slot) Weir

8.8 Proportional Weir

The proportional weir is assumed to be constructed in the side of the riser structure with the top of weir intersecting the riser crest (Figure 8.7) and would not be subject to tailwater conditions. The proportional weir has curved sides such that the discharge through the weir varies linearly with head[16] (Equation 8.6).

$$Q = C_d L \sqrt{2gb}(H - b/3) \qquad 8.6$$

Where: Q is the discharge at a given pond water surface elevation,
C_d is a coefficient of discharge (0.60),
L is the weir length at the base,
g is the acceleration due to gravity,
H is the head above the weir invert, and
b is the height of the vertical portion of the weir sidewall.



The curved portion of the weir is defined by the following equation (computed in radians):

$$x = b\left(1 - \frac{2}{\pi}\mathrm{Tan}^{-1}\sqrt{\frac{Z}{L}}\right)$$

Figure 8.7 – Proportional Weir

When the water surface elevation in the pond exceeds the riser crest elevation, the entrance to the weir becomes submerged and the orifice equation (Equation 8.1) is used to compute the discharge through the weir opening. In this situation, the head on the orifice is measured from the pond water surface elevation to the centroid of the weir opening.

8.9 Trapezoidal Broad Crested Weir

The trapezoidal broad crested weir[3,4,6] is commonly used as an emergency overflow structure and is assumed to be constructed on the pond/vault rim and does not interact with the riser structure. Discharge from the broad crested weir is assumed to be conveyed to the discharge point for the pond and is added to the discharge from any other structures associated with the pond.

The governing discharge relationships (Equations 8.7a,b,c) for the broad crested weir are based on critical depth occurring on the weir crest[4] (Figure 8.8).

$$Q = A_c \sqrt{\frac{gD_c}{\alpha}} \qquad 8.7a$$

$$\text{and} \quad D_c = A_c/T \qquad 8.7b$$

$$\text{and} \quad H = Y_c + D_c/2 \qquad 8.7c$$

Where: Q is the discharge at a given pond water surface elevation,
Y_c is critical depth on the weir,
A_c is the area of discharge at critical depth,
T is the top width of flow at the weir opening for critical depth,
D_c is the hydraulic depth at the weir opening,
H is the head on the weir, as measured from the water surface elevation of the pond to the invert elevation of the weir,
g is the acceleration due to gravity, and
α is a type of discharge coefficient (1.20)

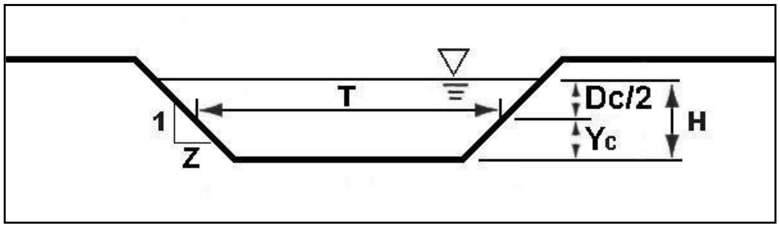


Figure 8.8 – Broad Crested Trapezoidal Weir

The critical depth-equations can be reformulated into the general form of the weir equation (Equation 8.5a) for a broad-crested weir with a rectangular cross-section. In this format, an alpha value (α) of 1.00 yields a weir coefficient (Cw) of 3.09, the maximum value for rectangular broad-crested weirs. Use of an alpha value (α) of 1.20 results in a weir coefficient of 2.82, which is applicable for broad-crested weirs operating at shallow depths. This would represent typical conditions for a broad-crested weir operating as an emergency spillway for a detention pond.

8.10 Riser Structures

A single riser structure can be specified for each pond analyzed. The riser can be either circular or rectangular in cross section with the top either closed (capped) or open. When the top is open, discharge is allowed to occur over the riser crest effectively functioning as an overflow spillway. If the riser top is open, discharge over the riser rim is computed according to Equations 8.8a,b.

Hydraulic structures that intersect the riser crest, such as V-notch or rectangular sharp crested weirs, are accounted for by entering a *common length*. This ensures that the discharge from the hydraulic structure is not double counted when flow passes over the riser crest. For the example shown in Figure 8.9, the 1.5-foot wide rectangular sharp crested weir intersects the riser crest. A value of 1.5 feet would be input as the *common length* under the Riser Structure Parameters.

For narrow devices that intersect the riser crest, there is little difference between the slot width (chord length on circle) and the arc length. Thus, the slot width can be entered for the common length. For structures that are wide relative to the diameter of the riser, the arc length should be computed and entered for the common length (Figure 8.10).

$$Q = C_w B H^{1.5} \tag{8.8a}$$

$$B = L - \textit{common length} \tag{8.8b}$$

Where: Q is the discharge at a given pond water surface elevation,
C_w is a weir coefficient that is initially 3.33, and decreases with increasing head (H) on the weir[28,29],
L is the weir length as measured along the circumference of the riser top,
B is the effective weir length, which is a function of the weir length (L) and reduced by any common length with other discharge devices according to Equation 8.8b, and
H is the head as measured above the riser lip.

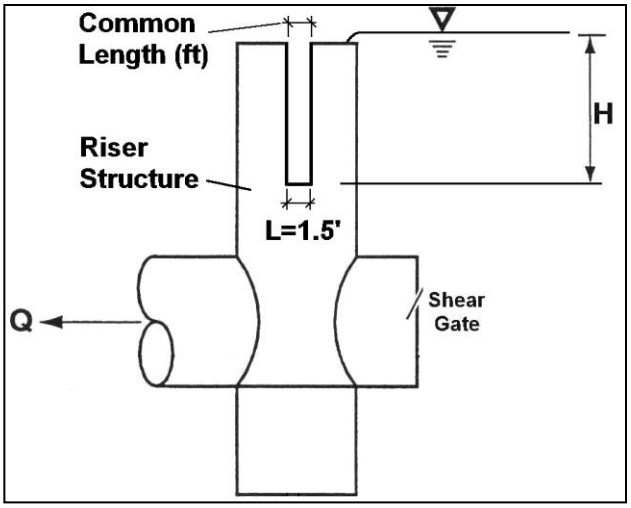
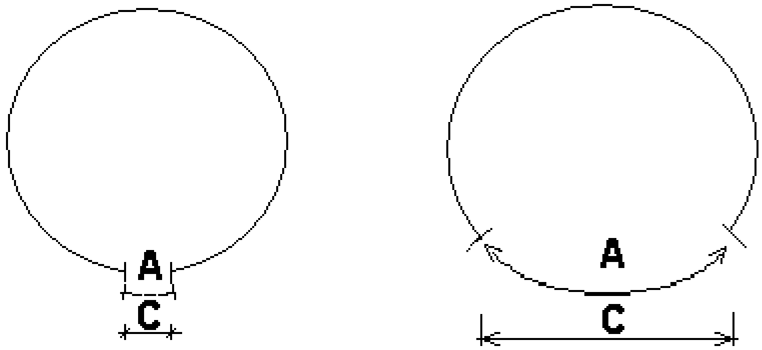


Figure 8.9 – Riser Structure



Arc Length (A) ≈ Chord Length (C)
(Chord Acceptable as Common Length)

Arc Length (A) > Chord Length (C)
(Input Arc Length as Common Length)

Figure 8.10 – Plan View of Circular Riser Showing Common Length for Narrow and Wide Weir

8.11 Sand Filter

A sand filter functions much like an infiltration pond except that instead of infiltrating into native soils, stormwater filters through a constructed sand bed with an underdrain system. Unlike infiltration from the pond bottom, the underdrain system is connected to the stream network and any discharge from the sand filter is added to the discharge from the outlet structure associated with the pond. The sand filter offers a mechanism for release of very small quantities of discharge as an alternative to a very small low-level circular orifice that is susceptible to debris blockage. A sand filter also removes pollutants by filtration. As stormwater passes through the sand, pollutants are trapped in the interstices between the sand grains.

It is assumed that discharges via a sand filter would be minimal relative to other discharge devices, and there would be no tailwater conditions present when the pond depth is above the sand filter but below any intermediate hydraulic device.

The discharge rate through a sand filter is computed using Darcy's law[6] (Equations 8.9a,b).

$$Q=KiA \qquad 8.9a$$

$$i=H/L \qquad 8.9b$$

Where: Q is the discharge through the filter for a given pond water surface elevation,
K is the saturated hydraulic conductivity (permeability)
i is the hydraulic gradient through the filter
H is the head on the filter at a given pond water surface elevation
L is the filter thickness, and
A is the surface area of the filter perpendicular to the direction of flow.

The percentage of pond inflow that passes through the sand filter is computed by the program and listed in the project report. Sizing the sand filter area is a trial and error procedure whereby different filter surface areas are tried; flows are routed through the facility and the percentage of runoff treated by the filter is noted from the project report. The process is repeated until the required level of treatment is achieved.

8.12 Automatic Pond and Outlet Works Sizing Routine

The proprietary version of MGSFlood includes routines for computing pond hydraulics and automatically sizing the pond and outlet works to meet the Washington State Department of Ecology Flow Duration Standard[9]. Designing stormwater ponds to this standard is a laborious, iterative process whereby the runoff timeseries (typically 40-years or more) is routed through the pond, flow-duration statistics computed and then compared with pre-developed flow-duration statistics. The automatic pond sizing routine performs this pond design procedure automatically.

The automatic pond sizing optimization routine in the MGSFlood will determine the pond size and outlet configuration for two pond types; a detention pond with minor infiltration and an infiltration pond (the routine will also automatically size infiltration trenches, see Section 10.3). The characteristics of these two pond types are listed in Table 8.1

Table 8.1 – Characteristics of Detention and Infiltration Ponds Sized using Optimization Routine

Characteristic	Detention Pond	Infiltration Pond
Pond Configuration	Riser Structure with Low Level Circular Orifice and Vertical Rectangular Upper Orifice	Overflow Riser Only
Valid Infiltration Rates*	0.00 – 0.10 inches/hour	0.05-50 inches/hour
Optimization Levels	Quick or Full	Quick Only

* Note: Infiltration occurs through the pond bottom only, not including the side slopes.

The pond sizing optimization routine uses general input about the pond geometry including;

- Pond length to width ratio,
- Pond side slope,
- Pond floor elevation,
- Riser crest elevation, and
- Pond infiltration information.

The pond sizing routine uses the information listed above to establish the geometric relationships for the pond configuration. The program establishes a parameter space of possible solutions by varying the pond bottom area, and sizes and elevations of hydraulic devices for the outlet structure. The program then routes the developed runoff timeseries through the pond and seeks to find a solution that provides the minimum pond size to meet the duration design standard.

The standard outlet configuration used for detention ponds consists of a circular low-level orifice and a vertical rectangular orifice (slot). If a different outlet configuration is desired, the volume-discharge characteristics of the desired configuration can be set to match the volume-discharge characteristics returned by the program for the orifice/slot weir configuration. The low-level circular orifice is assumed to be free of tailwater effects. If tailwater conditions are present, first use optimization to determine the pond configuration without tailwater. Then, include the tailwater rating table and manually adjust the pond configuration to meet the duration design criteria.

There are a wide variety of combinations of hydraulic devices, device sizes and invert heights, and pond configurations that can be used to match the flow duration standard. However, it is difficult to find a pond configuration that minimizes the pond volume and meets the duration standard using a manual trial and error approach. The automatic pond sizing routine searches the parameter space of possible solutions and seeks to find the minimum pond size to meet the flow duration standard.

The following steps describe the pond design process using the Hydraulic Structures, Optimization routine.

> Step 1. Input land use and drag a Structure icon to the Post Development Scenario screen.
>
> Step 2. Right click the structure and select Edit to bring up the structure editor screen. Select the *Optimization Input* tab and enter the general pond geometry. The geometry consists of the pond length, pond width, pond side slope, bottom of live storage elevation, riser crest elevation, and infiltration rate.
>
> Step 3. Select *Quick Optimization* or *Full Optimization* . Quick Optimization will determine a pond configuration, usually in 30 seconds or less, that meets or comes close to meeting the duration design criteria. Quick Optimization is the only option available if sizing an infiltration pond. The full optimization option takes longer and will converge to a solution for most project sites.
>
> Step 4. Click Ok to save the changes to the Structure Input.
>
> Step 5. Click the *Simulate* Tab and then Click the *Route* button. The program will compute runoff, route flows through the network, and then use the optimization routine to size the pond. When finished, the performance will be displayed on the Graphs tab.

The full optimization option takes longer and will converge to a solution for most project sites. In some situations, usually when precipitation with outliers or precipitation data of poor quality is used, the pond design may not meet all of the design criteria. In these cases, the pond design determined by the program is returned to the Hydraulic Structure Input Screen for manual refinement. Modifications can be made to the design by the user and flows routed through the pond using manual mode. Guidelines for adjusting the pond size and outlet works are discussed in Section 18.

9 Channel Routing Link

Channel routing is performed using a Modified Puls routing routine developed by the US Army Corps of Engineers for the HEC-1[25] flood hydrograph package. The user inputs the left overbank, main channel and right overbank channel cross sectional geometry, roughness, slope, and channel length. The program develops an elevation-volume-discharge rating table assuming normal depth at each discharge level and computes discharge according to the Manning Equation[4]. This rating table is then utilized by the Modified Puls routing routine to route flows .

MGSFlood includes two options for simulating infiltration; Massmann[30] equations and fixed infiltration. The Massmann equations are based on field observations of infiltration ponds in western Washington (See Section 16). This infiltration approach accounts for the side slope geometry of the pond, pond aspect (length to width ratio), the proximity of the pond to the regional groundwater table, and the potential for soil clogging and fouling. Inputs include; Soil Hydraulic Conductivity (inches/hour), Depth to the Regional Water Table (ft) whether bio-fouling potential is low, and whether average or better maintenance is performed. Infiltrated moisture is lost from the system and does not contribute to the discharge at the downstream end of the channel. The fixed infiltration option uses a constant user defined infiltration rate.

10 Infiltration Trench Link

MGSFlood includes two options for simulating infiltration; Massmann[30] equations and fixed infiltration. The Massmann equations are based on field observations of infiltration ponds in western Washington (See Section 16). This infiltration approach accounts for the side slope geometry of the pond, pond aspect (length to width ratio), the proximity of the pond to the regional groundwater table, and the potential for soil clogging and fouling. Inputs include; Soil Hydraulic Conductivity (inches/hour), Depth to the Regional Water Table (ft), whether bio-fouling potential is low, and whether average or better maintenance is performed. Infiltrated moisture is lost from the system and does not contribute to the discharge rate at the downstream end of the link. The fixed infiltration option uses a constant user defined infiltration rate.

The program routes flow for two types of infiltration trenches as shown in Figures 10.1 and 10.2; a trench located on the embankment side slope, or an infiltration trench located at the base of the embankment.

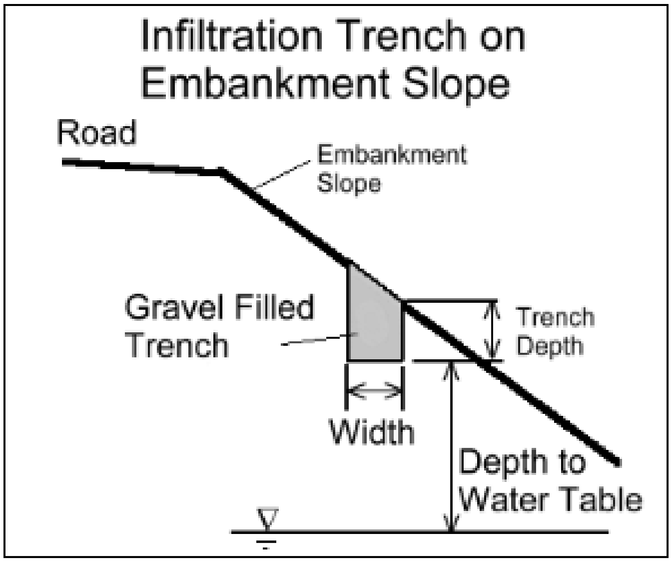


Figure 10.1 – Infiltration Trench Located on Embankment Slope Option

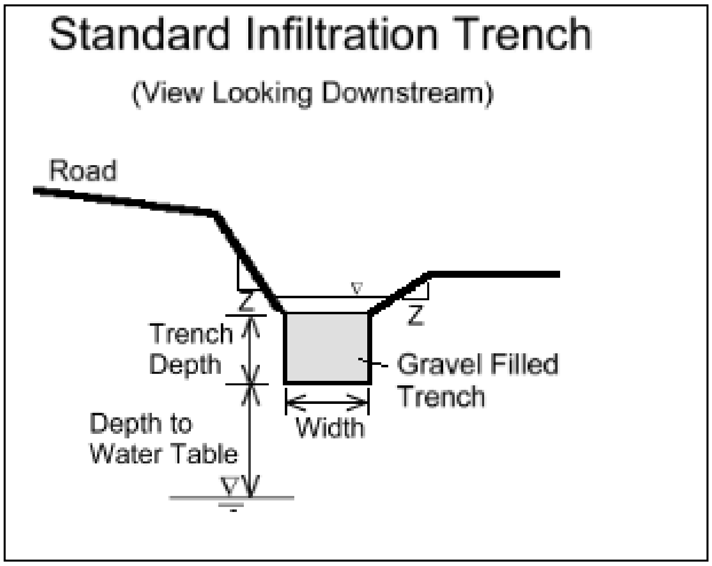


Figure 10.2 – Standard Infiltration Trench Option

10.1 Infiltration Trench Located on Embankment Slope

A trench is constructed along the roadway embankment and filled with gravel (Figure 10.1). Runoff from the roadway is directed to the gravel trench where it percolates through the gravel and infiltrates through the trench bottom. When the runoff rate exceeds the infiltration capacity, the gravel saturates from the bottom up with the voids in the gravel providing runoff storage, similar to a detention pond. If the storm is sufficiently large, the saturation will reach the ground surface and runoff from the road will pass over the gravel surface and continue down the embankment. Runoff not infiltrated in the trench is routed to the link outflow point.

It should be noted that the saturated hydraulic conductivity of the embankment fill will likely be different from the native material beneath the fill. The hydraulic conductivity estimates of the different layers can be combined using the harmonic mean (Massmann[30]):

$$K_{Equiv} = \frac{d}{\sum \frac{d_i}{K_i}} \qquad 10.1$$

Where: K_{Equiv} is the equivalent hydraulic conductivity,
d is the depth of the soil column above the regional groundwater table or limiting permeability layer,
d_i is the thickness of layer i,
K_i is the hydraulic conductivity of layer i

Note that the saturated hydraulic conductivity of the gravel in the trench is not included in Equation 10.1.
For sites with very deep groundwater tables (>100 feet), it is recommended that the total depth of the soil column in Equation 10.1 be limited to 20 times the trench depth.

10.2 Standard Infiltration Trench

The standard infiltration trench would be constructed at the base of the roadway embankment and would receive runoff from the adjacent roadway or from an upstream ditch. Runoff from the roadway is directed to the gravel trench where it percolates through the gravel and infiltrates through the trench bottom. When the runoff rate exceeds the infiltration capacity of the soil, the gravel saturates from the bottom up with the voids in the gravel providing runoff storage, similar to a detention pond. If the storm is sufficiently large, the saturation will reach the ground surface and runoff will occur down the ditch along the gravel surface. The program routes flow along the gravel surface to the link outflow according to the Manning Equation[4].

The infiltration trench routine may also be used to simulate a natural stream channel with infiltration through the channel bottom. The geometry of the channel is defined as a trapezoidal section and depth of gravel is input as zero.

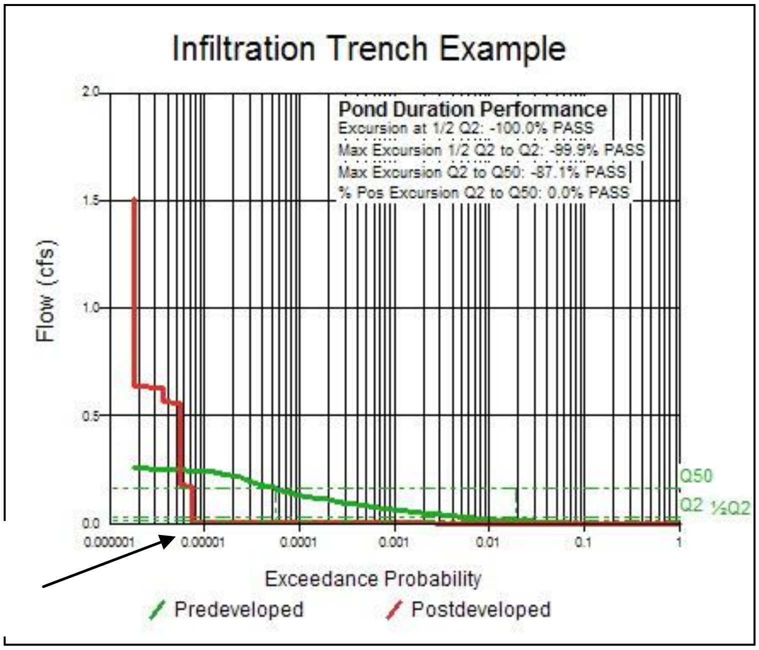
10.3 Automatic Infiltration Trench Sizing Routine

The automatic pond sizing optimization routine in the MGSFlood will automatically determine the size of infiltration trench required to meet the goals of the Ecology flow duration standard. The user inputs two of the three trench dimensions (length, width, or depth) and the optimizer solves for the third dimension. The input supplied by the user includes:

- The type of infiltration trench to be sized (Embankment Slope or Standard),
- The trench bottom elevation at the downstream end,
- Two of Three Trench Dimensions (Length, Width, or Depth)
- Rock fill porosity,
- Depth to water table,
- Saturated hydraulic conductivity of soil beneath trench.

The optimization routine uses the information listed above to establish the geometric relationships for the trench configuration. The program establishes a parameter space of possible solutions by varying the bottom width. The program then routes the developed runoff timeseries through the trench and seeks to find a solution that provides the minimum trench size to meet the duration design standard.

Flow duration curves computed for infiltration trenches typically plot along the horizontal axis and then bend sharply upward. This indicates that all runoff is infiltrated to a point and then the infiltration capacity is exceeded resulting in surface flow (Figure 10.3).

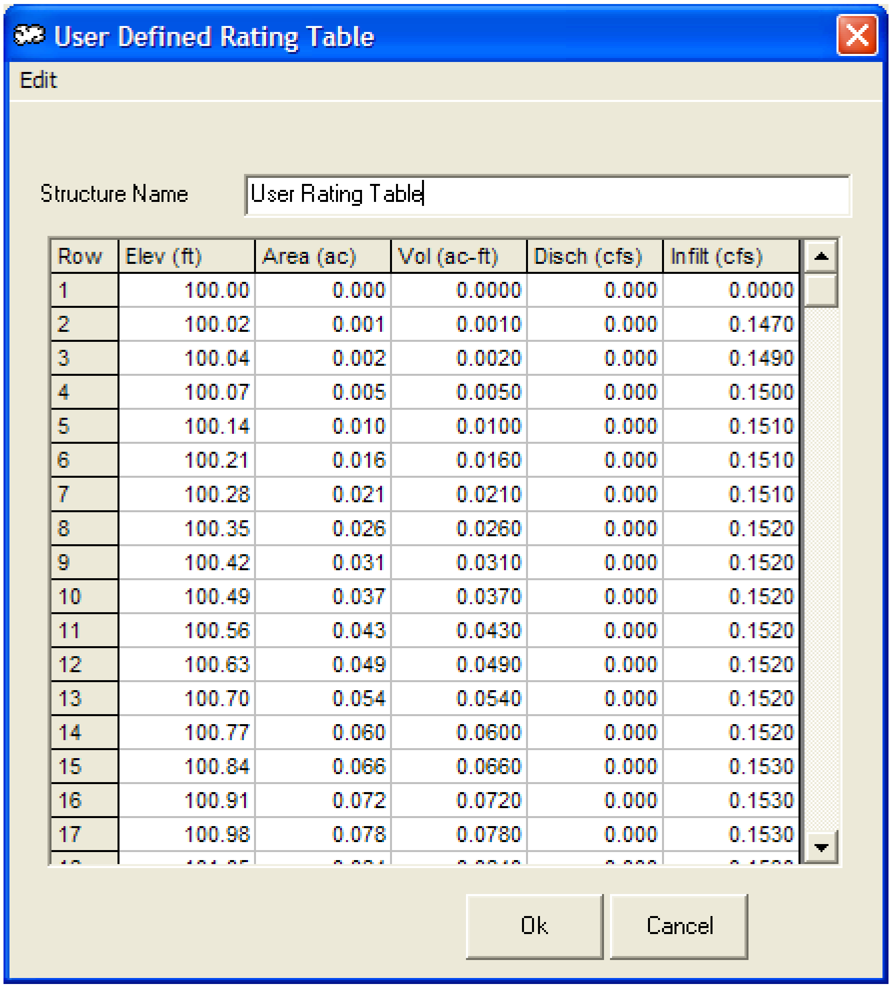


Point where Surface Runoff Begins

Figure 10.3 – Example Infiltration Trench Flow Duration Curve Performance

11 User Defined Rating Table Link

Structure hydraulics are specified using a stage-surface area-volume-discharge rating table (Figure 11.1). The pond storage (acre-feet), surface area (acres), discharge (cfs), and infiltration discharge (cfs) are computed by the user and entered in the table. Information may be copied from an external spreadsheet program and pasted into the input table using the Windows Clipboard utility.



Structure Name: User Rating Table

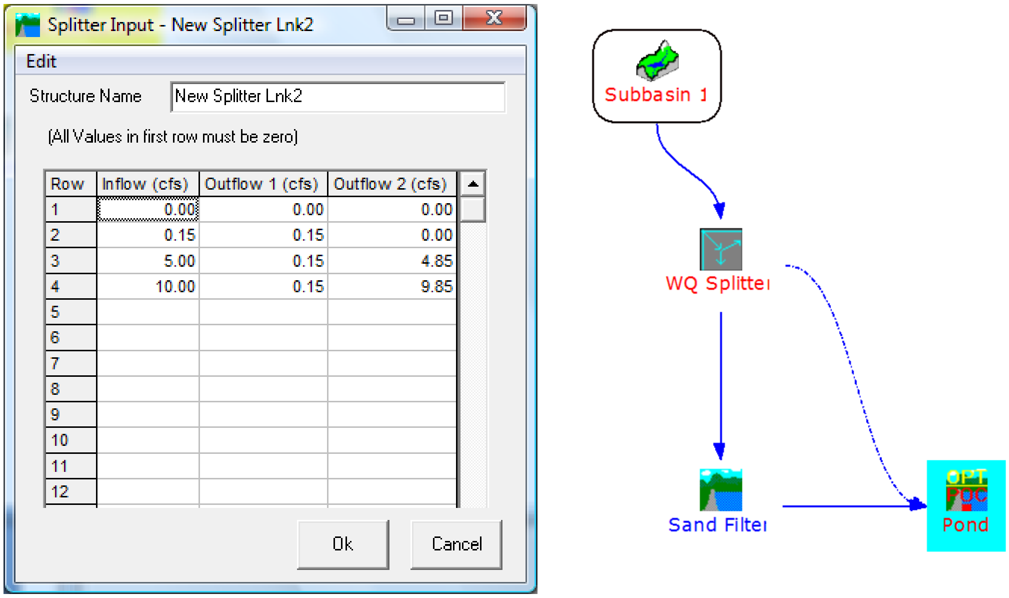
Row	Elev (ft)	Area (ac)	Vol (ac-ft)	Disch (cfs)	Infilt (cfs)
1	100.00	0.000	0.0000	0.000	0.0000
2	100.02	0.001	0.0010	0.000	0.1470
3	100.04	0.002	0.0020	0.000	0.1490
4	100.07	0.005	0.0050	0.000	0.1500
5	100.14	0.010	0.0100	0.000	0.1510
6	100.21	0.016	0.0160	0.000	0.1510
7	100.28	0.021	0.0210	0.000	0.1510
8	100.35	0.026	0.0260	0.000	0.1520
9	100.42	0.031	0.0310	0.000	0.1520
10	100.49	0.037	0.0370	0.000	0.1520
11	100.56	0.043	0.0430	0.000	0.1520
12	100.63	0.049	0.0490	0.000	0.1520
13	100.70	0.054	0.0540	0.000	0.1520
14	100.77	0.060	0.0600	0.000	0.1520
15	100.84	0.066	0.0660	0.000	0.1530
16	100.91	0.072	0.0720	0.000	0.1530
17	100.98	0.078	0.0780	0.000	0.1530

Figure 11.1 – User Rating Table Input

12 Flow Splitter Link

Flow splitter structures divert a portion of the flow at the splitter link inflow to a second link. Input consists of a table that specifies the inflow to the splitter link and the discharge to the splitter link outflow (Outflow 1) and the downstream link (Outflow 2). The secondary outflow from the splitter is denoted on the Postdeveloped Scenario screen as a dashed blue line (Figure 12.1). The program evaluates the inflow to the structure at each time step and determines the outflow to each downstream link by interpolation between rows of the table. The user should enter values in the table that extend beyond the maximum expected inflow to the link.

An example is shown in Figure 12.1 where a flow splitter is used to divert flows in excess of the water quality treatment discharge (0.15 cfs) around a sand filter link. In this example, the amount of runoff discharged to the sand filter structure and the size of the filter would be determined iteratively until the desired runoff percentage was treated by the sand filter (typically 91% of the total runoff volume).



Structure Name: New Splitter Lnk2

(All Values in first row must be zero)

Row	Inflow (cfs)	Outflow 1 (cfs)	Outflow 2 (cfs)
1	0.00	0.00	0.00
2	0.15	0.15	0.00
3	5.00	0.15	4.85
4	10.00	0.15	9.85
5			
6			
7			
8			
9			
10			
11			
12			

Figure 12.1 – Example Flow Splitter Input

13 Compost Amended Vegetated Filter Strip (CAVFS)

Compost Amended Vegetated Filter Strips (CAVFS) are land areas of planted vegetation and amended soils situated between the pavement surface and a surface water collection system, pond, wetland, stream, or river. These structures are used primarily for stormwater quality treatment; however, they do provide runoff attenuation via storage in the compost and infiltration into the underlying soils.

MGSFlood simulates flow through CAVFS using Darcy's Equation (Figure 13.1) where K is the saturated hydraulic conductivity. Note that the width dimension corresponds to the CAVFS width along the slope. Different hydraulic conductivity values are specified for the gravel spreader and the compost. Infiltration is accounted for using a constant infiltration rate into the underlying soils. During large storms, the voids in the CAVFS may become full (the CAVFS saturates) and additional flow directed to the CAVFS will run down the surface.

$$Q = K_c \frac{\Delta h}{w} Ld$$

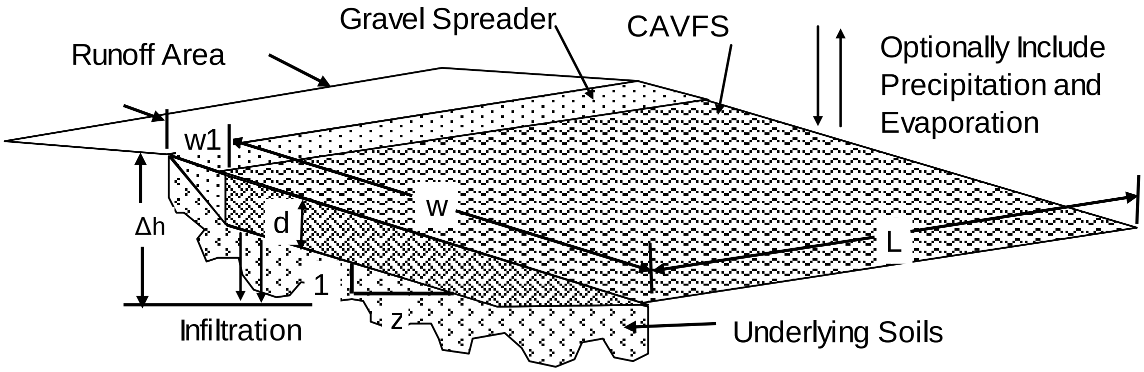
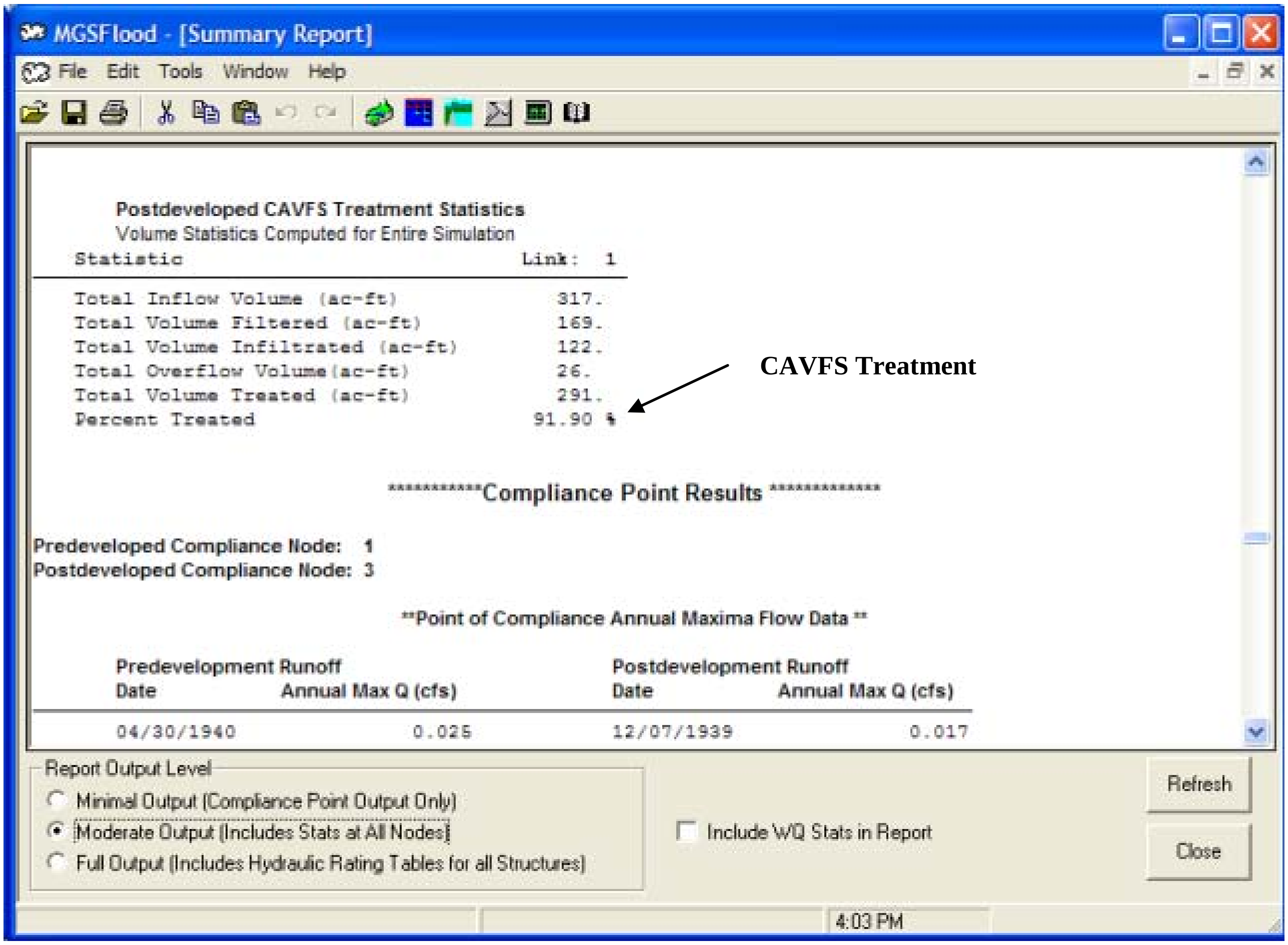


Figure 13.1 – CAVFS Definition Sketch

Precipitation and evapotranspiration may optionally be applied to the CAVFS. If precipitation and evapotranspiration are applied in the CAVFS link, the area of the CAVFS should not be included in the Subbasin Area input.

The size of CAVFS required for water quality treatment is determined via a trial and error procedure. Trial CAVFS dimensions are entered under the Link Definition. Runoff is then routed by clicking the Route button on the Simulate tab. When routing is completed, view the project report and locate the volume treated by the CAVFS. The runoff treated by the CAVFS is the sum of the filtered and infiltrated water and should be greater than or equal to 91 percent (Figure 13.2).



```
     Postdeveloped CAVFS Treatment Statistics
     Volume Statistics Computed for Entire Simulation
  Statistic                                   Link:  1
____________________________________________________________
  Total Inflow Volume (ac-ft)                   317.
  Total Volume Filtered (ac-ft)                 169.
  Total Volume Infiltrated (ac-ft)              122.
  Total Overflow Volume(ac-ft)                  26.
  Total Volume Treated (ac-ft)                  291.
  Percent Treated                             91.90 %


                    ***********Compliance Point Results *************

Predeveloped Compliance Node:   1
Postdeveloped Compliance Node:  3

                     **Point of Compliance Annual Maxima Flow Data **

     Predevelopment Runoff                    Postdevelopment Runoff
     Date              Annual Max Q (cfs)     Date              Annual Max Q (cfs)
____________________________________________________________________________________
     04/30/1940                0.025          12/07/1939                0.017
```

Report Output Level

○ Minimal Output (Compliance Point Output Only)
● Moderate Output (Includes Stats at All Nodes)
○ Full Output (Includes Hydraulic Rating Tables for all Structures)

☐ Include WQ Stats in Report

Figure 13.1- Project Report Showing Performance of CAVFS Designed to Meet the 91-Percent Water Quality Treatment Goal

14 Filter Strip

Filter Strips are land areas of planted vegetation situated between the pavement surface and a surface water collection system, pond, wetland, stream, or river. These structures are used primarily for stormwater quality treatment, however, they do provide some runoff attenuation via hydraulic routing and infiltration into the underlying soils. The user should refer to the appropriate stormwater design manual for information regarding sizing filter strips for water quality treatment.

MGSFlood performs routing through Filter Strips performed using a Modified Puls routing routine developed by the US Army Corps of Engineers for the HEC-1 flood hydrograph package. The user inputs the filter cross geometry and the program develops an elevation-volume-discharge rating table assuming normal depth at each discharge level and computes discharge according to the Manning Equation. This rating table is then utilized by the Modified Puls routing routine to route flows from the upstream to the downstream end. The user may also define infiltration according to Massmann’s Method (See Section 16), which uses the saturated hydraulic conductivity and depth to water table to compute infiltration losses.

Precipitation and evapotranspiration may optionally be applied to the Filter Strip. If precipitation and evapotranspiration are applied in the Filter Strip link, do not include the area of the Filter Strip in the Subbasin Area input.

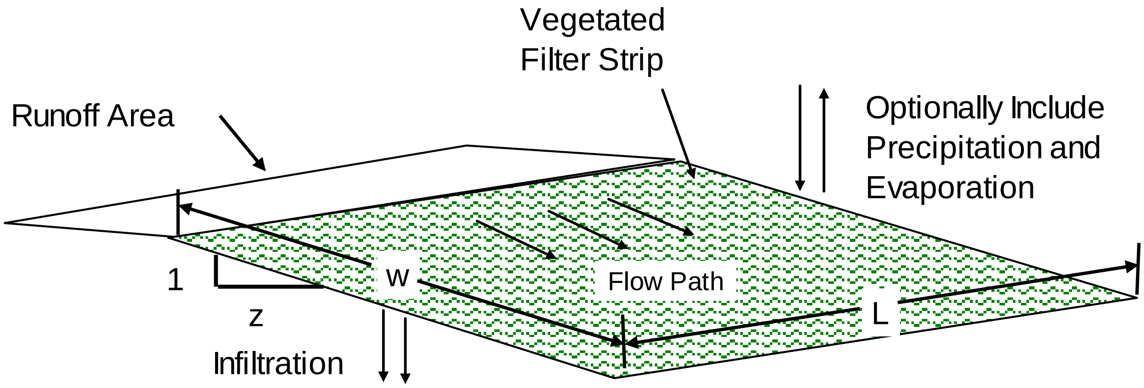


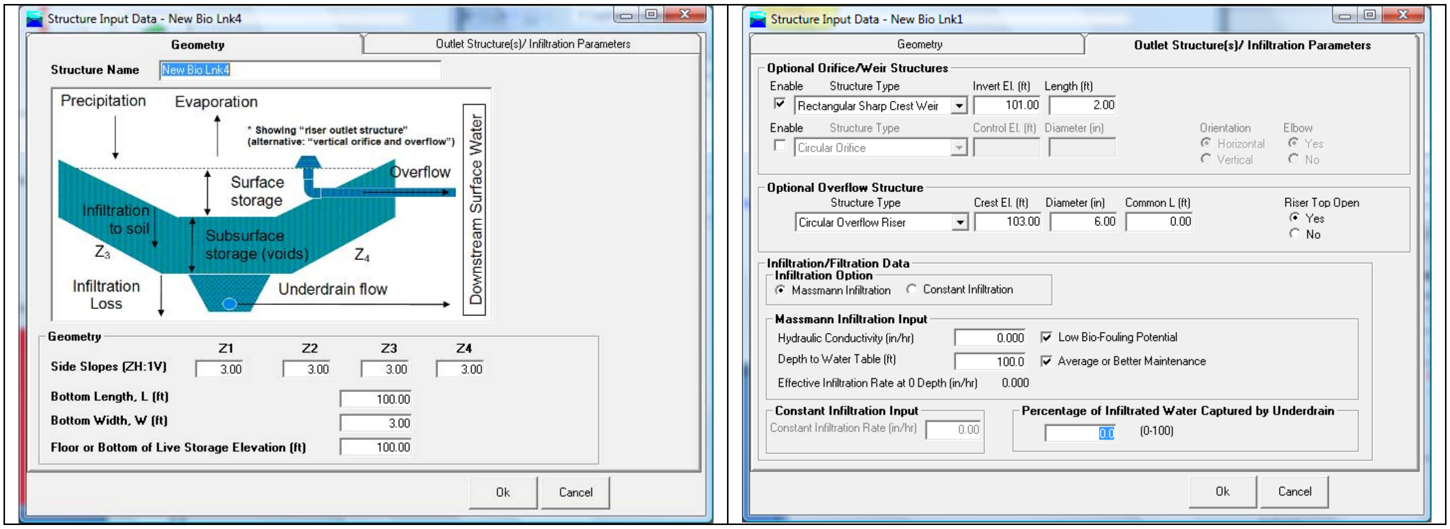
Figure 14.1 – Filter Strip Definition Sketch

15 Bioretention Facility

Bioretention areas are landscaping features adapted to treat stormwater runoff on the development site. These structures are used primarily for both stormwater quality and quantity treatment. For quality treatment, 91-percent of the simulated runoff volume from the site must be filtered or infiltrated by the facility.

MGSFlood simulates surface detention, surface outflow, infiltration, and return flow from an underdrain (Figure 15.1). The underdrain return flow is entered as a percentage of the infiltrated moisture. This percentage is then added to the link outflow. Infiltration can either be simulated using a constant rate or by using Massmann's equations.

Precipitation and evapotranspiration are applied to the facility so the area occupied by the bioretention facility should not be included in the Subbasin Area input.



Structure Input Data - New Bio Lnk4

Structure Name: New Bio Lnk4

Geometry

	Z1	Z2	Z3	Z4
Side Slopes (ZH:1V)	3.00	3.00	3.00	3.00

Bottom Length, L (ft): 100.00

Bottom Width, W (ft): 3.00

Floor or Bottom of Live Storage Elevation (ft): 100.00

Structure Input Data - New Bio Lnk1

Optional Orifice/Weir Structures

Enable ☑ Structure Type: Rectangular Sharp Crest Weir; Invert El. (ft): 101.00; Length (ft): 2.00

Enable ☐ Structure Type: Circular Orifice; Control El. (ft): ; Diameter (in): ; Orientation: ● Horizontal ○ Vertical; Elbow: ● Yes ○ No

Optional Overflow Structure

Structure Type: Circular Overflow Riser; Crest El. (ft): 103.00; Diameter (in): 6.00; Common L (ft): 0.00; Riser Top Open: ● Yes ○ No

Infiltration/Filtration Data

Infiltration Option: ● Massmann Infiltration ○ Constant Infiltration

Massmann Infiltration Input

Hydraulic Conductivity (in/hr): 0.000 ☑ Low Bio-Fouling Potential

Depth to Water Table (ft): 100.0 ☑ Average or Better Maintenance

Effective Infiltration Rate at 0 Depth (in/hr): 0.000

Constant Infiltration Input

Constant Infiltration Rate (in/hr): 0.00

Percentage of Infiltrated Water Captured by Underdrain: 0.0 (0-100)

Figure 15.1 – Bioretention Facility Input Screens

16 Infiltration Computed Using Massmann Approach

MGSFlood includes two options for simulating infiltration; Massmann[30] equations and fixed infiltration. The Massmann equations are based on field observations of infiltration ponds in western Washington. This infiltration approach accounts for the side slope geometry of the pond, pond aspect (length to width ratio), the proximity of the pond to the regional groundwater table, and the potential for soil clogging and fouling. Inputs include; Soil Hydraulic Conductivity (inches/hour), Depth to the Regional Water Table (ft), whether bio-fouling potential is low, and whether average or better maintenance is performed. Infiltrated moisture is lost from the system and does not contribute to the discharge rate downstream of the link.

> Soil Hydraulic Conductivity – Is the saturated hydraulic conductivity of the soil beneath the infiltration trench according to Darcy's Equation. It may be specified as either inches/hour or feet/day depending settings on the default menu under Tools-Options. It can be estimated using regression equations that use grain size distribution as input (Massmann[30]) or from literature (e.g. Freeze and Cherry[31], Fetter[32]).
>
> Depth to Regional Groundwater Table (ft) – Represents the depth from the bottom of the facility to the regional groundwater table or the first low-permeability layer. For shallow groundwater sites, groundwater mounding reduces the hydraulic gradient and the infiltration rate is significantly less than the saturated hydraulic conductivity. For deep groundwater sites where the effects of mounding will be small, the gradient will not typically be reduced by infiltration from the facility. Increasing the depth to groundwater greater than 100 feet ceases to have an influence on pond infiltration according to this approach.
>
> Bio-fouling Potential – Bio-fouling occurs from organic material blanketing the soil surface and reducing the infiltration rate. Bio-fouling is more likely to occur if the trench is located beneath trees and other vegetation or in shaded locations.
>
> Maintenance – Siltation is more likely to occur if there is not sufficient pre-treatment of the storm water or in locations where the drainage basin is prone to erosion because of recent land disturbances or steep slopes. The user should consider the potential for siltation and the level of maintenance when determining the effects of maintenance on pond infiltration performance.

Links that include infiltration according to Massmann's methods report the effective infiltration rate of the link in inches per hour on the input screen. This provides the user with an indication of the amount of infiltration that will be simulated by the program according to the Massmann input parameters.

The following equations by Massmann are used in MGSFlood to simulate infiltration.

Ponds, Channels, Filter Strips

$$f = K\left[\frac{D_{WT} + D_{Pond}}{138.62(K^{0.1})} CF_{Size}\right] CF_{Silt/Bio} CF_{Aspect} \qquad 15.1$$

Where:

f is the infiltration rate in feet per day,
K is the saturated hydraulic conductivity of the soil in feet per day,
D_{WT} is the depth to the regional water table or first low permeability layer (feet),
D_{Pond} is the ponding depth at the ground surface (feet)
CF_{Size} is a correction factor for the size of the facility, computed using equation 15.2
$CF_{silt/bio}$ is the infiltration correction for siltation, biofouling and maintenance (Table 15.1),
CF_{aspect} is the infiltration correction for structure aspect (Equation 15.3),

$$CF_{Size} = 0.73(A_{Pond})^{-0.76} \qquad 15.2$$

Where:

A_{Pond} is the area of the facility in acres.

$$CF_{aspect} = 0.02\, A_{Ratio} + 0.98 \qquad 15.3$$

Where:

A_{Ratio} is the length to width ratio of the facility.

Table 15.1 – Pond Infiltration Rate Reduction Factors to Account for Effects of Biofouling and Siltation (Massmann[30])

Potential for Biofouling	Degree of Long-Term Maintenance and Monitoring	Infiltration Rate Correction Factor ($CF_{silt/bio}$)
Low	Average or Better	0.9
Low	Low	0.6
High	Average or Better	0.5
High	Low	0.2

Infiltration Trenches

$$f = K\left[\frac{D_{WT} + D_{Trench}}{78(K^{0.05})}\right]CF_{Silt/Bio} \tag{15.4}$$

Where:

f is the infiltration rate in feet per day,
K is the saturated hydraulic conductivity of the soil in feet per day,
D_{WT} is the depth to the regional water table or first low permeability layer (feet),
D_{Trench} is the ponding depth at the ground surface (feet)
$CF_{silt/bio}$ is the infiltration correction for siltation, biofouling and maintenance (Table 15.2),

Table 15.2 – Trench Infiltration Rate Reduction Factors to Account for Effects of Biofouling and Siltation (Massmann[30])

Potential for Biofouling	Degree of Long-Term Maintenance and Monitoring	Infiltration Rate Correction Factor ($CF_{silt/bio}$)
Low	Average or Better	0.9
Low	Low	0.8
High	Average or Better	0.75
High	Low	0.6

17 Runoff/Network Routing Computation

17.1 Overview

After inputting land use, connecting subbasins to links, and defining link connections, runoff and routing computations are performed from the *Runoff/Optimize* tab. MGSFlood computes runoff using the impervious (IMPLND) and pervious (PERLND) land segment subroutines from the HSPF model. Precipitation and evaporation are read from the MGSRegion.mdb file, runoff is computed for predevelopment and postdevelopment conditions, and saved to FORTRAN, binary, direct access files. Routing through the predeveloped and postdeveloped networks is then performed with output saved to a separate binary FORTRAN direct access file called for each link. Statistics are then performed automatically and the results are plotted on the *Graphics* tab.

Runoff computations are performed on a *water year* basis, that is, they begin on October 1 and end on September 30. This is done because the soils are typically driest at the beginning of fall and a single set of antecedent conditions can be used for all regions of western Washington upon startup for the first year of the simulation. The user can define a time period shorter than the full record for the runoff computations, although the full period of record should be used in facility design to provide the most accurate design. The same FORTRAN direct access files are overwritten for each project analyzed by the flood model, i.e. the computed runoff timeseries are not saved for each project.

The program will automatically determine the size of pond or infiltration trench selected for optimization on the *Postdeveloped Scenario* Window. Only one structure may be optimized per simulation run. To optimize multiple structures, start with the furthest upstream structure and optimize each structure working downstream.

Statistics may be computed for the compliance locations only or all subbasins and links in the project . Computed statistics are available for graphing and are saved in the project report.

17.2 Governing Equations for Routing

Network routing is performed using a Modified Puls routing routine developed by the US Army Corps of Engineers for the HEC-1[25] flood hydrograph package. A storage indication function (Equation 17.1) is computed from storage and outflow data developed by the program for each structure in the network.

$$STRI(I) = C * \frac{STOR(I)}{\Delta t} + \frac{OUTFL(I)}{2} \qquad (17.1)$$

Where: *STRI* is the storage indication in cfs, *STOR* is the storage for a given outflow in acre-ft, *OUTFL* is the outflow in cfs, *C* is the conversion factor from acre-ft/hour to cfs, Δt is the time step in hours, and *I* is a subscript indicating corresponding values of storage and outflow.

Storage indication at the end of each time interval is given by:

$$STR(2)=STRI(1)+QIN-Q(1) \tag{17.2}$$

Where: QIN is the average inflow in cfs, and Q is the outflow in cfs, and subscripts 1 and 2 indicate beginning and end of the current time step.

The outflow at the end of the time interval is interpolated from a table of storage indication versus outflow. Storage is then computed from:

$$STR = \left(STRI - \frac{Q}{2} \right) * \frac{\Delta t}{C}$$

18 Flood Frequency and Duration Statistics

MGSFlood contains routines for computing flood-frequency and flow duration statistics on streamflow and water surface elevation timeseries computed by the program. The following sections describe the flow duration and flow frequency statistics, and the flow duration pond design criteria as required by the Washington State Department of Ecology[9].

18.1 Flow Duration Statistics

Flow duration statistics provide a convenient tool for characterizing streamflow computed with a continuous hydrologic model. Duration statistics are computed by tracking the fraction of time that a specified flow rate is equaled or exceeded. The program does this by dividing the range of flows simulated into discrete increments and then tracks the fraction of time that each flow is equaled or exceeded. For example, Figure 18.1a shows a one-year flow timeseries computed at hourly time steps from a ten acre forested site and Figure 18.1b shows the flow duration curve computed from this timeseries.

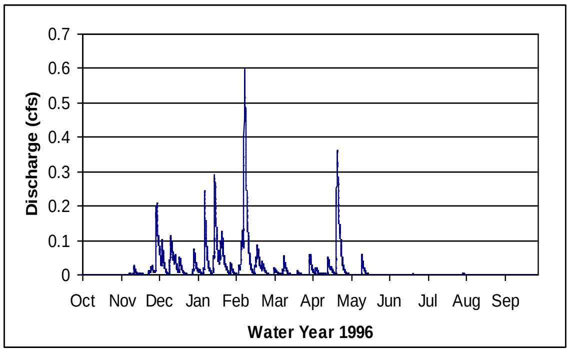


Figure 18.1a – Runoff from 10-Ac Forested Site

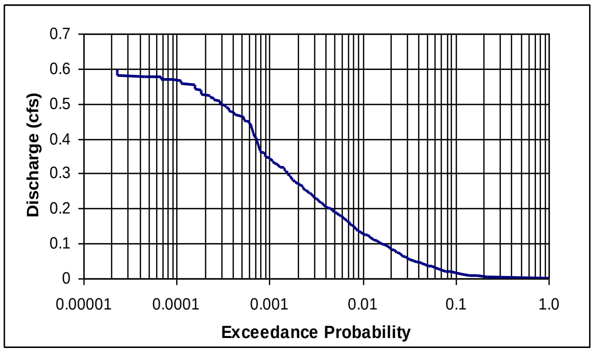


Figure 18.1b – Flow Duration Curve Computed Using Timeseries in Figure at left

The fraction of time that a particular flow is equaled or exceeded is called *exceedance probability*. It should be noted that exceedance probability for duration statistics is different from the *annual exceedance probability* associated with flood frequency statistics and there is no practical way of converting/relating annual exceedance probability statistics to flow duration statistics.

18.2 Flood/Water Surface Elevation Frequency Statistics

Flood-frequency analysis seeks to determine the flood flow or water surface elevation with a probability (p) of being equaled or exceeded in any given year. Return period (Tr) or recurrence interval is often used in lieu of probability to describe the frequency of exceedance of a flood of a given magnitude. Return period and annual exceedance probability are reciprocals (Equation 18.1) and the two are used interchangeably in this section. Flood-frequency analysis is most commonly conducted for flood peak discharge and peak water surface elevation but can also be computed for maximum or minimum values for various durations. Flood-frequency analysis as used here refers to analysis of flood peak discharge or peak water surface elevation.

$$Tr = \frac{1}{p} \qquad 18.1$$

Where:

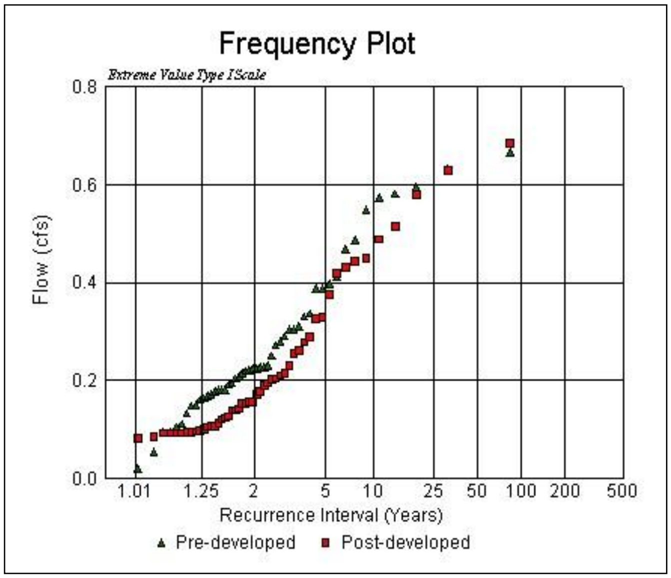
Tr is the average recurrence interval in years, and
p is the annual exceedance probability.

The exceedance probability for compute runoff and water surface elevations is estimated using the Gringorten[12] plotting position formula (Equation 18.2), which is a non-parametric approach. An example probability plot comparing forested land use with the pond outflow is shown in Figure 18.2 and a pond water surface elevation frequency plot is shown in
Figure 18.3.

$$Tr = \frac{N + 0.12}{i - 0.44} \qquad 18.2$$

Where: Tr is the recurrence interval of the peak flow or peak elevation in years,
i is the rank of the annual maxima peak flow, ranked from highest to lowest, and N is the total number of years simulated.

A probability distribution, such as the Generalized Extreme Value or Log-Pearson III[15], is not used for estimating the frequency characteristics because these and other three-parameter distributions typically do a very poor job of fitting annual maxima flows regulated by stormwater ponds and can produce grossly inaccurate estimates of the flow for rare recurrence intervals.



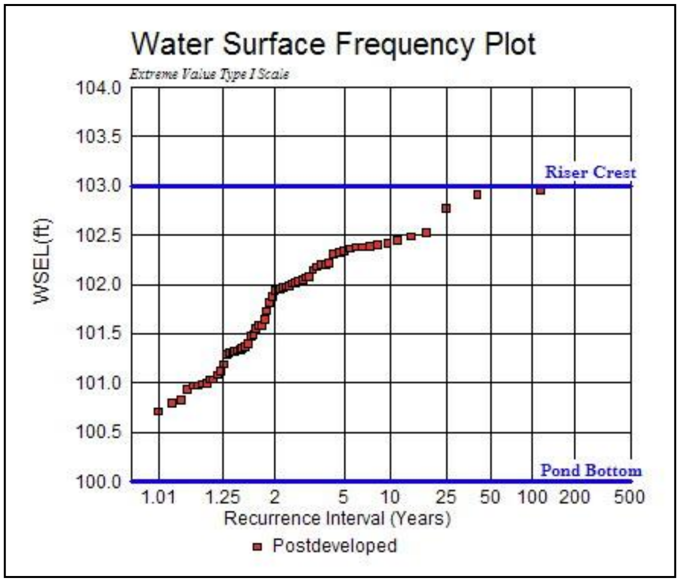


Figure 18.2 – Example Probability Plot Comparing Pond Outflow (Postdeveloped) with Predeveloped

Figure 18.3 – Example Pond Water Surface Elevation Probability Plot

19 Pond Design to Flow Duration Standard

In the past, stormwater pond design criteria have focused on flood control by regulating peak flow rates. Even if the design goal for controlling peak discharge is successful, the aggregate duration that flows occupy the stream channels is greater than under predeveloped conditions because the overall runoff volume is greater under postdevelopment conditions. This increased runoff volume results in increased erosive work being done on the receiving channels, and results in streams that are incised and devoid of the characteristics needed to support fish habitat.

The *flow duration standard* seeks to maintain predevelopment levels of the magnitude and duration of streamflow for those streamflows that exceed the threshold for bedload movement. The threshold for bedload movement is assumed to be 50-percent of the 2-year flow computed for predevelopment conditions[16,2]. The intent of this standard is to prevent increases in the rate of stream channel erosion over that which occurs under predeveloped conditions.

19.1 Flow Duration Standard

The following is the flow duration standard required by the Department of Ecology *Stormwater Management Manual for Western Washington*[9]:

> *Stormwater discharges shall match developed discharge duration to predeveloped durations for the range of predeveloped discharge rates from 50-percent of the 2-year peak flow up to the full 50-year peak flow.*
>
> *The pre-developed condition to be matched shall be a forested land cover unless reasonable, historic information is provided that indicates the site was prairie prior to settlement (modeled as pasture). This standard requirement is waived for sites that will reliably infiltrate all the runoff from impervious surfaces and converted pervious surfaces.*

The flow duration standard can be viewed graphically as shown in Figure 19.1. The flow duration curve for the site under predeveloped conditions (forested land cover in this example) is computed and is the target to which the postdeveloped flow duration curve is compared. The flow duration curve for the pond discharge must match the predeveloped curve between ½ of the predeveloped 2-year (1/2 Q2) and the predeveloped 50-year (Q50). The postdeveloped curve must match the predeveloped within the tolerance levels specified in Table 19.1 and shown graphically in Figure 19.2.

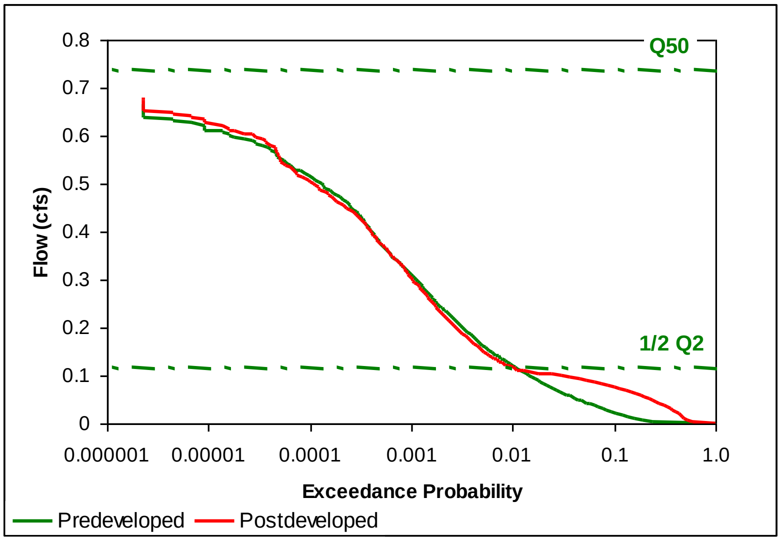


Figure 19.1 – Comparison of Predeveloped and Postdeveloped Flow Duration Curves

Table 19.1 – Tolerance Criteria for Matching Postdevelopment Flow Duration Curves to Predevelopment Levels
1. The exceedance probability of postdeveloped flow duration values must not exceed the predeveloped values between ½ of the 2-year and the 2-year discharge.
2. The exceedance probability of postdevelopment flow duration values must not exceed the predeveloped exceedance probability by more than 10% between the 2-year and 50-year discharge.
3. No more than 50-percent of the postdeveloped flow duration values can be greater than the predeveloped values between ½ Q2 and Q50.

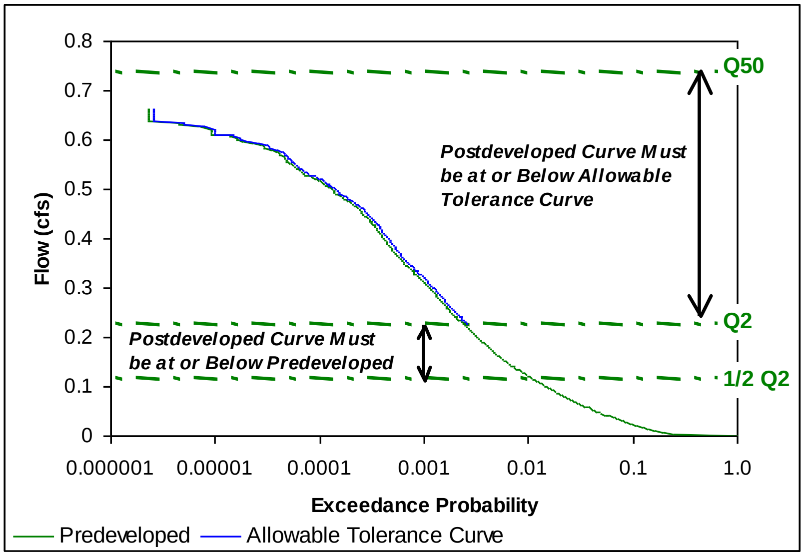


Figure 19.2 – Criteria for Matching Postdevelopment (Pond Outflow) Duration Curve to Predevelopment Flow Duration Curve

In the example shown in Figure 19.3, Tolerance Criterion 1 is met because the postdeveloped flow duration curve is at or below the predeveloped between ½ of the 2-year and the 2-year. Tolerance Criterion 2 is not met, because postdeveloped flow duration curve exceeds the tolerance curve above 0.45 cfs. Tolerance Criterion 3 is met because more than 50-percent of the postdeveloped duration values are at or below the predeveloped curve. Because not all three of the criteria are met, the pond does not meet the flow duration standard and modifications would be needed to the pond size and/or outlet works to meet the standard.

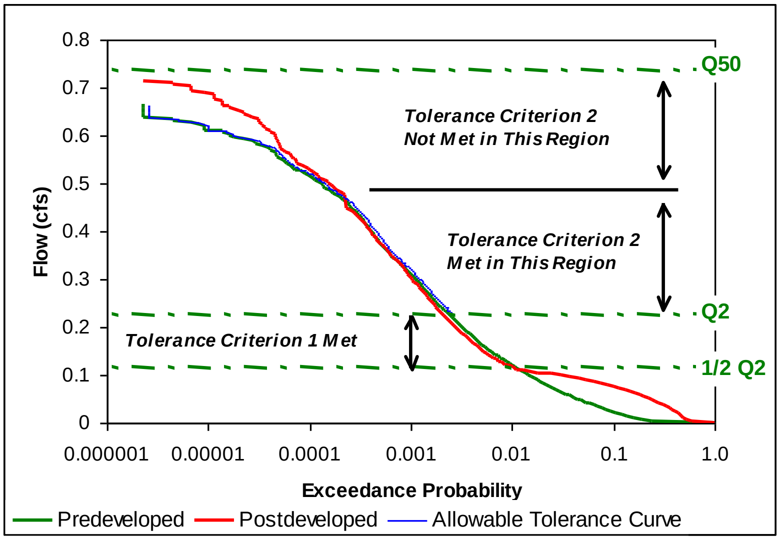


Figure 19.3 – Predevelopment and Postdevelopment (Pond Outflow) Flow Duration Curves and Flow Duration Standard Performance Criteria (Pond Fails Criterion 2, and Does not Meet Flow Duration Standard)

19.2 Pond/Infiltration Trench Design Procedure

The procedure for designing a stormwater pond or infiltration trench to meet the flow duration standard discussed in the previous section is described in the following sections.

Step 1. Define the watershed configurations for the predeveloped and postdeveloped conditions on the predeveloped and postdeveloped scenario input screens accessed from the *Scenario* tab.

Step 2. Enter land use and parameters for each subbasin or structure by right clicking on the icon and selecting *Edit.*

Step 3. On the post development scenario screen, define which link is to be optimized by right clicking the icon and selecting *Use Optimizer to Size this Structure*. The Icon will turn blue and the letters OPT appears indicating the link is set for optimization (Figure 19.4). Only structures and infiltration trenches may be optimized. Right click the icon and select *Edit* to input optimization information for the link to be optimized.

Step 4. Define the predeveloped compliance location. The program will automatically size the pond or infiltration trench such that the flow duration standard is met at the outflow of the optimized link. On the Predeveloped Scenario screen, right click the icon denoting the location of the point of compliance. Click *Select Point of Compliance* to set the predeveloped compliance point.

Step 7. On the *Simulate* tab, click the *Route* button (Figure 19.5). The program will compute runoff, route flows through the network for pre and post developed conditions, then iterate and determine the size of structure to meet the flow duration standard. When the iterations are complete, the program will plot duration statistics for the pond outflow for comparison with the compliance duration curve. Compliance criteria will also be displayed on the graph.

Step 8. If any of the criteria are not met, then the pond configuration must be modified and routing repeated. Subsequent routing to refine the pond design should be performed with the Optimized Structure toggled off on the Post Developed Scenario screen. Guidelines for adjusting the pond size and outlet works are discussed in the following section.

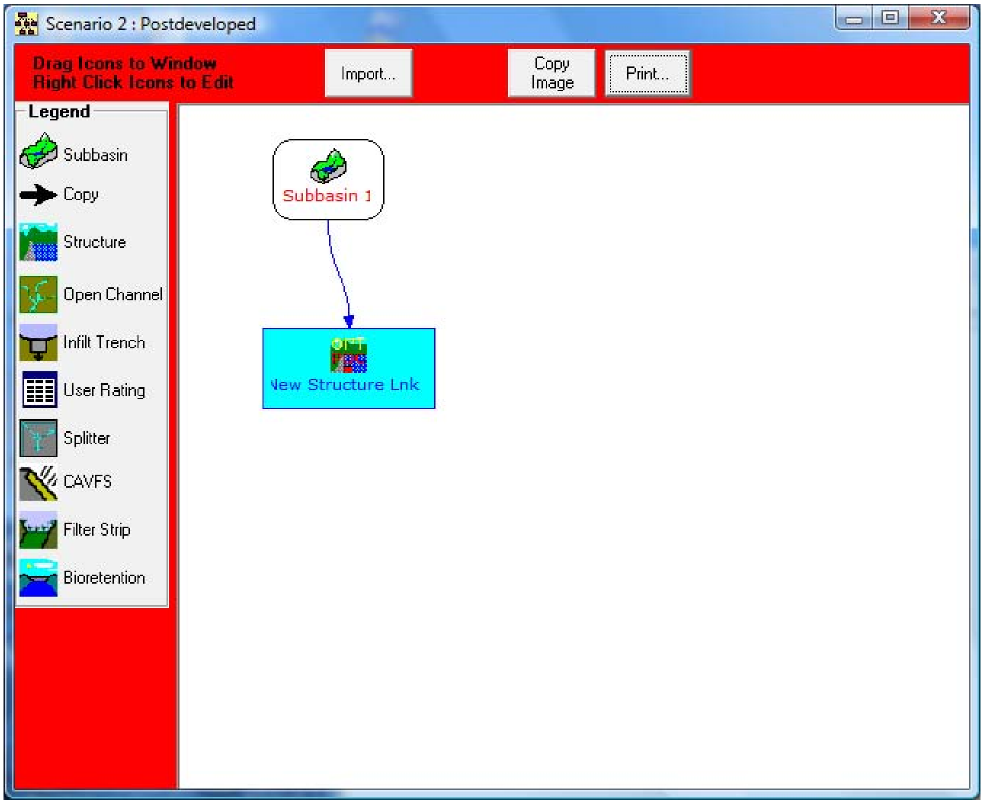
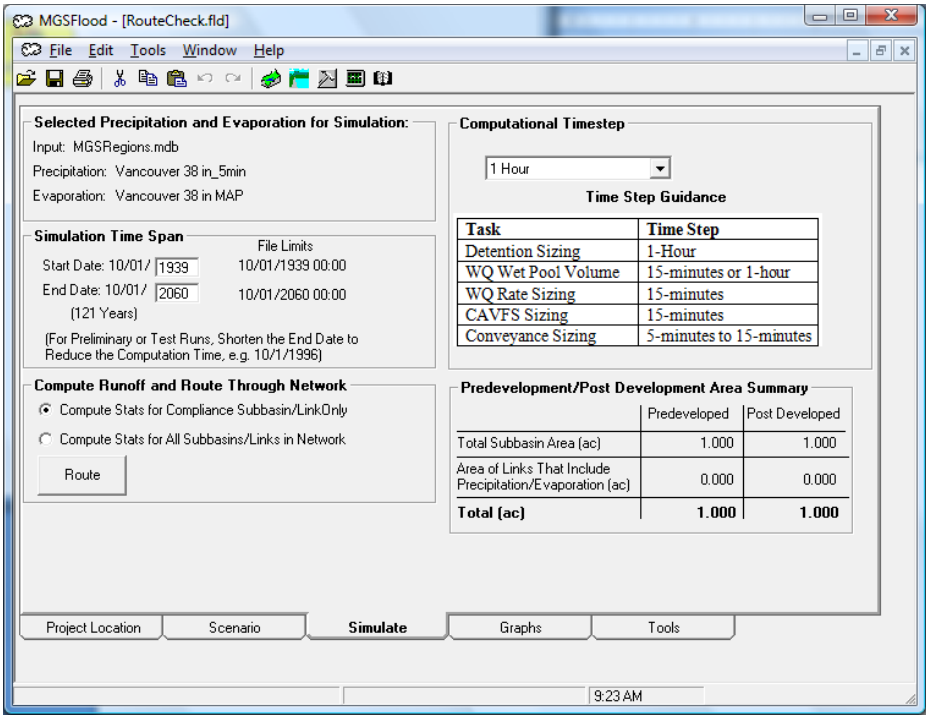


Figure 19.4 – Postdeveloped Scenario Screen with Pond Optimization Set



Selected Precipitation and Evaporation for Simulation:

Input: MGSRegions.mdb

Precipitation: Vancouver 38 in_5min

Evaporation: Vancouver 38 in MAP

Simulation Time Span

		File Limits
Start Date: 10/01/	1939	10/01/1939 00:00
End Date: 10/01/	2060	10/01/2060 00:00
	(121 Years)	

(For Preliminary or Test Runs, Shorten the End Date to Reduce the Computation Time, e.g. 10/1/1996)

Compute Runoff and Route Through Network

● Compute Stats for Compliance Subbasin/LinkOnly

○ Compute Stats for All Subbasins/Links in Network

Route

Computational Timestep

1 Hour

Time Step Guidance

Task	Time Step
Detention Sizing	1-Hour
WQ Wet Pool Volume	15-minutes or 1-hour
WQ Rate Sizing	15-minutes
CAVFS Sizing	15-minutes
Conveyance Sizing	5-minutes to 15-minutes

Predevelopment/Post Development Area Summary

	Predeveloped	Post Developed
Total Subbasin Area (ac)	1.000	1.000
Area of Links That Include Precipitation/Evaporation (ac)	0.000	0.000
Total (ac)	**1.000**	**1.000**

Figure 19.5 – Simulate Tab, Clicking the Route Button will Route all Flows and Optimize the Structure Indicated on the Postdeveloped Scenario Tab

19.3 Guidelines for Adjusting Pond Performance

General guidance for adjusting the geometry and outlet works of stormwater ponds to meet the duration standard were developed by King County[16], are summarized in Figure 19.6, and described below. Refinements should be made in small increments with one refinement at a time.

1. *Bottom Orifice Size* – Adjust the bottom orifice to control the lowest arc of the postdeveloped flow duration curve. Increase the orifice size to raise the arc, decrease it to lower the arc.

2. *Height of Second Orifice* – The invert elevation of the second orifice affects the point on the flow duration curve where the transition (break in slope) occurs from the curve produced by the low-level orifice. Lower the invert elevation of the second orifice to move the transition point to the right on the lower arc. Raise the height of the second orifice to move the transition point to the left on the lower arc.

3. *Second Orifice Size* – Adjust to control the arc of the curve for postdeveloped conditions. Increase the size to raise the arc, decrease it to lower the arc.

4. *Pond Volume* – Adjust the pond volume to control the upper end of the duration curve. Increase the volume to prevent overflow, decrease the volume if the duration curve is substantially below the overflow level.

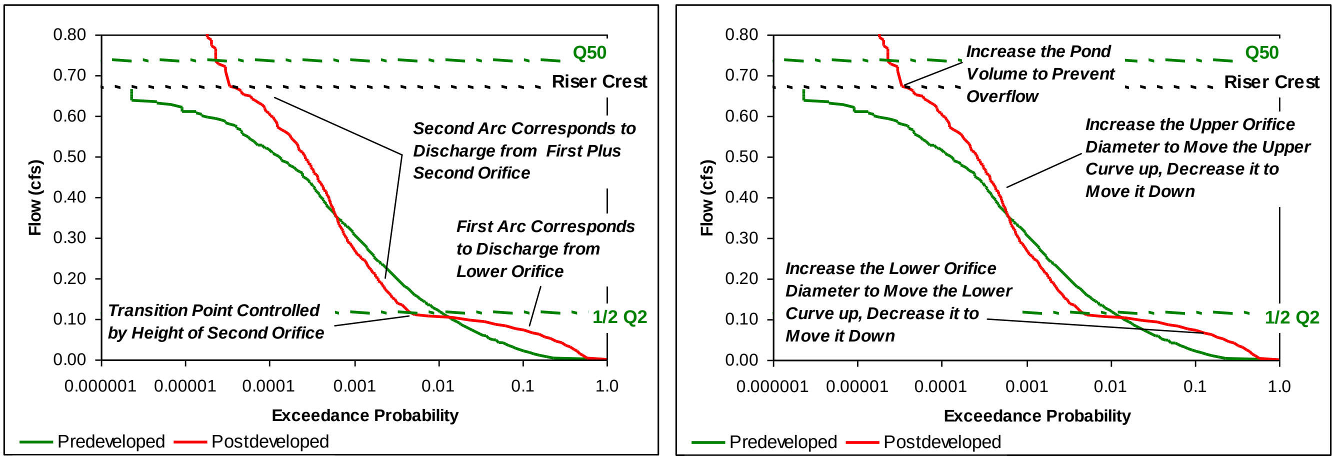


Figure 19.6 – General Guidance for Adjusting Pond Performance

- Analyze the duration curve from bottom to top, and adjust orifices from bottom to top.

- The bottom arc corresponds with the discharge from the bottom orifice. Reducing the bottom orifice discharge lowers and shortens the bottom arc while increasing the bottom orifice raises and lengthens the bottom arc.

- Inflection points in the outflow duration curve occur when additional structures (orifices, notches, overflows) become active.

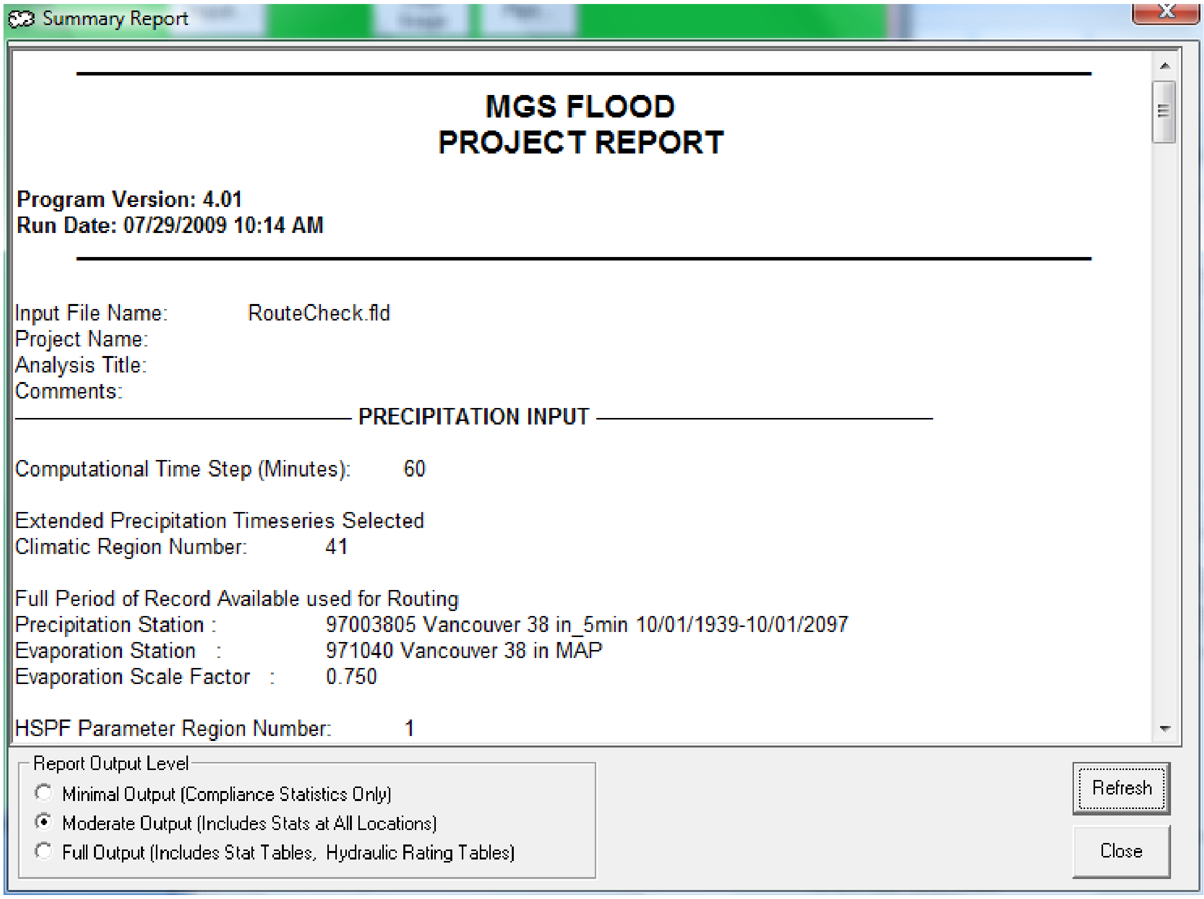
- Lowering the upper orifice moves the transition right on the lower arc and raising the upper orifice moves the breakpoint left of the lower arc

- The upper arc represents the combined discharge of both orifices. Adjustments are made to the second orifice as described above for the bottom orifice.

- Increasing the facility volume moves the entire curve down and to the left. This is done to control riser overflow conditions. Decreasing facility volume moves the entire curve up and to the right. This is done to ensure that the outflow duration curve extends up to riser overflow.

20 Project Documentation/Reporting

The project reporting utility creates a report that documents all model inputs, stormwater pond design information, and frequency and duration statistics. The report is created and viewed on screen by selecting *View Report* from the *File* menu or from the *View Report* icon (📖) on the tool bar. Note that the View Report utility only becomes active after saving the project file for the first time. The report can be printed by selecting *Print Report* from the File menu. When the project report is printed, the user is prompted to print the Predeveloped and Postdeveloped watershed schematics. Each time the report is viewed or printed, a copy of the report is stored in a file with the name <*ProjectName.rtf*> in the project data directory. This file is a Windows Rich Text Format (RTF) and can be edited with Microsoft Word or Word Pad. A partial listing of a project report is shown below.



MGS FLOOD
PROJECT REPORT

Program Version: 4.01
Run Date: 07/29/2009 10:14 AM

```
Input File Name:          RouteCheck.fld
Project Name:
Analysis Title:
Comments:
                        PRECIPITATION INPUT

Computational Time Step (Minutes):      60

Extended Precipitation Timeseries Selected
Climatic Region Number:           41

Full Period of Record Available used for Routing
Precipitation Station :           97003805 Vancouver 38 in_5min 10/01/1939-10/01/2097
Evaporation Station   :           971040 Vancouver 38 in MAP
Evaporation Scale Factor  :       0.750

HSPF Parameter Region Number:           1
```

Figure 20.1 – Project Report Output (Partial Listing)

21 Exporting Runoff Timeseries

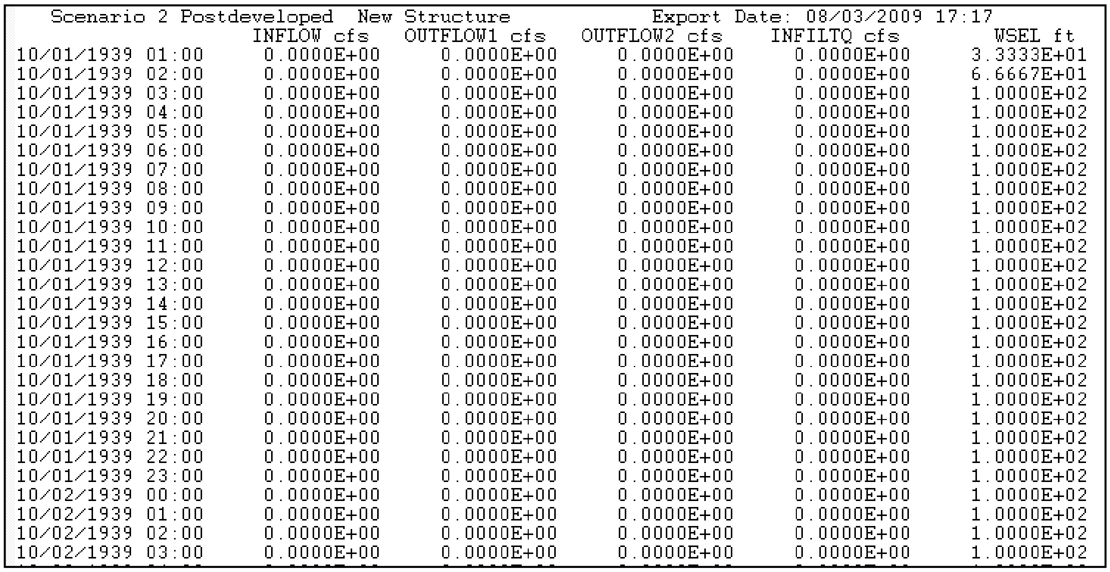
21.1 Exporting Timeseries

Timeseries computed by the program are stored in binary direct access files. These timeseries can be exported to an ASCII formatted file from the *Tools* tab. The output frequency option defines the number of time intervals to be aggregated before output is written to the file. For example, if the *Daily* option button is selected, then the timeseries will be aggregated and saved to the file once per day. For runoff computed on an hourly time-step, 24 values will be aggregated according to the option selected in the *Display* box. If *Maximum* was selected, then the maximum daily flow would be output, *Minimum* would result in the minimum daily flow, and *Average* would result in the average daily flow.

The output file format consists of the end of period date and time followed by the pre and post developed flows at each subbasin (Figure 21.1). Link inflow, outflow, infiltrated moisture and water surface elevation can also be output for each link in the project (Figure 21.2).

```
      Scenario 1 Predeveloped  Subbasin 1     Export Date: 08/03/2009 17:15
                   Runoff cfs
10/01/1939 01:00     0.0000E+00
10/01/1939 02:00     0.0000E+00
10/01/1939 03:00     0.0000E+00
10/01/1939 04:00     0.0000E+00
10/01/1939 05:00     0.0000E+00
10/01/1939 06:00     0.0000E+00
10/01/1939 07:00     0.0000E+00
10/01/1939 08:00     0.0000E+00
10/01/1939 09:00     0.0000E+00
```

Figure 21.1 – Example Output Produced by Export Utility (Subbasin Output)



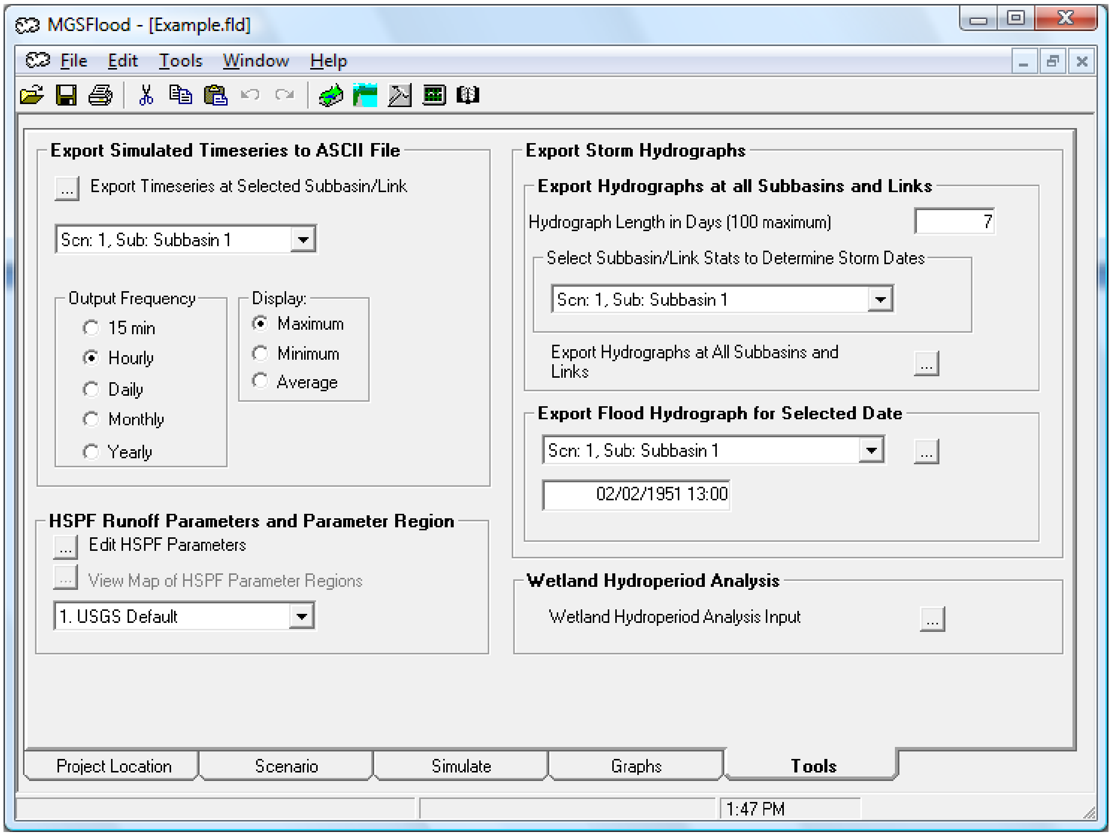
```
   Scenario 2 Postdeveloped  New Structure          Export Date: 08/03/2009 17:17
                   INFLOW cfs   OUTFLOW1 cfs   OUTFLOW2 cfs    INFILTQ cfs        WSEL ft
10/01/1939 01:00    0.0000E+00     0.0000E+00     0.0000E+00     0.0000E+00     3.3333E+01
10/01/1939 02:00    0.0000E+00     0.0000E+00     0.0000E+00     0.0000E+00     6.6667E+01
10/01/1939 03:00    0.0000E+00     0.0000E+00     0.0000E+00     0.0000E+00     1.0000E+02
10/01/1939 04:00    0.0000E+00     0.0000E+00     0.0000E+00     0.0000E+00     1.0000E+02
10/01/1939 05:00    0.0000E+00     0.0000E+00     0.0000E+00     0.0000E+00     1.0000E+02
10/01/1939 06:00    0.0000E+00     0.0000E+00     0.0000E+00     0.0000E+00     1.0000E+02
10/01/1939 07:00    0.0000E+00     0.0000E+00     0.0000E+00     0.0000E+00     1.0000E+02
10/01/1939 08:00    0.0000E+00     0.0000E+00     0.0000E+00     0.0000E+00     1.0000E+02
10/01/1939 09:00    0.0000E+00     0.0000E+00     0.0000E+00     0.0000E+00     1.0000E+02
10/01/1939 10:00    0.0000E+00     0.0000E+00     0.0000E+00     0.0000E+00     1.0000E+02
10/01/1939 11:00    0.0000E+00     0.0000E+00     0.0000E+00     0.0000E+00     1.0000E+02
10/01/1939 12:00    0.0000E+00     0.0000E+00     0.0000E+00     0.0000E+00     1.0000E+02
10/01/1939 13:00    0.0000E+00     0.0000E+00     0.0000E+00     0.0000E+00     1.0000E+02
10/01/1939 14:00    0.0000E+00     0.0000E+00     0.0000E+00     0.0000E+00     1.0000E+02
10/01/1939 15:00    0.0000E+00     0.0000E+00     0.0000E+00     0.0000E+00     1.0000E+02
10/01/1939 16:00    0.0000E+00     0.0000E+00     0.0000E+00     0.0000E+00     1.0000E+02
10/01/1939 17:00    0.0000E+00     0.0000E+00     0.0000E+00     0.0000E+00     1.0000E+02
10/01/1939 18:00    0.0000E+00     0.0000E+00     0.0000E+00     0.0000E+00     1.0000E+02
10/01/1939 19:00    0.0000E+00     0.0000E+00     0.0000E+00     0.0000E+00     1.0000E+02
10/01/1939 20:00    0.0000E+00     0.0000E+00     0.0000E+00     0.0000E+00     1.0000E+02
10/01/1939 21:00    0.0000E+00     0.0000E+00     0.0000E+00     0.0000E+00     1.0000E+02
10/01/1939 22:00    0.0000E+00     0.0000E+00     0.0000E+00     0.0000E+00     1.0000E+02
10/01/1939 23:00    0.0000E+00     0.0000E+00     0.0000E+00     0.0000E+00     1.0000E+02
10/02/1939 00:00    0.0000E+00     0.0000E+00     0.0000E+00     0.0000E+00     1.0000E+02
10/02/1939 01:00    0.0000E+00     0.0000E+00     0.0000E+00     0.0000E+00     1.0000E+02
10/02/1939 02:00    0.0000E+00     0.0000E+00     0.0000E+00     0.0000E+00     1.0000E+02
10/02/1939 03:00    0.0000E+00     0.0000E+00     0.0000E+00     0.0000E+00     1.0000E+02
```

Figure 21.2 – Example Output Produced by Export Utility (Link Output)

21.2 Exporting Storm Hydrographs

The *Export Storm Hydrographs* feature is used to extract hydrographs from the time series file(s) with peak flow corresponding to a user specified recurrence interval. Time series that have had peak flow frequency statistics computed on the tab are available for export (Figure 21.3). The length of the hydrograph is specified in the *Hydrograph Length* box, which can range from 1 to 100 days. Flow recurrence intervals of 2-years, 10-years, 25-years, 50-years, and 100-years are exported. The program uses the time series specified in the *Subbasin/Link Stats* box to determine the dates of storms with recurrence intervals closest to the recurrence interval of the exported storms (2-year, 10-years, 25-years, 50-years, and 100-years). The same dates are used for all time series in the model. The files are saved to a data file with the format: <sublinkname>_xx.dat

Where: <sublinkname> is the name of the subbasin or link time series,
xx is the recurrence interval of the storm exported.



Scn: 1, Sub: Subbasin 1

Output Frequency:
○ 15 min
● Hourly
○ Daily
○ Monthly
○ Yearly

Display:
● Maximum
○ Minimum
○ Average

1. USGS Default

Hydrograph Length in Days (100 maximum) 7

Select Subbasin/Link Stats to Determine Storm Dates: Scn: 1, Sub: Subbasin 1

Scn: 1, Sub: Subbasin 1

02/02/1951 13:00

1:47 PM

Figure 21.3 – Hydrograph Export Feature on Tools Tab

22 Water Quality Treatment Design Data

MGSFlood determines water quality treatment design parameters from the computed runoff timeseries according to methods defined in the 2005 Department of Ecology Stormwater Management Manual for Western Washington[9]. The user should refer to the Ecology Stormwater Manual for specific information regarding water quality treatment requirements and design methods.

Three types of water quality treatment parameters are computed by MGSFlood;

- Water Quality Design Volume, used for sizing wet ponds,
- Infiltration and filtration statistics,
- Water Quality Design Flow Rate, used for sizing flow rate dependent facilities such as biofiltration swales and filter strips.

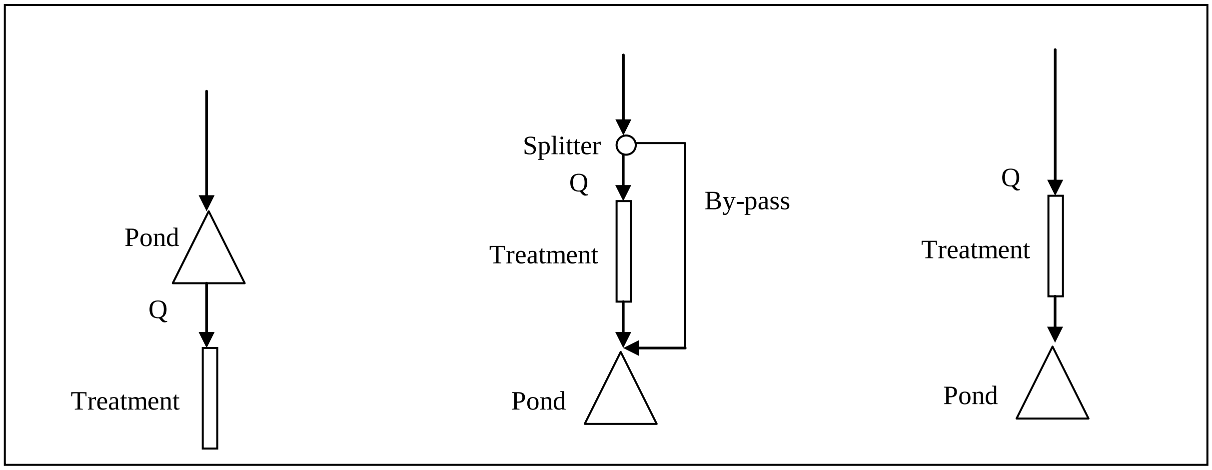
22.1 Water Quality Design Volume

The water quality design volume for sizing wet ponds is computed as the 91% non-exceedance 24-hour runoff volume. The program develops a daily runoff timeseries from the link inflow timeseries and scans the computed daily timeseries to determine the 24-hour volume that is greater than or equal to 91% of all daily values in the timeseries. According to the Ecology Stormwater Management Manual, this value is then used as the volume for a “Basic Wet Pond” and 1.5 times this value is used for sizing a “Large Wet Pond." These values are computed automatically at the time runoff is computed for the detention facility inflow and are listed on the Water Quality Data tab and in the project summary report.

22.2 Water Quality Design Discharge

The flow rate used to design flow rate dependent treatment facilities depends on whether or not the treatment is located upstream of a stormwater detention facility and whether it is an *on-line* or *off-line* facility (Figure 22.1).

Figure 22.1



Downstream of Detention Facility | **Upstream of Detention Facility, Off-Line** | **Upstream of Detention Facility, On-Line**

<ins>*Downstream of Detention Facilities*</ins> – If the treatment facility is located downstream of a stormwater detention facility, then the full 2-year release rate from the stormwater pond should be used to design the stormwater treatment facility.

<ins>*Upstream of Detention Facilities, Off-Line*</ins> – *Off-line* water quality treatment located upstream of the detention facility includes a high-flow by-pass that routes the incremental flow in excess of the water quality design rate around the treatment facility. It is assumed that flows from the bypass enter the system downstream of the treatment facility but upstream of the detention facility. If an hourly time step is used, the program determines the hourly water quality treatment design flow rate as the rate corresponding to the runoff volume that is greater than or equal to 91% of the hourly runoff volumes (Figure 22.2). The 15-minute water quality treatment design flow rate is then computed from an adjustment factor provide by Ecology for estimation of maximum 15-minute flow rates based on hourly timeseries. If a 15-minute time step is used, then the same procedure is used, except that the adjustment factors are not applied.

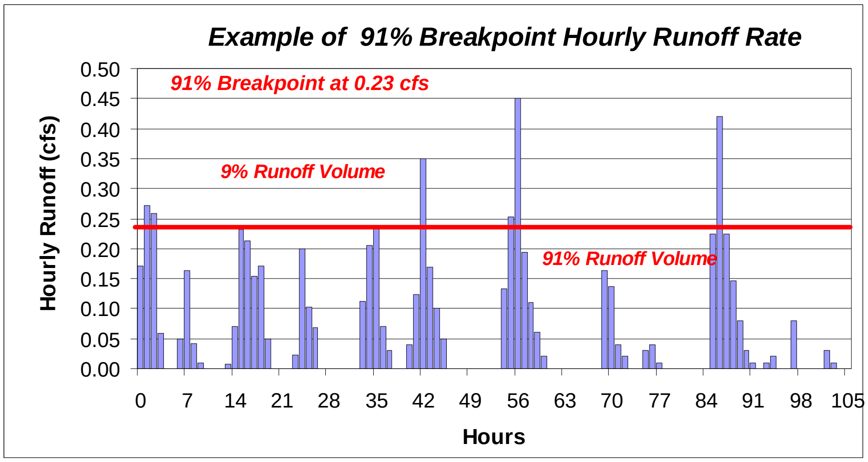


Figure 22.2 – Example showing calculation of Off-Line Water Quality Treatment Discharge Off-line Hourly Discharge of 0.23 cfs (in this case) is Automatically Adjusted by the Program to Obtain 15-minute Discharge Rate Used for Design (If a 15-minute time step is used, then no adjustment is applied)

Upstream of Detention Facilities, On-Line – *On-line* water quality treatment does not include a high-flow bypass for flows in excess of the water quality design flow rate and all runoff is routed through the facility. The program determines the water quality treatment design flow rate as the rate corresponding to the runoff volume that is greater than or equal to 91% of the runoff volume entering the treatment facility, however, those flows that exceed the water quality design flow are not included in the calculation (Figure 22.3). Thus, the design flow rate for on-line facilities is higher than for off-line facilities. As discussed above, if a 1-hour time step is used in the runoff computation, then the 15-minute water quality treatment design flow rate is determined by applying an adjustment factor provide by Ecology. If a 15-minute time step is used, then no adjustment factor is applied.

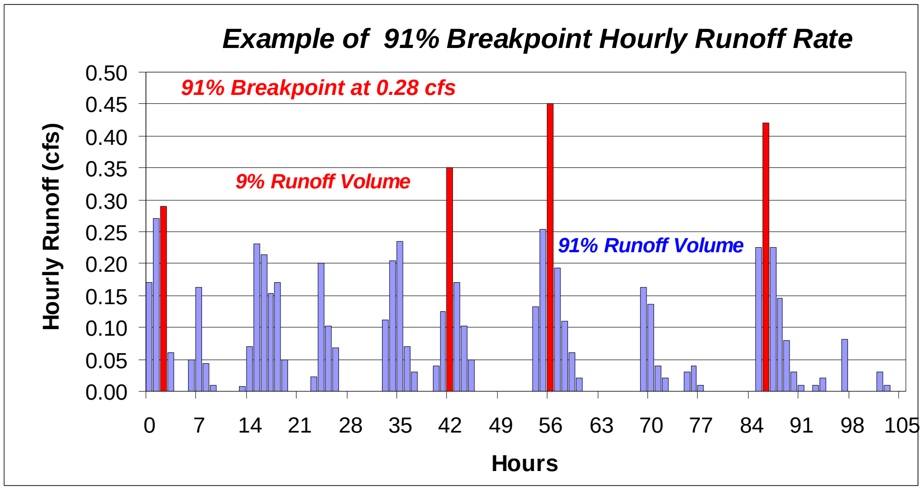


Figure 22.3 – Example showing calculation of Off-Line Water Quality Treatment Discharge Off-line Hourly Discharge of 0.28 cfs (in this case) is Automatically Adjusted by the Program to Obtain 15-minute Discharge Rate Used for Design (If a 15-minute time step is used, then no adjustment is applied)

22.3 Filtration/Infiltration Statistics

Water quality treatment statistics are computed for facilities that infiltrate or filter water through media. The total volume infiltrated and/or filtered is compared with the total volume entering the facility. For quality treatment, 91-percent of the simulated runoff volume from the site must be filtered or infiltrated by the facility. These values are reported on the Water Quality Calculation Window and the project report.

22.4 Water Quality Flow Splitter Design

When an *off-line* treatment approach is used, a flow-splitter is needed for bypassing flows that exceed the design flow rate. MGSFlood computes the geometry of the splitter structure according to guidelines listed in the Ecology Stormwater Management Manual. The splitter structure includes an orifice and an overflow weir (Figure 22.4). The design guidelines are listed below.

- The maximum head on the overflow weir must be minimized for flow in excess of the water quality design flow. Specifically, flow to the water quality facility at the 100-year water surface must not increase the design water quality flow by more than 10-percent.

- The splitter structure requires an orifice plate upstream of the discharge pipe that leads to the water quality treatment facility. The design water surface should be set to provide a minimum headwater/diameter ratio of 2.0.

The splitter design is a trial and error procedure whereby the orifice diameter is selected by the user. The program then computes the height of the baffle wall, the length of the overflow weir, and the ratio of the baffle wall height to orifice diameter. There is not a unique solution and the user should select an orifice size that produces a baffle wall height and overflow length that will conveniently fit in a standard manhole (or other structure) and meets the required headwater/diameter ratio of 2.0.

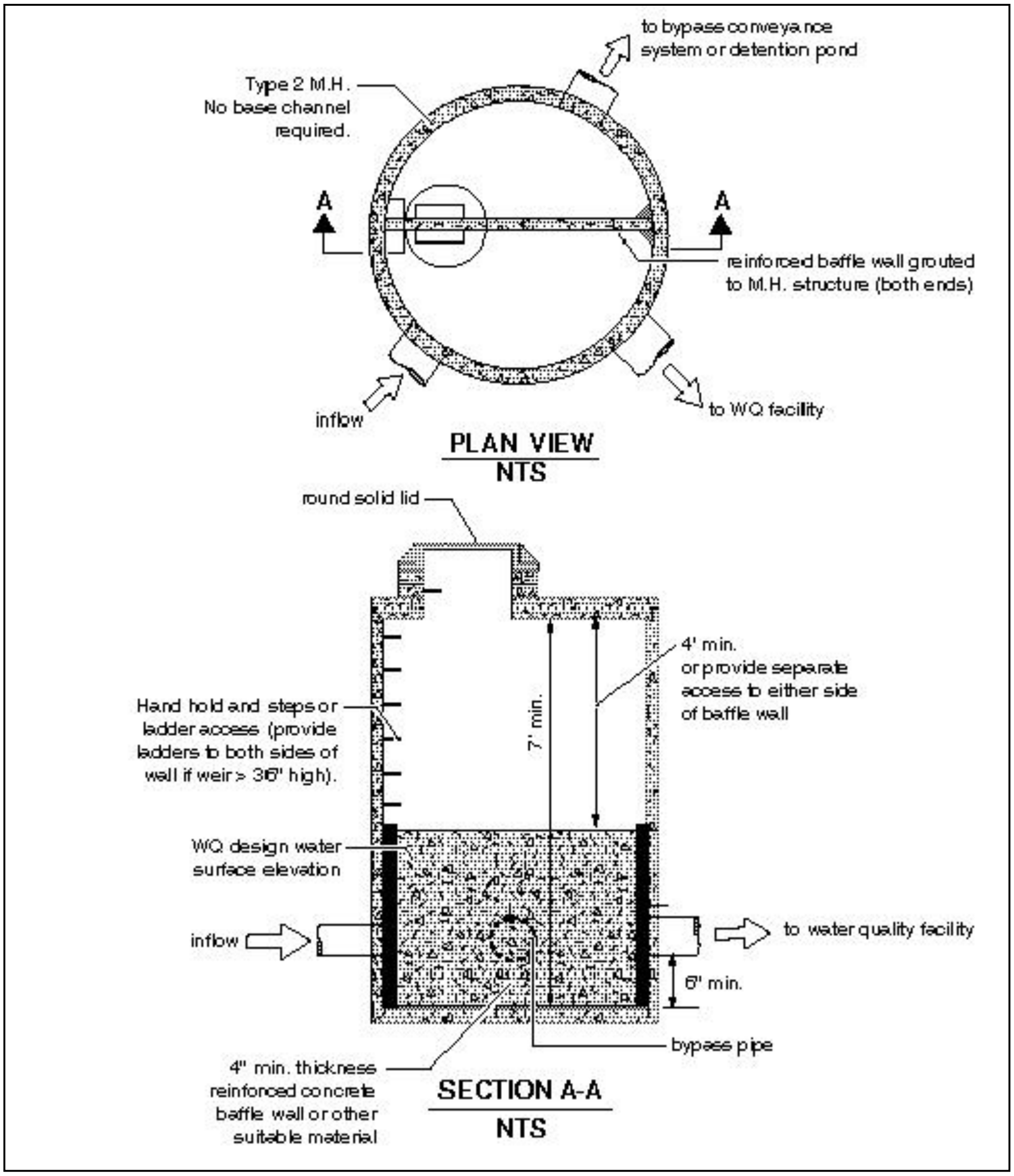


Figure 22.4 – Flow Splitter Geometry (per Ecology Stormwater Management Manual)

23 Wetland Water Level Analysis

23.1 Introduction

Protection of wetland plant and animal communities depends on controlling the wetland’s *hydroperiod*, meaning the pattern of fluctuation of water depth and the frequency and duration of exceeding certain levels, including the length and onset of drying in the summer.

MGSFlood computes hydroperiod statistics according to the guidance developed by the Puget Sound Wetlands and Stormwater Management Research Program[33]. The wetland water level fluctuation guidelines (Guide Sheet 2: Wetland Protection Guidelines) were adopted by Ecology and are listed in Appendix D of the Volume I of Ecology’s Stormwater Management Manual[9]. The following sections summarize the water level fluctuation statistics computed by MGSFlood.

23.2 Water Level Fluctuation (WLF)

Methods for computing Water Level Fluctuation (WLF) were not defined for continuous flow hydrologic models by the Puget Sound Stormwater Management Research Program. Instead, WLF was defined in terms of data collected using a crest stage gage. A crest stage gage consists of a staff gage for observing the instantaneous water surface elevation in the wetland. The gage also indicates the maximum water surface that occurs between observations.

WLF was defined by the Puget Sound Stormwater Management Research Program in terms of crest stage observations made no more than one month apart as follows:

$$\text{WLF} = \text{Crest stage} - \text{Average base stage} \qquad 23.1$$

Where: Crest stage= Maximum stage during interval
Average base stage = $(\text{Stage}_1 + \text{Stage}_2)/2$
S_1= Instantaneous stage at beginning of interval
S_2= Instantaneous stage at end of interval

This definition was adapted for use with the continuous flow model by using wetland water surface elevation information simulated by the model. Each month was divided into four periods with WLF computed according to Equation 23.1 for the entire simulation period. Average monthly and average annual WLF statistics are then computed and printed in the project report.

23.3 Stage Excursions

Stage excursions are defined as the difference between the predeveloped and postdeveloped water surface elevation above a specified threshold. The default threshold is 15 cm (0.5 feet). Thus, each time that the absolute value of the difference between simulated predeveloped and postdevelopment water surface elevation exceeds the threshold, then an excursion begins. When the difference drops below the threshold, then the excursion ends. Figure 23.1 shows a portion of the simulated predeveloped and postdeveloped wetland water surface elevation timeseries for an example wetland. Three excursions are indicated. Each excursion denotes a period here the difference between the predeveloped and postdeveloped timeseries exceeds the 0.5 foot threshold.

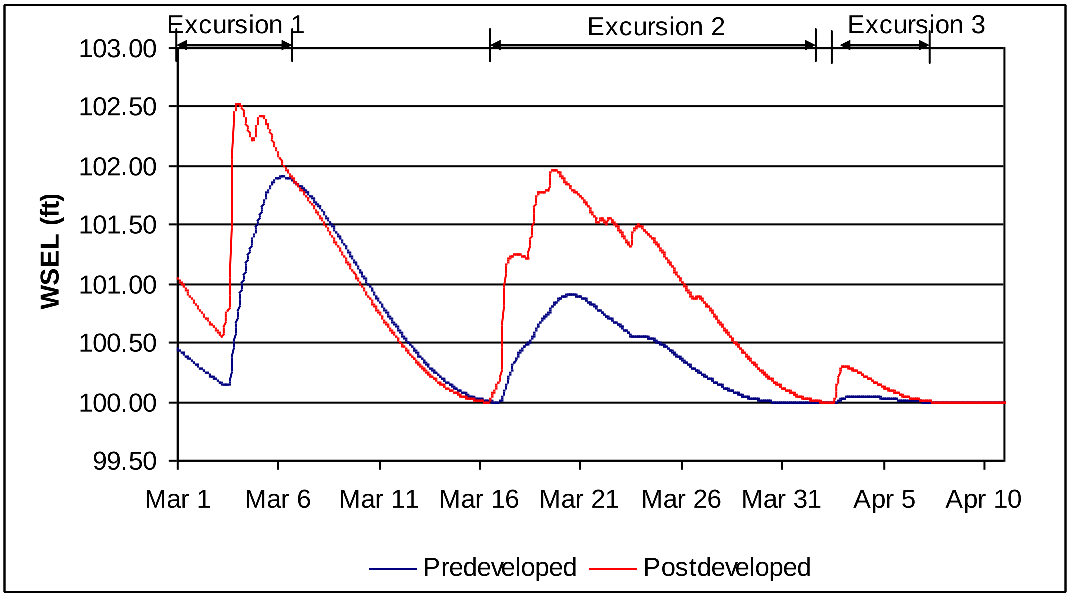


Figure 23.1 – Example Predeveloped and Postdeveloped Wetland Water Surface Elevation with Stage Excursions Noted (Stage Excursion Threshold = 0.5 feet)

The program computes stage excursions for the entire simulation period and outputs several excursion statistics in the project report. These include:

- Number of stage excursions per year,
- The total duration of excursions per year,
- The average duration of each excursion per year,
- The maximum excursion for each year,
- The duration of the longest excursion during the year.

23.4 Dry Period Analysis

The program tracks the number of hours per year that the water surface elevation drops below a user specified value. The default value for “dry” conditions is a depth less than 0.01 feet. The statistic is computed for both predeveloped and postdeveloped conditions and reported for each year simulated.

23.5 Amphibian Breeding Period Analysis

The program computes hydroperiod limits for a user specified amphibian breeding period (default February 1st through May 31). The program reports the duration of stage excursions above or below the predevelopment level in continuous 30-day periods during the breeding months. The default stage excursion threshold is 8 cm (0.25 feet). These statistics allow for the evaluation amphibian criteria which states that the magnitude of stage excursions above or below the predevelopment stage should not exceed 8 cm for more than 24 hours in any 30-day period.

Example wetland hydroperiod statistics computed by the program are shown in Figure 23.2.

Figure 23.2 – Example Wetland Hydroperiod Analysis Output

```
Predeveloped Wetland Location: Link 1: Predeveloped Test Wetland
Postdeveloped Wetland Location: Link 1: Post Developed Condition Wetland
                ***********Mean Water Level Fluctuation Results (ft) *************
    Month           Predeveloped              Postdeveloped
    Oct             0.0033                    0.2971
    Nov             0.0162                    0.4894
    Dec             0.0943                    0.4932
    Jan             0.1663                    0.5036
    Feb             0.1294                    0.4206
    Mar             0.0797                    0.3413
    Apr             0.0253                    0.2208
    May             0.0047                    0.1517
    Jun             0.0009                    0.1458
    Jul             0.0001                    0.0698
    Aug             0.0000                    0.0802
    Sep             0.0001                    0.1808
    Ann             0.0433                    0.2829
```

```
***********Stage Excursion Results  *************

Stage Excursions Threshold (ft): 0.500
Avg Number of Stage Excursions Per Year: 13.824
        WY                No. Excursions          Max (ft)        Max Dur (hrs)      Avg Duration (hrs)
        1940                  16                   2.4519              430.0              111.1
        1941                  15                   1.7901              240.0               70.8
        1942                  12                   2.1804              310.0               89.6
        1943                  10                   2.4195              661.0              162.9
        1944                  11                   1.5237              179.0               50.1
        1945                  18                   2.5822              257.0               66.0
        1946                  15                   1.6662              391.0               95.5
      ...
```

```
***********No Water (Dry) Excursion Results  *************

Wetland Dry when Stage Drops Below (ft): 0.010
        Dry Excursion Duration (hrs)
        WY                Predeveloped           Postdeveloped
        1940            .0                         4568.0
        1941            .0                         4096.0
        1942            .0                         4727.0
        1943            .0                         5244.0
        1944            .0                         5692.0
        1945            .0                         4792.0
        1946            .0                         4583.0
      ...
```

```
***********Amphibian Season Analysis*************

Season Begins : 02/01
Season Ends   : 05/31
Amphibian Stage Excursions Threshold (ft): 0.250
       WY              Max Excursion (ft)         Max 30-Day Excursion (hrs)
        1940                 1.173                        630.0
        1941                 0.935                        163.0
        1942                 0.826                        289.0
        1943                 1.938                        444.0
        1944                 0.713                        158.0
        1945                 2.582                        454.0
        1946                 1.666                        480.0
      ...
```

24 References

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PART II – PROGRAM OPERATION AND DATA INPUT

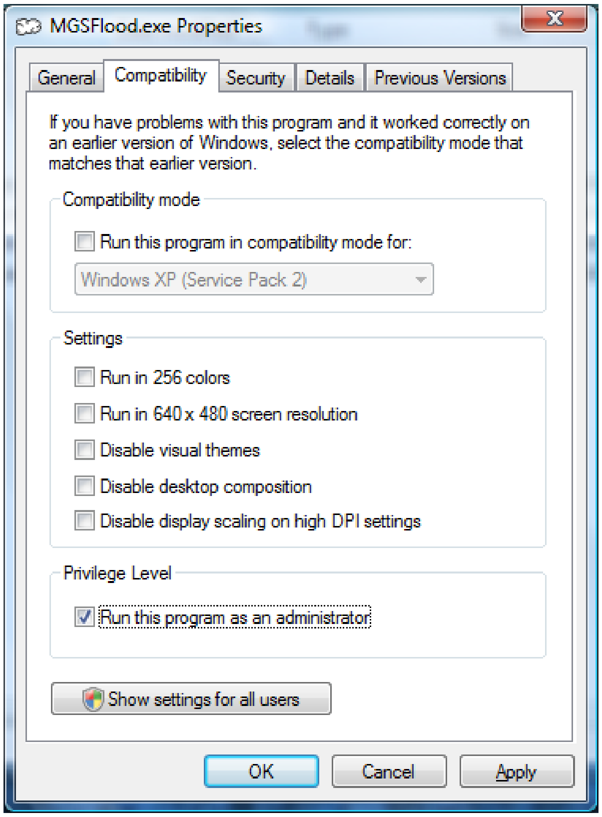
1 Purpose

MGS Flood is a general, continuous, rainfall runoff computer model developed for stormwater facility design in western Washington. Specifically, the program is intended to size stormwater detention ponds to meet the requirements of the 2005 Washington State Department of Ecology Stormwater Management Manual for Western Washington[9]. The program uses the Hydrological Simulation Program-Fortran (HSPF)[26] routine for computing runoff from rainfall.

2 Computer Requirements

- Windows XP/Vista with 1 GB uncompressed hard drive space.
- The program is designed to be installed and operated from a single computer and not run from a network.

For Vista operating systems running with User Account Control On, the program executable may need to be set to run as administrator. Right click on the MGSFlood.exe file in the Program Files\MGSFloodV4 folder and check Run this program as an administrator.



If you have problems with this program and it worked correctly on an earlier version of Windows, select the compatibility mode that matches that earlier version.

Compatibility mode

☐ Run this program in compatibility mode for:

Windows XP (Service Pack 2)

Settings

☐ Run in 256 colors

☐ Run in 640 x 480 screen resolution

☐ Disable visual themes

☐ Disable desktop composition

☐ Disable display scaling on high DPI settings

Privilege Level

☑ Run this program as an administrator

3 Stormwater Analysis Overview

The MGSFlood input screen (Figure 3.1) is organized as a series of tabs that follows the sequence of steps to analyze stormwater runoff. These steps include:

- Entering the project information and determining the precipitation and runoff parameters,
- Entering the subbasin land use, defining Links (ponds, trenches, etc),
- Routing Flows,
- Plotting detention performance graphs,
- Computing water quality treatment parameters.

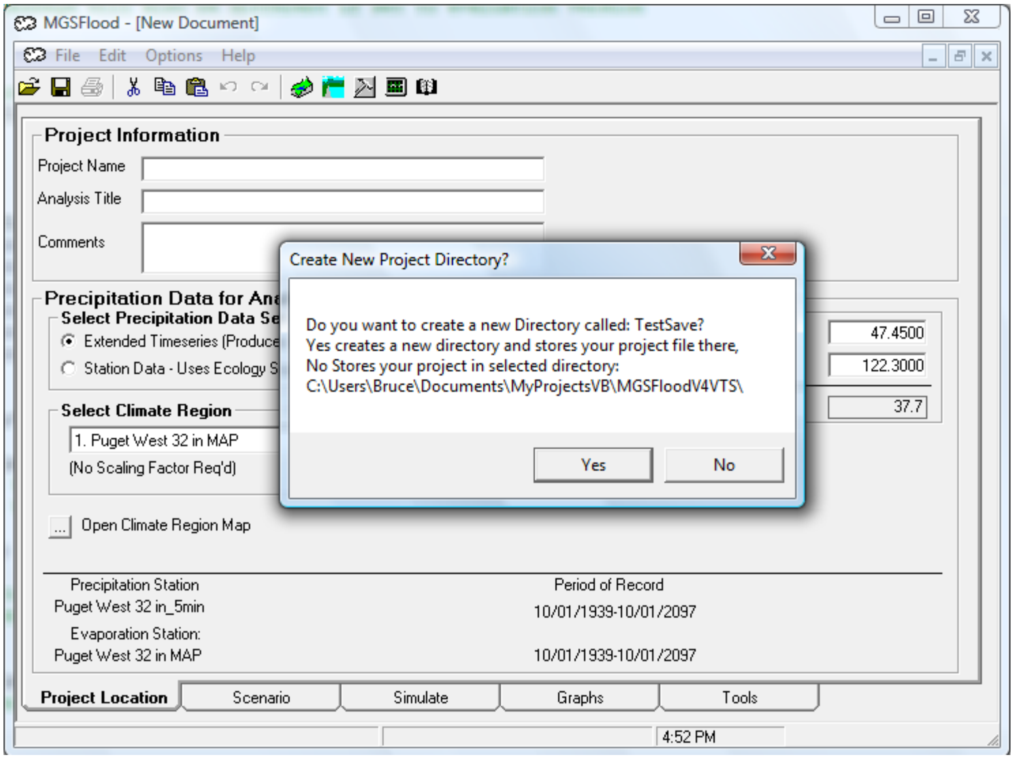
Figure 3.1 – MGSFlood Tabs at Bottom of Input Screen

4 Starting Program, Saving Data

MGSFlood is installed to a default folder in the \Program Files directory. A shortcut created under the Start menu in the *Programs-MGSSoftware* folder can be used to start the program. Graphics Server is a graphics package used by MGSFlood to plot statistics and hydrographs and is installed with MGSFlood. When MGSFlood terminates, Graphics Server is unloaded from memory.

MGSFlood creates a number of files on disk for each project so it is recommended that a separate folder be created for each project. This can be accomplished automatically when saving the project for the first time. The program will prompt for the creation of a new folder with the project name (Figure 4.1). Responding yes to this prompt will create a new folder with the project file stored in it. All subsequent files created by the program for the project will be stored in this directory.

The default directory for saving data files can be set from the Options menu at the top of the main screen. The default directory can be any directory mapped to the computer, including network drives.



Do you want to create a new Directory called: TestSave?
Yes creates a new directory and stores your project file there,
No Stores your project in selected directory:
C:\Users\Bruce\Documents\MyProjectsVB\MGSFloodV4VTS\

● Extended Timeseries (Produce
○ Station Data - Uses Ecology S

1. Puget West 32 in MAP
(No Scaling Factor Req'd)

47.4500
122.3000
37.7

Precipitation Station	Period of Record
Puget West 32 in_5min	10/01/1939-10/01/2097
Evaporation Station:	
Puget West 32 in MAP	10/01/1939-10/01/2097

Figure 4.1 – Prompt to Create a new Project Folder when Saving a Project for the First Time

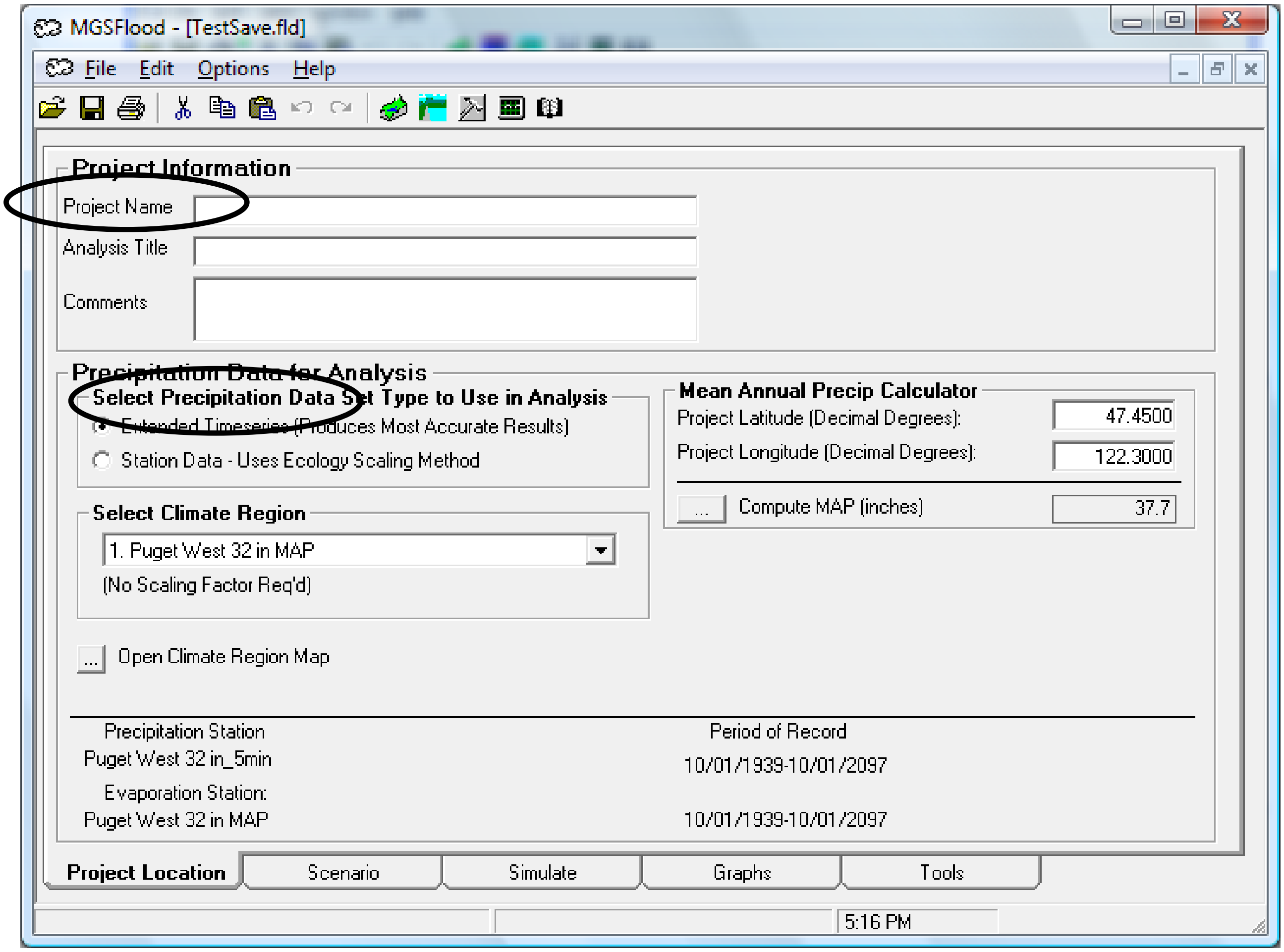
5 Getting Help

Context sensitive help is available by pressing F1 or by selecting Help from the command menu at the top of the screen.

6 Project Location Tab

The project location tab contains two different types of data; *Project Information* and *Precipitation Data Used in Analysis* (Figure 6.1). Data fields in the Project Information section are used for identifying the project. Information entered here is printed on the project reports.

The program contains two options for selecting precipitation input for project analysis; *Extended Precipitation Timeseries* and *Station Data*. The two options are discussed in the following sections.



Project Information

Project Name

Analysis Title

Comments

Precipitation Data for Analysis

Select Precipitation Data Set Type to Use in Analysis

● Extended Timeseries (Produces Most Accurate Results)

○ Station Data - Uses Ecology Scaling Method

Select Climate Region

1. Puget West 32 in MAP

(No Scaling Factor Req'd)

Mean Annual Precip Calculator

Project Latitude (Decimal Degrees): 47.4500

Project Longitude (Decimal Degrees): 122.3000

Compute MAP (inches) 37.7

Precipitation Station	Period of Record
Puget West 32 in_5min	10/01/1939-10/01/2097
Evaporation Station:	
Puget West 32 in MAP	10/01/1939-10/01/2097

Figure 6.1 – Project Location Tab

6.1 Extended Precipitation Timeseries Selection

Extended Precipitation timeseries utilizes a family of pre-scaled precipitation and evaporation timeseries. The extended precipitation time series have a time step of 5-minutes and were developed by combining and scaling hourly precipitation records from widely separated stations resulting in record lengths in excess of 100-years. The hourly time series were then disaggregated to a 5-minute time step using data collected from the Seattle Public Utilities precipitation gage network. Extended precipitation and evaporation timeseries have been developed using this method for most of the lowland areas of western Washington where stormwater projects will be constructed. These timeseries should be used for facility design for projects located in the region shown in Figure 6.2.

To select the precipitation and evaporation input for a project, open the Precipitation Map from the Project Location Tab. Locate the project site on the map and note the zone and the mean annual precipitation for the project site. The mean annual precipitation may also be determined by entering the project latitude and longitude in the Mean Annual Precipitation Calculator (in decimal degrees) and clicking the *Compute MAP* button.

From the Climatic Region drop down box on the *Project Location* tab, select the precipitation timeseries corresponding to the region and mean annual precipitation noted from the map or computed from the calculator. For project sites located in the City of Seattle, utilize the Seattle 38 in time series.

Separate time series were also developed for Pierce County, however, these time series are at a 1-hour time step. If conveyance design is of interest for projects in Pierce County, then the Puget East or Puget West extended time series should be used.

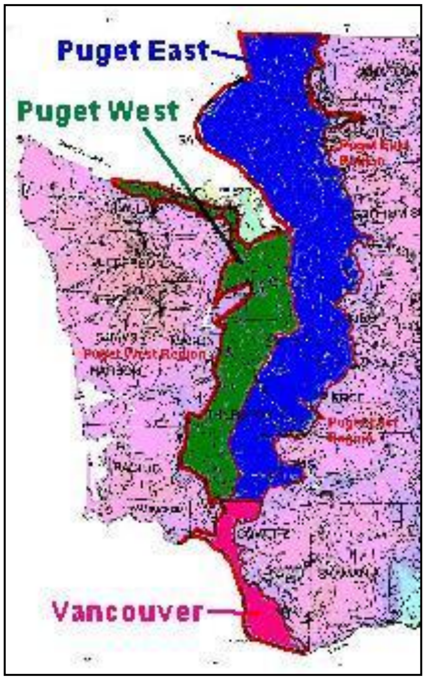


Figure 6.2 – Extended Precipitation Timeseries Regions

The example project site shown in Figure 6.3 is located in the western Puget Sound Region and the project mean annual precipitation is 51 inches. The precipitation timeseries for the western Puget Sound Region with mean annual precipitation closest to the project site should be selected from the drop down box. In this case, Puget Sound West Region, 52 inches MAP should be used.

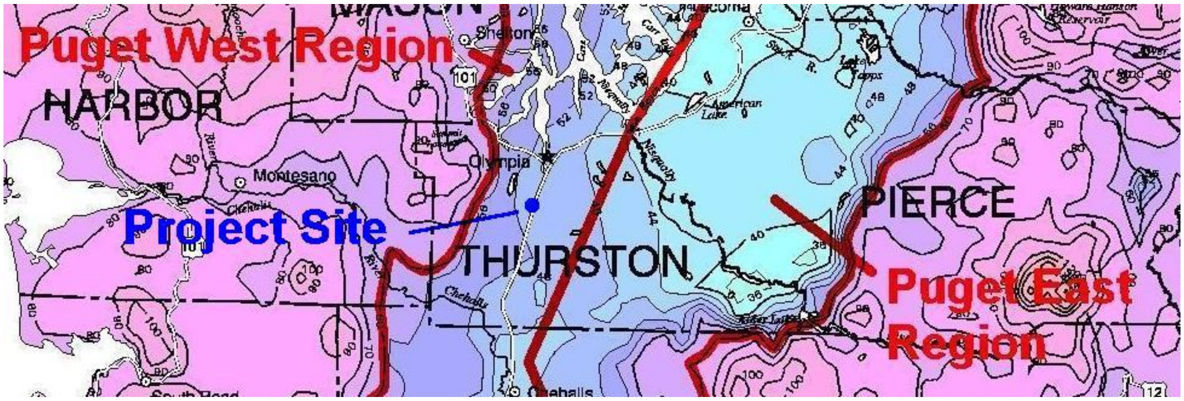


Figure 6.3 – Extended Precipitation Timeseries Selection Example

6.2 Precipitation Station Selection

For projects sites located outside of the extended timeseries region, data from hourly precipitation stations are used and a single scaling factor is applied to transpose the hourly record to the site of interest (target site). The current approach for single factor scaling, as recommended in the *Stormwater Management Manual for Western Washington*[9], is to compute the scaling factor as the ratio of the 25-year 24-hour precipitation[21] for the target and source sites.

To select the precipitation and evaporation input for a project location outside the area where the extended precipitation timeseries apply, check the *Station Data* option button and open the Precipitation Map from the Project Location Tab. Choose the precipitation region where the project site is located. Read the project site 25-year 24-hour precipitation from the map and enter it in the appropriate field on the Project Location Tab. The project 25-year 24-hour precipitation may also be computed by entering the project latitude and longitude in the Precip Calculator (in decimal degrees) and clicking the *Compute 25-Yr. 24-Hr* button.

For the example project site shown in Figure 6.4, the Clearwater gage should be selected as the source gage, and a project site 25-year, 24-hour precipitation of 6.0 inches should be entered in the appropriate field on the Project Location tab. The Scale factor would be computed by the program as the ratio of the project site to station 25-year, 24-hour precipitation, or 6.0 inches divided by 7.9 inches equals 0.759 (MGSFlood limits the scale factor to a minimum of 0.80 and no constraint on the maximum scaling factor is imposed). This value would be displayed in the *Scale Factor* field and all precipitation values subsequently read by the program would be multiplied by this value.

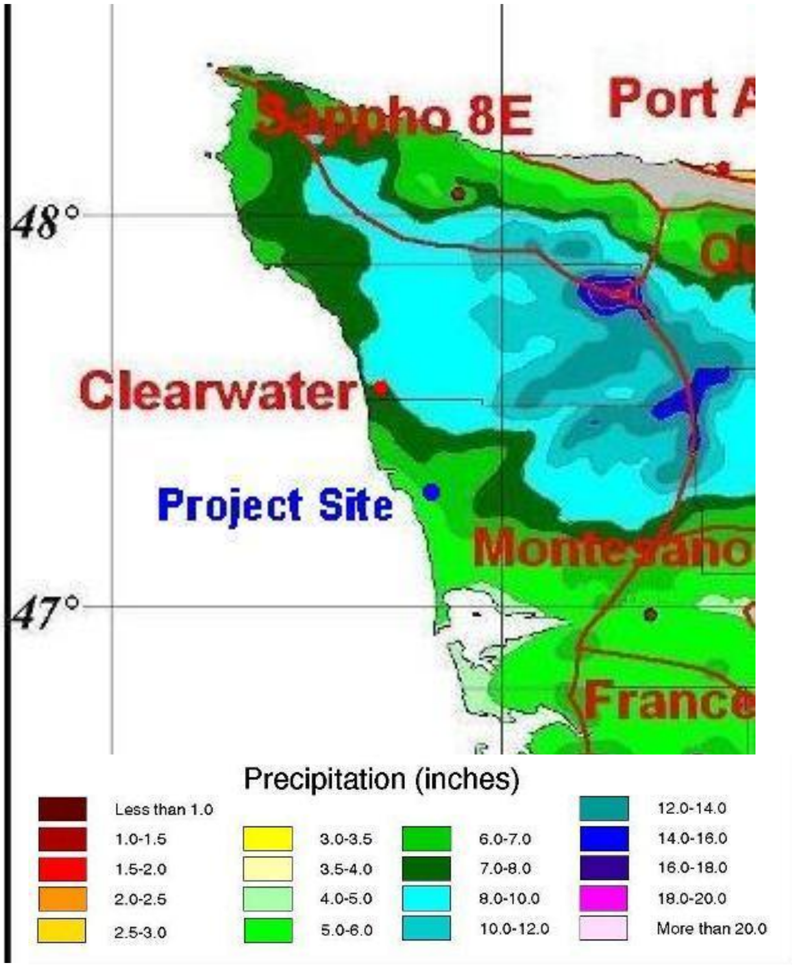
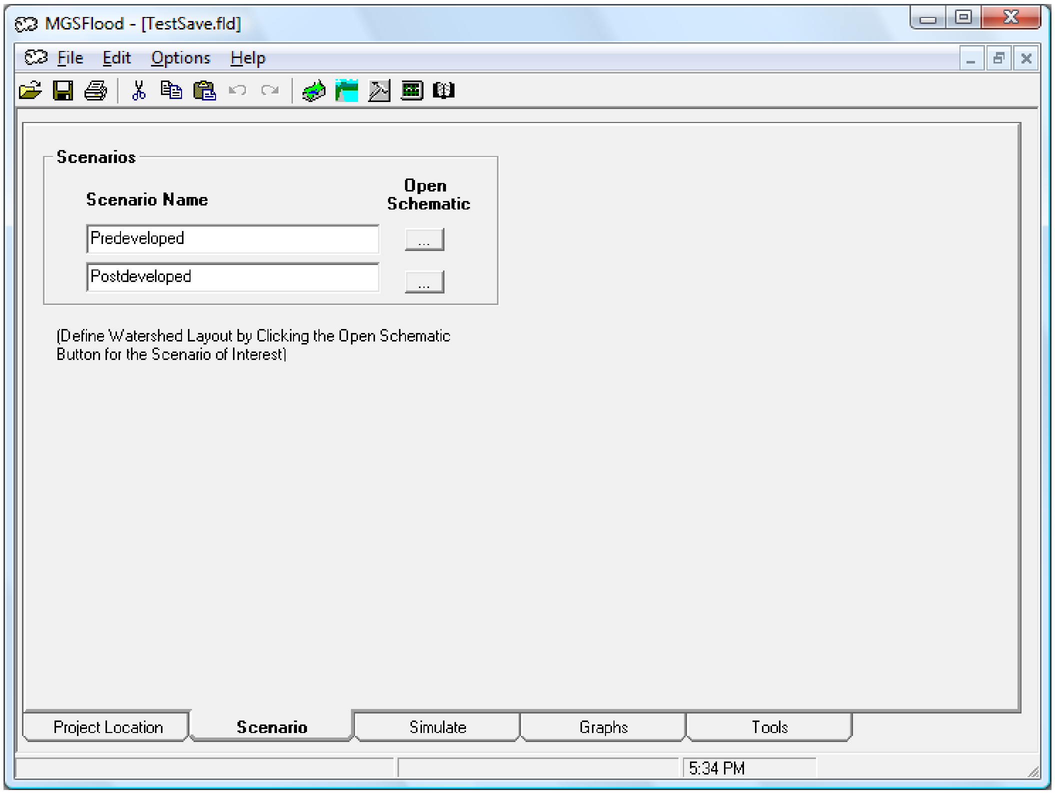


Figure 6.4 – Precipitation Input Selection Example for Project Sites Located Outside of Region Covered by Extended Timeseries

7 Scenario Tab

MGSFlood uses a graphical interface to define the predeveloped and postdeveloped watershed layouts accessed from the Scenario tab (Figure 7.1). Clicking the *Open Schematic* buttons display the Predeveloped (Scenario 1) and Postdeveloped (Scenario 2) input screens (Figure 7.2).



Scenario Name	Open Schematic
Predeveloped	...
Postdeveloped	...

(Define Watershed Layout by Clicking the Open Schematic Button for the Scenario of Interest)

Figure 7.1 – Scenario Tab with Buttons to Access the Predeveloped and Postdeveloped Graphical Input Screens

7.1 Watershed Input Screens

The program allows for different watershed configurations for predeveloped and postdeveloped conditions. This allows for structures, such as stormwater ponds, to be added in the post-developed condition without having to specify a corresponding dummy reach in the predeveloped condition.

Separate input screens are used to define the predeveloped and postdeveloped watersheds. To create an object in a watershed, such as a subbasin or stormwater pond, click and drag an icon from the Object box onto the watershed definitions window (Figure 7.2). To connect subbasins to links, and links to other links, right click the icon to display the menu and then click *Link Connection Primary* (Figure 7.3).

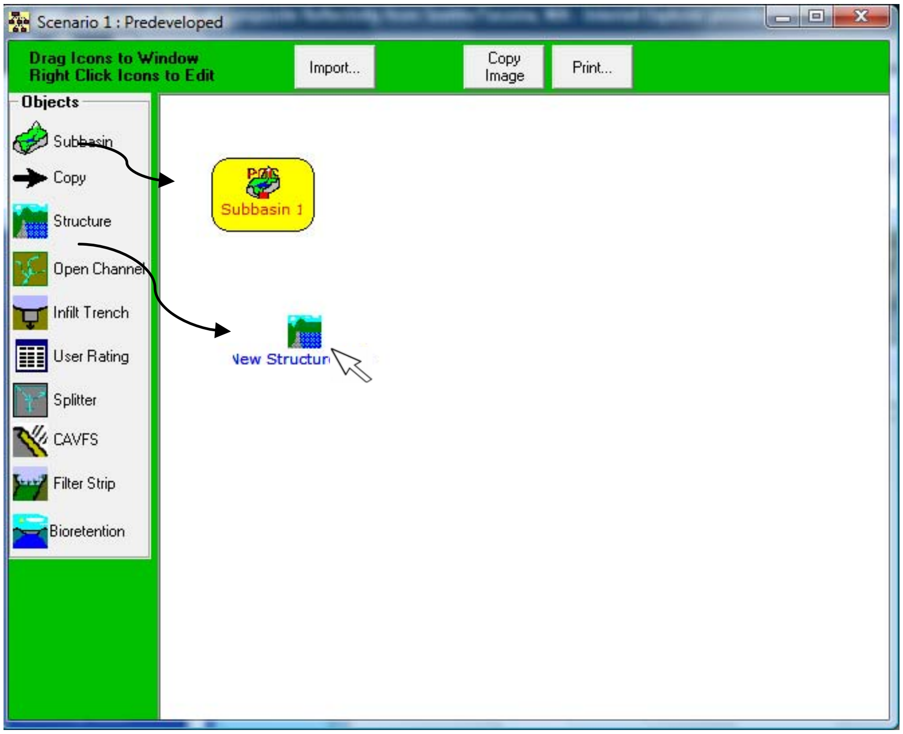


Figure 7.2 – Drag Icons onto Window to Create Objects to Define Watershed

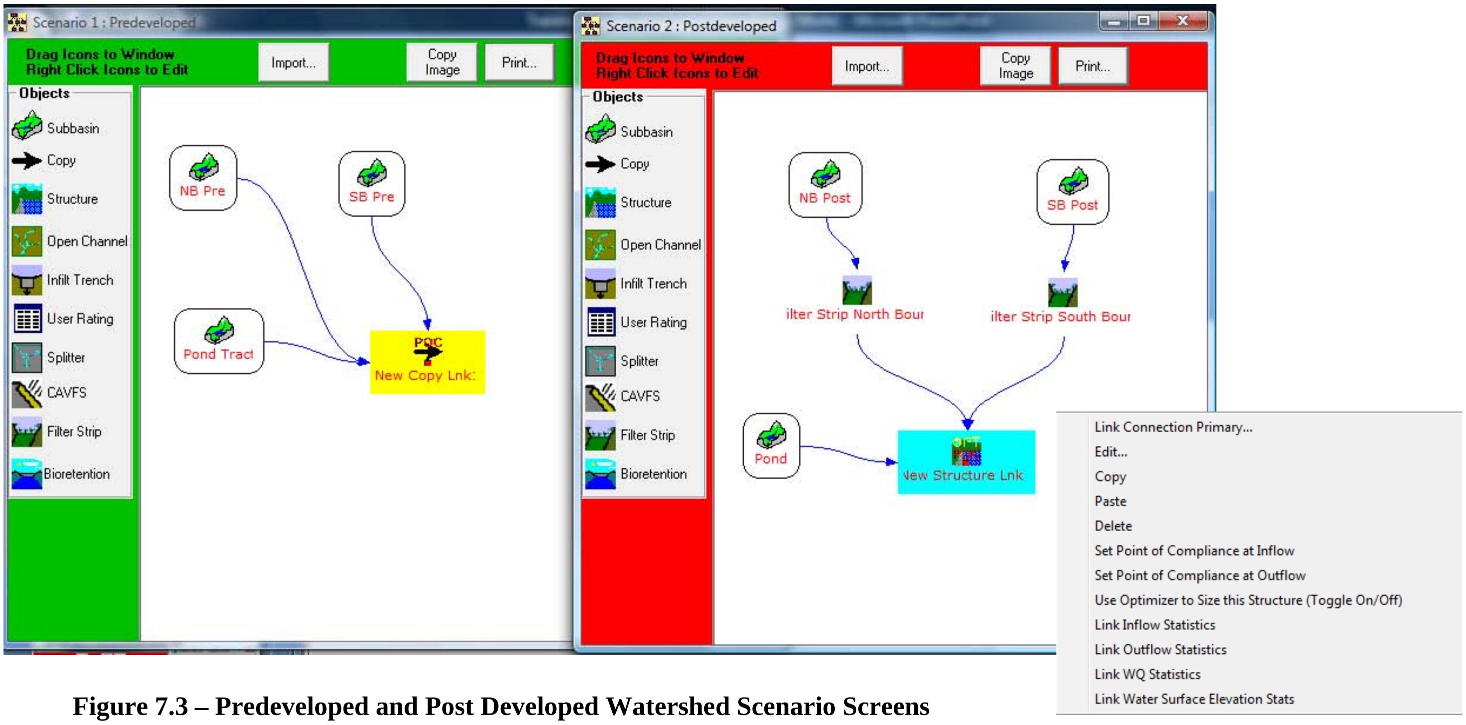


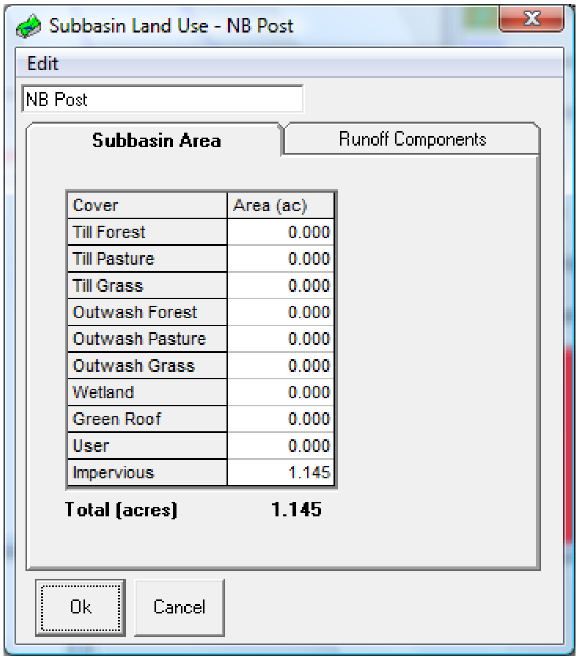
Figure 7.3 – Predeveloped and Post Developed Watershed Scenario Screens Showing Right Click Menu

Other items on the menu include:

- Edit – Opens the parameter screen for the current subbasin or link
- Copy – Copies the current subbasin or link to the Windows Clipboard,
- Paste – Pastes the subbasin or link.
- Set Point of Compliance – Sets the point of compliance for the current subbasin or Link,
- Use Optimizer to Size this Structure – Toggles the optimizer on or off for Structure or Infiltration Trench links,
- Flow Statistics – Computes duration and peak flow statistics. The statistics can be viewed in the project report or on the graphs tab,
- WQ Statistics – Opens the water quality statistics calculation Window for the Link.

7.2 Subbasin Area Input Screen

To create a subbasin, drag a subbasin icon onto the screen of the desired scenario (predeveloped or post developed). Right click the subbasin icon and then click *Edit*. The subbasin area input screen will appear (Figure 7.4).



NB Post

Cover	Area (ac)
Till Forest	0.000
Till Pasture	0.000
Till Grass	0.000
Outwash Forest	0.000
Outwash Pasture	0.000
Outwash Grass	0.000
Wetland	0.000
Green Roof	0.000
User	0.000
Impervious	1.145

Total (acres) 1.145

Figure 7.4 – Subbasin Land Use Input Screen

For each subbasin, land use is defined in acres for each soil/geologic group and land cover. The *Stormwater Management Manual for Western Washington*[9] relates SCS hydrologic soil groups to HSPF soil/geologic groups as shown in Table 7.1

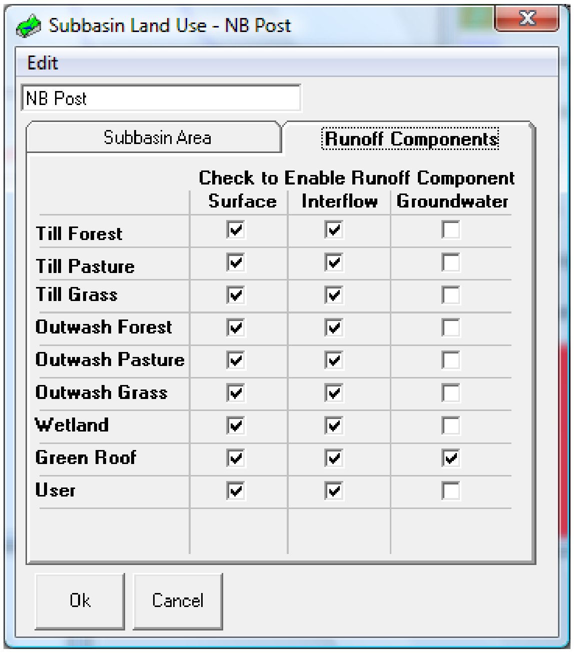
Table 7.1 – Relationship between SCS and HSPF Soil Groups

SCS Hydrologic Soil Group	MGSFlood/HSPF Soil/Geologic Group
A/B	Outwash
C	Till
D	Wetland

Note: The surface area of the pond must be included under the land use for the subbasin because precipitation is not applied to the pond surface by the program. This can be accomplished by adding impervious surface equal to the maximum pond surface area under the Subbasin Definitions window.

7.3 Subbasin Runoff Components Input Screen

MGSFlood simulates runoff as three components; surface overland flow, interflow and groundwater flow. The program sets these by default with the surface and interflow components connected, and the groundwater flow disconnected (except for the green roof parameters, where all three should be connected). The Runoff Components Input Screen allows the user to turn on or off any of the three runoff components for each land use/soil type. Except for very unusual circumstances, the runoff components should be left in their default configuration (Figure 7.5).



NB Post

Check to Enable Runoff Component

	Surface	Interflow	Groundwater
Till Forest	☑	☑	☐
Till Pasture	☑	☑	☐
Till Grass	☑	☑	☐
Outwash Forest	☑	☑	☐
Outwash Pasture	☑	☑	☐
Outwash Grass	☑	☑	☐
Wetland	☑	☑	☐
Green Roof	☑	☑	☑
User	☑	☑	☐

Figure 7.5 – Runoff Component Input Screen

7.4 Ecology Requirements for Land Cover

Consult the stormwater management manual for the local regulatory jurisdiction and the Washington State Stormwater Management Manual for Western Washington[9] (SWMMWW) regarding possible regulatory restrictions for:

<u>Predeveloped Forest Cover</u> - There are restrictions concerning the designation of the predeveloped land use as anything other than forest (SWMMWW Volume I, Minimum Requirement 7),

<u>Post Developed Forest Or Pasture Cover</u> - Assurances are required when designating an area as forest or pasture for the postdevelopment state to ensure that the area will not be disturbed in the future (SWMMWW Volume III, Appendix B),

<u>Off-Site Run-On To Project</u> - There are limits to offsite inflow discharging to a stormwater detention facility (SWMMWW Volume III, Appendix B),

<u>On-Site Stormwater Bypass</u> - There are restrictions to the size of development area from which stormwater runoff may bypass a detention facility (SWMMWW Volume III, Appendix B).

7.5 Including Bypass Area

Local topographic constraints often make it impractical to direct all runoff from developed areas to a detention facility. To bypass a portion of the subbasin area, include the bypass area as a separate subbasin and omit it from the primary subbasin. Connect the bypass subbasin to a link downstream of the detention facility. Figure 7.6 shows a subbasin connected to a link configured as a detention pond. A bypass subbasin was also included and is connected to a link downstream of the detention link. The downstream link is a Copy link and functions as a dummy reach to combine the pond outflow with the bypass. The inflow to the Copy link is set as point of compliance.

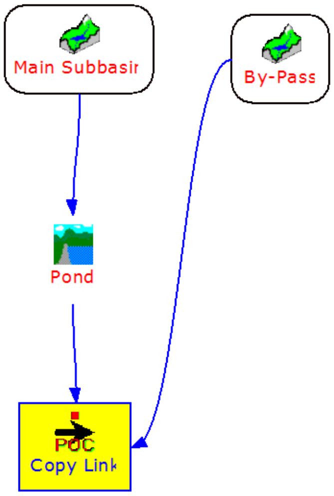


Figure 7.6 – Model Setup for Bypass Configuration

7.6 Defining the Point of Compliance

The *Point of Compliance* is the point in the watershed under predeveloped and post developed conditions where flow control compliance is measured. The predeveloped and post developed compliance points must be specified before performing a simulation.

To define the point of compliance, right click on the subbasin or link icon where the compliance point is to be located and then click set point of compliance. The icon will change to yellow and include the letters POC (Point of Compliance). For subbasins, the point of compliance is always the subbasin outflow and includes all runoff from the subbasin. For links, the point of compliance may be either the link inflow (prior to routing) or the link outflow (after routing) (Figure 7.7).

Subbasin Point of Compliance **Link Point of Compliance at Inflow** **Link Point of Compliance at Outflow**

Figure 7.7 – Watershed Icons denoting Compliance Point

7.7 Defining Links for Automatic Sizing (Optimization)

MGSFlood includes the ability to automatically size stormwater ponds, infiltration ponds, and infiltration trenches to meet the 2005 Ecology flow duration standard. The Link must be set for optimization by right clicking the link icon on the Postdeveloped Scenario window and selecting *Use Optimizer to Size this Structure (Toggle On/Off)*. Link selected for optimization is shown in blue on the Scenario input screen (Figure 7.8). The link will automatically be optimized when routing is performed on the *Simulate* tab.

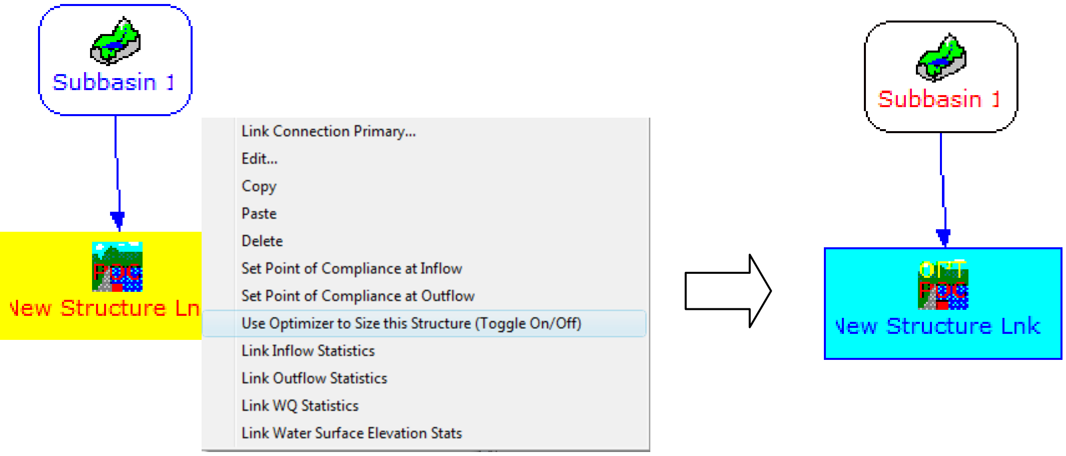


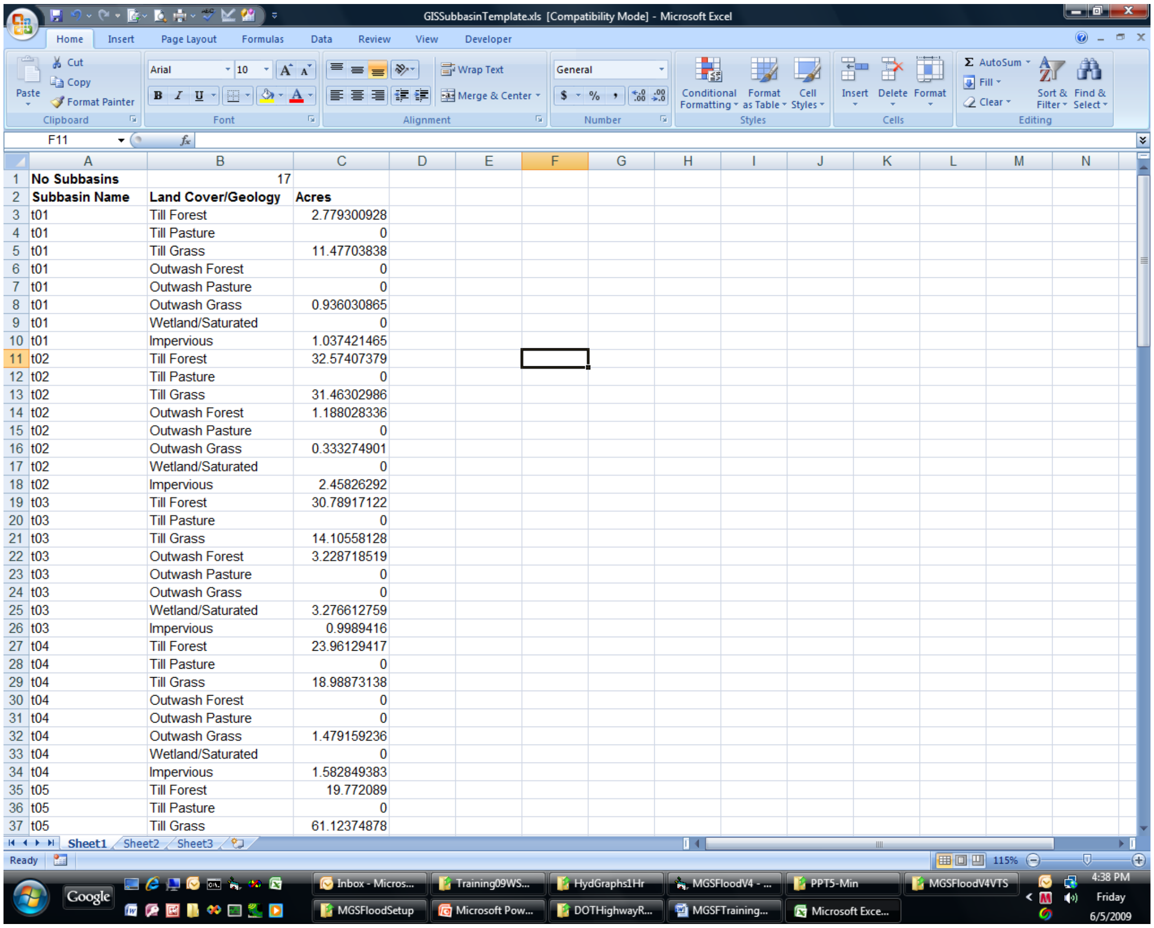
Figure 7.8 – Setting Pond Link for Optimization Icon turns Blue when Set for Optimization

7.8 Importing Subbasin Areas from Excel CSV Files

For projects with a large number of subbasins, it can be tedious and error prone to create each subbasin and enter the land use manually. An alternative is to use the feature that imports land use from a comma delimited file that can be created with Excel (.CSV file).

An Excel file included in the MGSFlood program directory was developed for creating CSV files in the format needed for MGSFlood. The Excel file is called GISSubbasinTemplate.xls and has the format shown in Figure 7.9. The number of subbasins to be imported is entered on the first line followed by the land use for each subbasin. The soil type/land cover combination must be in the order shown and repeats for each subbasin. Note that green roof areas are not included on the spreadsheet. Green roof areas must be entered manually using the MGSFlood subbasin area input screen. The subbasin area is entered in acres. The steps for using the Excel file to import subbasin areas is described below.

1. Open GISSubbasinTemplate.xls. Data from GIS can be exported to Excel and then reformatted to match the configuration shown below.

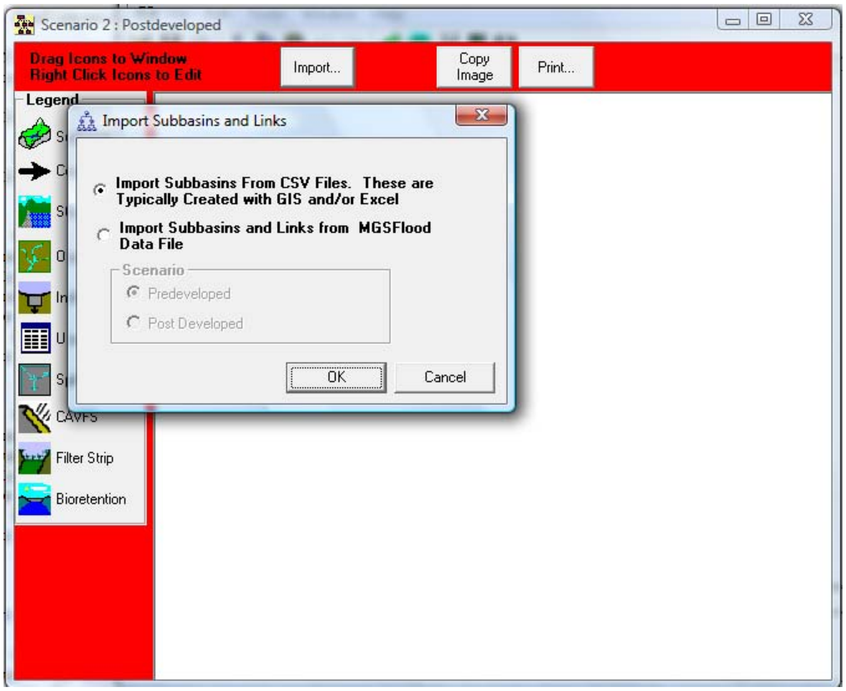


	A	B	C
1	**No Subbasins**	17	
2	**Subbasin Name**	**Land Cover/Geology**	**Acres**
3	t01	Till Forest	2.779300928
4	t01	Till Pasture	0
5	t01	Till Grass	11.47703838
6	t01	Outwash Forest	0
7	t01	Outwash Pasture	0
8	t01	Outwash Grass	0.936030865
9	t01	Wetland/Saturated	0
10	t01	Impervious	1.037421465
11	t02	Till Forest	32.57407379
12	t02	Till Pasture	0
13	t02	Till Grass	31.46302986
14	t02	Outwash Forest	1.188028336
15	t02	Outwash Pasture	0
16	t02	Outwash Grass	0.333274901
17	t02	Wetland/Saturated	0
18	t02	Impervious	2.45826292
19	t03	Till Forest	30.78917122
20	t03	Till Pasture	0
21	t03	Till Grass	14.10558128
22	t03	Outwash Forest	3.228718519
23	t03	Outwash Pasture	0
24	t03	Outwash Grass	0
25	t03	Wetland/Saturated	3.276612759
26	t03	Impervious	0.9989416
27	t04	Till Forest	23.96129417
28	t04	Till Pasture	0
29	t04	Till Grass	18.98873138
30	t04	Outwash Forest	0
31	t04	Outwash Pasture	0
32	t04	Outwash Grass	1.479159236
33	t04	Wetland/Saturated	0
34	t04	Impervious	1.582849383
35	t05	Till Forest	19.772089
36	t05	Till Pasture	0
37	t05	Till Grass	61.12374878

Figure 7.9 – Excel File used to Format CSV files for Import to MGSFlood

2. Save the Workbook as a .CSV file. Click File, Save As and Select *CSV Comma Delimited (*.csv)* as the file type.

3. Open MGSFlood. Open the scenario that you want to import the subbasins to. Check the Import CSV File option and click OK.



● Import Subbasins From CSV Files. These are Typically Created with GIS and/or Excel

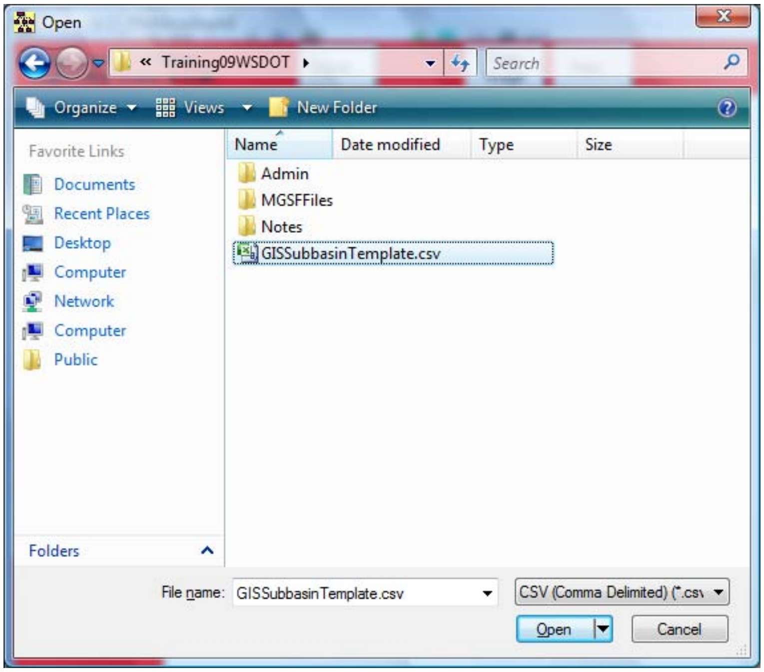
○ Import Subbasins and Links from MGSFlood Data File

Scenario

● Predeveloped

○ Post Developed

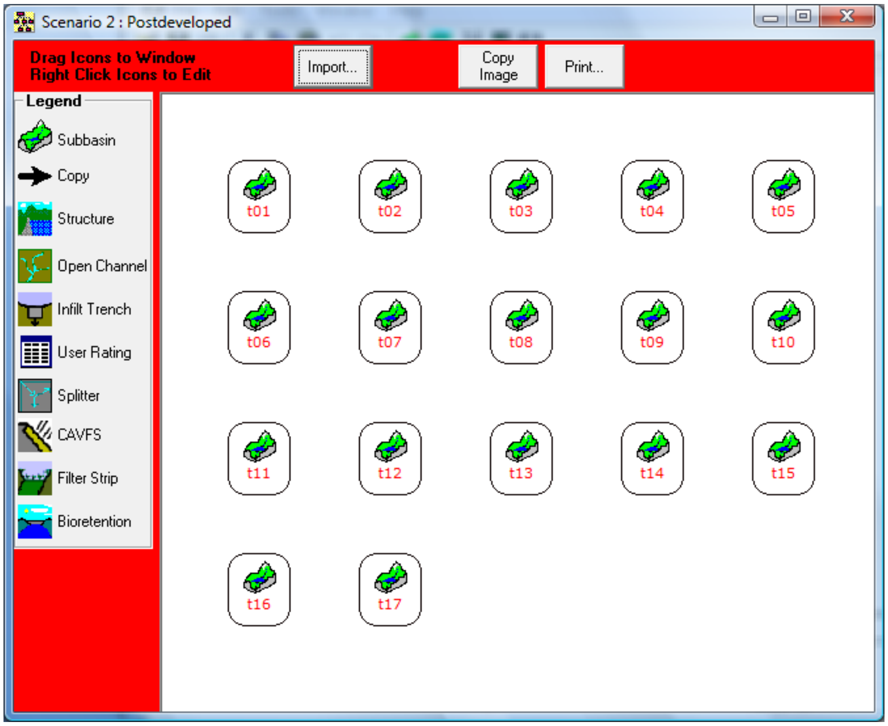
4. Navigate to the GISSubbasinTemplate.csv file and click Open



File name: GISSubbasinTemplate.csv

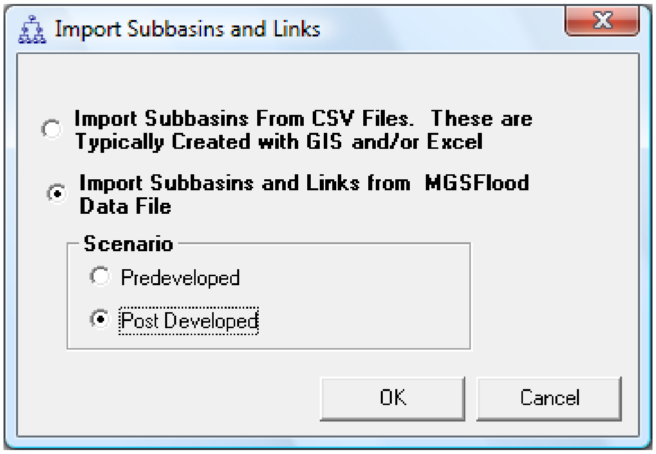
CSV (Comma Delimited) (*.csv

5. The subbasins will be automatically created with the land use from the CSV file.



7.9 Importing Subbasins and Links from Another MGSFlood File

Subbasins and links can be imported from another MGSFlood data file by clicking the Import button on the top of the Scenario input screen. Select Import Subbasins and Links from MGSFlood Data File option button and then select the scenario you would like to import (predeveloped or post developed). Click OK and navigate to the MGSFlood data file you wish to read subbasins and links from. Note, the files must have been created using MGSFlood Version 4 or later. Click Open to Import the subbasins and links from the other file.



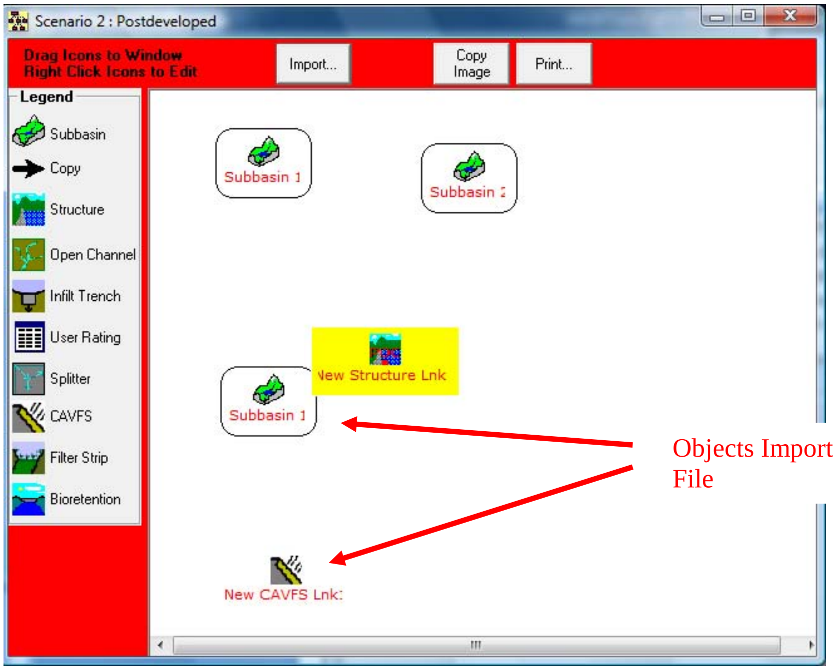
Import Subbasins and Links

○ Import Subbasins From CSV Files. These are Typically Created with GIS and/or Excel

● Import Subbasins and Links from MGSFlood Data File

Scenario
○ Predeveloped
● Post Developed

After importing, we now have the subbasins and links from the imported data file. In the example below, Subbasin 1 and a CAVFS link were imported from a separate project file.



Drag Icons to Window
Right Click Icons to Edit

Objects Imported from Second MGSFlood File

8 Link Definitions and Parameters

Links are used to connect one part of the watershed to another; subbasins to links and links to other links. The following summarizes the type of links currently in the model:

1. Copy – The Copy Link may be thought of as a dummy reach because is copies discharge from the inflow point to the outflow point without routing or lagging,
2. Structure – Includes detention and infiltration ponds, and sand filters,
3. Channel – Performs routing in open channels,
4. Infiltration Trench – Performs routing through infiltration trenches,
5. Rating Table – User defined stage storage discharge table,
6. Flow Splitter – Splits a fraction of the discharge from one link to another.
7. CAVFS – Compost amended vegetated filter strip,
8. Filter Strip – Similar to CAVFS, but doesn't include compost amendment,
9. Bioretention – Simulates bioretention facility with surface detention storage, infiltration, and underdrain return flow.

Information for each type of Link is discussed in the following Sections.

8.1 Copy Link

The copy link copies timeseries from the upstream subbasin or link and adds it to the inflow at the downstream link. Hydrographs are transferred to the outflow without attenuation or lagging. The copy link is appropriate for small watersheds where there is little attenuation of the flood hydrograph from routing. If the conveyance channel is long with large overbank storage, then the link should be defined as an open channel. As a general rule, channel routing may be neglected for watersheds smaller than about ½ square mile (320 acres) and the link may be defined using the copy routine.

8.2 Structure Link

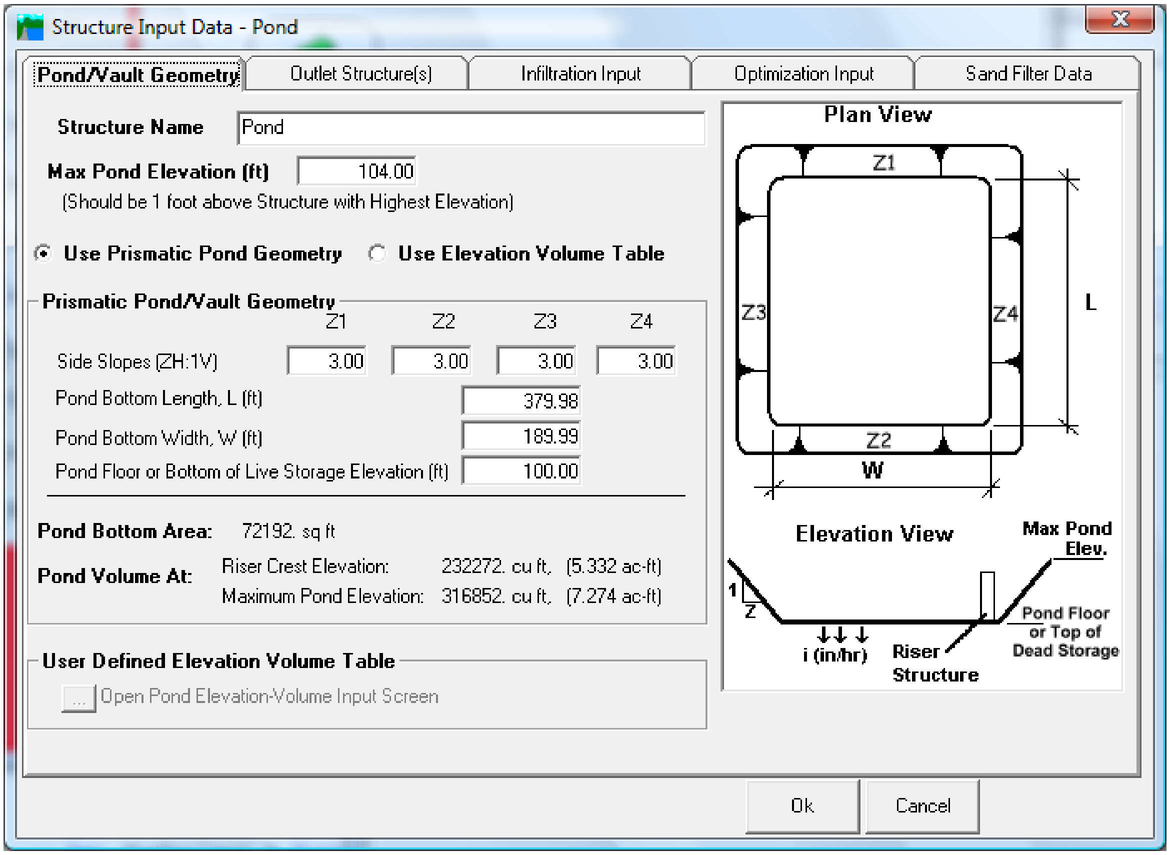
Structure links are used to define stormwater ponds, infiltration ponds, and sand filters. Pond optimization information for post-development condition ponds is also input on the structure link input screens.

A variety of hydraulic devices can be included in the design of stormwater treatment facilities. Devices attached to the riser structure include; circular orifices, circular orifices under backwater influence, rectangular orifices, rectangular weirs, V-notch weirs, and proportional weirs. In addition, the riser structure can also be defined with an open top to function as an overflow weir, or the top may be capped. Any combination of up to six devices plus the riser structure and a sand filter can be included for each structure. A trapezoidal broad crested weir may also be specified to function as an emergency overflow.

The following sections describe the input for Structure Links.

8.2.1 Pond/Vault Geometry Input

Two options are available for specifying pond or vault geometry. The first assumes a prismatic geometry with pond length, width, depth, and side slopes as shown in Figure 8.1.



Structure Name Pond

Max Pond Elevation (ft) 104.00

(Should be 1 foot above Structure with Highest Elevation)

● **Use Prismatic Pond Geometry** ○ **Use Elevation Volume Table**

Prismatic Pond/Vault Geometry

	Z1	Z2	Z3	Z4
Side Slopes (ZH:1V)	3.00	3.00	3.00	3.00

Pond Bottom Length, L (ft) 379.98

Pond Bottom Width, W (ft) 189.99

Pond Floor or Bottom of Live Storage Elevation (ft) 100.00

Pond Bottom Area: 72192. sq ft

Pond Volume At:

Riser Crest Elevation:	232272. cu ft,	(5.332 ac-ft)
Maximum Pond Elevation:	316852. cu ft,	(7.274 ac-ft)

User Defined Elevation Volume Table

... Open Pond Elevation-Volume Input Screen

Figure 8.1 – Hydraulic Structures Input Screen

where:

L – is the pond length in feet,

W – is the pond width in feet,

Z1, Z2, Z3, Z4 - are side slopes for each side of the pond where Z is the number of feet in the horizontal plane for every foot of rise,

Pond Floor Elevation – Represents the bottom of the *live* pond storage. Live storage is defined as the storage used to detain stormwater runoff and eventually flows through the outlet structure. Dead storage is retained in the pond below the elevation of the outlet structure. The pond floor elevation should be input if the pond is <u>not</u> a combined wet pond. If the pond is a combined wet pond, then enter the elevation of the top of the dead storage, i.e. the elevation where water begins to discharge from the pond

Riser Crest Elevation – The elevation at which water begins to flow into the overflow riser. The maximum flood recurrence interval detained by the pond generally corresponds with this elevation (or slightly above this elevation). For example, the Ecology flow duration standard requires control of the flow duration between ½ of the 2-year and the 50-year recurrence interval. Water will begin to spill into the riser structure near the 50-year recurrence interval. It is acceptable for water to spill into the riser structure for floods smaller than the 50-year provided that the flow duration standard is met.

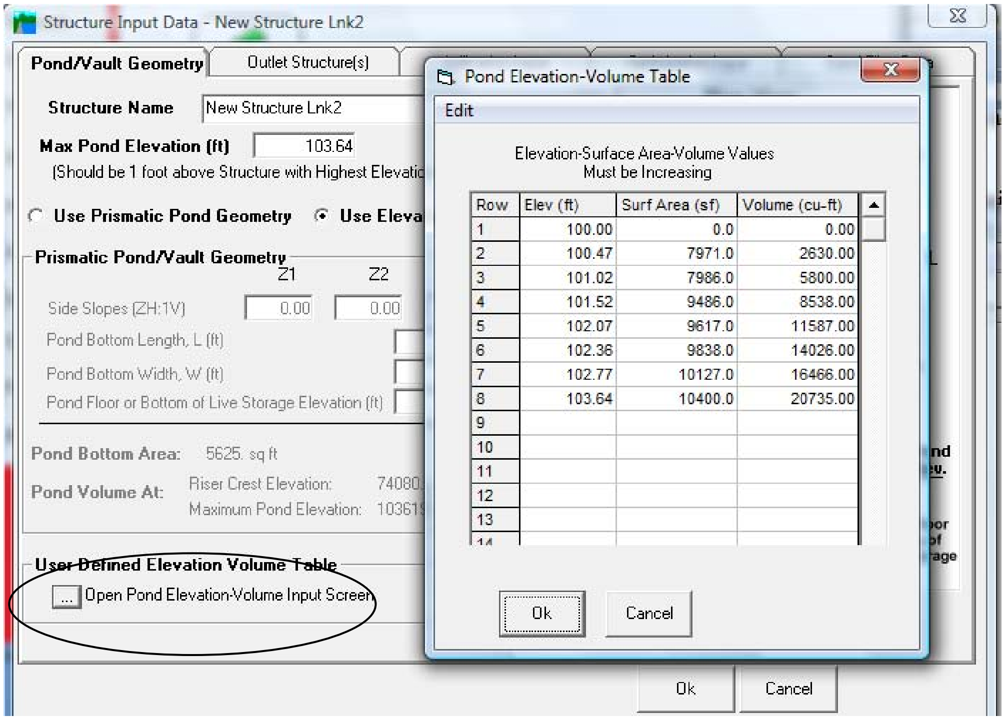
Max Pond Elevation – Is the maximum elevation used in pond routing calculations and typically extends above the riser crest elevation a sufficient distance to accommodate large floods or to allow for flood passage if one or more of the lower level outlets become blocked. The required maximum pond elevation depends on the design standards of the local jurisdiction.

The automatic pond sizing routine (optimizer) in MGSFlood determines the riser diameter and maximum pond elevation so that the 100-year peak inflow will pass through the riser structure assuming the lower level outlets are blocked. The user is advised to check the maximum pond elevation returned by the optimizer with the design standards of the local jurisdiction including any freeboard requirements.

If a vault is to be analyzed, then side slopes (*Z1, Z2, Z3, Z4*) of zero are input denoting vertical sides. The pond volume for elevations ranging from the floor to one foot above the maximum pond elevation is computed according to this geometry.

The second method for specifying pond geometry is with a user defined elevation-volume table as shown in Figure 8.2. This is useful for specifying the geometry of irregularly shaped ponds. The elevation-volume relationship can be computed using a spreadsheet program and pasted into the form using the Windows Clipboard utility.

Note: Precipitation falling on the surface of the detention pond is not automatically computed by MGSFlood. This approach was taken to allow use of both ponds and vaults. The difference being ponds are open to collection of precipitation, and vaults are closed to precipitation input. To include precipitation on the pond surface in the computations, the surface area of the pond must be included under the land use for the subbasin where the pond resides. This can be accomplished by adding impervious surface equal to the maximum pond surface area under the *Subbasin Definitions* window for the sub-basin where the pond resides. A simple approach to get an initial estimate of the pond surface area would be to run the *Quick Optimization routine* after the tributary subbasins have been defined.


Structure Input Data - New Structure Lnk2

Pond/Vault Geometry | Outlet Structure(s)

Structure Name: New Structure Lnk2

Max Pond Elevation (ft): 103.64

(Should be 1 foot above Structure with Highest Elevatio

○ Use Prismatic Pond Geometry ● Use Eleva

Prismatic Pond/Vault Geometry

Side Slopes (ZH:1V): Z1 0.00, Z2 0.00

Pond Bottom Length, L (ft)

Pond Bottom Width, W (ft)

Pond Floor or Bottom of Live Storage Elevation (ft)

Pond Bottom Area: 5625. sq ft

Pond Volume At: Riser Crest Elevation: 74080

Maximum Pond Elevation: 10361

User Defined Elevation Volume Table

Open Pond Elevation-Volume Input Screen

Pond Elevation-Volume Table

Edit

Elevation-Surface Area-Volume Values
Must be Increasing

Row	Elev (ft)	Surf Area (sf)	Volume (cu-ft)
1	100.00	0.0	0.00
2	100.47	7971.0	2630.00
3	101.02	7986.0	5800.00
4	101.52	9486.0	8538.00
5	102.07	9617.0	11587.00
6	102.36	9838.0	14026.00
7	102.77	10127.0	16466.00
8	103.64	10400.0	20735.00
9			
10			
11			
12			
13			

Figure 8.2 – Hydraulic Structures Input Screen, User Defined Elevation/Volume Input

8.2.2 Pond Infiltration

MGSFlood includes two options for simulating infiltration; Massmann[30] equations and fixed infiltration. The Massmann equations are based on field observations of infiltration ponds in western Washington. This infiltration approach accounts for the side slope geometry of the pond, pond aspect (length to width ratio), the proximity of the pond to the regional groundwater table, and the potential for soil clogging and fouling. Inputs include; Soil Hydraulic Conductivity (inches/hour), Depth to the Regional Water Table (ft) (Figure 8.3), whether bio-fouling potential is low, and whether average or better maintenance is performed. Infiltrated moisture is lost from the system and does not contribute to the discharge rate through the riser or orifices.

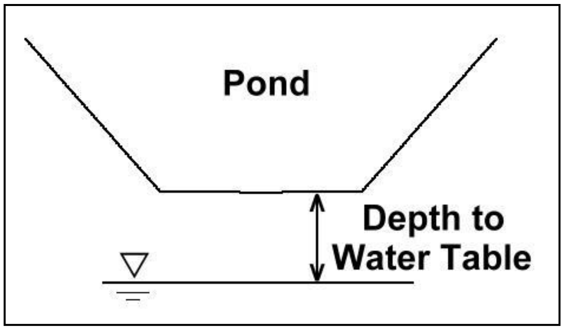


Figure 8.3 – Infiltration Pond Depth to Water Table
(Accounts for Groundwater Mounding Beneath Pond)

Soil Hydraulic Conductivity (in/hr) – Is the saturated hydraulic conductivity of the soil beneath the pond in inches per hour according to Darcy's Equation. It can be estimated using regression equations that use grain size distribution as input (Massmann[30]) or from literature (e.g. Freeze and Cherry[31], Fetter[32]).

Depth to Regional Groundwater Table (ft) – Represents the depth from the bottom of the pond to the regional groundwater table or the first low-permeability layer. For shallow groundwater sites, groundwater mounding reduces the hydraulic gradient and the infiltration rate is significantly less than the saturated hydraulic conductivity. For deep groundwater sites where the effects of mounding will be small, the gradient will not typically be reduced by infiltration from the facility. Increasing the depth to groundwater greater than 100 feet ceases to have an influence on pond infiltration according to this approach.

Bio-fouling Potential – Bio-fouling occurs from organic material blanketing the soil surface and reducing the infiltration rate. Bio-fouling is more likely to occur if the pond is located beneath trees and other vegetation or in shaded locations.

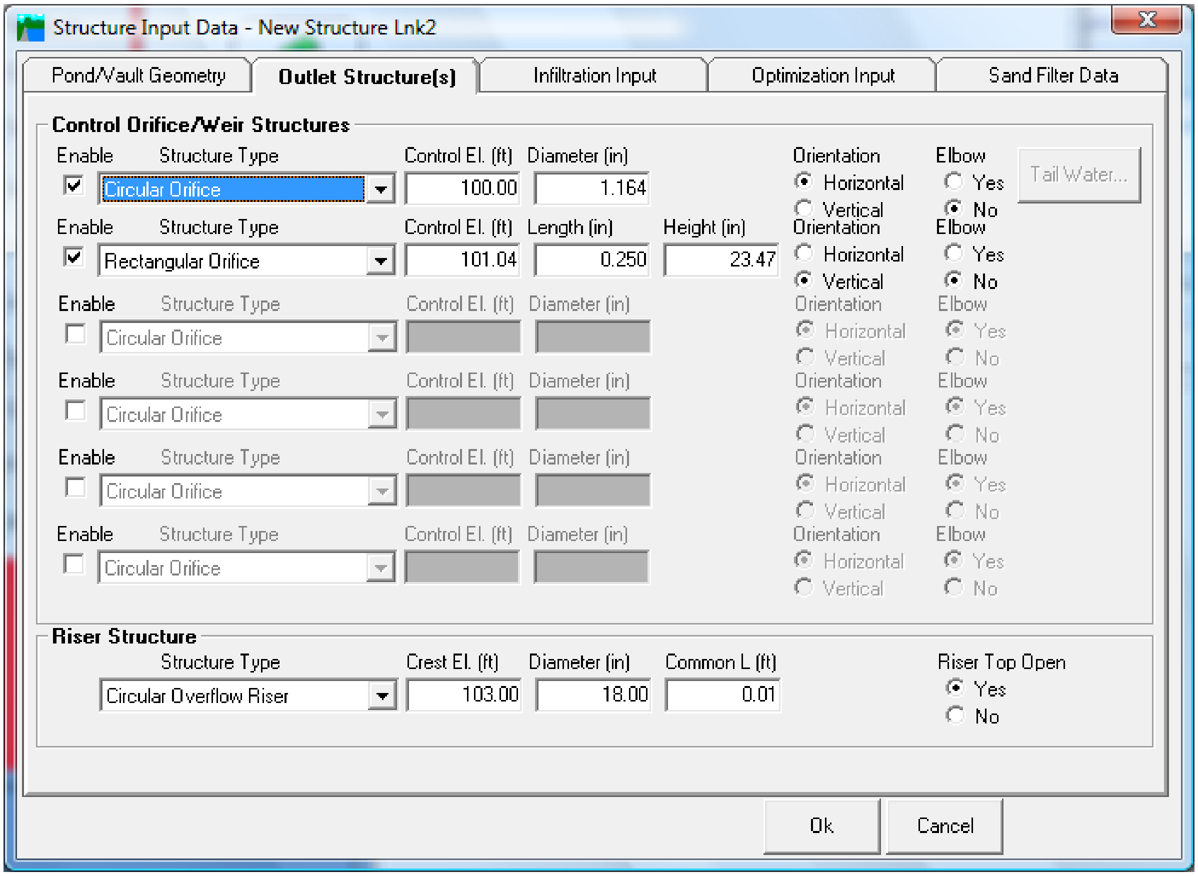
Maintenance – Siltation is more likely to occur if there is not sufficient pre-treatment of the storm water or in locations where the drainage basin is prone to erosion because of recent land disturbances or steep slopes. The user should consider the potential for siltation of the infiltration pond and the maintenance program when determining the effects of maintenance on pond infiltration performance.

8.2.3 Outlet Structures

The *Outlet Structures* Tab defines type, size, and elevation of pond outlets (Figure 8.4). Up to eight outlet devices consisting of any combination of the following can be defined:

- Circular orifice with or without tailwater,
- Rectangular orifice or slot,
- V-notch sharp crested weir,
- Rectangular sharp crested weir,
- Proportional sharp crested weir
- Trapezoidal broad crested weir

If an orifice subjected to tailwater is selected, then an elevation-discharge rating table must be entered by clicking the *Tailwater* button (Figure 8.5). A minimum of four elevation-discharge pairs must be entered, and discharges must be entered in an increasing order of magnitude. See Part I, Section 8 for more information regarding the geometry and hydraulic equations governing each structure, and guidance for backwater conditions.



Structure Input Data - New Structure Lnk2

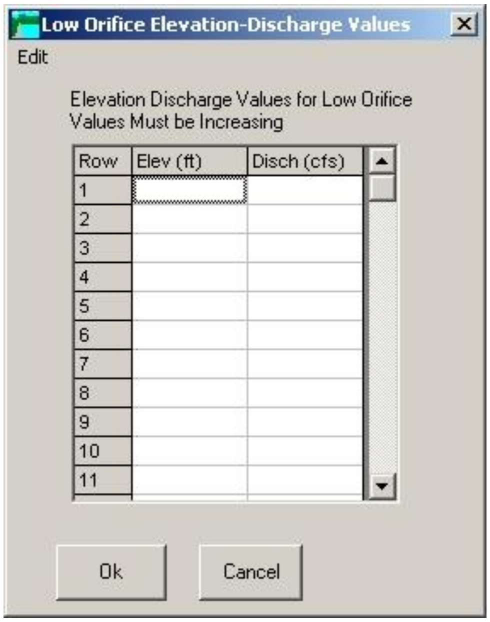
Control Orifice/Weir Structures

Enable	Structure Type	Control El. (ft)	Diameter (in)	Length (in)	Height (in)	Orientation	Elbow
☑	Circular Orifice	100.00	1.164			● Horizontal ○ Vertical	○ Yes ● No
☑	Rectangular Orifice	101.04		0.250	23.47	○ Horizontal ● Vertical	○ Yes ● No
☐	Circular Orifice					● Horizontal ○ Vertical	● Yes ○ No
☐	Circular Orifice					● Horizontal ○ Vertical	● Yes ○ No
☐	Circular Orifice					● Horizontal ○ Vertical	● Yes ○ No
☐	Circular Orifice					● Horizontal ○ Vertical	● Yes ○ No

Riser Structure

Structure Type	Crest El. (ft)	Diameter (in)	Common L (ft)	Riser Top Open
Circular Overflow Riser	103.00	18.00	0.01	● Yes ○ No

Figure 8.4 – Hydraulic Structures Input Screen



Elevation Discharge Values for Low Orifice
Values Must be Increasing

Row	Elev (ft)	Disch (cfs)
1		
2		
3		
4		
5		
6		
7		
8		
9		
10		
11		

Figure 8.5 – Input Screen for Specifying Tailwater Conditions for Low Orifice

8.2.4 Riser Structure

The pond riser structure is defined at the bottom of the *Outlet Structures* tab (Figure 8.4). A single riser structure is defined for each pond and can be either circular or rectangular in cross section. If the *Riser Top Open* option button is selected, then the riser functions as an overflow structure. The *Common Length* field defines the sum of any outlet structures that intersect the riser crest. Specifying the common length ensures that the discharge from the hydraulic structure is not double counted when flow passes over the riser crest (See Part I, Section 8 for more details).

8.2.5 Automatic Pond and Outlet Works Sizing Routine/Optimization

The pond sizing optimization routine automatically determines the size of the pond, and size and elevation of the outlet works needed to meet the Washington State Department of Ecology Flow Duration Standard[9].

Designing stormwater ponds to this standard is a laborious, iterative process when performed manually. In addition, because of the number of variables involved in designing a pond to the flow duration standard, it is difficult to find a pond configuration that minimizes the pond volume and meets the duration standard using manual trial and error. The automatic pond sizing routine seeks to determine a minimum pond size that meets the flow duration standard.

The automatic pond sizing optimization routine will determine the pond size and outlet configuration for two pond types; a detention pond with minor infiltration and an infiltration pond. The characteristics of these two pond types are listed in Table 8.1

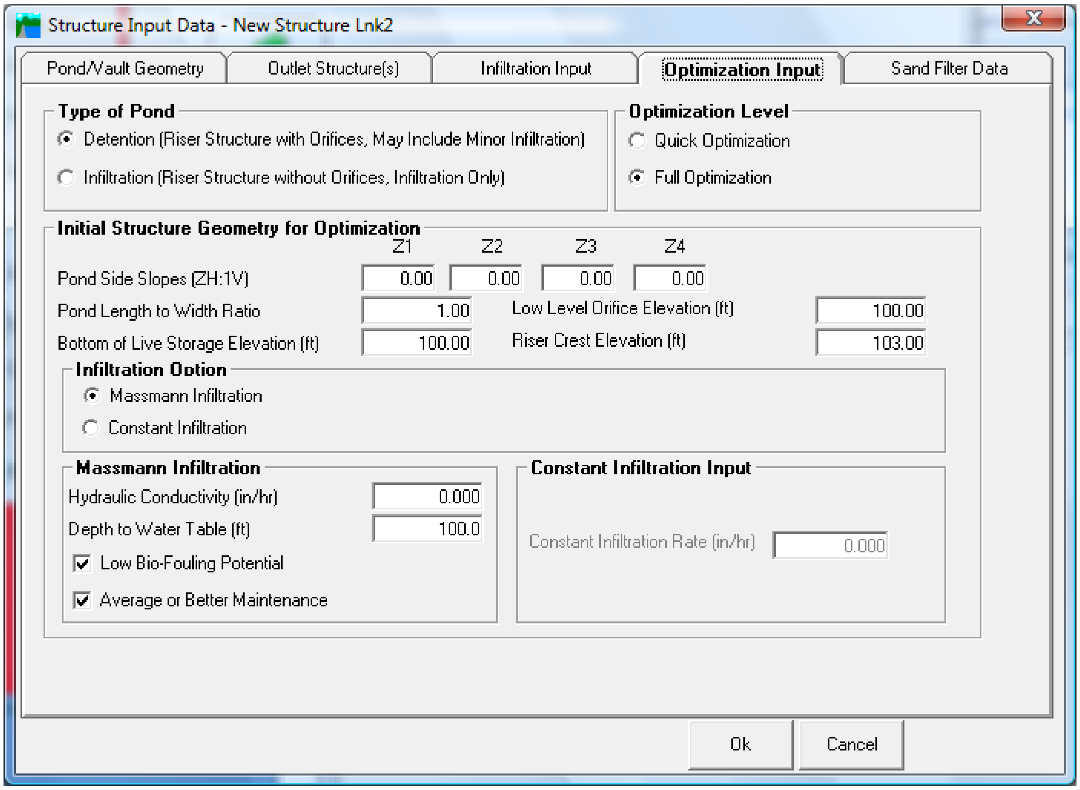
Table 8.1 – Characteristics of Detention and Infiltration Ponds Sized using Optimization Routine		
Characteristic	**Detention Pond**	**Infiltration Pond**
Pond Configuration	Riser Structure with Low Level Circular Orifice and Vertical Rectangular Upper Orifice	Overflow Riser Only
Valid Infiltration Rates*	0.00 – 0.10 inches/hour	0.05-50 inches/hour
Optimization Levels	Quick or Full	Quick Only

* Note: Infiltration occurs through the pond bottom only, not including the side slopes.

The pond sizing optimization routine uses general input about the pond geometry including;

- Pond length to width ratio,
- Pond side slopes (Z1, Z2, Z3, Z4),
- Pond floor elevation,
- Riser crest elevation,
- Hydraulic Conductivity (used to simulate infiltration),
- Depth to Water Table (used to simulate infiltration),
- Bio-fouling Potential and Maintenance Level (used to simulate infiltration).

These variables are entered on the Optimization tab (Figure 8.6). The Link is then set for optimization by right clicking the link icon on the Scenario window and selecting *Use Optimizer to Size this Structure*. Links selected for optimization are shown in blue on the Scenario input screen (Figure 8.7). The link will automatically be optimized when routing is performed on the *Simulate* tab.



Type of Pond

● Detention (Riser Structure with Orifices, May Include Minor Infiltration)

○ Infiltration (Riser Structure without Orifices, Infiltration Only)

Optimization Level

○ Quick Optimization

● Full Optimization

Initial Structure Geometry for Optimization

	Z1	Z2	Z3	Z4
Pond Side Slopes (ZH:1V)	0.00	0.00	0.00	0.00

Pond Length to Width Ratio 1.00

Bottom of Live Storage Elevation (ft) 100.00

Low Level Orifice Elevation (ft) 100.00

Riser Crest Elevation (ft) 103.00

Infiltration Option

● Massmann Infiltration

○ Constant Infiltration

Massmann Infiltration

Hydraulic Conductivity (in/hr) 0.000

Depth to Water Table (ft) 100.0

☑ Low Bio-Fouling Potential

☑ Average or Better Maintenance

Constant Infiltration Input

Constant Infiltration Rate (in/hr) 0.000

Figure 8.6 – Automatic Pond Design and Optimization Input Screen

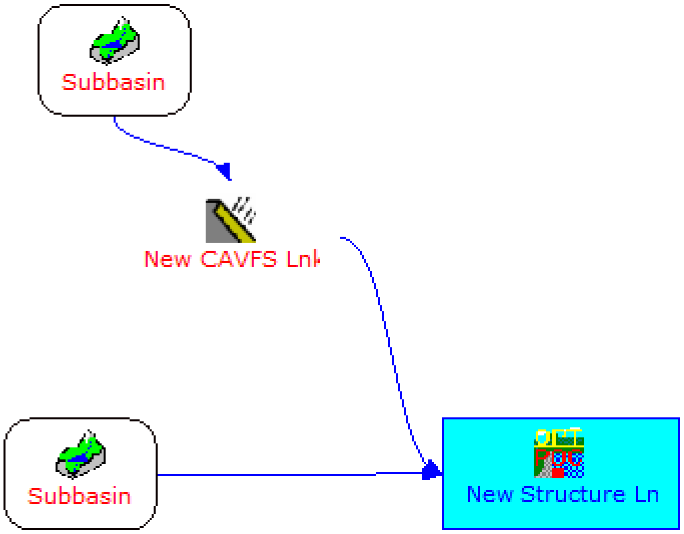


Figure 8.7 – Post Development Network Showing Pond Structure Set for Optimization

The optimization routine is currently configured to handle modest amounts of infiltration for detention ponds, with infiltration rates less than 0.10 in/hr. If a larger infiltration rates are required, it is recommended that the quick optimization routine be used to obtain a rough starting point and then proceed with the pond design using the manual adjustment.

To manually edit the pond configuration determined by the *Optimization* routine, reopen the *Hydraulic Structures* input screen by right clicking on the structure icon and then clicking Edit from the Post Developed scenario window. Make changes to the outlet works or geometry returned by the optimization routine program and click OK. Right click the icon and toggle off the optimizer. Click the Route button on the Simulate tab to route flows through the network. Guidelines for manually adjusting the outlet works and pond geometry to achieve compliance with the flow duration standard are listed in Section 18 of Part I.

Optimization Level

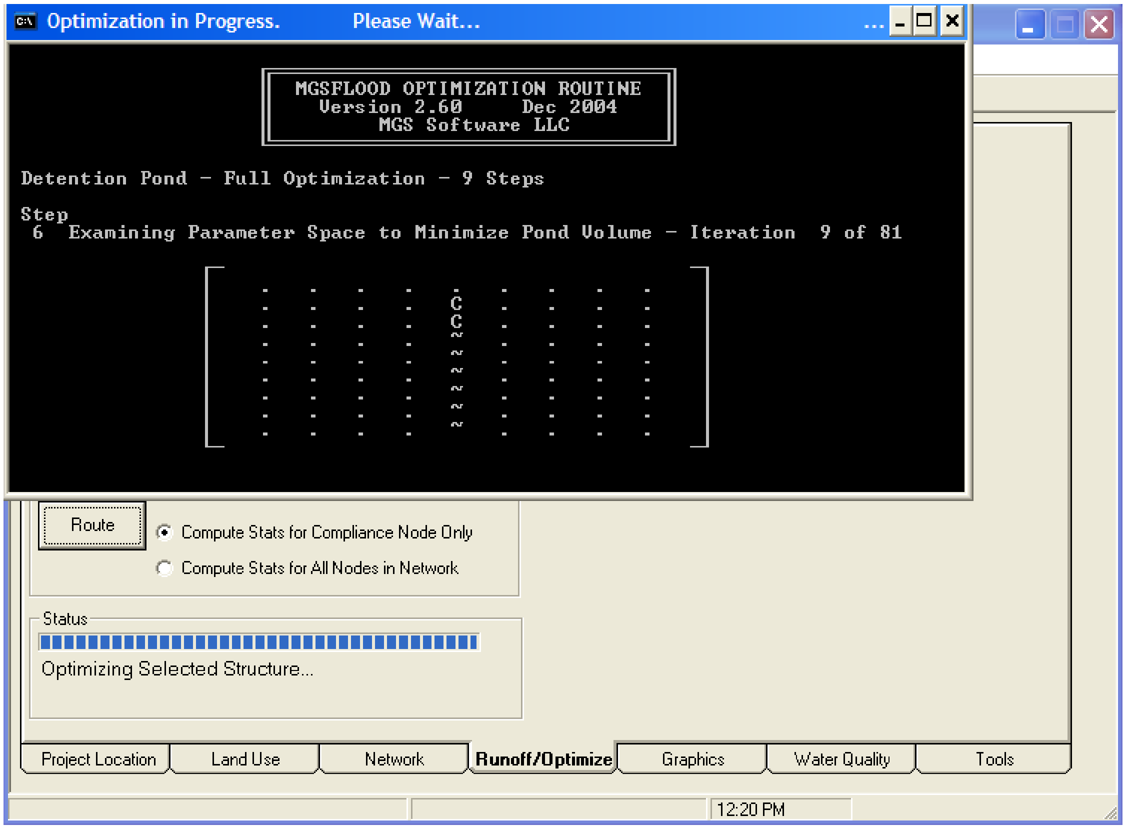
Two levels of optimization are available for detention pond sizing; Quick Optimization and Full Optimization. *Quick Optimization* determines a "ballpark" solution in a relatively short time (usually less than one minute). *Full Optimization* does an exhaustive search of potential solutions in searching for a configuration for the minimum pond size required to meet the flow duration standard. The full optimization routine usually converges on a solution in less than ten minutes (depending on the speed and memory of the computer).

For infiltration ponds, only *Quick Optimization* is available. Infiltration pond optimization is much less computationally demanding than detention pond optimization and the Quick Optimization routine typically produces an optimal pond design in a short period of time.

8.2.6 Running the Pond Optimization Routine

Optimization of selected ponds or infiltration trenches on the postdevelopment network will be performed when routing is performed by clicking the *Route* button on the *Runoff/Optimize* tab. The *Optimize Structure Indicated on Network Tab* check box must be selected for optimization to occur. Only one structure may be optimized per simulation run. To optimize multiple structures, start with the furthest upstream structure and optimize each structure working downstream.

When the Pond Optimization routine is executed, a second window opens that displays progress messages from the routine (Figure 8.8). If the *Full Optimization* option is checked when sizing a detention pond, a matrix will be displayed on the screen and filled with symbols indicating the progress of the routine. When the routine is finished, the pond size and outlet information is placed on *Structure Input* screen for the link being optimized replacing any previously entered information. The program then automatically computes and plots the pond performance duration statistics. If the resulting pond does not meet all of the duration design criteria, then manual edits to the pond design must be made. The procedure for making manual edits to a pond returned by the optimization is described in the next section.



```
        MGSFLOOD OPTIMIZATION ROUTINE
         Version 2.60      Dec 2004
              MGS Software LLC

Detention Pond - Full Optimization - 9 Steps

Step
 6  Examining Parameter Space to Minimize Pond Volume - Iteration  9 of 81

               [    .   .   .   .   .   .   .   .   .    ]
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```

● Compute Stats for Compliance Node Only
○ Compute Stats for All Nodes in Network

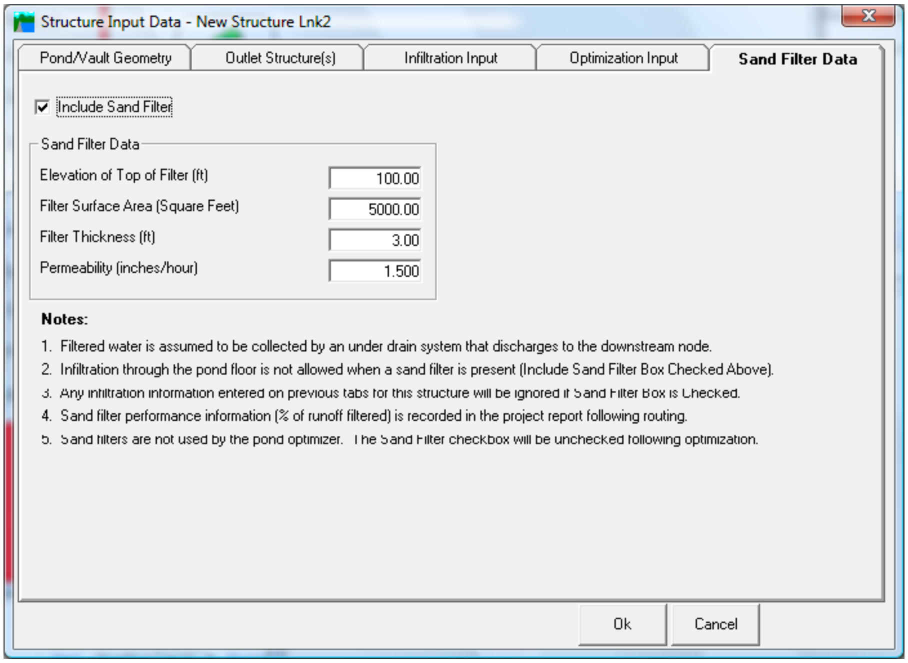
Status
Optimizing Selected Structure...

Figure 8.8 – MGSFlood Pond Optimization Status Screen

8.2.7 Sand Filter

A sand filter functions much like an infiltration pond except that instead of infiltrating into native soils, stormwater filters through a constructed sand bed with an underdrain system to remove pollutants. The underdrain system is assumed connected to the discharge conduit from the pond and flows from the sand filter are added to the total discharge from the pond. The program treats the sand filter as an additional structure associated with the stormwater pond. The filter surface area is used by the program to determine the rate of water infiltrated through the filter. The pond length and width entered on the Pond/Vault geometry are used to establish the pond storage volume. The pond bottom area may be larger than the sand filter area to allow placement of the sand filter in just a portion of the pond bottom.

To include a sand filter, check the *Include Sand Filter* box on the *Sand Filter Data* tab (Figure 8.9). The elevation of the top of the filter, filter surface area, thickness, and permeability are entered on the input screen (See Section I for more information regarding these parameters).



☑ Include Sand Filter

Sand Filter Data

Elevation of Top of Filter (ft)	100.00
Filter Surface Area (Square Feet)	5000.00
Filter Thickness (ft)	3.00
Permeability (inches/hour)	1.500

Notes:

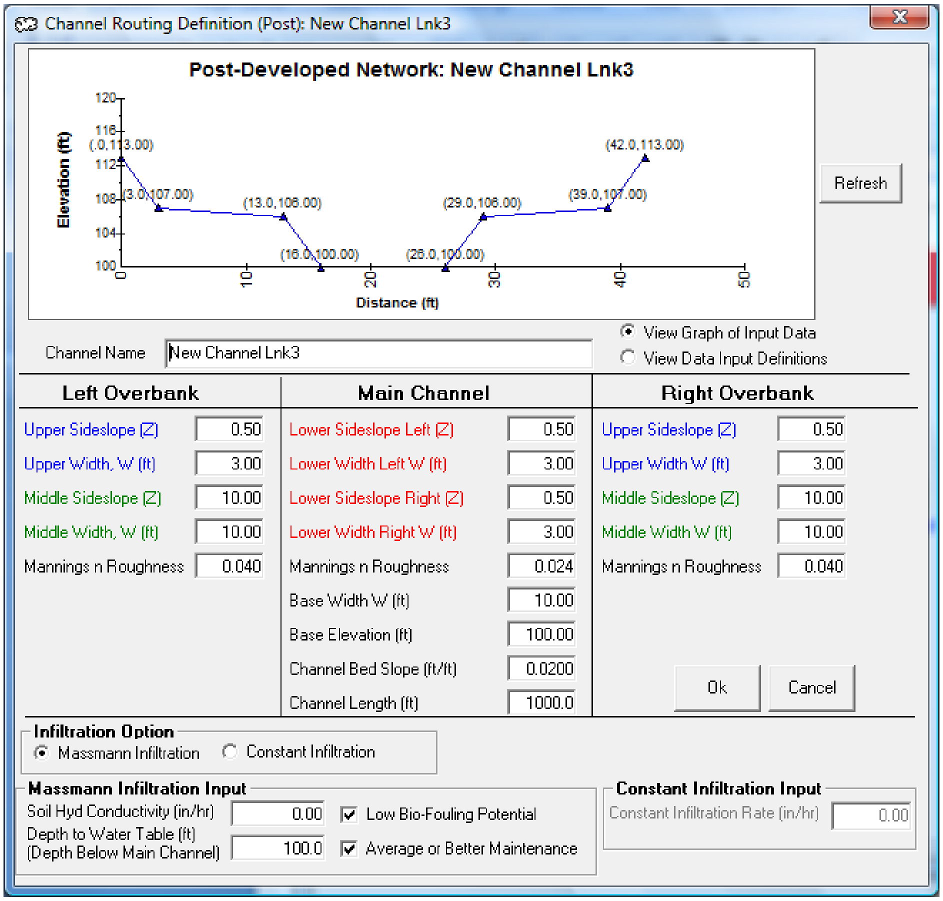
1. Filtered water is assumed to be collected by an under drain system that discharges to the downstream node.
2. Infiltration through the pond floor is not allowed when a sand filter is present (Include Sand Filter Box Checked Above).
3. Any infiltration information entered on previous tabs for this structure will be ignored if Sand Filter Box is Checked.
4. Sand filter performance information (% of runoff filtered) is recorded in the project report following routing.
5. Sand filters are not used by the pond optimizer. The Sand Filter checkbox will be unchecked following optimization.

Figure 8.9 – Sand Filter Input Screen

8.3 Channel Routing

Channel routing is performed using a Modified Puls routing routine developed by the US Army Corps of Engineers for the HEC-1[25] flood hydrograph package. The user inputs the left overbank, main channel and right overbank channel cross sectional geometry, roughness, slope, and channel length (Figure 8.10). The program develops an elevation-volume-discharge rating table assuming normal depth at each discharge level and computes discharge according to the Manning Equation[4]. This rating table is then utilized by the Modified Puls routing routine to route flows through the link.

MGSFlood includes two options for simulating infiltration; Massmann[30] equations and fixed infiltration. The Massmann equations are based on field observations of infiltration ponds in western Washington. This infiltration approach accounts for the proximity of the channel to the regional groundwater table, and the potential for soil clogging and fouling. Inputs include; soil hydraulic conductivity (inches/hour or feet/day), depth to the regional water table (ft) whether bio-fouling potential is low, and whether average or better maintenance is performed. Infiltrated moisture is lost from the system and does not contribute to the discharge at the downstream end of the channel



● View Graph of Input Data
○ View Data Input Definitions

Channel Name: New Channel Lnk3

Left Overbank		Main Channel		Right Overbank	
Upper Sideslope (Z)	0.50	Lower Sideslope Left (Z)	0.50	Upper Sideslope (Z)	0.50
Upper Width, W (ft)	3.00	Lower Width Left W (ft)	3.00	Upper Width W (ft)	3.00
Middle Sideslope (Z)	10.00	Lower Sideslope Right (Z)	0.50	Middle Sideslope (Z)	10.00
Middle Width, W (ft)	10.00	Lower Width Right W (ft)	3.00	Middle Width W (ft)	10.00
Mannings n Roughness	0.040	Mannings n Roughness	0.024	Mannings n Roughness	0.040
		Base Width W (ft)	10.00		
		Base Elevation (ft)	100.00		
		Channel Bed Slope (ft/ft)	0.0200		
		Channel Length (ft)	1000.0		

Infiltration Option
● Massmann Infiltration ○ Constant Infiltration

Massmann Infiltration Input
Soil Hyd Conductivity (in/hr) 0.00
Depth to Water Table (ft) (Depth Below Main Channel) 100.0
☑ Low Bio-Fouling Potential
☑ Average or Better Maintenance

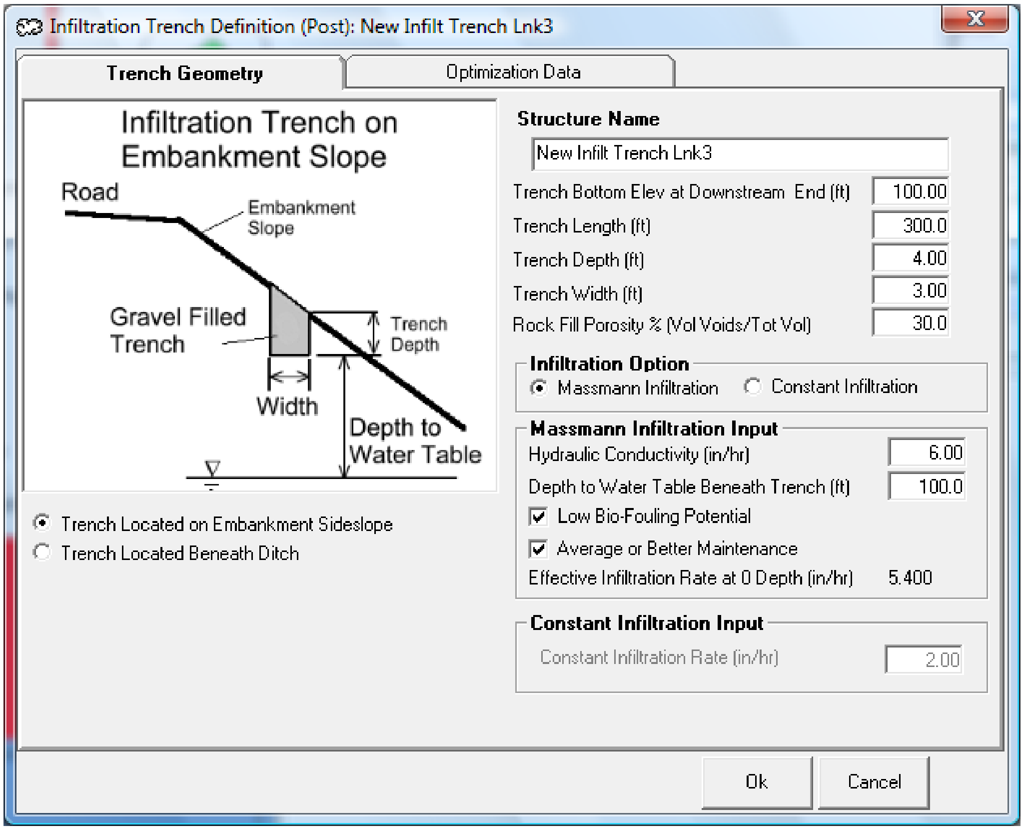
Constant Infiltration Input
Constant Infiltration Rate (in/hr) 0.00

Figure 8.10 – User Input for defining Open Channel Routing

8.4 Infiltration Trench

MGSFlood includes two options for simulating infiltration; Massmann[30] equations and fixed infiltration. The Massmann equations are based on field observations of infiltration ponds in western Washington. This approach accounts for the side slope geometry of the structure, the aspect (length to width ratio), the proximity to the regional groundwater table, and the potential for soil clogging and fouling. Inputs include; Soil Hydraulic Conductivity (inches/hour), Depth to the Regional Water Table (ft), whether bio-fouling potential is low, and whether average or better maintenance is performed. Infiltrated moisture is lost from the system and does not contribute to the discharge rate at the downstream end of the link.

The program routes flow for two types of infiltration trenches as shown in Figures 8.11 and 8.12; a trench located on the embankment side slope, or an infiltration trench located at the base of the embankment.



Infiltration Trench Definition (Post): New Infilt Trench Lnk3

Structure Name: New Infilt Trench Lnk3

Trench Bottom Elev at Downstream End (ft): 100.00

Trench Length (ft): 300.0

Trench Depth (ft): 4.00

Trench Width (ft): 3.00

Rock Fill Porosity % (Vol Voids/Tot Vol): 30.0

● Trench Located on Embankment Sideslope

○ Trench Located Beneath Ditch

Infiltration Option

● Massmann Infiltration ○ Constant Infiltration

Massmann Infiltration Input

Hydraulic Conductivity (in/hr): 6.00

Depth to Water Table Beneath Trench (ft): 100.0

☑ Low Bio-Fouling Potential

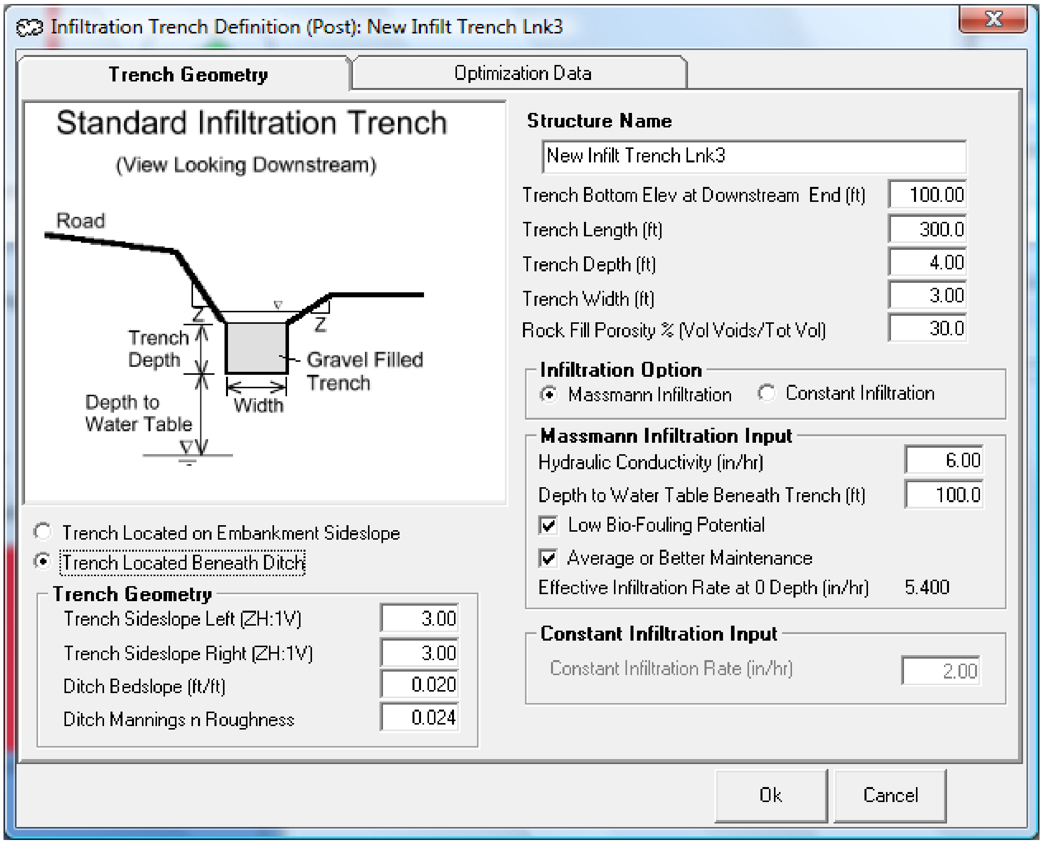
☑ Average or Better Maintenance

Effective Infiltration Rate at 0 Depth (in/hr): 5.400

Constant Infiltration Input

Constant Infiltration Rate (in/hr): 2.00

Figure 8.11– Infiltration Trench Located on Embankment Slope Option



Infiltration Trench Definition (Post): New Infilt Trench Lnk3

Trench Geometry | Optimization Data

Structure Name

New Infilt Trench Lnk3

Trench Bottom Elev at Downstream End (ft)	100.00
Trench Length (ft)	300.0
Trench Depth (ft)	4.00
Trench Width (ft)	3.00
Rock Fill Porosity % (Vol Voids/Tot Vol)	30.0

Infiltration Option

● Massmann Infiltration ○ Constant Infiltration

Massmann Infiltration Input

Hydraulic Conductivity (in/hr)	6.00
Depth to Water Table Beneath Trench (ft)	100.0

☑ Low Bio-Fouling Potential

☑ Average or Better Maintenance

Effective Infiltration Rate at 0 Depth (in/hr) 5.400

Constant Infiltration Input

Constant Infiltration Rate (in/hr) 2.00

○ Trench Located on Embankment Sideslope

● Trench Located Beneath Ditch

Trench Geometry

Trench Sideslope Left (ZH:1V)	3.00
Trench Sideslope Right (ZH:1V)	3.00
Ditch Bedslope (ft/ft)	0.020
Ditch Mannings n Roughness	0.024

Figure 8.12 – Standard Infiltration Trench Option

Soil Hydraulic Conductivity (in/hr) – Is the saturated hydraulic conductivity of the soil beneath the infiltration trench in inches per hour according to Darcy's Equation. It can be estimated using regression equations that use grain size distribution as input (Massmann[30]) or from literature (e.g. Freeze and Cherry[31], Fetter[32]).

Depth to Regional Groundwater Table (ft) – Represents the depth from the bottom of the trench to the regional groundwater table or the first low-permeability layer. For shallow groundwater sites, groundwater mounding reduces the hydraulic gradient and the infiltration rate is significantly less than the saturated hydraulic conductivity. For deep groundwater sites where the effects of mounding will be small, the gradient will not typically be reduced by infiltration from the facility. Increasing the depth to groundwater greater than 100 feet ceases to have an influence on pond infiltration according to this approach.

Bio-fouling Potential – Bio-fouling occurs from organic material blanketing the soil surface and reducing the infiltration rate. Bio-fouling is more likely to occur if the trench is located beneath trees and other vegetation or in shaded locations.

Maintenance – Siltation is more likely to occur if there is not sufficient pre-treatment of the storm water or in locations where the drainage basin is prone to erosion because of recent land disturbances or steep slopes. The user should consider the potential for siltation and the level of maintenance when determining the effects of maintenance on pond infiltration performance.

8.5 Infiltration Trench Located on Embankment Slope

A trench is constructed along the roadway embankment and filled with gravel (Figure 8.11). Runoff from the roadway is directed to the gravel trench where it percolates through the gravel and infiltrates through the trench bottom. When the runoff rate exceeds the infiltration capacity, the gravel saturates from the bottom up with the voids in the gravel providing runoff storage, similar to a detention pond. If the storm is sufficiently large, the saturation will reach the ground surface and runoff from the road will pass over the gravel surface and continue down the embankment. Runoff not infiltrated in the trench is passed to the downstream link without routing.

It should be noted that the saturated hydraulic conductivity of the embankment fill will likely be different from the native material beneath the fill. The hydraulic conductivity estimates of the different layers can be combined using the harmonic mean (Massmann[30]):

$$K_{Equiv} = \frac{d}{\sum \frac{d_i}{K_i}} \qquad 8.1$$

Where: K_{Equiv} is the equivalent hydraulic conductivity,
d is the depth of the soil column above the regional groundwater table or limiting permeability layer,
d_i is the thickness of layer i,
K_i is the hydraulic conductivity of layer i

Note that the saturated hydraulic conductivity of the gravel in the trench is not included in Equation 8.1. For sites with very deep groundwater tables (>100 feet), it is recommended that the total depth of the soil column in Equation 8.1 be limited to 20 times the trench depth.

8.6 Standard Infiltration Trench

The standard infiltration trench would be constructed at the base of the roadway embankment and would receive runoff from the adjacent roadway or from an upstream ditch. Runoff from the roadway is directed to the gravel trench where it percolates through the gravel and infiltrates through the trench bottom. When the runoff rate exceeds the infiltration capacity of the soil, the gravel saturates from the bottom up with the voids in the gravel providing runoff storage, similar to a detention pond. If the storm is sufficiently large, the saturation will reach the ground surface and runoff will occur down the ditch along the gravel surface. The program routes flow along the gravel surface to the downstream link according to the Manning Equation[4].

The infiltration trench routine may also be used to simulate a natural stream channel with infiltration through the channel bottom. The geometry of the channel is defined as a trapezoidal section and depth of gravel is input as zero.

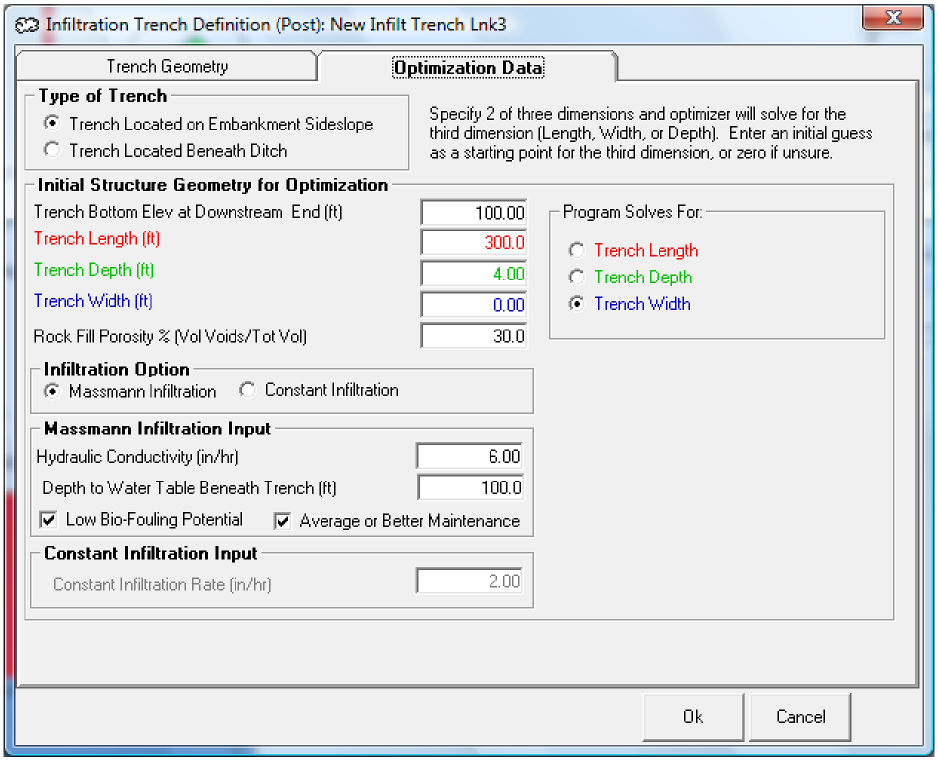
8.7 Automatic Infiltration Trench Sizing Routine

The automatic pond sizing optimization routine in the MGSFlood will automatically determine the size of infiltration trench required to meet the goals of the Ecology flow duration standard. The user inputs two of the three trench dimensions (length, width, or depth) and the optimizer solves for the third dimension. The input supplied by the user includes:

- The type of infiltration trench to be sized (Embankment Slope or Standard),
- The trench bottom elevation at the downstream end,
- Two of Three Trench Dimensions (Length, Width, or Depth)
- Rock fill porosity,
- Depth to water table,
- Saturated hydraulic conductivity of soil beneath trench.

The optimization routine uses the information listed above to establish the geometric relationships for the trench configuration. The program establishes a parameter space of possible solutions by varying the bottom width. The program then routes the developed runoff timeseries through the trench and seeks to find a solution that provides the minimum trench size to meet the duration design standard.

These variables are entered on the Optimization tab (Figure 8.13). The user must specify which trench dimension (length, width, or depth) is to be optimized. If a value is entered for the dimension to be optimized, the program will use this as a starting point for the optimization run. A value of zero is also acceptable. The Optimize Box is checked for the link to be optimized on the *Network* tab. The link will automatically be optimized when routing is performed on the *Runoff/Optimize* tab.



Type of Trench

● Trench Located on Embankment Sideslope
○ Trench Located Beneath Ditch

Specify 2 of three dimensions and optimizer will solve for the third dimension (Length, Width, or Depth). Enter an initial guess as a starting point for the third dimension, or zero if unsure.

Initial Structure Geometry for Optimization

Trench Bottom Elev at Downstream End (ft)	100.00
Trench Length (ft)	300.0
Trench Depth (ft)	4.00
Trench Width (ft)	0.00
Rock Fill Porosity % (Vol Voids/Tot Vol)	30.0

Program Solves For:

○ Trench Length
○ Trench Depth
● Trench Width

Infiltration Option

● Massmann Infiltration ○ Constant Infiltration

Massmann Infiltration Input

Hydraulic Conductivity (in/hr)	6.00
Depth to Water Table Beneath Trench (ft)	100.0

☑ Low Bio-Fouling Potential ☑ Average or Better Maintenance

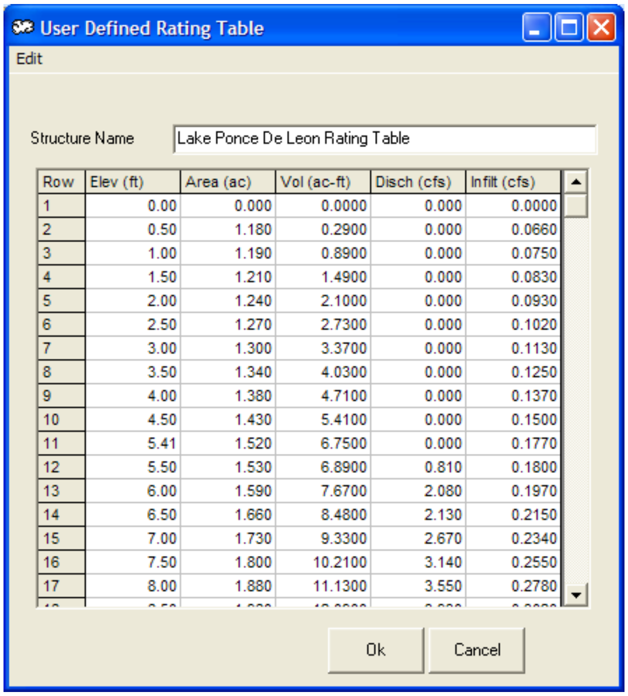
Constant Infiltration Input

Constant Infiltration Rate (in/hr)	2.00

Figure 8.13 – Optimization Input Screen for Infiltration Trenches

8.8 User Defined Rating Table

Structure hydraulics may be specified using a user-defined stage-surface area-volume-discharge rating table (Figure 8.14). The pond storage (acre-feet), surface area (acres), discharge (cfs), and infiltration discharge (cfs) are computed by the user and entered in the table. Information may be copied from an external spreadsheet program and pasted into the input table using the Windows Clipboard utility.



Structure Name: Lake Ponce De Leon Rating Table

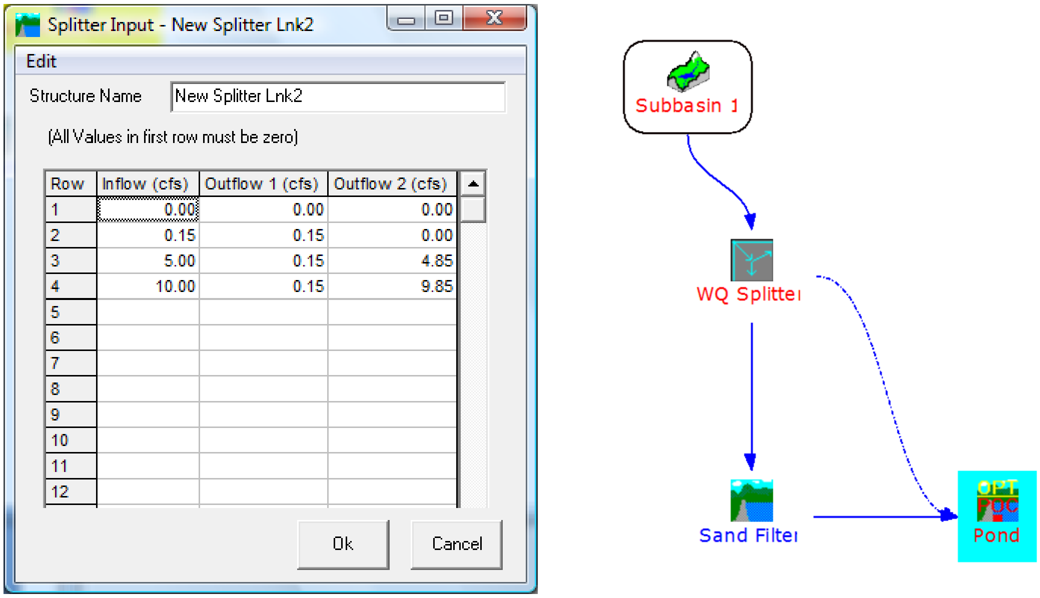
Row	Elev (ft)	Area (ac)	Vol (ac-ft)	Disch (cfs)	Infilt (cfs)
1	0.00	0.000	0.0000	0.000	0.0000
2	0.50	1.180	0.2900	0.000	0.0660
3	1.00	1.190	0.8900	0.000	0.0750
4	1.50	1.210	1.4900	0.000	0.0830
5	2.00	1.240	2.1000	0.000	0.0930
6	2.50	1.270	2.7300	0.000	0.1020
7	3.00	1.300	3.3700	0.000	0.1130
8	3.50	1.340	4.0300	0.000	0.1250
9	4.00	1.380	4.7100	0.000	0.1370
10	4.50	1.430	5.4100	0.000	0.1500
11	5.41	1.520	6.7500	0.000	0.1770
12	5.50	1.530	6.8900	0.810	0.1800
13	6.00	1.590	7.6700	2.080	0.1970
14	6.50	1.660	8.4800	2.130	0.2150
15	7.00	1.730	9.3300	2.670	0.2340
16	7.50	1.800	10.2100	3.140	0.2550
17	8.00	1.880	11.1300	3.550	0.2780

Figure 8.14 – Input Screen for User Defined Rating Table

8.9 Flow Splitter Link

Flow splitter structures divert a portion of the flow at the splitter link inflow to a second link. Input consists of a table that specifies the inflow to the splitter link and the discharge to the splitter link outflow (Outflow 1) and the downstream link (Outflow 2). The program evaluates the inflow to the structure at each time step and determines the outflow to each downstream link by interpolation between rows of the table. The user should enter values in the table that extend beyond the maximum expected inflow to the link.

An example is shown in Figure 8.15 where a flow splitter is used to divert flows in excess of the water quality treatment discharge (0.15 cfs) around a sand filter link. In this example, the amount of runoff discharged to the sand filter structure and the size of the filter would be determined iteratively until the desired runoff percentage was treated by the sand filter (typically 91% of the total runoff volume).



Structure Name: New Splitter Lnk2

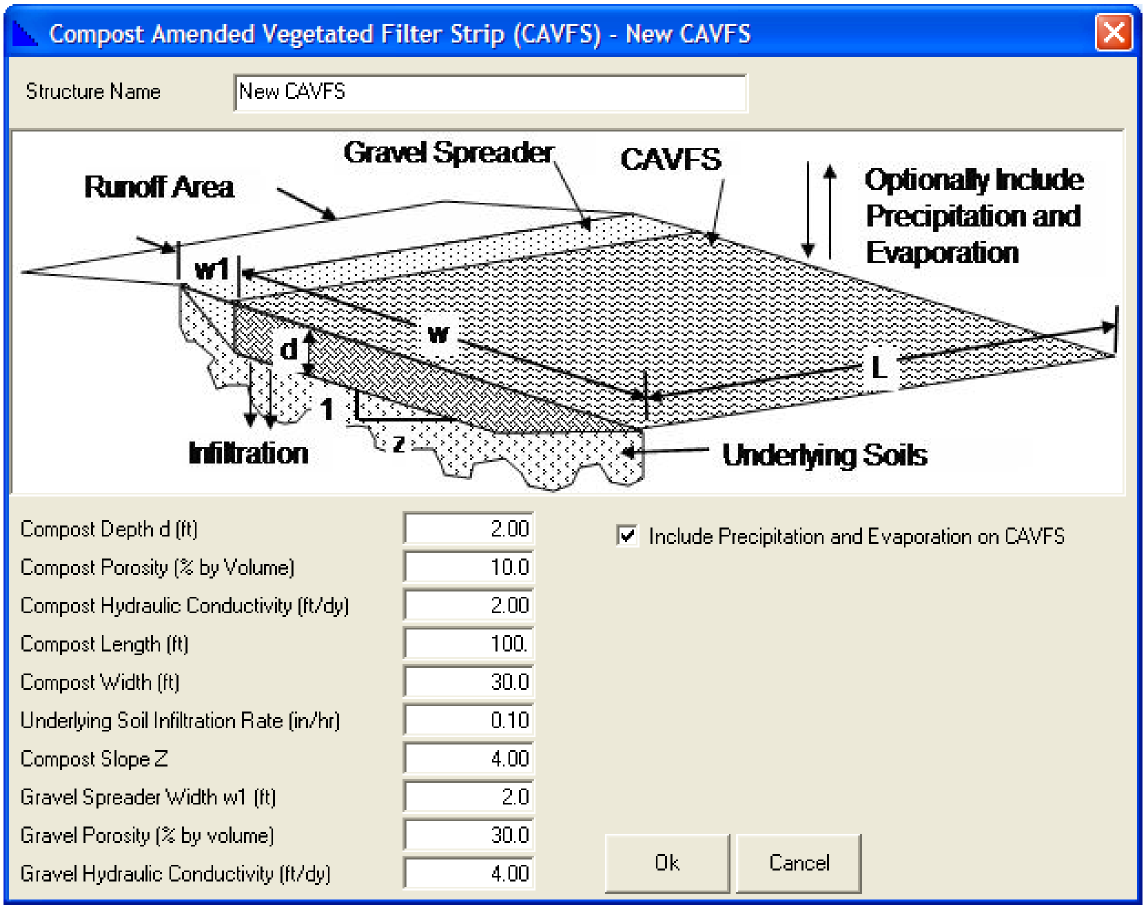
(All Values in first row must be zero)

Row	Inflow (cfs)	Outflow 1 (cfs)	Outflow 2 (cfs)
1	0.00	0.00	0.00
2	0.15	0.15	0.00
3	5.00	0.15	4.85
4	10.00	0.15	9.85
5			
6			
7			
8			
9			
10			
11			
12			

Figure 8.15– Example Flow Splitter Input Table

8.10 Compost Amended Vegetated Filter Strip (CAVFS)

Flow through the compost soil mix along the slope is simulated using Darcy's Equation. Note that the width dimension corresponds to the CAVFS width along the slope. Infiltration is accounted for using a constant infiltration rate into the underlying soils. During large storms, the voids in the CAVFS may become full (the CAVFS saturated) and runoff is simulated as overflow down the surface of the CAVFS. Runoff volume filtered by the CAVFS, total volume infiltrated, and total volume flowing over the CAVFS surface are listed in the project report.



Structure Name: New CAVFS

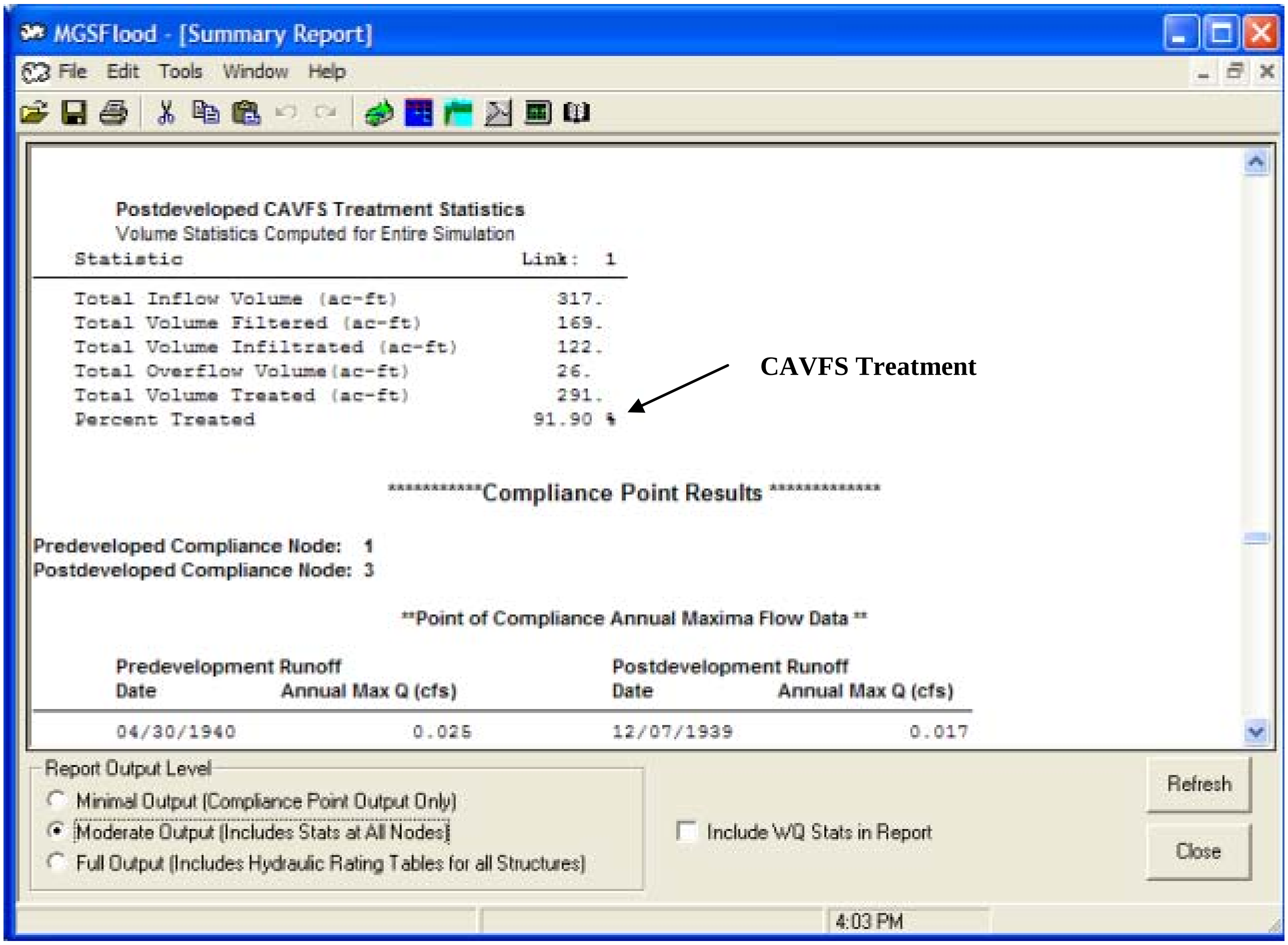
Compost Depth d (ft)	2.00
Compost Porosity (% by Volume)	10.0
Compost Hydraulic Conductivity (ft/dy)	2.00
Compost Length (ft)	100.
Compost Width (ft)	30.0
Underlying Soil Infiltration Rate (in/hr)	0.10
Compost Slope Z	4.00
Gravel Spreader Width w1 (ft)	2.0
Gravel Porosity (% by volume)	30.0
Gravel Hydraulic Conductivity (ft/dy)	4.00

☑ Include Precipitation and Evaporation on CAVFS

Figure 8.16 – CAVFS Input Screen

Precipitation and evapotranspiration may optionally be applied to the CAVFS. <u>If precipitation and evapotranspiration are applied in the CAVFS link, do not include the area of the CAVFS in the Subbasin Area input.</u>

The size of CAVFS required for water quality treatment is determined via a trial and error procedure. Trial CAVFS dimensions are entered under the Link Definition. Runoff is then routed by clicking the Route button on the Runoff/Optimize tab. When routing is completed, view the project report and locate the volume treated by the CAVFS. The runoff treated by the CAVFS is the sum of the filtered and infiltrated water and should be greater than or equal to 91 percent (Figure 8.17).



```
        Postdeveloped CAVFS Treatment Statistics
        Volume Statistics Computed for Entire Simulation
    Statistic                                   Link:  1
  ------------------------------------------------------
    Total Inflow Volume (ac-ft)                  317.
    Total Volume Filtered (ac-ft)                169.
    Total Volume Infiltrated (ac-ft)             122.
    Total Overflow Volume(ac-ft)                  26.
    Total Volume Treated (ac-ft)                 291.
    Percent Treated                             91.90 %


                 ***********Compliance Point Results *************

Predeveloped Compliance Node:   1
Postdeveloped Compliance Node:  3

                  **Point of Compliance Annual Maxima Flow Data **

        Predevelopment Runoff                     Postdevelopment Runoff
        Date           Annual Max Q (cfs)         Date           Annual Max Q (cfs)
  ------------------------------------------------------------------------------
        04/30/1940              0.025             12/07/1939              0.017
```

CAVFS Treatment

Report Output Level

○ Minimal Output (Compliance Point Output Only)
● Moderate Output (Includes Stats at All Nodes)
○ Full Output (Includes Hydraulic Rating Tables for all Structures)

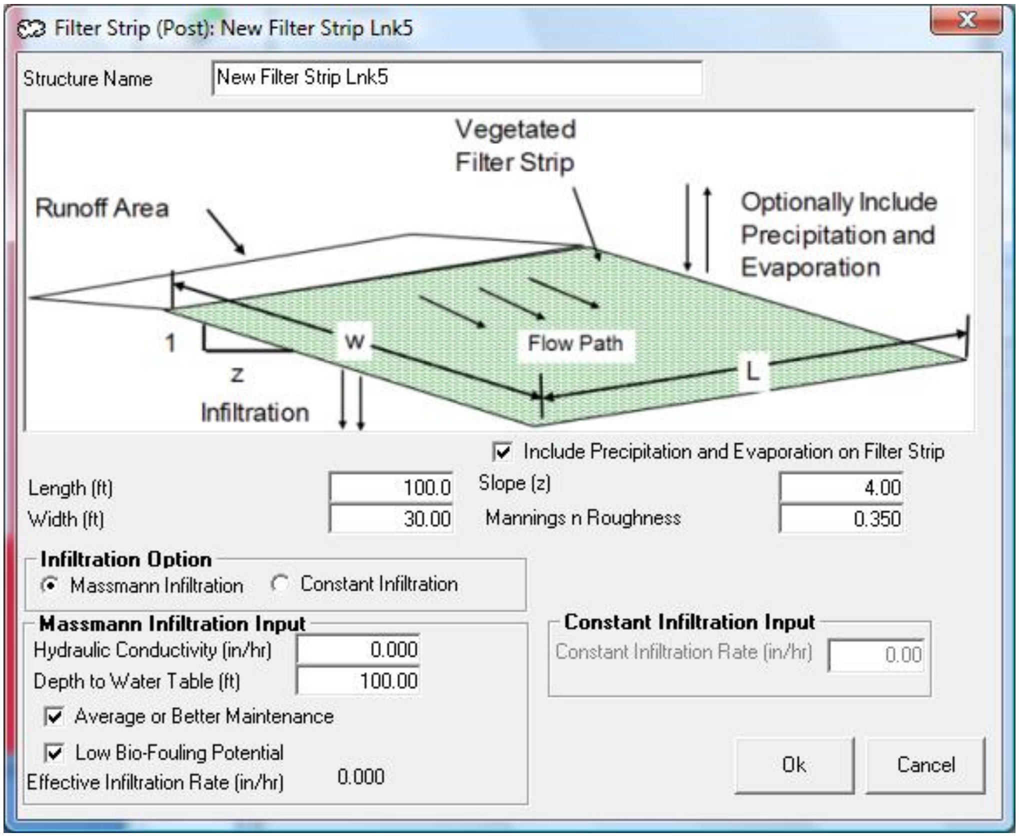
☐ Include WQ Stats in Report

Figure 8.17- Project Report Showing Performance of CAVFS Designed to Meet the 91 Percent Water Quality Treatment Goal

8.11 Filter Strip

Routing across filter strips is performed using a Modified Puls routing routine. The user inputs the filter geometry and the program develops an elevation-volume-discharge rating table assuming normal depth at each discharge level and computes discharge according to the Manning Equation. This rating table is then utilized by the Modified Puls routing routine to route flows through the link. Infiltration can either be simulated using a constant rate or by using Massmann’s equations.

Precipitation and evapotranspiration may optionally be applied to the filter strip. If precipitation and evapotranspiration are applied in the Filter Strip link, do not include the area of the filter strip in the Subbasin Area input.



Structure Name: New Filter Strip Lnk5

☑ Include Precipitation and Evaporation on Filter Strip

Length (ft): 100.0
Width (ft): 30.00
Slope (z): 4.00
Mannings n Roughness: 0.350

Infiltration Option
● Massmann Infiltration ○ Constant Infiltration

Massmann Infiltration Input
Hydraulic Conductivity (in/hr): 0.000
Depth to Water Table (ft): 100.00
☑ Average or Better Maintenance
☑ Low Bio-Fouling Potential
Effective Infiltration Rate (in/hr): 0.000

Constant Infiltration Input
Constant Infiltration Rate (in/hr): 0.00

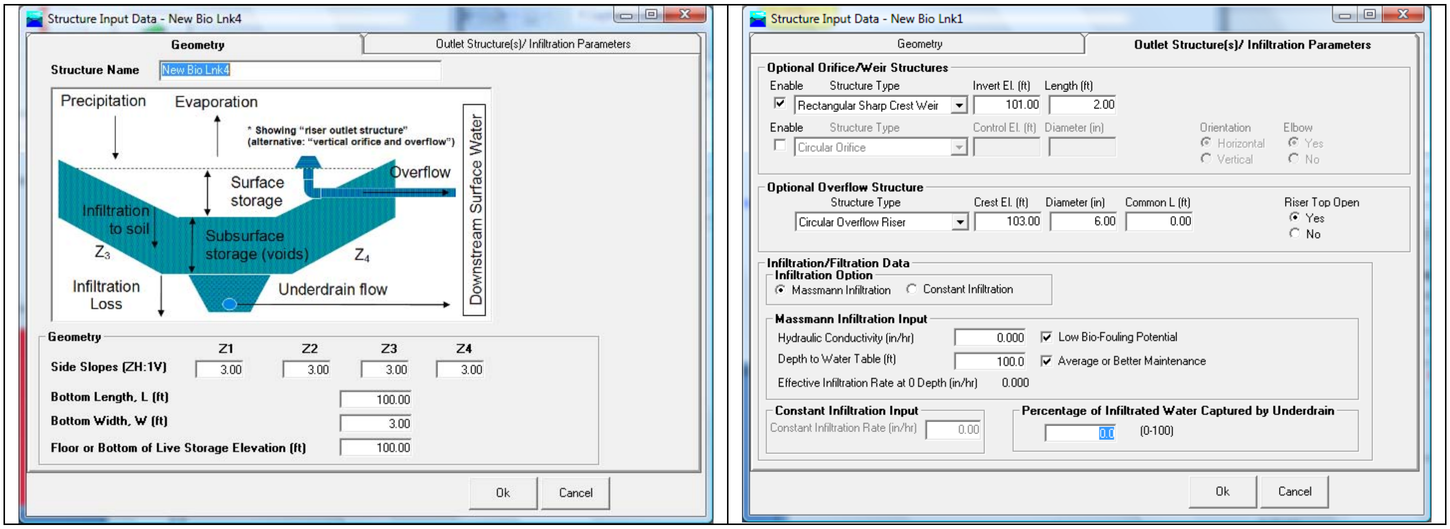
Figure 8.18- Filter Strip Input Screen

8.12 Bioretention Facility

MGSFlood simulates the following hydrologic features of bioretention facilities; surface detention, surface outflow, infiltration, and return flow from an underdrain (Figure 8.19). The underdrain return flow is entered as a percentage of the infiltrated moisture. This percentage is then added to the link outflow. Infiltration can either be simulated using a constant rate or by using Massmann's equations.

A variety of surface detention outflow structures may be specified and include orifices, weirs, and riser structures. Infiltration may be specified as a fixed value or using Massmann's method. Underdrains are simulated by entering a percentage of the infiltrated moisture that is returned to the system downstream of the facility. A value 100 means that all infiltrated moisture is captured by an underdrain. A value of 0 means that no underdrain is present.

Precipitation and evapotranspiration are applied to the facility so the area occupied by the bioretention facility should not be included in the Subbasin Area input.



Structure Input Data - New Bio Lnk4

Geometry | Outlet Structure(s)/ Infiltration Parameters

Structure Name New Bio Lnk4

Geometry

	Z1	Z2	Z3	Z4
Side Slopes (ZH:1V)	3.00	3.00	3.00	3.00

Bottom Length, L (ft) 100.00
Bottom Width, W (ft) 3.00
Floor or Bottom of Live Storage Elevation (ft) 100.00

Structure Input Data - New Bio Lnk1

Geometry | **Outlet Structure(s)/ Infiltration Parameters**

Optional Orifice/Weir Structures

Enable	Structure Type	Invert El. (ft)	Length (ft)
☑	Rectangular Sharp Crest Weir	101.00	2.00

Enable	Structure Type	Control El. (ft)	Diameter (in)	Orientation	Elbow
☐	Circular Orifice			● Horizontal ○ Vertical	● Yes ○ No

Optional Overflow Structure

Structure Type	Crest El. (ft)	Diameter (in)	Common L (ft)	Riser Top Open
Circular Overflow Riser	103.00	6.00	0.00	● Yes ○ No

Infiltration/Filtration Data

Infiltration Option
● Massmann Infiltration ○ Constant Infiltration

Massmann Infiltration Input
Hydraulic Conductivity (in/hr) 0.000 ☑ Low Bio-Fouling Potential
Depth to Water Table (ft) 100.0 ☑ Average or Better Maintenance
Effective Infiltration Rate at 0 Depth (in/hr) 0.000

Constant Infiltration Input
Constant Infiltration Rate (in/hr) 0.00

Percentage of Infiltrated Water Captured by Underdrain
0.0 (0-100)

Figure 8.19 – Bioretention Facility Input Screens

9 Simulate Tab

After defining the predeveloped and postdeveloped watershed layouts, rainfall runoff simulation is performed from the *Simulate* tab (Figure 9.1). MGSFlood computes runoff using the impervious (IMPLND) and pervious (PERLND) land segment subroutines from the HSPF model. Precipitation and evaporation data are read from the MGSRegion.mdb file and runoff is computed for predevelopment and postdevelopment conditions. Routing through the predeveloped and postdeveloped networks is then performed followed by calculation of statistics with results displayed on the *Graphics* tab.

9.1 Specify Time Period for which Runoff is to be Computed

Runoff computations are performed on a *water year* basis, that is, they begin on October 1 and end on September 30. The user can define a time period shorter than the full record for preliminary design computations, although the full period of record should be used for the final design to provide the most accurate streamflow computations.

9.2 Time Step Guidance

Extended precipitation time series are stored at a 5-minute time step, which allows the user to select the computational time step most appropriate for the feature being analyzed or designed. For the design of project elements dependent on runoff volume, such as detention facilities, a 1-hour time step has been the accepted standard of practice. Conveyance facilities upstream of detention facilities can be sensitive to short duration bursts of rainfall that can produce high peak discharge rates. A 5-minute to 15-minute time step is appropriate for design of conveyance structures depending on the time of concentration of the basin being analyzed. Ideally, the time-step should be on the order of one-fourth to one-third of the time of concentration. For very small basins with very short time-of-concentration, standard practice has been to use a 5-minute time-step.

Table 9.1 lists recommended time steps for the design or analysis of various hydrologic features. This table is included for guidance purposes and where it conflicts with local stormwater guidelines, the local stormwater guidelines should take precedence.

For projects where multiple facilities are to be designed requiring different time steps, it is recommended that multiple MGSFlood data files be created, one for each time step used in the analysis.

Station Data precipitation (used for areas outside of the extended precipitation time series coverage) are stored at a 1-hour time step and can only be simulated using a 1-hour computational time step. Peak discharge rates computed using MGSFlood with a 1-hour time step should not be used for the design of conveyance facilities upstream of detention. They may be appropriate for facilities downstream of detention, provided that the detention facility does not overflow at a recurrence interval more common than the conveyance design recurrence interval.

Table 9.1 – Recommended Time Step for Various Analyses

	Computational Time Step	
Task	**Extended Time Series**	**Station Data**
Detention Sizing	1-Hour	1-Hour
WQ Wet Pool Volume Sizing	15-minutes or 1-hour	1-Hour
WQ Rate Sizing	15-minutes	1-Hour (Program uses Adjustment Factors to compute 15-minute rate)
CAVFS Sizing	15-minutes	1-Hour acceptable
Bioretention Facility Sizing	15-minutes	1-Hour acceptable
Conveyance Sizing Upstream of Detention	5-minutes to 15-minutes	(Cannot Use MGSFlood, Use Single Event Model or Rational Method)

9.3 Variable Time Step Algorithm

MGSFlood utilizes a variable time step algorithm when computing runoff and routing time series using the extended precipitation time series. The algorithm automatically uses a longer computational time step (up to 6-hours) during dry periods without sacrificing computational accuracy. During storms, the time step reverts back to that selected by the user in the *Computational Time Step* selection box. The Variable Time Step Algorithm greatly reduces the simulation time when using the Extended Precipitation time series, especially with time steps less than 1-hour.

9.4 Predevelopment/Post Development Area Summary

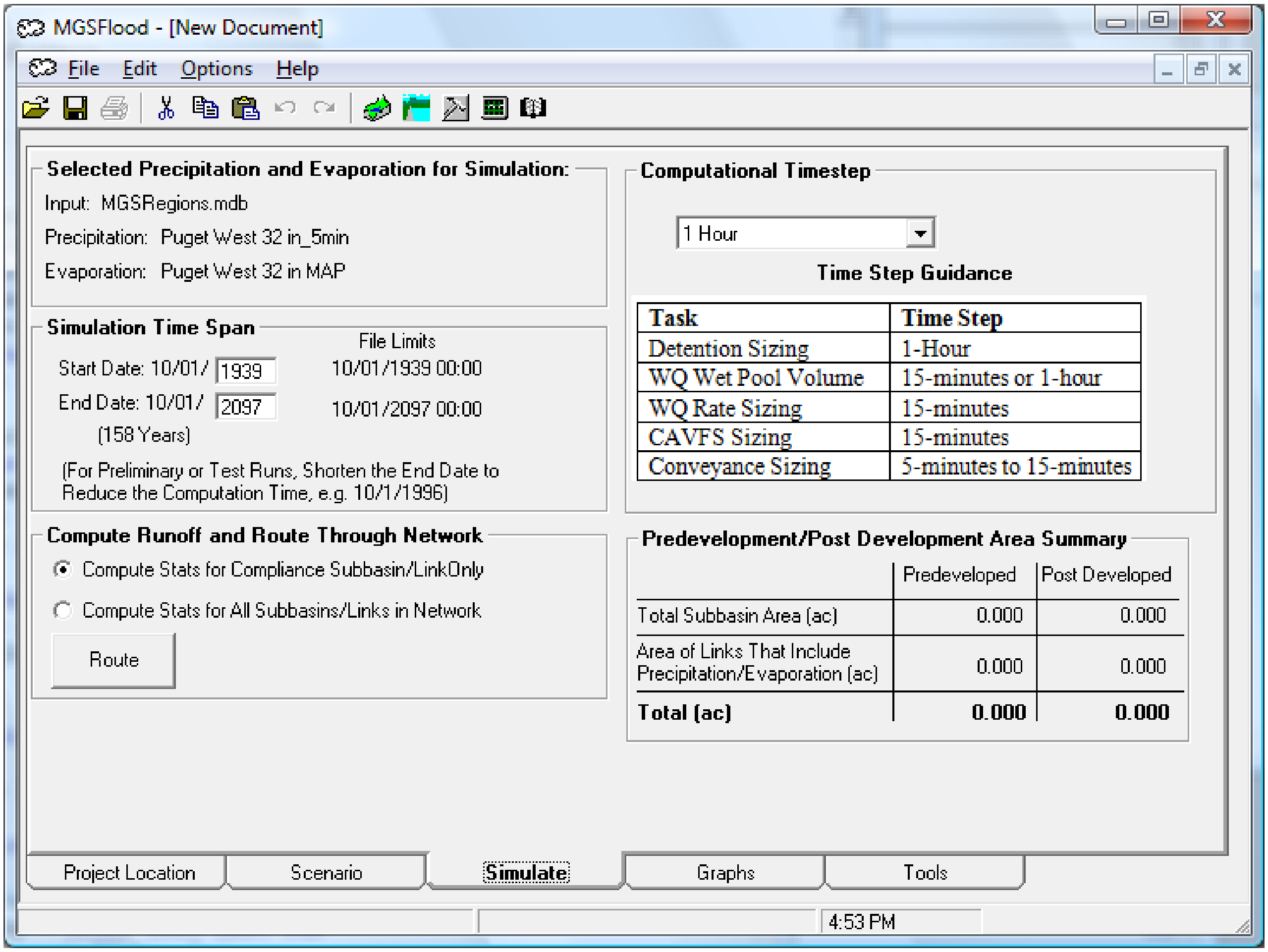
The total area (Subbasin and Links with Precipitation applied) is summarized on the Runoff/Optimize tab. This provides the user with an overall check of the total land use in the model before beginning the simulation. Note that the area computed for CAVFS and Filter Strips is the projection of their area onto the horizontal plane. This is necessary because of the slopes of these structures. It is not required that the predeveloped and post developed areas match before starting a simulation.

9.5 Compute Statistics Option Buttons

Two options are available for calculating statistics in the project. Statistics may be computed for the compliance point only (lower output level) or at all subbasins and links (higher output level). Water surface elevation statistics are also computed for the compliance link (lower output level) or all links if the higher output level is selected.

9.6 Route Button

Clicking the *Route* button causes runoff to be computed for the period selected and runoff to be saved in the direct access file for all subbasins and links. The program then reads the runoff stored in this file for all future pond sizing calculations for the project. If the land use is subsequently changed, the runoff is automatically recomputed. If a link is set for optimization, the program will prompt before starting the optimization run.



Selected Precipitation and Evaporation for Simulation:

Input: MGSRegions.mdb
Precipitation: Puget West 32 in_5min
Evaporation: Puget West 32 in MAP

Simulation Time Span

File Limits
Start Date: 10/01/ 1939 — 10/01/1939 00:00
End Date: 10/01/ 2097 — 10/01/2097 00:00
(158 Years)

(For Preliminary or Test Runs, Shorten the End Date to Reduce the Computation Time, e.g. 10/1/1996)

Compute Runoff and Route Through Network

● Compute Stats for Compliance Subbasin/LinkOnly
○ Compute Stats for All Subbasins/Links in Network

Computational Timestep

1 Hour

Time Step Guidance

Task	Time Step
Detention Sizing	1-Hour
WQ Wet Pool Volume	15-minutes or 1-hour
WQ Rate Sizing	15-minutes
CAVFS Sizing	15-minutes
Conveyance Sizing	5-minutes to 15-minutes

Predevelopment/Post Development Area Summary

	Predeveloped	Post Developed
Total Subbasin Area (ac)	0.000	0.000
Area of Links That Include Precipitation/Evaporation (ac)	0.000	0.000
Total (ac)	**0.000**	**0.000**

Figure 9.1 – Simulate Input Tab

9.7 Manual Editing of Pond Configuration Obtained from the Optimization Routine

To manually edit the configuration of a pond or infiltration trench determined by the *Optimization* routine, toggle the optimizer off by right clicking on the optimized link icon and select *Use Optimizer to Size this Structure (Toggle On/Off)*. This will route flows without rerunning the optimization routine. Guidelines for manually adjusting the outlet works and pond geometry to achieve compliance with the flow duration standard are listed in Section 19 of Part I.

10 Graphs Tab

The *Graphs* tab is used for plotting runoff statistics for selected subbasins and links, plotting the performance of a stormwater detention facilities or plotting hydrographs.

The type of graph to be plotted is determined by the *Plot Type* option buttons;

- Flood Frequency,
- Flow Duration,
- Water Surface Elevation (WSEL) Frequency in ponds,
- Hydrographs.

The subbasin or link to be plotted is selected using the drop down list boxes for the predeveloped and postdeveloped condition (Figure 10.1).

10.1 Flood Frequency Statistics Graphs

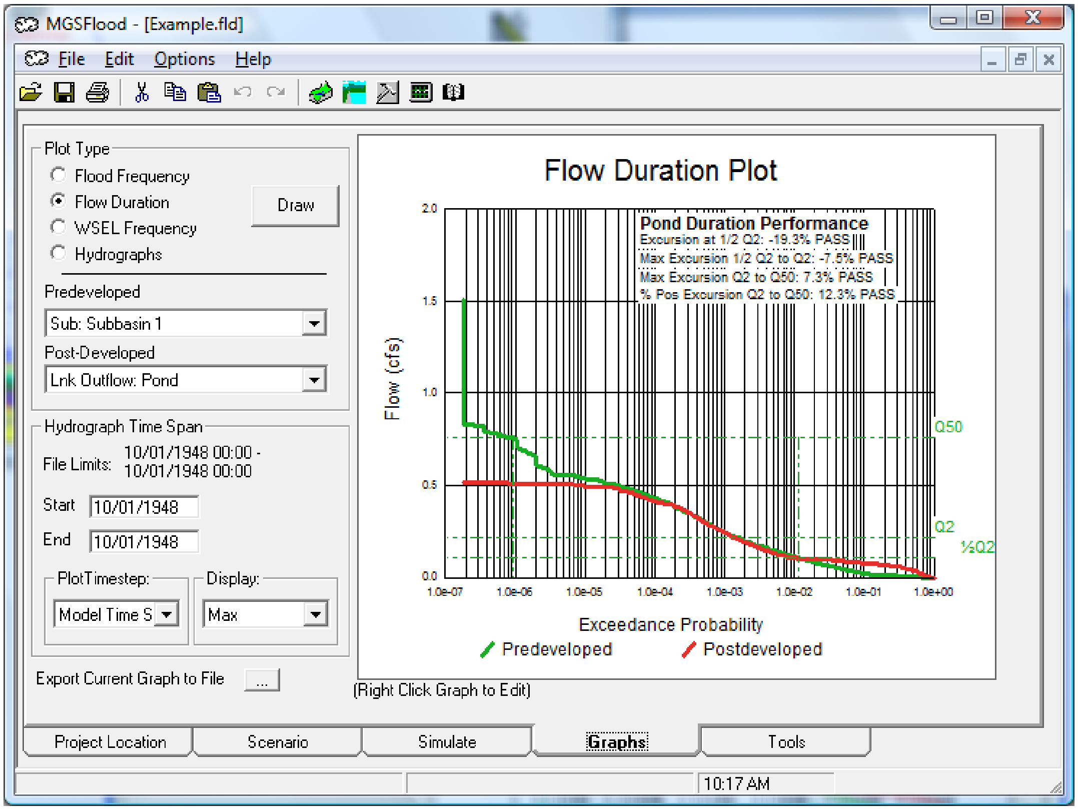
Flood frequency statistics are plotted by selecting the *Flood Frequency* option button and clicking the *Draw* button. Each time the draw button is clicked, the graph on the screen and the jpeg file on disk are each updated.

10.2 Water Surface Elevation Statistics

Water surface elevation statistics are available for any link defined as a pond. Flood frequency statistics are plotted by selecting the *Flood Frequency* option button and clicking the *Draw* button. The pond bottom and riser crest elevations are noted on the graphs. Each time the draw button is clicked, the graph on the screen and the jpeg file on disk are each updated.

10.3 Flow Duration Statistics Graphs

Flow duration statistics are plotted by selecting the *Flow Duration* option button and clicking the *Draw* button. Each time the draw button is clicked, the graph on the screen is updated and the graph is stored onto disk as a jpeg file. For the compliance locations, the graph includes annotations noting the predeveloped ½ of the 2-year, 2-year and the 50-year flows, the exceedance probability corresponding to these flows, and whether the Department of Ecology Flow Duration criteria[9] have been met (Figure 10.1).



○ Flood Frequency
● Flow Duration
○ WSEL Frequency
○ Hydrographs

Predeveloped: Sub: Subbasin 1
Post-Developed: Lnk Outflow: Pond

Hydrograph Time Span
File Limits: 10/01/1948 00:00 - 10/01/1948 00:00
Start: 10/01/1948
End: 10/01/1948
PlotTimestep: Model Time S
Display: Max

Pond Duration Performance
Excursion at 1/2 Q2: -19.3% PASS
Max Excursion 1/2 Q2 to Q2: -7.5% PASS
Max Excursion Q2 to Q50: 7.3% PASS
% Pos Excursion Q2 to Q50: 12.3% PASS

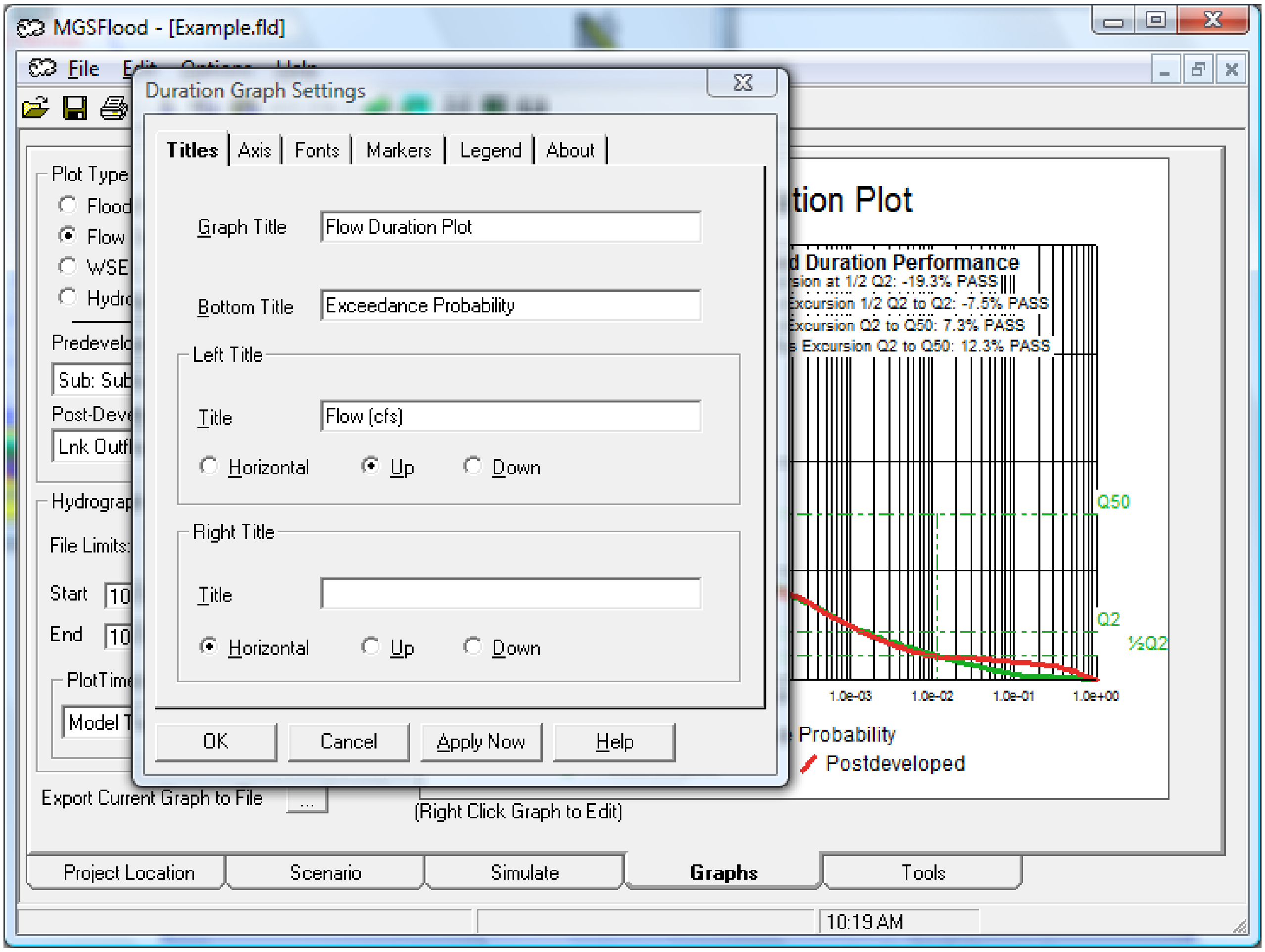
Figure 10.1 – Flow Duration Graph Showing Pond Performance

10.4 Hydrographs

Runoff from all subbasins and links defined on the Predeveloped and Postdeveloped Watershed Scenario Screens are available for display as hydrographs on the *Graphs* tab. One predeveloped and one postdeveloped timeseries can be displayed on the graph. Any time period, within the period of record saved in the direct access file, can be plotted. The *Plot Timestep* defines the number of time intervals to be aggregated before output is written to the file. For example, if *Daily* is selected then runoff for each day will be aggregated before outputting. If the runoff was computed at a 1-hour time step, then 24 values will be aggregated according to the *Aggregate* option selected. If *Maximum* was selected, then the maximum flow would be plotted, *Minimum* would result in the minimum flow, and *Average* would result in the average flow.

10.5 Customizing Graphs

Graph titles, line styles, colors, fonts, legends, etc, can be changed or modified by clicking the right mouse button on the graph. This will display the *Graph Settings* screen where the graph titles and other settings can be customized (Figure 10.2). Changed graph settings are saved in the project directory in a file with a .*GSP* extension and are applied each time the project is loaded.



MGSFlood - [Example.fld]

Duration Graph Settings

Graph Title: Flow Duration Plot

Bottom Title: Exceedance Probability

Left Title

Title: Flow (cfs)

○ Horizontal ● Up ○ Down

Right Title

Title:

● Horizontal ○ Up ○ Down

Figure 10.2 – Graphs Settings Screen Displayed by Clicking the Right Mouse Button on the Graph

10.6 Saving Graphs to Disk

The current graph displayed on the Graphics tab may be exported to a .jpg file by clicking the *Export Current Graph to File* button on the Graphics Tab. The program will prompt for a file name and directory location to save the currently displayed graph. Note, the file name length (including the path) is limited to a maximum of 128 characters. If a file name longer than this limit is input, then the program will prompt the user and a shorter file name and/or different path location may be entered.

11 Water Quality Parameter Calculation

Water quality treatment design parameters are computed for Links using the *Water Quality Data* calculation window according to methods defined in the 2005 Department of Ecology Stormwater Management Manual for Western Washington^9. To open the Water Quality Data window, right click on the Link of interest and click *Link WQ Statistics* (Figure 11.1). The water quality calculation window for the selected link will then appear (Figure 11.2).

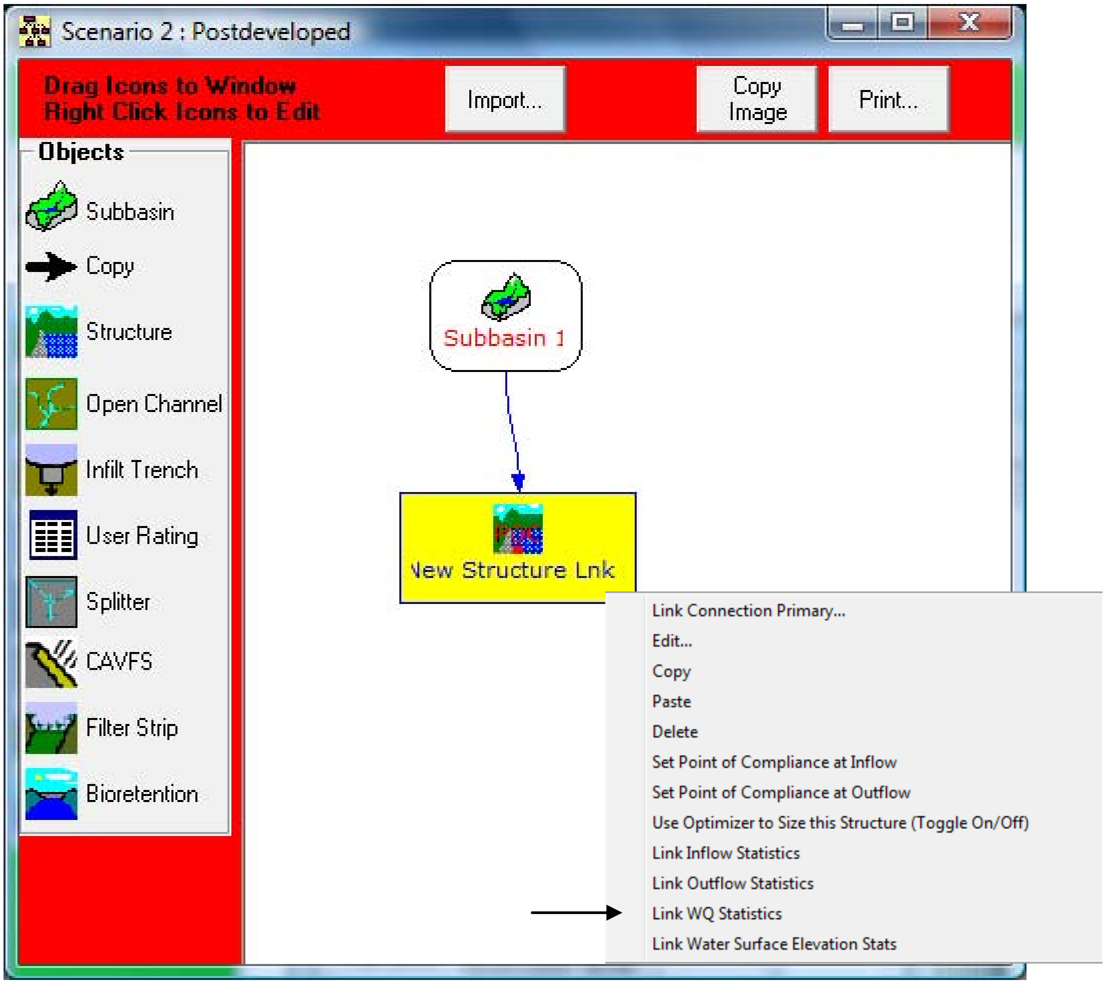
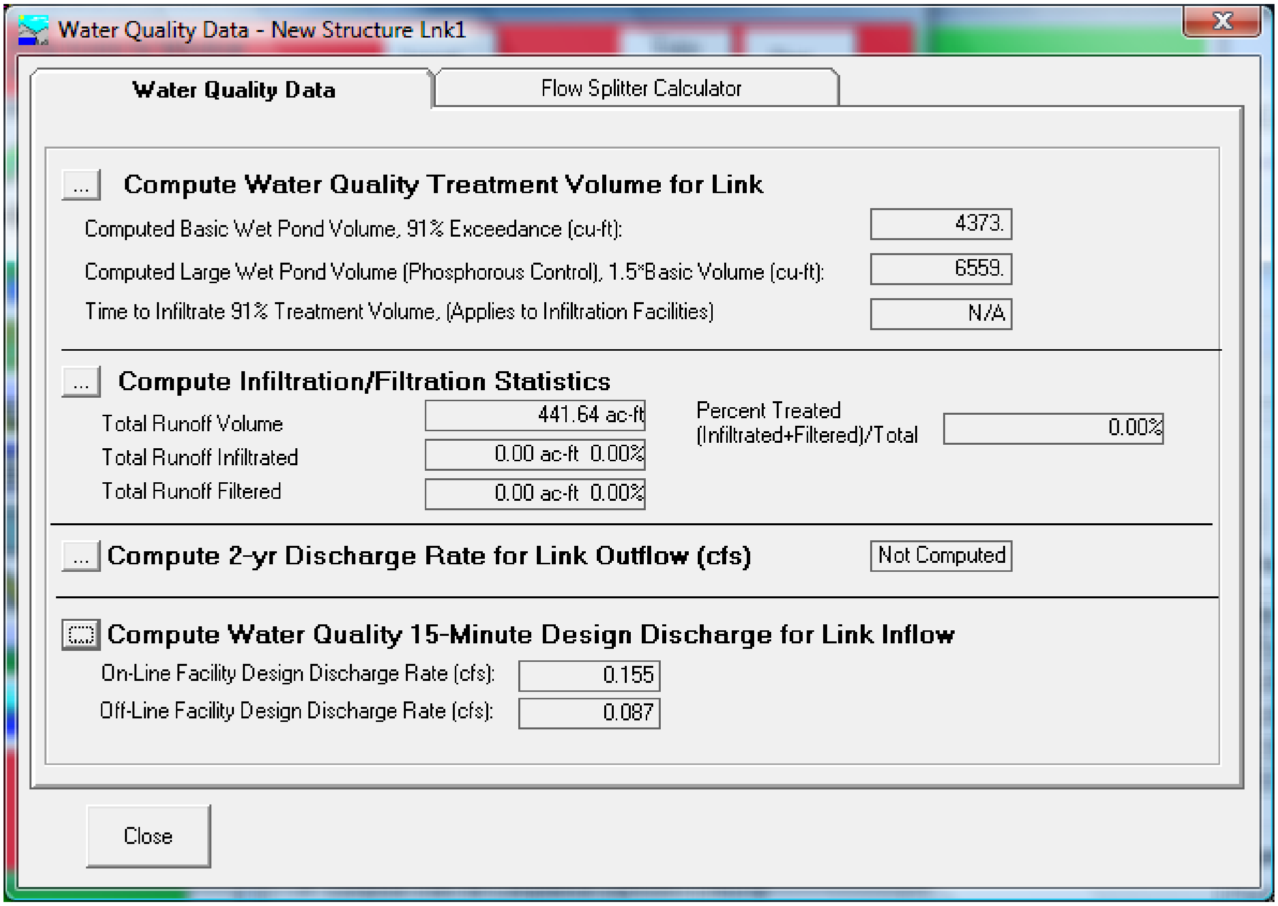


Figure 11.1 – Opening Water Quality Calculation Window



Compute Water Quality Treatment Volume for Link

Computed Basic Wet Pond Volume, 91% Exceedance (cu-ft): 4373.

Computed Large Wet Pond Volume (Phosphorous Control), 1.5*Basic Volume (cu-ft): 6559.

Time to Infiltrate 91% Treatment Volume, (Applies to Infiltration Facilities) N/A

Compute Infiltration/Filtration Statistics

Total Runoff Volume 441.64 ac-ft

Total Runoff Infiltrated 0.00 ac-ft 0.00%

Total Runoff Filtered 0.00 ac-ft 0.00%

Percent Treated (Infiltrated+Filtered)/Total 0.00%

Compute 2-yr Discharge Rate for Link Outflow (cfs) Not Computed

Compute Water Quality 15-Minute Design Discharge for Link Inflow

On-Line Facility Design Discharge Rate (cfs): 0.155

Off-Line Facility Design Discharge Rate (cfs): 0.087

Figure 11.2 – Water Quality Calculation Window for Selected Link

Three types of water quality treatment parameters are computed by MGSFlood;

- Water Quality Design Volume, used for sizing wet ponds,
- Infiltration and filtration statistics,
- Water Quality Design Flow Rate, used for sizing flow rate dependent facilities such as biofiltration swales and filter strips.

The user should refer to the Ecology Stormwater Manual for specific information regarding water quality treatment requirements and design methods.

11.1 Water Quality Design Volume

The water quality design volume for sizing wet ponds is computed as the 91% non-exceedance 24-hour runoff volume. The program develops a daily runoff timeseries from the hourly pond inflow timeseries and scans the computed daily timeseries to determine the 24-hour volume that is greater than or equal to 91% of all daily values in the timeseries. According to the Ecology Stormwater Management Manual, this value is then used as the volume for a “Basic Wet Pond” and 1.5 times this value is used for sizing a “Large Wet Pond." These values are computed automatically at the time runoff is computed for the compliance point and are reported on the Water Quality Calculation Window and the project report.

11.2 Filtration/Infiltration Statistics

Water quality treatment statistics are computed for facilities that infiltrate or filter water through media. The total volume infiltrated and/or filtered is compared with the total volume entering the facility. For quality treatment, 91-percent of the simulated runoff volume from the site must be filtered or infiltrated by the facility. These values are reported on the Water Quality Calculation Window and the project report.

11.3 Water Quality Design Discharge

The water quality design discharge rate is computed using the link inflow time series. The program returns both *off-line* and *on-line* design discharge rate for facilities located upstream of the detention facility. If the treatment facility is located downstream of the detention facility, then the pond outflow 2-year discharge rate is used for treatment design.

Off-line water quality treatment located upstream of the detention facility includes a high-flow by-pass that routes the incremental flow in excess of the water quality design rate around the treatment facility. It is assumed that flows from the bypass enter the system downstream of the treatment facility but upstream of the detention facility. The program determines the hourly water quality treatment design flow rate as the rate corresponding to the runoff volume that is greater than or equal to 91% of the hourly runoff volumes. The 15-minute water quality treatment design flow rate is then computed from an adjustment factor provide by Ecology for estimation of maximum 15-minute flow rates based on hourly timeseries. If a 15-minute time step is used, then the same procedure is used, except that the adjustment factors are not applied.

On-line water quality treatment does not include a high-flow bypass for flows in excess of the water quality design flow rate and all runoff is routed through the facility. The program determines the hourly water quality treatment design flow rate as the rate corresponding to the runoff volume that is greater than or equal to 91% of the hourly runoff volume entering the treatment facility, however, those flows that exceed the water quality design flow are not included in the calculation. Thus, the design flow rate for on-line facilities is higher than for off-line facilities. As discussed above, the 15-minute water quality treatment design flow rate is then computed from an adjustment factor provide by Ecology for estimation of maximum 15-minute flow rates based on hourly timeseries. If a 15-minute time step is used, then the same procedure is used, except that the adjustment factors are not applied.

11.4 Water Quality Flow Splitter Design

When an *off-line* treatment approach is used, a flow-splitter is needed for bypassing flows that exceed the design flow rate. The Flow Splitter Calculator tab is used to compute the geometry of the splitter structure according to guidelines listed in the Ecology Stormwater Management Manual. The splitter structure includes an orifice and an overflow weir, and the design guidelines are listed below.

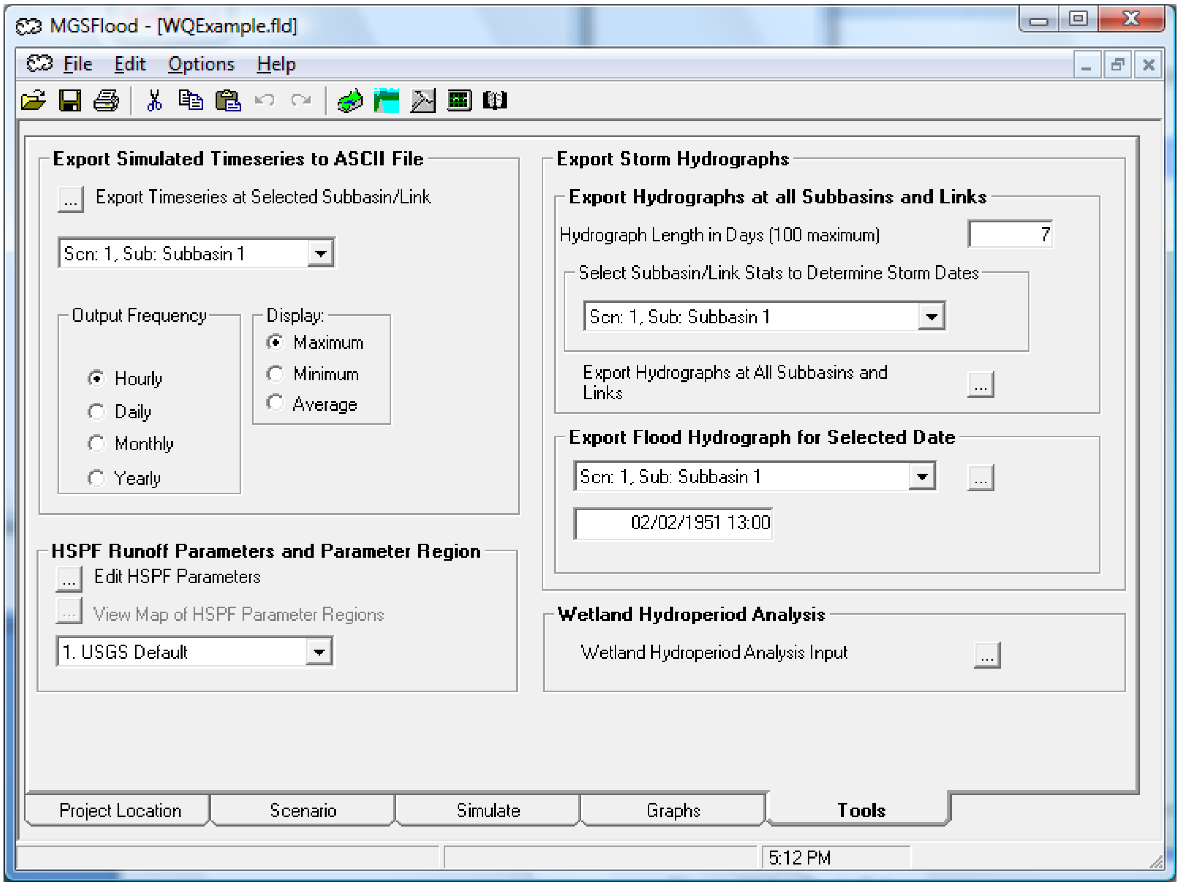
- The maximum head on the overflow weir must be minimized for flow in excess of the water quality design flow. Specifically, flow to the water quality facility at the 100-year water surface must not increase the design water quality flow by more than 10-percent.

- The splitter structure requires an orifice plate upstream of the discharge pipe that leads to the water quality treatment facility. The design water surface should be set to provide a minimum headwater/diameter ratio of 2.0.

The splitter design is a trial and error procedure whereby the orifice diameter is selected by the user. The program then computes the height of the baffle wall, the length of the overflow weir, and the ratio of the baffle wall height to orifice diameter. There is not a unique solution and the user should select an orifice size that produces a baffle wall height and overflow length that will conveniently fit in a standard manhole (or other structure) and meets the required headwater/diameter ratio of 2.0.

12 Tools Tab

The *Tools* tab provides a means to export time series computed by the program, perform wetland hydroperiod analyses, or modify the default HSPF runoff parameters (Figure 12.1).



Export Simulated Timeseries to ASCII File

Export Timeseries at Selected Subbasin/Link: Scn: 1, Sub: Subbasin 1

Output Frequency: ● Hourly ○ Daily ○ Monthly ○ Yearly

Display: ● Maximum ○ Minimum ○ Average

HSPF Runoff Parameters and Parameter Region

Edit HSPF Parameters

View Map of HSPF Parameter Regions

1. USGS Default

Export Storm Hydrographs

Export Hydrographs at all Subbasins and Links

Hydrograph Length in Days (100 maximum): 7

Select Subbasin/Link Stats to Determine Storm Dates: Scn: 1, Sub: Subbasin 1

Export Hydrographs at All Subbasins and Links

Export Flood Hydrograph for Selected Date

Scn: 1, Sub: Subbasin 1

02/02/1951 13:00

Wetland Hydroperiod Analysis

Wetland Hydroperiod Analysis Input

5:12 PM

Figure 12.1 – Tools Tab

12.1 Exporting Timeseries

Timeseries computed by the program are stored in binary direct access files. These timeseries can be exported to an ASCII formatted file from the *Tools* tab. The output frequency option defines the number of time intervals to be aggregated before output is written to the file. For example, if the *Daily* option button is selected, then the timeseries will be aggregated and saved to the file once per day. For runoff computed on an hourly time-step, 24 values will be aggregated according to the option selected in the *Display* box. If *Maximum* was selected, then the maximum daily flow would be output, *Minimum* would result in the minimum daily flow, and *Average* would result in the average daily flow.

12.2 Exporting Storm Hydrographs

The *Export Storm Hydrographs* feature is used to extract hydrographs from the time series file(s) with peak flow corresponding to a user specified recurrence interval. Time series that have had peak flow frequency statistics computed on the tab are available for export. The length of the hydrograph is specified in the *Hydrograph Length* box, which can range from 1 to 100 days. Flow recurrence intervals of 2-years, 10-years, 25-years, 50-years, and 100-years are exported. The program uses the time series specified in the *Subbasin/Link Stats* box to determine the dates of storms with recurrence intervals closest to the recurrence interval of the exported storms (2-year, 10-years, 25-years, 50-years, and 100-years). The same dates are used for all time series in the model. The files are saved to a data file with the format: <*sublinkname*>_xx.dat

Where <*sublinkname*> is the name of the subbasin or link time series,
xx is the recurrence interval of the storm exported.

12.3 Wetland Hydroperiod Analysis

Protection of wetland plant and animal communities depends on controlling the wetland's *hydroperiod*, meaning the pattern of fluctuation of water depth and the frequency and duration of exceeding certain levels, including the length and onset of drying in the summer.

MGSFlood computes hydroperiod statistics according to the guidance developed by the Puget Sound Wetlands and Stormwater Management Research Program[33]. The statistics quantify the difference in wetland water level between predeveloped and post developed conditions. A predeveloped and postdeveloped timeseries must be selected from the drop down list boxes prior to performing the analysis.

The wetland water level fluctuation guidelines (Guide Sheet 2: Wetland Protection Guidelines) were adopted by Ecology and are listed in Appendix D of the Volume I of Ecology's Stormwater Management Manual[9]. Default values listed on the Wetland Hydroperiod input fields were obtained from Guide Sheet 2. More information regarding the calculation of hydroperiod statistics can be found in Part I, Section 22 or by referring to Guide Sheet 2.

Hydroperiod statistics can be computed for ponds or high groundwater land segments. Any pond link present in the project may be selected from the drop down list boxes. Hydroperiod results are written to the project report.

12.4 Runoff Parameter Region, HSPF Parameters

12.4.1 Runoff Parameter Region

MGSFlood can accommodate unique sets of runoff parameters for different regions of western Washington. Currently, only one set of runoff parameters, defined by the USGS, has been defined for use for all of western Washington.

12.4.2 HSPF Parameters

Clicking the Open HSPF Parameters button will display the default runoff parameters for the currently selected region. These parameters should only be modified by those users experienced with HSPF. Any changes to the default runoff parameters will be identified on the project documentation report.

12.4.3 User Defined Land Use

An additional Pervious Land Segments (PERLND) may be specified in addition to the default parameter set. This is useful for defining a land cover/soil combination unique to a watershed that is not defined in the default parameter set.

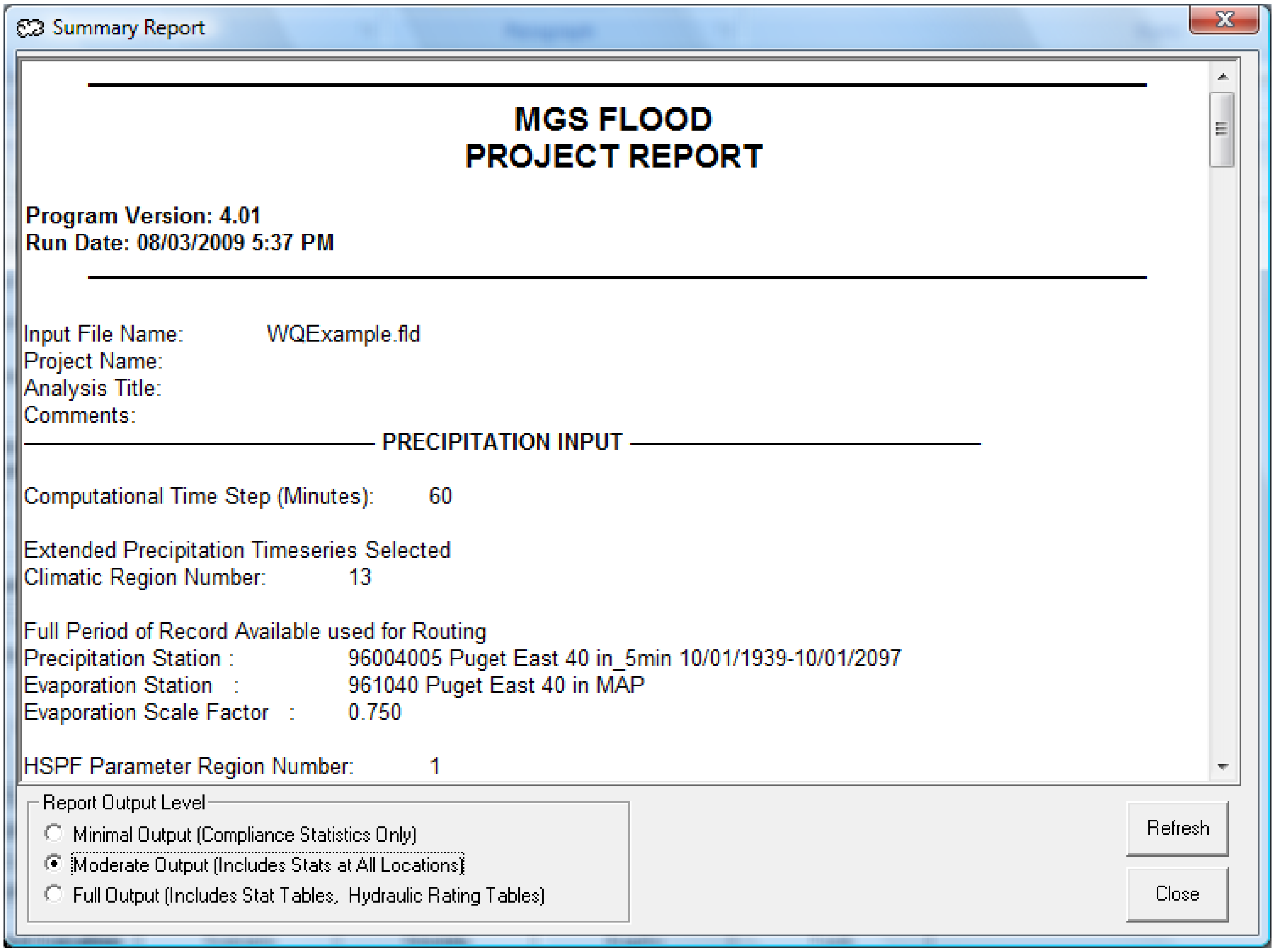
The new parameter set is defined by opening the HSPF Parameter sheet and clicking the *User* button at the bottom of the page. A window will appear with parameter fields for an additional user-defined PERLND. The user can specify the name as well as the HSPF parameters. Defining a new PERLND will require knowledge of HSPF model parameters and including should only be undertaken by those familiar with the HSPF runoff routine.

13 Creating/Viewing the Project Documentation Report

The project reporting utility creates a report that documents all model inputs, stormwater pond design information, and frequency and duration statistics. The report is created and viewed on screen by selecting *View Report* from the File menu or from the *View Report* icon (📖) on the tool bar. Each time the report is viewed or printed, a copy of the report is stored in a file with the name <*ProjectName.rtf*> in the project data directory. This file can be viewed or edited with Microsoft Word or WordPad or printed by MGSFlood. Three levels of output may be selected; minimal which includes land use input and compliance results only, moderate which includes statistics available from all subbasins and links, and full output which includes hydraulic rating tables for all structures and detailed statistics.

13.1 Printing Project Report

The report can be printed by selecting *Print Report* from the File menu or from the Printer Icon on the tools menu. Only text selected on the screen by highlighting with the mouse will be printed. If no text is selected, then the entire report file will be printed (Figure 13.1).



MGS FLOOD
PROJECT REPORT

Program Version: 4.01
Run Date: 08/03/2009 5:37 PM

Input File Name: WQExample.fld
Project Name:
Analysis Title:
Comments:

PRECIPITATION INPUT

Computational Time Step (Minutes): 60

Extended Precipitation Timeseries Selected
Climatic Region Number: 13

Full Period of Record Available used for Routing
Precipitation Station : 96004005 Puget East 40 in_5min 10/01/1939-10/01/2097
Evaporation Station : 961040 Puget East 40 in MAP
Evaporation Scale Factor : 0.750

HSPF Parameter Region Number: 1

Report Output Level
○ Minimal Output (Compliance Statistics Only)
● Moderate Output (Includes Stats at All Locations)
○ Full Output (Includes Stat Tables, Hydraulic Rating Tables)

Figure 13.1 – Project Report Screen

13.2 Printing Watershed Schematic and Performance Graphics

When the project report is printed, the program prompts the user to print the predeveloped and postdeveloped watershed schematic images. Alternatively, the user could copy the images to the Windows Clipboard from the Watershed Schematic Window and print them through a word processing program.

Graphs created on the Graphs tab can be printed by first saving them using the save button on the Graphs Tab. They can subsequently be opened and printed using image viewing software such as Microsoft Picture Manger.

User Notes