The Design, Use, and Evaluation of Wet Detention Ponds for Stormwater Quality Management

Using WinDETPOND

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Abstract
Wet detention ponds are probably the most common management practice for the control of stormwater runoff quality. If properly designed, constructed, and maintained, they can be very effective in controlling a wide range of pollutants and peak runoff flow rates. There is probably more information concerning the design and performance of detention ponds in the literature than for any other stormwater control device. Wet detention ponds are a very robust method for reducing stormwater pollutants. They typically show significant pollutant reductions as long as a few design-related attributes are met. Many details are available to enhance performance, and safety, that should be followed. Many processes are responsible for the pollutant removals observed in wet detention ponds. Physical sedimentation is the most significant removal mechanism. However, biological and chemical processes can also contribute important pollutant reductions. The extensive use of aquatic plants, in a controlled manner, can provide additional pollutant removals. Wet detention ponds are also suitable for enhancement with chemical and advanced physical processes.

Introduction
This chapter discusses one of the most often used and most effective stormwater control practice: wet detention ponds. There are many stormwater control practices, but all are not suitable in every situation. It is important to understand which controls are suitable for the site conditions and can also achieve the required goals. This will assist in the realistic evaluation for each practice of: the technical feasibility, implementation costs, and long-term maintenance requirements and costs. It is also important to appreciate that the reliability and performance of many of these controls have not been well established, with some still in the development stage. This is not to say that emerging controls cannot be effective, however, they do not have a large amount of historical data on which to base designs or to be confident that performance criteria will be met under the local conditions. The most promising and best understood stormwater control practices are wet detention ponds. Less reliable in terms of predicting performance, but showing promise, are stormwater filters, wetlands, and percolation basins (Roesner, *et al.* 1989). Grass swales also have shown great promise during the EPA's Nationwide Urban Runoff Program (NURP) (EPA 1983) and other research projects. During the last 10 to 30 years, much experience has been gained with many stormwater practices, especially source controls and stream restoration efforts. An effective stormwater management program likely must contain elements of many control practices to be most cost-effective. The combinations of practices that are most efficient for a specific area must be selected based on many site-specific conditions and local objectives. In many cases, wet detention ponds can be an important stormwater control that should be given serious consideration.

Wet detention ponds are also one of the most robust stormwater control practices available. Although a good maintenance program is necessary to ensure the best performance and minimize associated problems, many stormwater ponds have functioned well with minimal maintenance. In addition, as long as certain design guidelines are followed, many design details that are worthwhile to consider do not create critical problems if incorrectly implemented. Finally, it is possible to retrofit stormwater ponds and correct many of these problems as experience dictates. These robust attributes are rare for most stormwater control practices. As an example, a study of 11 types of stormwater quality and quantity control practices used in Prince George's County, Maryland (Metropolitan

Washington Council of Governments 1992) was conducted to examine their performance and longevity. They concluded that several types of stormwater control practices had either failed or were not performing as well as intended. Generally, wet ponds, artificial marshes, sand filters, and infiltration trenches achieved moderate to high levels of removal for both particulate and soluble pollutants. Only wet ponds and artificial marshes were found to function for a relatively long time without frequent maintenance. Control practices, which were found to perform poorly, included infiltration basins, porous pavements, grass filters, swales, smaller "pocket" wetlands, extended detention dry ponds, and oil/grit separators. Infiltration stormwater controls had high failure rates that could often be attributed to poor initial site selection and/or lack of proper maintenance. The poor performance of some of the controls was likely a function of poor design, improper installation, inadequate maintenance, and/or unsuitable placement of the control. Greater attention to these details would probably reduce the failure rate of these practices. The wet ponds and artificial marshes were much more robust and functioned adequately under a wider range of marginal conditions.

The majority of stormwater treatment practices are most effective for the removal of particulate forms of pollutants only, especially the settleable solids fraction. Removal of dissolved, or colloidal, pollutants is minimal and therefore pollution prevention or control at the sources offers a more effective way to control the dissolved pollutants. Fortunately, most toxic stormwater pollutants (heavy metals and organic compounds) are mostly association with stormwater particulates (Pitt, *et al*. 1995). Therefore, the removal of the solids will also remove much of the pollutants of interest. Notable exceptions of potential concern include: nitrates, chlorides, zinc, pathogens, 1,3-dichlorobenzene, fluoranthene, and pyrene. Stormwater ponds mostly utilize sedimentation as the main pollutant removal mechanism. However, chemical and biological mechanisms are also available, especially when the pond is appropriately planted with wetland vegetation. Stormwater ponds, while costly, also generally add substantial value to adjacent property, if designed and maintained well. The following are general conclusions pertaining to stormwater detention facilities.

Expected Detention Pond Performance

- Dry ponds have little documented direct water quality benefits due to scouring of bottom sediments. Decreased receiving water velocities will decrease receiving water bank erosion and will improve aquatic habitat, however.
- Wet ponds have been extensively monitored under a wide variety of conditions. If well designed and properly maintained, suspended solids removals of 70 to 90% can be obtained. BOD_5 and COD removals of about 70%, nutrient removals of about 60 to 70%, and heavy metal removals of about 60 to 95% can also be obtained. Limited bacteria control (maybe up to 50%) can be expected in the absence of disinfection. Wet ponds can also be designed to obtain significant flood control benefits.

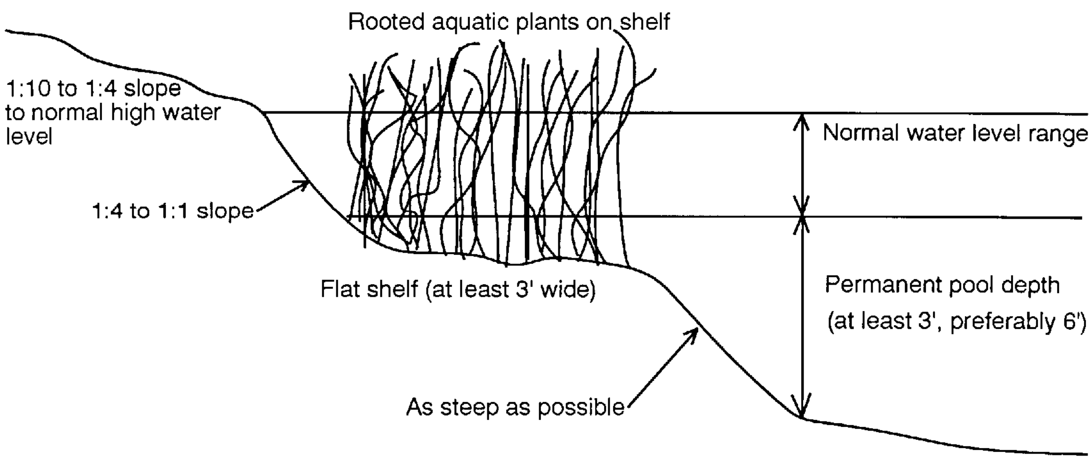
Potential Detention Pond Problems

- Wet ponds can require about three to six years to obtain an ecological balance. During the initial unstable period, excessive algal growths, fish kills, and nuisance odors may occur.
- Wet ponds can have poor water quality and water contact recreation and consumptive fishing should be discouraged.
- Careful watershed-wide planning is needed to insure composite flood control benefits from many ponds in a watershed.

Wet Detention Pond Design Guidelines to Minimize Potential Problems

- Keep pond shape simple to encourage good water circulation. The length should be about three to five times the width for maximum detention efficiency and the inlets and outlets need to be widely spaced to minimize short-circuiting.
- Need at least three and preferably six feet of permanent standing water over most of the pond to protect sediments from scouring, to decrease light penetration (to minimize rooted aquatic plant growths), and to increase winter survival of fish.
- Increase flushing during extended dry periods, possibly with groundwater, to improve water quality. Reduce contaminated baseflows from entering the pond through source controls.
- Proper pond side slopes are very important to improve safety and aesthetics and to minimize mosquito problems and excessive rooted plant growths. An underwater shelf near the pond edge needs to be planted with rooted aquatic plants to prevent children's access to deep water, to improve pond aesthetics, to increase pollutant

removals through biochemical processes, and to improve aquatic habitat. If waterfowl are desired users of the pond, then no more than one-half of the pond perimeter should be heavily planted. The following general dimensions for pond side slopes are suggested:



• Outlet structures should be designed for low outflows during low pond depths to maximize particulate retention. Place underwater dams or deeper sediment trapping forebays near pond inlets to decrease required dredging areas. Provide a drain to completely de-water the pond for easier maintenance.

• Protect the inlet and outlet areas from scour erosion and cover the inlets and outlets with appropriate safety gratings. Provide an adequate emergency spillway. Minimize water elevation changes to discourage mosquito problems.

Required Stormwater Detention Pond Maintenance

• If the pond does not require any maintenance, it is not producing very many water quality benefits. Ponds need to be periodically dredged to remove contaminated bottom sediments.

• Plan extra pond depth for sacrificial volume to lengthen dredging intervals (approximately one inch per year, much more in forebays). Also plan for heavy equipment access to pond edges.

• Remove excessive algae and other aquatic plants to prevent decomposition and nutrient cycling and associated nuisance conditions.

Basic Wet Detention Pond Design Guidelines

• Engineering design guidelines (covering such things as foundations, fill materials, embankments, gratings, anti-seep collars, and emergency spillway construction), such as published by the U.S. Natural Resources Conservation Service, the Bureau of Reclamation, and the Army Corps of Engineers must be followed.

• Pond size is dictated mostly by desired particle size control and water outflow rate. The following table is an estimate of pond surface requirements for different land uses and conditions. A target for the worst-case control of 5 μm will remove all particles greater than 5 μm under almost all conditions and will result in a long-term median removal of about 2 μm. This control goal corresponds to about 90% suspended solids reductions in urban runoff. A worst-case goal of 20 μm control will result in about 65% suspended solids reductions.

Percent of drainage area required as pond for:

Land Use	5 µm control	20 µm control
Totally paved areas	3.0 percent	1.1 percent
Freeways	2.8	1.0
Industrial areas	2.0	0.8
Commercial areas	1.7	0.6
Institutional areas	1.7	0.6
Residential areas	0.8	0.3
Open space areas	0.6	0.2
Construction sites	1.5	0.5

Wet Detention Pond Costs

• Initial wet detention pond construction costs are roughly estimated to be about $40,000 per acre of pond surface (excluding land costs).

• Maintenance costs are estimated to be about $1500 per pond surface acre per year.

Pond Size Calculation

• The following table shows the minimum pond surface area (acres) required for different freeboard elevations above the invert of 60 degree and 90 degree V-notch weirs, for both 5 and 20 µm particle control:

	60^{o} V-notch weir			90^{o} V-notch weir		
Head	Discharge	Min. surface acres for:		Discharge	Min. surface acres for:	
(feet)	(cfs)	5 µm	20 µm	(cfs)	5 µm	20 µm
0.5	0.25	0.044	0.004	0.45	0.08	0.006
1	1.4	0.25	0.02	2.4	0.42	0.03
1.5	3.9	0.69	0.06	6.7	1.2	0.1
2	8.0	1.4	0.11	14	2.5	0.2
3	22	3.9	0.32	40	7.1	0.6
4	45	7.9	0.65	81	14	1.2

A review of wet detention pond design procedures must include four very important publications that all stormwater managers should have. Tom Schueler's *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban Best Management Practices* (1987) includes many alternative wet pond designs for various locations and conditions. *Watershed Protection Techniques* is a periodical published by Schueler at the Center for Watershed Protection (Ellicott City, Maryland) and includes many summaries of current stormwater management research, including new developing design procedures and performance data for detention ponds. In addition, Peter Stahre's and Ben Urbonas's book on *Stormwater Detention for Drainage, Water Quality and CSO Management* (1990) includes in-depth discussions on many detention pond design and operational issues. Also, Gary Minton recently published a comprehensive manual on stormwater treatment, *Stormwater Treatment; Biological, Chemical & Engineering Principles* (2002) that stormwater managers should also have access to. In addition, the on-going ACSE BMP database contains a growing number of case studies documenting stormwater control performance from many US locations. This database is located at:
http://www.asce.org/community/waterresources/nsbmpdb.cfm


Orlando, FL, convention center pond


Gulfport, MS, shopping center pond


Dayton, OH, shopping mall pond (multiple ponds surrounding parking area)


Birmingham, AL, shopping center pond, showing vegetated safety ledge

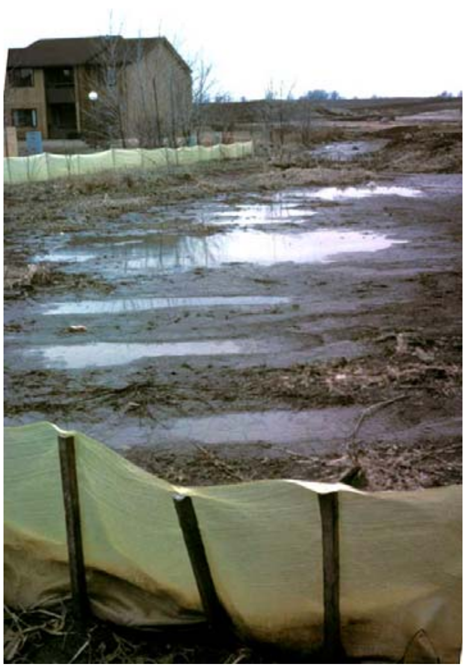

Downtown Austin pond, following oil and grease trap


Monroe St. pond in Madison, WI (WI DNR photo)

Figure 1. Ponds located in commercial areas.

Auckland, NZ, pond with multi-stage riser

Birmingham, AL, land fill pond



Middleton, WI, pond (heavy erosion filled pond)

Birmingham, AL, land fill pond

Birmingham, AL, land fill pond

Birmingham, AL, construction site pond

Wisconsin DOT pond at work area

Figure 2. Temporary ponds at construction sites.


Middleton, WI, pond at conservation design subdivision


Auckland, NZ, pond in cemetery serving surrounding residential area


Milwaukee, WI, apartment complex pond

Charleston, South Carolina, pond in restricted residential development


Large Alameda County, CA, pond serving surrounding residential, office, and commercial area


Hoover, AL, pond located in new park treating runoff from adjacent residential area


Lake Oswego, OR, pond treating runoff from adjacent shopping center and apartment complex

Figure 3. Residential area ponds


Birmingham, AL, industrial area pond
Figure 4. Industrial area ponds.

Snowmass, CO, pond

Snowmass, CO, pond

Mountain Home, Telluride, CO, pond

Figure 5. Ponds at resort communities.

Background

Detention ponds are probably the most common management practice for the control of stormwater sediment. If properly designed, constructed, and maintained, they can be very effective in controlling a wide range of pollutants and peak runoff flow rates. In an early 1980 survey of cities in the U.S. and Canada, the American Public Works Association found more than 2,000 wet ponds, more than 6,000 dry ponds, more than 3,000 parking lot multi-use detention areas, and more than 500 rooftop storage facilities (Smith 1982). About half of the wet detention ponds were publicly owned. In some areas of the U.S., detention ponds have been required for some time and are therefore much more numerous than elsewhere. In Montgomery County, Maryland, as an example, detention ponds were first required in 1971, with more than 100 facilities planned during that first year, and about 50 actually constructed. By 1978, more than 500 detention facilities had been constructed in Montgomery County alone (Williams 1982). In DuPage County, Illinois, near Chicago, more than 900 stormwater detention facilities (some natural) receive urban runoff (McComas and Sefton 1985).

There is probably more information concerning the design and performance of detention ponds in the literature than for any other stormwater control device. Wet detention ponds are also a very robust method for reducing stormwater pollutants. They typically show significant pollutant reductions as long as a few basic design-related attributes are met (most important being size). Many details are available to enhance performance, and safety, that should be followed. Many processes are responsible for the pollutant removals observed in wet detention ponds. Physical sedimentation is the most significant removal mechanism. However, biological and chemical processes can also contribute important pollutant reductions. The extensive use of aquatic plants, in a controlled manner, can provide additional pollutant removals. Magmedov, *et al*. (1996), for example, report on the use of wetlands for treatment of stormwater runoff in the UK and in the Ukraine, including design guidelines. Wet detention ponds also are suitable for enhancement with chemical and advanced physical processes. Lamella plate and tube separators, air floatation, filtration, and UV disinfection are examples of treatment enhancements being investigated in France, and elsewhere (Bernard, *et al*. 1996; Delporte 1996; Pitt, *et al.* 1999).

Ellis (1993) describes design guidelines for a reed bed wetland for the treatment of stormwater. These are compact control practices that have little standing water. Most of the removal of pollutants occurs in the root zone of the selected wetland plants, with pretreatment provided by a grit chamber and possibly a grass filter. A small micropool can also be used after the reed bed. It is expected that these wetlands would provide from 50 to 90% reductions of suspended solids, and up to 90% removal of heavy metals.

Little information has been provided in the literature on the performance of artificial wetlands in cold climates for stormwater treatment. Dormant plants provide ineffective mechanisms for pollutant removal, plus scour of previously retained pollutants may increase during periods of dormancy. It is recommended that stormwater wetlands be used as polishing treatment devices, after pretreatment with more robust devices (such as wet detention ponds), in areas having severe weather. Flows should also be diverted around wetland treatment systems whenever the plants are dormant, except for necessary flows to sustain natural moisture conditions. Harvesting of aquatic plants is also probably needed in wetland treatment systems. Decomposition of plants readily release nutrients and other organic material that may degrade water quality.

Multiple Benefits of Detention Facilities

The most common multiple benefit of detention facilities built for water quality improvements is flood control. If appropriately designed, wet detention ponds can provide significant peak flow rate reductions. Ponds by themselves provide little runoff volume reductions, but can be designed in conjunction with infiltration devices to provide water quality in addition to peak flow rate and water volume reduction benefits. In order to provide flood control benefits, substantial freeboard storage above the normal wet pond elevation must be provided. This has been commonly done in open space land uses such as parks and golf courses where periodic short-term flooding does not detract from the other uses of the land.

Many people enjoy wetlands (including wet detention ponds) in urban settings. Adams, *et al*. (1982) reports a typical comment from a resident living near a wet detention pond in Columbia, Maryland: "...now that they've matured, we're reaping rewards from all the wildlife using the ponds." Numerous ducks, herons, egrets, songbirds, mammals, and amphibians have been observed and highly prized by residents living near these small artificial wetlands. Establishing natural aquatic vegetation (rooted macrophytes) on the shallow shelf edges of the ponds make them more attractive to wildlife and enhances their beauty.

Fishing is also popular in many wet detention facilities, especially by children, although fish consumption should usually be discouraged due to the possibility of accumulations of toxic substances. Many currently enjoy recreational fishing in wet detention facilities using catch-and-release.

The integration of properly designed, constructed, and maintained wet detention ponds into parks and linear green (and blue) belts can provide substantial community benefits, even if the water quality in the ponds is less than "good" (Jones and Jones 1982). Flood control, non-contact recreation, non-consumptive fishing, education, and aesthetics benefits have all been achieved at many wet detention ponds. Stormwater reuse has also been incorporated into the design of wet-detention systems constructed in Florida. Stormwater reuse reduces the volume of stormwater discharged downstream thereby decreasing the loss of potentially valuable freshwater resource. Additionally, by reusing the detained stormwater instead of discharging it, the treatment efficiency of the stormwater detention pond was increased thereby decreasing the pollutant load delivered downstream (Livingston 1999).

The direct and nearby benefits of wet detention facilities on in-stream biological conditions is not clear. Maxted and Shaver (1996) studied the use of wet detention ponds, constructed to mitigate stormwater impacts on aquatic life. Physical habitat and biological measurements were taken below eight ponds. Two of the sites were in commercial areas, while six were in residential areas. These results were compared to 38 sites with no ponds that had been sampled in 1993. The ponds did not prevent the almost complete loss of sensitive species and they did not attenuate the impacts of urbanization once the watershed reached 20% impervious cover. They concluded that the data set size was too small for a conclusive evaluation of the effectiveness of stormwater controls to protect stream biota and habitat. It is also possible that the stormwater facilities studied were not used long enough to achieve improvements in habitat and to repopulate the species.

Crunkilton and Kron (1999) measured the toxicity of stormwater runoff before and after it had been allowed to flow through a pilot-scale wet-detention pond. Selected heavy metals and PAH compounds were measured in incoming and outgoing-settling pond water. *Daphnia magna* and *Pimephales promelas* (fathead minnow) were exposed to pre and post-settling pond treated stormwater runoff for three test periods of 14 days each in 1996 and 1997. The pond significantly reduced the toxicity of the stormwater with treatment, although there was still important residual toxicity remaining. Lieb and Carline (2000) examined the impact of runoff from a stormwater pond on the macroinvertebrate community in a small headwater stream downstream of the pond in central Pennsylvania. Invertebrate communities 98 m and 351 m downstream of the pond were highly degraded, while a community 798 m downstream was markedly less degraded. Despite downstream improvement, all three sites were considered impaired relative to a reference community. These results were found to generally be in agreement with those of similar studies in other states and reinforced the need for land-use planning that considers the potential negative effects of urbanization on headwater streams. Based on an investigation into phytoplankton and periphyton algal communities of two recently constructed stormwater management ponds, Olding (2000) suggested that stormwater impacts on biological communities are reduced during passage through wet stormwater treatment ponds, providing a degree of protection for biological communities in receiving waters by reducing harmful toxins and nutrients. The lack of blue-green algae suggested that stormwater facilities could be engineered to inhibit undesirable algal communities. Marsalek, *et al.* (2002) reported on an assessment of the impacts of urban development on a small creek with an on-stream stormwater pond. The assessment included creek-pond system hydrology, water and sediment chemistry and toxicity, and benthic communities. They concluded that the pond accumulated sediments and toxicants and thereby prevented further degradation of the creek condition downstream of the plaza drainage outfall.

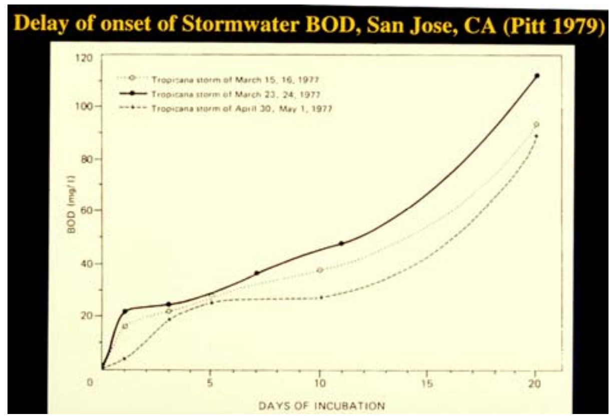
Side-stream bioassay laboratory, Lincoln Creek, Milwaukee, WI

Construction of bioassay test lab

Bioassay test lab ready to go

Pilot-scale detention pond for pre-treating water before bioassay tests

Delayed toxicity to stormwater



Older BOD tests showing delayed onset of oxygen demand (Pitt 1979)

Figure 6. Lincoln Creek bioassay test site and example test results.

Dry Ponds

Dry ponds have been extensively used throughout the U.S. and other countries (EPA 1983). These ponds have been constructed to reduce peak runoff rates (peak shaving), with typically little consideration given to runoff quality improvement. Their main purpose has therefore been in flood control by reducing flows and water elevations in the receiving waters. These flow reductions can also improve the aquatic habitat by reducing flushing of fish and other organisms from urban creeks (Pitt and Bissonnette 1984). Flow reductions also reduce downstream channel bank erosion and bottom scour. The use of many dry ponds in a watershed, without regard to their accumulative effect, can actually increase downstream flooding or channel scour problems (McCuen, *et al.* 1984). The delayed discharge of a mass of water from a dry pond may be superimposed on a more critical portion of the receiving water hydrograph.

Because these ponds are normally dry and only contain water for relatively short periods of time, they can be constructed as part of parking lots, athletic fields, tennis courts and other multi-use areas. Their outlets are designed to transmit all flows up to a specific design flow rate, after which excess flows are temporarily backed-up. In many cases, they only contain water during a few rains each year.

Several dry detention ponds were examined as part of the Nationwide Urban Runoff Program (NURP), with monitored pollutant removals ranging from insignificant to quite poor (EPA 1983). Sedimentation may occur in dry ponds, but only during the major storms when flows are retained in the pond. The deposited material must be removed after each treated rain, or it can easily be resuspended by later rains and washed into the receiving waters.

Adler (1981) found that new sediment deposits have little cohesion and without removal as part of a maintenance program, or without several feet of overlaying water, bottom scour is probable.

Because of the poor documented stormwater pollutant control effectiveness of dry detention ponds, they cannot be recommended as viable water quality control measures. However, they can be very effective when used in conjunction with other stormwater control practices (such as between a wet detention pond and an infiltration or grass filter area). They can also be very effective in reducing the energy associated with stormwater discharges and helping to stabilize degraded receiving water habitat.

East Lake Festival Center, Birmingham, AL, "bathtub" dry pond

Typical small dry pond (WI DNR photo)

East Lake Festival Center, Birmingham, AL, dry pond

Madison, WI, golf course dry pond with concrete pilot channel

Madison, WI, dry pond at apartment complex play field

Los Angeles River, CA, stormwater pumping station forebay

Austin, TX, dry pond prior to horizontal flow sand filter

Underground plastic pipe detention facility during construction

Typical dry pond with fence and steep slopes

Figure 7. Examples of dry detention ponds.

Wet Detention Ponds

Wet detention ponds maintain several feet of water in a permanent pool. The runoff water is detained for varying periods of time, depending on the pond detention volume and the storm runoff flow rate and duration. Detention times (residence) can vary from several minutes for small ponds receiving high flows to many days for large ponds receiving relatively small flows. Monitored performance of wet ponds during the NURP program ranged from poor to excellent, generally depending on the size of the detention pond relative to the watershed area served and storm characteristics (EPA 1983). Sedimentation is the main pollutant removal process, but biological processes can also substantially reduce concentrations of soluble nutrients by converting them into algae and by providing substrate for beneficial bacteria. If the algae is removed from the detention pond, nutrient discharges to the receiving waters can be reduced. If algae is not harvested from the ponds, dead algae can be decomposed back into soluble nutrient forms (and exert biochemical oxygen demand) either in the detention pond or in the receiving water. Wet ponds can be very effective in the control of stormwater runoff flows and pollutants, but must be carefully designed and maintained to prevent nuisance conditions from developing.

Extended Detention (Combination) Ponds

Extended detention, or combination wet/dry, ponds are normally dry, but have special outlets that cause the slow release of impounded water. They are therefore not as conveniently used for other uses, such as parking lots. Outlet modifications can be easily made to existing dry ponds to make them into extended detention ponds and significantly improve their stormwater pollutant control effectiveness (EPA 1983). Since they are normally dry and lack a protective water cover over the deposited sediment, they must be frequently maintained to remove accumulated sediment before a flushing rain occurs. Biological activity is restricted, reducing the potential of high nutrient removals, but they also have reduced potentials for nuisance algal growths and mosquito production. Depending on their design, extended detention ponds may behave as artificial wetlands, grass filters or percolation ponds, with much greater pollutant removal benefits, compared to dry ponds.

Figure 8. Long Island, NY, percolation pond.

Caltrans initiated a study in two Districts (Los Angeles and San Diego, California) to examine the benefits, technical feasibility, costs and operation and maintenance requirements of retrofitting extended detention facilities into existing highway and related infrastructure (Taylor, *et al*. 2001). Monitored constituents will include suspended solids, metals, nutrients, and organics (e.g., gasoline). Detailed records will also be kept for maintenance and operations requirements. Sampling results showed average suspended solids removal was 73%, total metals removal varied between 61% and 75%, while dissolved metals removal varied between 16% and 44%. Removal was lowest for nutrients, especially nitrate, which was about 17%. A concrete lined basin showed lower removal rates. Major removal of sediment is estimated to be required every 10 years.

Roof Storage

Specialized detention "ponds" include roof storage of water. These behave like dry ponds, as permanent standing water is not desirable. Roof water runoff rates can be substantially reduced by temporary detaining roof water. Very few particulates are found in roof runoff waters (Pitt and McLean 1986), so rooftop particulate sedimentation is not very important. The reduction of roof runoff flow rates can significantly reduce erosion near downspouts and "slower" roof runoff can be more easily treated by infiltration devices. Plastic rings with holes, or gravel, can be placed around roof drain inlets to slow water runoff from roofs. Water depths of two or three inches can be safely held on most roofs, with roof runoff rates reduced to about 0.6 cubic feet per second per acre of roof (Ontario 1984).

Rospond (1976) studied the effects of roof storage on site hydrology and found it to be very effective in reducing peak flows. He found that substantial cost savings resulted because of reduced pipe savings, even when considering the extra structural costs associated with strengthening the roofing systems. Controlling roof runoff rates also allows significant savings when infiltration devices are also needed. By storing runoff on the roof, infiltration trenches to store runoff from periods of peak rain intensities are not needed. Simple surface percolation areas created by site grading and landscaping may be sufficient for most cases. Substantial cost savings would then be realized because excavation of trenches and purchased filter fabric and rock fill would not be needed. Long term maintenance of the infiltration area would also be less of a problem with a surface percolation area as compared with an infiltration trench system.

"Green roofs" can be effective for stormwater management, but they usually have limited storage. Their benefits are normally associated with increased evapotranspiration of water from the multiple layers of plants and light-weight soil placed on a roof.

Figure 9. Naturally occurring rooftop storage of rainwater on flat commercial roofs.

Up-Sized Pipes

Enlarged pipe sections have been used to create in-line detention within the storm drainage system. These large pipe sections slow the water velocity and provide a sump for sediment. They remove suspended sediment through sedimentation and bed load sediment by trapping. An up-sized pipe section was monitored in Lansing, Michigan as part of a NURP project (Luzkow, *et al*. 1981). This device had a 54" inlet pipe entering a 144' section of 96" up-sized pipe. A 48" outlet pipe was used. All pipes had their crowns aligned, were made of reinforced concrete, and were at slightly less than a 1% slope. The performance of this device was variable, but much larger in-line systems (such as the deep tunnels in Chicago and Milwaukee for the control of combined sewer overflows) can be expected to have much more consistent and better performance. The required maintenance of underground devices that collect large amounts of sediment may be difficult, however. The Lansing, Michigan, tests of the up-sized pipes found particulate residue removals of about 30 to 50 percent. Large quantities of trapped bed load were also retained (not captured in the automatic samplers), but BOD_5 and nutrient removals were quite low (Luzkow, *et al*. 1981).

Underground Rock-Filled Detention

Another form of underground sedimentation, rock filled detention reservoirs, has been used in very high density commercial areas in the New York City and Boston areas (Heimbuch 1981). These are created under buildings during building excavation and are designed for peak flow reduction and not for pollutant removal. Collected stormwater is distributed through finger type perforated galleries that are rock filled. The stormwater is detained underground and slowly released through a control orifice. These devices are most suitable where excessive rock is produced during excavation. Even though the volumes of the galleries are about three times the volumes required for cisterns that are not rock filled, the rock filled system is substantially less expensive because of the structural support provided and the unfinished walls. Maintenance to remove deposited sediment is not possible, but the distribution system minimizes clogging. Excess volume must be provided for sediment storage for the life of the project. Sediment removal performance may possibly decrease and sediment scour may increase with time.

Use with Other Controls

Detention facilities can be easily used in conjunction with other stormwater control devices. Upland infiltration can be used to treat parking lot and roof runoff, substantially reducing the size of "downstream" detention facilities. Even with source area controls, detention faculties can be very important in industrial areas to help treat dry weather urban runoff. Hawley, *et al*. (1981) described a "treatment-train" that was a series of control devices that used a preliminary sedimentation trap, followed by a grass filter strip and a wet detention pond. This arrangement would substantially decrease sedimentation (and required maintenance) and substantially reduce nuisance conditions in the detention facility.

Beyerlin (1999) described the short-comings of relying on stormwater-detention facilities for complete mitigation of urban runoff problems. Increased winter flood flows, decreased summer low flows, and a general degradation of the stream systems have occurred with development. It was concluded that these problems persisted because of the attempt of replacing the complex interactions of the hydrologic cycle with a pond, which was not possible.

Moody, AL, storage tank for capture of SSOs prior to treatment plant

Austin, TX, MoPac Highway treatment unit using accident discharge diversion capture pond, dry pond, and horizontal flow sand filter

Gravity separation of wet weather solids in CSO swirl concentrators at Tangen, Germany

Retrofitted sump at inlet to provide capture of sediment as catchbasin

Sedimentation forebay before vertical flow sand filter at Austin, TX

Inclined tube settlers and sorbent socks at MCTT installation in Milwaukee, WI

Figure 10. Other controls utilizing sedimentation for the treatment of wet weather flows.

Examples of Detention Pond Performance

There have been many studies that have examined detention pond performance. They included laboratory settling column tests, pilot scale laboratory experiments, and full-scale field experiments. Colston (1974), during laboratory tests, found that fifteen minutes of quiescent settling removed about 80 percent of the suspended solids, 60 percent of the COD and 50 percent of turbidity from urban runoff. Davis (1979) found significant reductions in indicator and pathogenic bacteria with plain sedimentation. Dalrymde, *et al*. (1979), also found that one-hour of settling reduced suspended solids concentrations by 80 to 90 percent. Grizzard, *et al*. (1986) described a series of settling column studies which examined pollutant concentration changes with time for several types of runoff samples having various residue concentrations. This information related quite well with their limited field observations. The samples having high concentrations of suspended solids experienced very high percentage removals in short time periods (about 85% removal after only two hours). Samples having low initial suspended solids concentrations required much longer settling times to achieve the same percentage removals (about 48 hours of settling for 85% removal). Low particulate pollutant concentrations for all samples, however, were found after about 10 to 15 hours of settling.

Chemical Assisted Sedimentation

Chemical addition has been used for many years in water treatment, and in lake management. More recently, full-scale implementations of chemical assisted settling has been used for the treatment of stormwater in wet detention ponds or at outfalls into small urban lakes. The chemicals tested and used include alum (generally a complex of aluminum and sulfate), ferric chloride, and aluminum chloride compounds, plus various coagulant aids.

The addition of alum in Colston's (1974) tests further increased the reduction of particulate residue, COD, and turbidity to about 85 to 97 percent. Gietz (1981), in a series of laboratory tests in Ontario, found that an alum dosage of 4 to 6 mg/L was the most effective for highly polluted runoff. Over-dosages of alum and ferric chloride generally gave poor results. He found that it was difficult to add the correct dosage of coagulant because of the changing pollutant concentrations in the runoff. Low flow velocities also reduced mixing effectiveness and may require mechanical assistance. The flocs that were formed with the coagulants were also easily disturbed by runoff turbulence.

Kronis (1982), in a series of Ontario bench and pilot scale tests, found that disinfection of stormwater with NaOCl at 5 mg/L available chlorine reduced fecal coliform populations to less than 10 organisms per 100 mL. He identified alum dosages of 30 mg/L as a preferred flocculant, with 10 to 30 percent increases in removals of particulate residue, BOD_5, COD, and total phosphorus as compared to plain sedimentation. However, chemical assisted settling generally produced moderate and erratic reductions in bacteria populations. Disinfection in wet detention ponds may be expensive, but it may be the only feasible method of significantly reducing bacteria populations in areas with serious bacteria problems.

Heinzmann (1993) described the development of a coagulation and flocculation treatment procedure for stormwater in Berlin. He found that because the stormwater was weakly buffered and was very soft, a polyaluminum chloride, with a cationic coagulant aid (polyacrylamid), was most suitable. A constant dosage of 0.06 mmol/L (as Al) was used, resulting in pH levels always greater than 6. The constant dosage was possible because the pH and buffering capacity of the stormwater was relatively constant during storms. He found that the best enhanced stormwater treatment process used coagulation and flocculation in a pipe designed for both microfloc and macrofloc formation, and final separation by filtration. The filtration was much better than the one hour sedimentation typically used in Berlin sedimentation tanks. He did find that a six minute flocculation time was sufficient before filtration. He found significant removals of phosphorus (<0.2 mg/L), organic compounds (including PCB and PAHs), solids (<5 mg/L), lead and copper. However, very poor removal of zinc was noted, and pollution prevention (decreased use of galvanized metals) was recommended. In the one-hour sedimentation tanks, without any chemical addition, the phosphorus (about 0.5 mg/L) and solids (about 50 mg/L) effluent concentrations were not nearly as low. The costs for this enhanced treatment (7 to 10 DM/m^3 in 1990) was about 10 to 40% higher than with the ordinary one-hour sedimentation tanks alone.

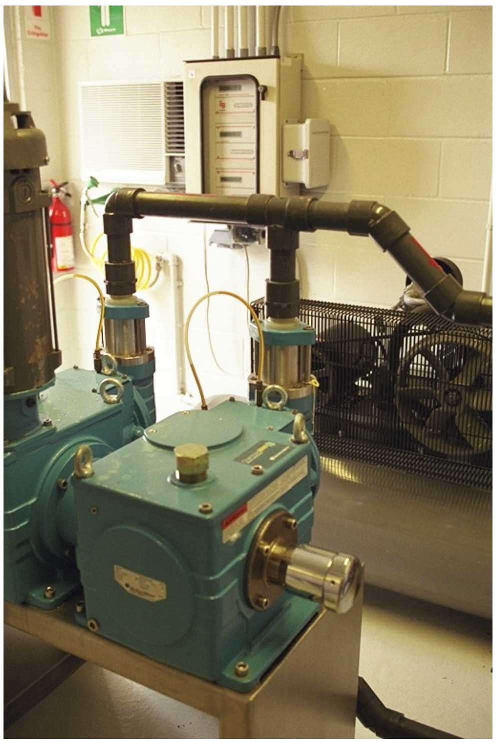
Pitt and Dunkers (1992 and 1995a) described a full-scale stormwater treatment plant, using the Karl Dunkers' system for treatment of separate stormwater and lake water. This system has been operating since 1981 in Lake Rönningesjön, near Stockholm, Sweden. The treatment facility uses ferric chloride and polymer precipitation and crossflow lamella clarifiers for the removal of phosphorus. Excess flows are temporarily stored before treatment inside an in-lake flow-balancing tank (the Flow Balancing Method, or FBM). The stored excess stormwater is then pumped from the flow balancing storage tanks to the treatment facility during dry weather. The overall phosphorus removal rate for the 11 years from 1981 through 1991 was about 17 kg/year. About 40% of the phosphorus removal occurred in the FBM from sedimentation processes, while the remaining occurred in the chemical treatment facility. This phosphorus removal would theoretically cause a reduction in phosphorus concentrations of about 10 μg/L per year in the lake, or a total phosphorus reduction of about 100 μg/L during the data period since the treatment system began operation. About 70% of this phosphorus removal was associated with the treatment of stormwater, while about 30% was associated with the treatment of lake water. The lake phosphorus concentration improvements averaged only about 50 μg/L. This was only about one-half of the theoretical improvement, probably because of sediment-water interchange of phosphorus, or other unmeasured and untreated phosphorus sources entering the lake.

FBM concept for pumpback of stormwater discharges and lake water to ferric chloride treatment plant (Karl Dunkers drawing)

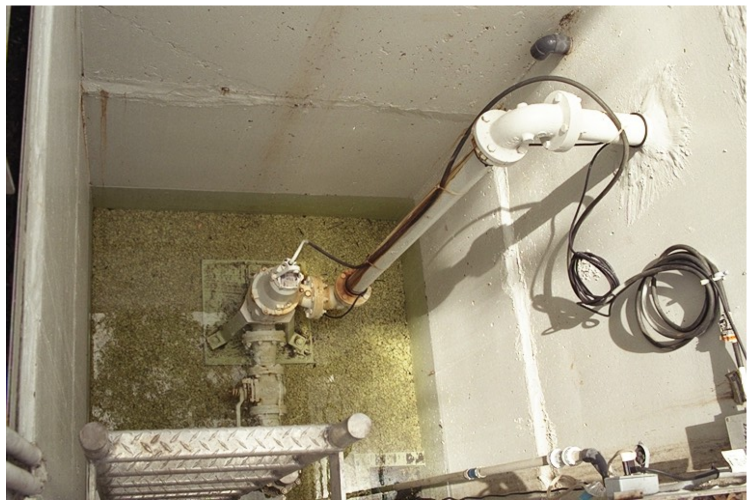
FBM installation in Sweden (Karl Dunkers photo)

Figure 11. Flow balancing method for the treatment of surface runoff.

The 1996 NALMS (North American Lakes Management Society) conference in Minneapolis/St. Paul included several presentations describing the use of alum for stormwater treatment. Harper and Herr (1996) describe the historical use of alum to treat stormwater entering Lake Ella in Tallahassee, FL, which began in 1986. A liquid slurry of alum is injected into the major storm drainage entering the lake, on a flow-weighted basis during rains. The alum forms precipitates with phosphorus, suspended solids, and heavy metals, which then settle in the lake. This treatment system resulted in immediate and substantial improvements to Lake Ella water quality. There were 23 alum stormwater treatment systems in Florida by 1995. Harper and Herr (1996) reported that alum treatment of stormwater has consistently achieved 90% reductions in total phosphors, 50 to 70% reductions in total nitrogen, 50 to 90% reductions in heavy metals, and >99% reductions in fecal coliform bacteria. The precipitates of the phosphorus and heavy metals have been shown to be extremely stable over a wide range of dissolved oxygen and pH conditions in the receiving waters.



Orlando, FL, alum injection system



Orlando, FL, alum injection system

Orlando, FL, settling ponds to remove floc after alum injection, before discharge to lake

Figure 12. Chemical-assisted sedimentation for stormwater treatment.

Herr and Harper (1996) also reported on a very large alum project at Lake Maggiore in St. Petersburg, FL. This 156 ha lake receives stormwater from a 927 ha watershed. Water quality problems were noted as early as the 1950s that included fish kills, algal blooms, nuisance macrophyte algal growths, and high bacteria levels. An environmental assessment determined that an 80% reduction in the annual phosphorus discharges from the stormwater and baseflow would result in an acceptable trophic status for the lake. Five alum treatment plants were then designed and were to be operational by August 1997, comprising the largest alum stormwater treatment system ever built.

Kloiber and Brezonik (1996) described an alum pilot-scale treatment system for stormwater, located in Minnesota. This system injected 1 mg/L (as Al) alum into a storm sewer at a pumping station just upstream of a 1.2 acre wet detention pond. The few minutes travel time between injection and the pond allowed 75 to 80% reductions in soluble reactive phosphorus. However, the pond retained only 40% of the added aluminum, increasing to 70% when a coagulant aid was used. The lowest total aluminum concentration in the pond effluent was 0.26 mg/L, still exceeding the water quality standard. They concluded that closer evaluations of the toxicity and bioavailability of the aluminum associated with alum stormwater treatment was needed. During treatability tests of stormwater from critical source areas, Pitt, *et al.* (1995) found that alum addition significantly increased the toxicity of the water (as indicated using the Microtox screening procedure).

Pitt recently conducted a series of chemical addition treatability tests for stormwater. He examined alum, ferric chloride, and ferric sulfate (all with and without organic polymers), and organic polymers alone. He also tested the benefits of adding a microsand (75 to 150 μm) as a coagulant aid. Initial findings indicate that ferric chloride with the microsand was the most effective chemical for treating stormwater. The concentrations of the ferric chloride were in the range of 30 to 80 mg/L, and the microsand was added to produce a turbidity of about 200 NTU. Heavy metals (copper, lead, and zinc, in both particulate and filterable forms) and toxicants (as indicated by the Microtox™ screening test) removals were greater than 80%, with many tests greater than 95%. Phosphates were also significantly reduced (by about 50%). Alum added toxicity (possibly through zinc contamination in the alum, or by the dissolved aluminum) and many of the polymers also added COD and toxicity. Figures 13 through 18 show typical heavy metal removals for several chemical addition tests over a range of dosages.

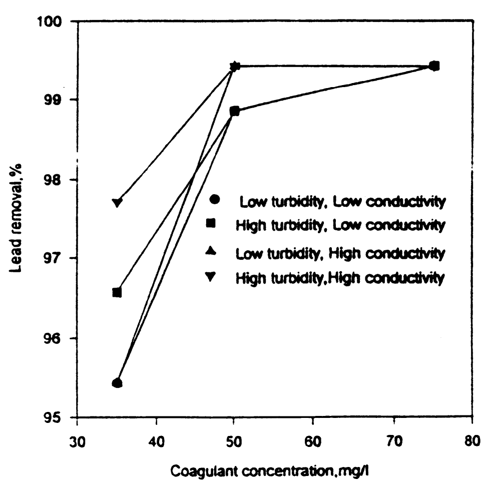


Figure 13. Lead removal using ferric chloride.

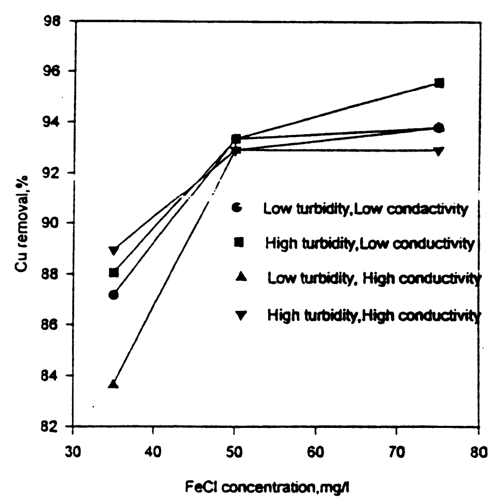


Figure 14. Copper removal using ferric chloride.

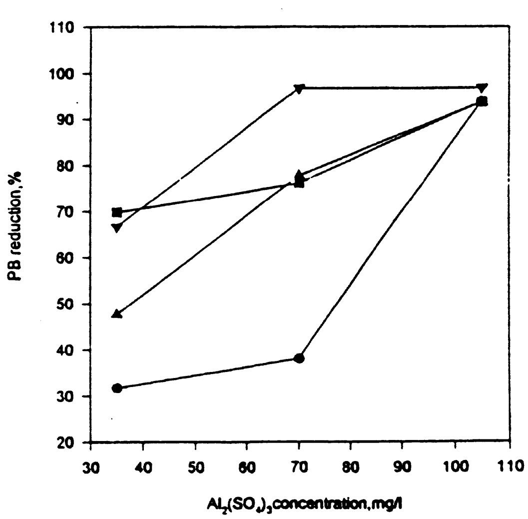


Figure 15. Lead removal using alum ($Al_2(SO_4)_3$

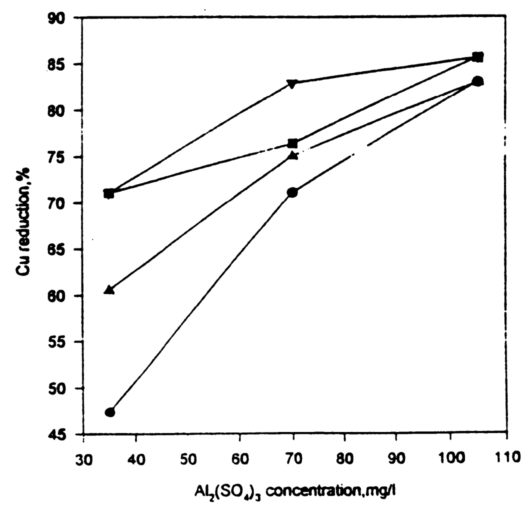


Figure 16. Copper removal using alum ($Al_2(SO_4)_3$

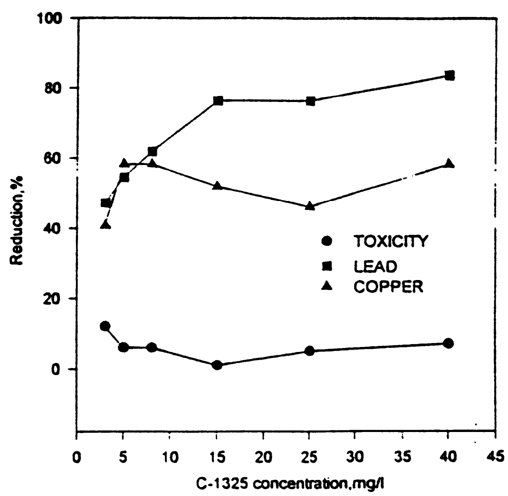


Figure 17. Metals and toxicant reductions using organic polymer (C-1325).

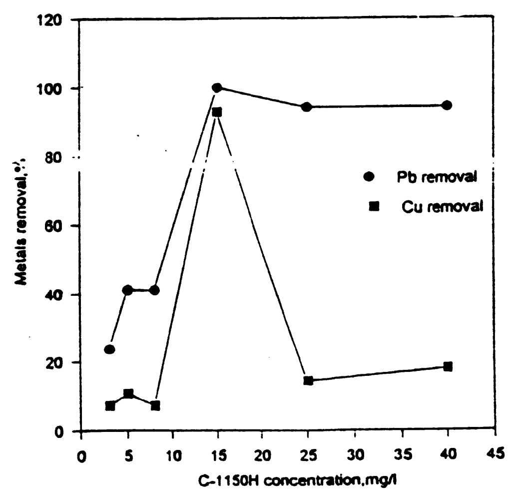


Figure 18. Metals removed using organic polymer (C-1150H).

Example Use of Chemical-Assisted Sedimentation at Construction Sites

Larcombe (1999) of the Auckland Regional Council (New Zealand) prepared a report (*Technical Publication on Chemical Removal of Sediment from Earthworks Stormwater*) describing the use of chemical-assisted sedimentation for the control of construction site sediment. They tested both solid forms of flocculation material (Magnasol Floc Blocs Allied Colloids, Australia Pty Ltd., NZ agent Chemiplas NZ Limited) and liquid chemicals at several construction areas. These included sites along the extension of the northern motorway (ALPURT), and at a residential subdivision development (Greenhithe). The extensive field trials using aluminum sulfate (Alum), and polyaluminum chloride (PAC) were carried out during construction of the initial stages of the northern motorway. They then developed a passive dosing system for the treatment of the construction site runoff. This system proved highly effective under a wide range of storm conditions. The following discussion is summarized from that report.

Conditions when Chemical Treatment may be Necessary

The requirements for sediment ponds at construction sites are given in the Auckland Regional Council guidance (TP 90, *Erosion and Sediment Control*, 1999). The performance of ponds constructed according to these specifications is generally good, but a number of situations have been identified where chemical treatment can provide a marked improvement in sediment removal. Chemical treatment is important when a pond of the required size cannot be constructed. This may occur because of topographical constraints, difficult soil conditions, or the presence of natural habitat of high value. In some situations, the design of the pond cannot be optimized in terms of shape, depth, location of inlet and outlet, or energy attenuation of the inflow. Some soil types produce suspended solids in construction site runoff that has very poor settling characteristics in a normal sediment pond. There is also a higher risk of increased erosion and sediment losses during rainstorms in areas having highly erodible soils, or very steep or long slope lengths. Some common uses of construction sites, such as repeated machinery movement on haul roads, can result in high sediment loadings in stormwater. Finally, chemical treatment provides a means of reducing the sediment discharge to highly sensitive receiving environments.

Initial Tests

Two types of chemicals were considered for the initial bench testing and field trials, polyelectrolyte flocculants (polymer or polyacrylamide) and aluminum coagulants (aluminum sulfate (alum) and polyaluminum chloride, (PAC)). Cationic polyelectrolytes have a greater toxicity to fish and other aquatic organisms than anionic, or nonionic, polyelectrolytes, because the gills of fish are anionic and the cationic polymer binds to them resulting in mechanical suffocation.

Polyelectrolyte Flocculants

Bench testing showed that a number of polyacrylamides resulted in good removal of suspended solids from the construction site runoff water. However, they identified several difficulties hindering the use of liquid polyacrylamides at construction sites. The most serious difficulty is that liquid polyacrylamide concentrates are highly viscous and would require onsite predilution with water to achieve a suitable consistency for dosing and mixing with construction site runoff. This would require mixing equipment and storage tanks, along with electric power. In addition, the diluted polyacrylamide has a limited storage life.

Three solid polyacrylamide products (Floc Bloc), marketed by Allied Colloids, were evaluated in bench-scale tests. The products were: Percol AN1 and AN2 (both anionic polyacrylamide blends) and Percol CN1 (a cationic polyacrylamide blend). The floc blocs were 300 x 100 x 85 mm and weighed 3 kg. AN2 performed best when using runoff from sites having either clay or limestone soils. AN2, an anionic polyacrylamide, also had a lower toxicity. Effective dose rates were between 1 and 4 mg/L of dry AN2. Higher concentrations led to reductions in flocculation and suspended sediment removal. AN2, even at excessive dosages of about 8 mg/L, did not affect pH.

Aluminum Coagulants

A major issue with aluminum coagulants is they contain large concentrations of ionic aluminum, the toxic form of aluminum. It is generally agreed that dissolved aluminum at concentrations as high as 50 to 100 μg/L and at pH values between 6.5 and 8.0 present little threat of toxicity. At lower pH, the toxicity increases due to possible mucus formations on the gills of fish. The toxic aluminum associated with the coagulant dose is very rapidly reduced by the precipitation and coagulation reactions. The insoluble precipitates (incorporating metals, nutrients, and solids) that form after aluminum coagulants are added to water are stable and denser than water. The alum floc that is formed is not toxic to benthic organisms. Most pollutants are tightly bound to the aluminum matrix with little likelihood of release from either dried or wet sludges within normal pH and redox ranges.

Solid Floc Blocks

The initial tests indicated advantages to the use of the solid floc blocks, particularly on sites with difficult access; sites with only small construction areas, or sites where there was a need for short term treatment only. They therefore followed up their initial tests with detailed field assessments to determine the best methods to use the blocs to obtain the most effective suspended solids removal in highly variable flow conditions.

Field trials using solid floc blocs:

Preliminary field trials used an AN2 floc bloc to treat sediment-laden runoff from a construction site having limestone soils. The first trials placed the floc blocs in plastic mesh bags in plywood flumes through which the runoff from the site was directed. Those trials encountered problems with the high bedload of solids in the runoff flow that accumulated against and partially buried the floc bloc and inhibited solubility of the chemical. The trial was then moved to a channel between a forebay and the settlement pond (for pre-treatment of the water to remove the large materials), and demonstrated that new floc blocs achieved good treatment for low flows (about 2 L/s) and when the suspended solids was between 10,000 to 20,000 mg/L. However, the high influent solids in the runoff continued to be a problem, and following an intense rainfall event, both the forebay and floc bloc channel were filled with sediment. As the construction site area was gradually stabilized, the quality of runoff improved. Additional tests in a new flume showed that effective treatment was achieved for new floc blocs at flows of about 2 L/s with suspended solids concentrations up to 5,000 mg/L.

The Auckland Regional Council concluded that a constant stormwater flow through a floc bloc treatment flume is best in terms of providing the optimum chemical dose for suspended solids removal. It was difficult to set up an array of floc blocs that provided optimal dosing for highly variable flows. They conclude that for any floc bloc system, it was desirable to restrict the maximum flow to about *2*0 L/s. The treatment capacity of the tested floc bloc (AN2) at a limestone soil site was about 2 L/s per bloc at 10,000 mg/L suspended solids, and about 1 L/s per bloc at 20,000 mg/L suspended solids. They concluded that floc bloc treatment has a good potential for removal of suspended solids, particularly for small catchments, when flow balancing can be achieved prior to treatment, and the stormwater is of consistent quality.

The preliminary results of the trials using floc blocs were encouraging despite the above noted problems. They therefore conducted further tests using a more comprehensive field trial in order to determine the effectiveness of the floc bloc form of polyacrylamide, and to design a robust dosing system that can be used for different types of sites. The trial site was four ha in area and included monitoring over a wide range of site development phases.

The preliminary trials stressed the importance of achieving a moderate degree of turbulence in the floc bloc area. The design of the flume included cages that held the floc blocs vertically and allowed the number of blocks to be easily changed. The flume was also roofed to shade the floc blocs to prevent their degradation when exposed to sun. A forebay of about $10m^3$ upchannel of the flume was intended to trap bedload and other heavy material that could interfere with floc bloc operation. However, because of site constraints, the forebay was only about $5m^3$ in volume.

Serious cracking of the floc blocs were noted during an initial dry period of several weeks in the summer. Large pieces fell from the blocs that eventually formed a sticky mass that blocked the bottom of the bloc cages and interfered with the flow paths during subsequent periods of runoff. An intensive rain (about 30 mm of rain during 40 minutes) caused extensive site erosion and the very high sediment loads filled the forebay and treatment flume, plus about 60 m^3 of sediment was trapped in the pond. Although the floc bloc treatment system was overwhelmed by bedload during this event, the treated pond had lower suspended solids concentrations in the discharge than the other two ponds (2,400 mg/L vs. 7,300 mg/L). During more moderate events, treated pond effluent concentrations were about 500 mg/L, compared to typical effluent concentrations of about 1,000 to 2,000 mg/L from untreated ponds.

They found that a construction site having saturated soils can produce runoff flows of more than 60 L/s per hectare under the intense rainfall conditions that may occur in the Auckland Region. Also, the runoff rates from construction sites can be extremely variable, making it difficult to provide an appropriate array of floc blocs that will provide optimal dosing for such variable flows. Also, with large numbers of blocs in a single channel system, there could be some potential for overdosing in low flow conditions.

Liquid Coagulants

An automatic liquid coagulant dosage system was designed to eliminate many of the problems found during the use of the floc blocs. This system could have included a flow measurement weir or flume, an ultrasonic sensor and signal generating unit, and a battery driven dosing pump. The estimated cost for this unit was about $6,000, including installation and shelter, but it would still require electricity at the site. An alternative system was therefore considered that would not be dependent on site power and direct flow measurements. This passive dosing system was estimated to cost about $1,000. Figure 19 shows an example of this system at a New Zealand construction site. Figure 20 shows the main internal components of this system.

Figure 19. Auckland Regional Council rainfall-driven dosing system.

Figure 20. Main components of Auckland Regional Council rainfall-driven chemical dosing system.

The rainfall volume collected from a small roof (area proportionate to the construction site drainage area and chemical dosage desired) is used to displace the liquid chemical from a storage tank into the runoff channel before a sediment pond. Their design (based on the field trails) assumes that 100% of the rainfall falling onto saturated disturbed areas and 60% of the rainfall falling onto stabilized areas, needs to be treated.

The roof runoff is drained by gravity into an elevated header tank that has a volume below an overflow equal to the detention storage of the site. Figure 20 shows a second overflow above the main overflow tube that causes an increased dosage rate for very high rain intensities. The overflow tubes from this elevated header tank are directed into a displacement tank that is floating in the main chemical tank. As the water flows into this floating displacement tank from the elevated header tank, the chemical is pushed out the reservoir tank and through the dosage line to the dosing location in the flow path.

Example of Volumetric Design

The following example is from the Auckland Regional Council report (Larcombe 1999), assuming a 1 ha (2.5 acre) site and using PAC. The target dosage is 8 mg/L (the actual dosage needs to be determined from bench-scale tests using actual site runoff, or runoff from a similar site). Liquid PAC obtained from Fernz Chemicals contains 10.1% Al_2O_3 by weight, equivalent to 53,500 mg/kg aluminum or 64,200 mg/L aluminum, as the density of PAC is 1.20. Therefore, 1L of PAC would treat 8,020 L of construction site runoff at a dose rate of 8 mg aluminum per liter.

• Roof runoff area calculation:

About 500 m^3 of runoff would result from each 50 mm of rainfall on saturated disturbed soil per 1 ha of catchment area. The volume of PAC required to treat 500 m^3 of runoff is 62.3 L at 8 mg/L. The density of PAC is 1.2. Therefore, 74.8 L of rainwater is needed to displace 62.3 L of PAC. This would require an area of 1.5 square meters for a 50 mm rain. Table 1 presents the rainfall catchment area required for different PAC dose rates (at 10.1% Al_2O_3 by weight).

Table 1. Rainfall Catchment Area Required for Different PAC Derived Aluminum Dose Rates

Aluminum dose required (mg/L)	Roof Catchment Area per hectare of Saturated Disturbed Ground (m^2)	Roof Catchment Area per hectare of Stabilized Catchment (m^2)
2	0.375	0.225
4	0.75	0.45
6	1.125	0.675
8	1.5	0.90
10	1.875	1.125
12	2.25	1.35

• Header tank size calculation:

The header tank allows initial abstraction losses on the site to be considered (provides a delayed dosage at the beginning of the rain) and continued dosing after the rain ends, but as the runoff continues. For the Auckland test sites, the header tank allows 15mm of rainfall before dosing commences. This would require a header tank volume below the lowest overflow of 15 L per m^2 of roof rainfall catchment area. The lowest overflow consists of a 4mm internal diameter tube, while the high rate outlet has sufficient capacity to carry the maximum predicted flow from the roof catchment during short-term rainfalls of about 40mm/hour.

• Displacement tank and chemical reservoir tank size calculation:

The displacement tank should fit neatly inside the reservoir tank when floating on the liquid chemical. A larger displacement tank and reservoir tank system will reduce the required frequency of servicing. Auckland Regional Council recommends that the minimum displacement tank capacity should be the 24-hour rainfall for a 2-year return period. In their field studies, this was about 86 mm of rain. With a 1.5 m^2 roof catchment area, this would result in a volume of 129 L. Their standard design used a 400 L displacement tank inside a 550 L reservoir tank, providing dosing of up to 320 L of PAC. Their standard design called for the outlet tubing to be placed at the 400 L chemical level in the reservoir tank so it could hold the contents of two standard 200 L drums of PAC. The outlet tubing level is determined with the floating displacement tank in place to account for the slight displacement associated with the weight of the empty displacement tank.

Setup and Servicing of the Rainfall Driven Dosing System

• Header tank setup and maintenance:

The level of the low capacity overflow from the header tank (the vertical position of the tubing exiting the tank) is set to allow for initial abstractions before chemical dosing starts. In the summer, after a week or more without rain, this was found to be about 15mm in the Auckland test areas. However, when a very intense rain of about 15 mm in 15 minutes fell on dry ground, substantial runoff occurred, and the delay in the start of dosing resulted in insufficient dosing. In wet weather, the header tank was set with no delay in dosing. During long dry periods, the header tank volume below the low capacity outlet is adjusted to provide for no dosing during the first part of the next rainfall. This is to prevent overdosing of the sediment pond which may cause reduced pH levels and associated increased free aluminum concentrations, plus it also conserves PAC. After each event, the water is removed from the header tank using a siphon. It also would be possible to install a drain valve in the bottom of the header tank for easier emptying.

• Displacement and chemical reservoir tank maintenance:

The chemical level in the reservoir tank and the water level in the displacement tank also need to be periodically checked. If the water level is too high, or the chemical level too low, then maintenance is needed. The displacement tank may be either emptied using a siphon, or baled out by hand. The chemical reservoir can be filled using a hand operated drum pump to refill the reservoir from the 200 L delivery drum.

• Monitoring and adjustment for changing site conditions:

The passive chemical dosing treatment system needs to be carefully monitored during the first few runoff events to check that the system is effective, and to ensure that overdosing is not occurring. If overdosing is suspected because the pond dead storage water is exceptionally clear, samples should be analyzed for pH and dissolved aluminum. If overdosing is occurring, reducing the size of the rainfall catchment tray can reduce the chemical dose. This can be done by placing a diagonal batten across the tray and directing some of the runoff through a waste hole.

Field Trials of Chemical-Assisted Sedimentation

Alum additions:

Initial tests indicated that alum additions (at 5.5 mg aluminum/L) worked well under a wide range of rain conditions at a site having limestone soils, including during one event having 25 mm of rain in 25 minutes. During this intense rain, the alum-treated pond had a 92% reduction in suspended solids, compared to only 10% in the same pond for a similar heavy rain during a period of no alum addition. The pH was reduced by about 0.5 pH units and the discharged dissolved aluminum concentration was about 0.1 mg/L during these tests. The pH did not undergo major reductions during bench-scale tests, even when the dosage approached 12.6 mg/L.

Polyaluminum Chloride (PAC) additions:

The runoff from test sites having clay soils had more acidic runoff than the sites that had limestone soils. At the clay sites, alum treated runoff (after the pond) had pH values that ranged from 4.3 to 5.9, while runoff treated with PAC had pH values ranging from 5.5 to 6.7. They therefore decided that PAC was a more suitable choice, especially for clayey soil conditions. Overall, the Auckland Regional Council has data from 21 different sediment ponds that used passive PAC additions, with drainage areas ranging from 0.5 to 15 ha (1.3 to 38 acres). The overall suspended solids treatment efficiency of PAC-treated ponds has been between 90 to 99 % for ponds having good physical designs. Lower treatment efficiencies have occurred where there have been problems with decants not operating properly, or physical problems such as multiple inflow points, high inflow energy, and poor separation of inlets and outlets. Figure 21 shows the typical multiple decant risers used at Auckland Regional Council sediment pond sites to allow more efficient settling of the floc.


Figure 21. Multi-level, perforated, floating discharges (decants) to better retain floc.

The influent concentrations of suspended solids for the PAC-treated ponds ranged from 746 to 26,300 mg/L (median of about 16,000 mg/), while the treated effluent ranged from 3 to 966 mg/L (median of about 50 to 100 mg/L). The percentage suspended solids reductions ranged from 77 to 99%, with a median of about 95%. The untreated pond had much poorer levels of treatment (about 10%). The dissolved aluminum concentrations in the outflow from the untreated pond were much higher (0.29 – 0.31 mg/L) than in the outflows from the treated ponds (0.010 – 0.084 mg/L). When the PAC was added at too high a concentration, the pH levels dropped to as low as 4.7, although the effluent dissolved aluminum was still low and the suspended solids concentrations were very low (as low as 10 mg/L). Typical effluent pH conditions were between 6 and 7.

Design of Sediment Ponds with Aluminum Coagulant Treatment

Although chemical treatment using aluminum coagulants is capable of achieving effective sediment removal from stormwater with relatively brief detention times for settlement in quiescent conditions, there are practical difficulties in achieving quiescent conditions in construction site ponds when high flows are being discharged into a small pond. The Auckland Regional Council recommends a minimum size of 1.5% (150 cubic meters per hectare) for aluminum coagulant treated ponds. Analysis of the long term rainfall and construction site suspended solids data obtained during the field trials shows that more than 60% of the sediment from a construction site occurs during the two or three rainstorms per construction season which exceed 30 mm in 24 hours.

Table 2 shows the expected advantages of using PAC assisted sedimentation for different sized wet sediment ponds in the Auckland, New Zealand, area. Chemical treatment results in a major improvement in the efficiency of sediment capture during rainstorms that exceed the hydraulic capacity of a sediment pond. This is indicated by the large improvements in sediment capture for the smaller ponds with PAC addition.

Table 2. Summary of Advantages of PAC Treatment of Construction Site Runoff for Normal Catchments during a Construction Season

	Wet Sediment Pond Size		
	3%	2%	1.5%
1. Without PAC treatment:			
Total sediment discharged to receiving water (tones dry wt per hectare)	5.8	9.2	12.0
Efficiency of sediment removal in pond (%)	81	69	60
2. With PAC treatment:			
Total sediment discharged to receiving water. (tones dry wt per hectare)	1.0	2.1	2.8
Efficiency of sediment removal in pond (%)	97	93	90

Full-scale Demonstrations of Detention Ponds

Wet Detention Ponds

The use of detention ponds for both water quality and quantity benefits is relatively new. Wet pond stormwater quality benefits have been commonly reported in the literature since the 1970s, while the water quality benefits of dry detention ponds have only recently been adequately described (Hall 1990).

The Nationwide Urban Runoff Program (NURP) included full-scale monitoring of nine wet detention ponds (EPA 1983). The Lansing project included two up-sized pipes, plus a larger detention pond. The NURP project located in Glen Ellyn (west of Chicago) monitored a small lake, the largest pond monitored during the NURP program. Ann Arbor, Michigan, monitoring included three detention ponds, Long Island, New York, studied one pond, while the Washington D.C. project included one pond. About 150 storms were completely monitored at these ponds, and the performances ranged from negative removals for the smallest up-sized pipe installation, to more than 90 percent removal of suspended solids at the largest wet ponds. The best wet detention ponds also reported BOD_5 and COD removals of about 70 percent, nutrient removals of about 60 to 70 percent, and heavy metal removals of about 60 to 95 percent.

The Lansing NURP project monitored a wet detention pond (Luzkow, *et al.* 1981). The monitored pond was located on a golf course (receiving urban runoff from an adjacent residential and commercial area). Suspended solids removals were about 70 percent for moderate rains (10 to 25 mm rains) while phosphorus removals were usually greater than 50 percent. Total Kjeldahl nitrogen removals ranged from about 30 to 50 percent. The removals of these pollutants increased with increasing storm size because of the larger quantities of pollutants carried by the larger storms. During small storms, most of the discharge water was displaced water from preceding storms that was still relatively polluted. For rains smaller than about 0.25 inches, the discharge pollutant yields were typically greater than the input yields for most of the pollutants.

Hey and Schaefer (1983), as part of a NURP project, reported substantial urban runoff improvements for a small Chicago area (Glen Ellyn) lake that was about ten percent of the residential area served. Lake monitoring indicated about 85 percent suspended solids removals, even with residence times substantially longer than the four hours reported to give 95 percent suspended solids removals during lab tests. They felt that flocculation was responsible for the differences between the lab tests and the observed field results. Total phosphorus removals were about 35 percent, while heavy metal (copper, lead, and zinc) removals were about 75 percent.

Two wet detention ponds near Toronto, Ontario, were monitored from 1977 through 1979 (Brydges and Robinson 1986). Lake Aquitaine is 1.9 ha in size and receives runoff from a 43 ha urban watershed. Observed pollutant reductions were about 70 to 90 percent for suspended solids, 25 to 60 percent for nitrogen, and about 80 percent for phosphorus. The much smaller Lake Wabukayne (0.8 ha) received runoff from a much larger urban area (186 ha).

The smaller Lake Wabukayne experienced much smaller pollutant reductions: about 30 percent for suspended solids, less than 25 percent for nitrogen, and 10 to 30 percent for phosphorus.

Oliver, *et al*. (1981), monitored a small lake detention facility in Rolla, Missouri. Suspended solids yield reductions averaged about 88 percent, with 54 and 60 percent yield reductions for COD and total phosphorus. Organic nitrogen yields were reduced by about 22 percent.

Gietz (1983) studied a 1.3 ha wet detention pond serving a 60 ha urban watershed near Ottawa, Ontario. Batch operation of the pond resulted in substantial pollutant control improvements for particulate residue, bacteria, phosphorus, and nitrate nitrogen. Continuous operation gave slightly better performance for BOD_5 and organic nitrogen. Suspended solids reductions were about 80 to 95 percent, BOD_5 reductions were about 35 to 45 percent, bacteria was reduced by about 50 to 95 percent, phosphorus by about 70 to 85 percent, and organic nitrogen by about 45 to 50 percent.

Numerous additional detention pond performance studies have been conducted in the years since the Nationwide Urban Runoff Program. Yousef, *et al.* (1986) reported some long-term nutrient removal information for a detention pond in Florida having very long residence times and substantial algal and rooted aquatic plant growths. He found 80 to 90 percent removals of soluble nutrients due to plant uptake. Particulate nutrient removals, however, were quite poor (about ten percent). These particulate nutrient forms were mostly nitrogen and phosphorus that were tied up with the plant cells and not the particulate nutrient forms that were discharged to the pond with the runoff (Driscoll 1986). It is difficult to design a detention pond to obtain a desired net removal of nutrients (soluble plus particulate forms) because of the plant uptake and conversion of soluble forms to particulate cellular forms. If the plants are not removed from the detention pond, the particulate cellular nutrients will be released back into the water as more available (soluble) forms during periods of plant die-off. The role of aquatic plants in nutrient (and other pollutant) removals for cold climatic conditions is not well understood. Substantial releases of pollutants that had been “removed” by aquatic plants during the growing season when the plants die back in the fall is expected, resulting in substantially less removals than indicated by warm weather monitoring alone.

Hvitved-Jacobsen, *et al.* (1987) along with Martin and Miller (1987) described pollutant removal benefits of wet detention ponds. Niemczynowicz (1990) described stormwater detention pond practices in Sweden. Van Buren, *et al*. (1996) also reported on the performance of an on-stream pond located in Kingston, Ontario. They describe their monitoring activities and measures taken to enhance performance.

Hvitved-Jacobsen, *et al.* (1994) examined the most effective treatment systems for treating urban and highway runoff in Denmark. They concluded that wet detention ponds were the most efficient and suitable solution for the removal of most pollutants of concern from both highway and urban runoff. Denmark does not have any effluent standards and the acceptable pollutant discharges are therefore determined based on specific receiving water requirements. They concluded that CSO problems were causing acute receiving water effects (hydraulic problems, oxygen depletion, high bacterial pollution, etc.), requiring treatment designs based on design storm concepts. However, both urban and highway runoff were mostly causing accumulative (chronic) effects (associated with suspended solids, toxicants, and nutrient discharges) and treatment designs therefore need to be based on long-term pollutant mass discharge reductions. It was evident that relatively low concentrations of pollutants must be reduced, and that large volumes of water must be treated in a short time period. For these reasons, and for the specific pollutants of concern, they concluded that wet detention ponds were the most effective option, even though the first wet detention pond was only constructed in Denmark in 1989. Their recommended design was based on: detention pond volume (about 250 m^3 per effective hectare of drainage area), water depth, pond shape, use of plants (covering at least 30% of the water surface), and the use of a grit removal forebay. This pond design was evaluated using the computer program MOUSE/SAMBA for long-term simulations using Aalborg, Denmark, rains. The resulting mass removals using this design were excellent for suspended solids (80 to 90%) phosphorus (60 to 70%) and heavy metals (40 to 90%).

Mayer, *et al*. (1996) examined sediment and water quality conditions in four wet detention ponds in Toronto. They found that poor water circulation in the summer months between rains decreased the pond water quality, especially for dissolved oxygen and nutrients. Anaerobic conditions near the pond water-sediment interface in two of the ponds caused elevated ammonia concentrations. They felt that decomposition of nitrogenous organic matter (from terrestrial and aquatic plant debris) was the likely source of the ammonia. They also found prolific algal growths in the same two ponds in the summer, with chlorophyll a concentrations of about 30 μg/L. The chlorophyll a concentrations in the other two ponds were much lower, between about 3 and 10 μg/L.

Petterson, *et al.* (1999) studied the pollutant-removal efficiency of two stormwater ponds in Sweden. Observed outflow pollutant conditions were independent of the influent conditions for the two ponds. They also found that pollutant removal efficiency increased for increasing values of ratio of the pond surface area to the watershed impervious area, up to about 250 m^2/ha, while the benefits of larger ponds were not as important.

The pollutant removal efficiency of three wet detention ponds was investigated by Mallin, *et al.* (2002). The results for solids removal were good for all ponds, but nutrient removal was variable. To achieve good reduction in a variety of pollutants, wet pond design should include maximizing the contact time of inflowing water with rooted vegetation and organic sediments. This could be achieved through a physical pond design that provides a high length to width ratio, and planting of native macrophyte species.

Dry Detention Ponds

Stanley (1996) examined the pollution removal performance at a dry detention pond in Greenville, NC, during eight storms. The pond was 0.7 ha in size and the watershed was 81 ha of mostly medium density single family residential homes, with some multifamily units, and a short commercial strip. The observed reductions were low to moderate for suspended solids (42 to 83%), phosphate (-5 to 36%), nitrate nitrogen (-52 to 21%), ammonia nitrogen (-66 to 43%), copper (11 to 54%), lead (2 to 79%), and zinc (6 to 38%). Stanley also summarized the median concentration reductions at dry detention ponds studied by others, shown in Table 3. In all cases, the removals of the stormwater pollutants were substantially less than would occur at well designed and operated wet detention ponds. The resuspension of previously deposited sediment during subsequent rains was typically noted as the likely cause of these low removals. The conditions at the Greenville pond were re-examined three years after its construction. The most notable changes were that the pond bottom and interior banks of the perimeter dike were covered with weeds and many sapling trees (mostly willows), indicating that the interior areas have been too wet to permit mowing. The perforated riser was also partially clogged and some pooling was occurring near the pond outlet. It seemed that the dry pond was evolving into a wetlands. The monitoring activity was conducted a few months after the pond was constructed and was not affected by these later changes. Stanley felt that the wetlands environment, with the woody vegetation, if allowed to spread, could actually increase the pollutant trapping performance of the facility. With continued no maintenance, the dry pond will eventually turn into a wet pond, with a significant permanent pool. The pollutant retention capability would increase, at the expense of decreased hydraulic benefits and less flood protection than originally planned. Maintenance problems in dry ponds had also been commonly noted in earlier Maryland surveys.

The benefits of off-line stormwater detention ponds were examined by Nix and Durrans (1996). Off-line ponds (side-stream ponds) are designed so that only the peak portion of a stream flow is diverted to the pond (by an in-stream diversion structure). They are designed to reduce the peak flows from developed areas, with no direct water quality benefits, and are typically dry ponds. Off-line ponds are smaller (by as much as 20 to 50%) than on-line ponds (where the complete storm flow passes through the pond) for the same peak flow reductions. However, the outflow hydrographs from the two types of ponds are substantially different. The off-line ponds produce peak outflows earlier and the peak flows do not occur for as long a period of time. If located in the upper portion of a watershed, off-line ponds may worsen flooding problems further downstream, whereas downstream on-line ponds tend to worsen basin outlet area flooding. Off-line dry ponds can be used in conjunction with on-line wet ponds to advantage to provide both water quality and flood prevention benefits. Off-line ponds have an advantage in that they do not interfere with the passage of fish and other wildlife and they do not have to dramatically affect the physical character of the by-passed stream itself. On-line dry ponds would substantially degrade the steam habitat by removing cover and radically changing the channel dimensions. The peak flow rate reductions can also have significant bank erosion benefits in the vicinity of the pond, although these benefits would be decreased further downstream.

Bartone and Uchrin (1999) compared the performance of two dry-stormwater-detention facilities, one having a concrete low-flow channel, and the other with a vegetated low-flow channel during four events. As they expected, the detention pond having the concrete channel was ineffective for stormwater quality control. However, the basin with the vegetated channel was also found essentially ineffective for water quality improvement, with flushing of previously captured pollutants being the most likely reason for the poor performance.

Table 3. Summary of Dry Detention Pond Pollutant Removal Capabilities (Stanley 1996)

	Detention pond name and location						
	Lakeridge, northern Virginia	London, northern Virginia	Stedwick, Montgomery Co., Md.	Maple Run, Austin, Texas	Oakhampton, Baltimore, Md.	Lawrence, Kansas	Greenville, N.C
Watershed, acres	88	11	34	28	17	12	200
Imperviousness, %						49	
Hours to drain after filling	1-2	<10	6-12	-9		6-16	75
Storms monitored	28	27	25	17		19	8
Removal efficiencies, %							
TSS	14	29	70	30	87	3	71
TP	20	40	13	18	26	19	14
PO_4-P	-6				-12	0	26
TN	10	25	24	35			26
NO_3-N	9			52	-10	20	-2
NH_4-N				55	54	69	9
TOC				30		-3	10
POC							45
DOC							-6
Cu				31			26
Pb		39	62	29		66	55
Zn	-10	24	57	-38		65	26

Guo, *et al.* (2000) experimented with modifications to the outlet structure of a dry detention basin to improve pollutant removal performance of the pond, and found no conclusive correlation between the pollutant removal efficiency and the detention time. Instead, pollutant removal efficiency in the field was strongly dependent on the inflow concentration.

Pond/Wetlands

Yu, *et al.* (1996) monitored seven wetlands in Virginia for the Virginia Department of Transportation (VDOT). A total of 25 events had been evaluated, with the best pollution retention being more than 50% for suspended solids, about 30% for COD, and over 80% for orthophosphate, at a well-designed and well-maintained facility. They found no harm to the wetland vegetation due to the highway runoff. They are also creating an Arc/Info GIS system to allow the VDOT to track the more than 200 mitigation wetlands that they have already constructed, plus the additional ones needed in the future. A stormwater model was also developed, specifically to predict pollution retention in the mitigation wetlands. They are using a modification of WASP4, with a multi-layered (sediment and water column) bucket wetland system.

Schueler (1996) summarized research on submerged bed wetland treatment systems for treating stormwater. Many wastewater treatment facilities have used submerged bed wetlands for polishing treatment. They have used rock or gravel media to grow emergent wetland plants. The wastewater slowly flows through a shallow rock-filled trench, where particulates settle and microbial and algal activity breakdown, and roots uptake, some of the pollutants. Schueler points out that most stormwater wetlands only treat surface flows and questions whether enhanced pollutant removal would occur with subsurface treatment also. He summarized a study conducted in Orlando, FL, by Tim Egan (of Dyer, Riddle, Mills and Precourt, Inc.) that tested several different submerged wetland cells for the treatment of stormwater, including two cells that were only filled with rock or crushed re-cycled concrete, and no plants. The stormwater was pretreated in a holding pond before being pumped into the cells. This pretreatment is necessary to reduce clogging and to equalize the flow rates through the cells. The reported average mass removal rates were: 81% for suspended solids, 78% for fecal coliforms, 75% for nitrate nitrogen, 14% for orthophosphate, 21% for copper, 73% for lead, and 55% for zinc. Interestingly, the crushed concrete filled cell performed better than any of the planted cells, probably because of the higher pH of the water in that cell. The rock surfaces were apparently more important than the root surfaces for pollutant removal by creating a larger surface area for epilithic algae and microbes.

Reed bed wetlands have been extensively used in the UK to treat CSO discharges at small treatment works. In Severn Trent, the local water department had more than 700 facilities serving less than 2,000 people (Green and Martin 1996). They had installed 55 reed bed systems by 1994, and planed to construct more, as resources allowed. Detailed monitoring and tracer studies have been initiated at some of these facilities to confirm the stringent discharge limits that apply. The beds are constructed as shallow excavations lined with plastic or clay and then are filled with 5 to 10 mm diameter gravel to a depth of about 0.6 m. The water levels are checked at least weekly, and any evaporation is made up with secondary effluent. In one critical location, the overflow concentration limits are: 40 mg/L for BOD_5, 60 mg/L for suspended solids, and 15 mg/L for ammonia nitrogen. They found that the reed beds provided consistent water quality improvements throughout the overflow hydrographs, although the initial improvement was mostly through dilution and dispersion. Continued pollutant reductions showed that pollutant uptake in the system was occurring.

The StormTreat™ system is a modular control device that includes sedimentation and plants (Allard, *et al.* 1996). One unit is 2.9 m in diameter and has a capacity of 5,260 L. The recommended detention time in the wetland portion of the unit (2,880 L) is 5 days. Multiple tanks are usually used at sites. Two tanks would be needed at a 0.4 ha paved site in order to capture 0.6 cm of runoff, if pre-treatment is provided. Five units would be needed otherwise. The units cost about $US 4,000 each, including installation. Four events have been monitored at one site and show high removals of bacteria (83%), suspended solids (95%), COD (75%), orthophosphate (32%), dissolved nitrogen (44%), lead (65%), and zinc (90%). Other modular units commercially available for source area treatment that rely mostly on sedimentation for pollutant removal include the Vortechs™ unit (from Vortechnics, Portland, ME), the Stormcepter™ (from Stormceptor Corp., Rockville, MD), and the CDS™ unit (from CDS Technologies, Alpharetta, GA). These units may be promising for source area control, however, long-term monitoring data is needed for these units before their actual performance and maintenance requirements can be determined with confidence.

Davies and Bavor (2000) compared the performances of a constructed wetland and a water pollution control pond in terms of their abilities to reduce stormwater bacterial loads to recreational waters. Bacterial removal was significantly less effective in the water pollution control pond than in the constructed wetland, likely because of the inability of the pond system to retain the fine clay particles (< 2 um) to which the bacteria were predominantly adsorbed. The key to greater bacterial longevity in the pond sediments appeared to be the adsorption of bacteria to fine particles, which protected them from predators.

Oil/Water Separators

This section briefly examines the most widely available oil/water separation technologies and their expected ability to treat stormwater, as they are commonly assumed to be equivalent to detention facilities, but on a small scale. These devices include gravity separators (including API separators and separation vaults), coalescing plates separators, and cartridge filters added to oil/water separators. These devices are extensively used to treat industrial wastewaters and have been shown to be effective in those applications for which they were designed. These units perform best at very high levels of oil contamination, such as may be found at some industrial locations. About 90% reductions in oil are possible if the influent oil concentrations are greater than about 10,000 mg/L. Reductions of about 50% would occur at influent oil concentrations of about 200 mg/L. Very little reduction is expected at levels less than about 100 mg/L. Little information is available demonstrating their effectiveness in treating stormwater, which usually has oil contamination levels of much less than 100 mg/L.

Other oil/water reduction technologies are used in some industrial applications, including separation tanks (typically small tanks used in shops that produce very small wastewater flows), and centrifuge separators (which require high energy demands and high maintenance, and are utilized in off-shore drilling operations). Neither of these technologies would be appropriate for the diffuse locations and highly irregular stormwater flows from critical source areas and are therefore not addressed here.

Figure 22. Oil and grease trap before downtown detention pond in Austin, TX.

Gravity Separation

Gravity separation relies on the density differences between oil and water. Oil will rise to the water surface unless some other contributing factor such as a solvent or detergent interferes with the process. For gravity units, this density difference is the only mechanism by which separation occurs. Other technologies, such as air flotation, coalescing plates, and impingement coalescing filters, enhance the separation process by mechanical means.

Gravity separators are the most basic type of separator and are the most widely used. They have few, if any, moving parts and require little maintenance with regard to the structure or operation of the device. Usually, separators are designed to meet the criteria of the American Petroleum Institute (API), and are fitted with other devices such as coalescing plate interceptors (CPI) and filters. Even though these separators are effective in removing free and unstable oil emulsions, they are ineffective in removing most emulsions and soluble oil fractions (Ford 1978). Furthermore, it is important to remember that no gravity oil/water separation device will have a significant impact on many of the other important stormwater pollutants, requiring additional treatment (Highland Tank).

Conventional American Petroleum Institute (API) Oil/Water Separator

The conventional API oil/water separator consists of a large chamber divided by baffles into three sections. The first chamber acts as an equalization chamber where grit and larger solids settle and turbulent flow slows before entering the main separation chamber. Often, manufacturers suggest the use of a catchbasin or interceptor tank as a pretreatment device so that coarse material will be kept from entering the oil/water separation tank. After entering the main chamber, solids settle to the bottom and oil rises to the top, according to Stokes' law. Larger API oil/water separators contain a sludge scraper which continually removes the captured settled solids into a sludge pit. The oil is also removed by an oil skimmer operating on the water surface. At the end of the separation chamber, all oil particles having a diameter of larger than the critical size have theoretically risen to the surface and have been removed by an oil skimmer. Small API units usually do not contain an oil skimmer, sludge scrapper, or sludge pit. While they are less costly due to the absence of moving parts, they require more frequent cleaning and maintenance. These smaller units have been shown to be as effective as the larger more expensive units, if they receive proper maintenance at regular intervals.

The API (1990) stipulates that if their design criteria are met, then the separator will remove all oil droplets greater than about 150 µm in diameter. The API reports that retention times are usually greater than the actual design values since actual flows are usually smaller than design flows, hence smaller droplets are removed most of the time. This finding is confirmed by Ruperd (1993) in a study of an oil/water separator treatment device in the community of Velizy, France. Also, API tanks are known to effectively remove large amounts of oil, including slugs of pure oil, and will not be overwhelmed (Tramier 1983). Studies have also shown that these separators can produce effluents down to 30 ppm (Delaine 1995), routinely at 30-150 ppm, with occasional concentrations above 150 ppm, depending upon the flow rate, and hence the retention times (Ford 1978).

The API has stated that very few separators with ratios of surface area to flow within the API design range achieved effluent oil concentrations lower that 100 ppm (API 1990). Therefore, the API separator is a recommended system for the removal of solids and gross oil as a pretreatment device upstream of another treatment system, if additional pollutants of concern are present, or if more stringent effluent standards are to be met.

The following is a partial list of oil/water separator manufacturers in the U.S.:

- Highland Tank and Manufacturing Co., One Highland, Rd. Stoystown, PA 15563
- McTighe Industries, P.O. Box 928, Mitchell, SD 57301-0928
- Xerxes Corp., 7901 Xerxes Rd. Minneapolis, MN 55431-1253

Separation Vaults

Separation vaults are variations on the API oil/water separator design. They are usually either septic tanks or utility vaults that have been fitted with baffles in the manner of an API separator. They are usually poured in place or manufactured locally. Surveys of these vaults in King County, Washington, revealed that they had main chamber depths of 1.2 – 1.5 m (4 – 5 ft), widths of 1.2 – 1.8 m (4 – 6 ft), and lengths of about 1.8 m (6 ft). These vaults are not necessarily designed according to the previously stated API methods and therefore are termed separation vaults to differentiate them from conventional API oil/water separators (King County 1995). These vaults can theoretically achieve removal of all oil droplets of 75 μm in size, or greater, however, practical removal sizes would probably be in excess of 150 μm.

Coalescing Plate Interceptor Oil/Water Separators

The coalescing plate interceptor (CPI) oil/water separators are simply conventional API oil/water separators and separator vaults with sets of parallel plates added to the main separation chamber. As small droplets of oil enter the plates, they rise until they encounter the next plate. Other drops also rise and coalesce. As the drops become larger, the buoyant forces acting on them become greater, eventually forcing the drops to slide off the plates and to rise quickly to the surface.

The total horizontal separator area requirement is reduced by the use of parallel plates by compacting the effective separation area into a limited space. The total area is the sum of the area of each plate projected on the horizontal plane, along with the open surface area of the separator itself. According to vendors, the use of coalescing plates can reduce spatial requirements of separators up to two-fold on width and ten-fold on length when used in place of a conventional separator without plates. Plates also help to dampen turbulence in the system, thus helping to maintain laminar flow. Oil collected from these systems has a lower water content than from conventional separators. The overall effluent oil content has been reported to be 60% lower for parallel-plate systems, with a higher proportion of small oil droplets recovered (Brunsmann 1962).

The earliest models of CPI separators used horizontal parallel plates. Currently, two types of parallel-plate separators are marketed: the cross-flow inclined plate separator and the down-flow inclined plate separator. In the cross-flow separator, flow enters the plates from the side and oil and sludge accumulates above and below the current. As oil and sludge build up, the oil then breaks free and rises, while the sludge descends to the separator bottom. In a down-flow separator, the water flows downward while oil rises to the above plate, and after coalescence, rises counter to the current to the top, while sludge will descend, helped along by the current.

The plates themselves are corrugated to improve oil and sludge collection. Vertical gutters are placed along the sides of the plates themselves at the influent and effluent points to aid in the collection of oils and solids. The plates are tilted at an angle of 45° - 60°, allowing sludge and oil to slide off, preventing clogging and resulting in lower maintenance requirements. A 45° angle has been found to be most effective for oil removal (Thanh and Thipsuwan 1978), but a 60° angle would reduce maintenance requirements further by insuring less clogging. However, a greater angle would also reduce the effective surface area as the effective surface is equal to the projection of the plates onto the horizontal plane (Branion 1978).

CPI separators have been found to remove droplets down to 30 to 60 μm size (Ryan 1986; Romano 1990), and have been found to produce effluent concentrations in the range of 10 to 20 ppm (Delaine 1995; Dull 1984; Ryan 1986). CPI separators are a good treatment choice if the wastewater contains smaller droplets and possibly some unstable emulsions with larger diameter droplet sizes. Dissolved oil, stable emulsions, or a large amount of unstable emulsions would decrease the performance of the coalescing plate interceptor separators.

The API notes that it is difficult to describe the separation process in a parallel plate separator due to the variability of plate size, spacing, and inclination. They recommend that users rely on the empirically-derived recommendations of the plate unit vendors when selecting a coalescing plate interceptor separator.

Impingement Coalescers and Filtration Devices
Filtration devices are used as post-treatment after separation in coalescing plate separators, and greatly improves the removal efficiency of a system. Many systems utilize these devices for treatment of industrial runoff; however, they are occasionally used in stormwater applications as well (Aires 1995). The most common type used is a vertical tube coalescer which has a random matrix of vertical tubes made of polypropylene fitted together in bundles. These bundles are placed towards the end of the separation tank before the outlet and after the coalescing plates; however, some manufacturers use these devices in place of plate systems. Oleophilic (oil-loving) filters provide a maximum coalescing surface, as well as helping to create a more laminar flow. These types of devices can provide better oil removal than a tank fitted only with coalescing plates, often with effluents suitable for direct discharge into surface waters.

Solids are trapped in sharp turns or crevices while oils are removed by two mechanisms occurring within the filters. First, the small passages in the filters allow the oil droplets to come in contact with each other and coalesce together. Second, the oleophilic properties of the media attract oil droplets and hold them until they coalesce with other trapped droplets until they eventually break free and rise to the surface.

The cartridge bundles can be removed and cleaned for reuse, although disposable filters are sometimes used. Disposable cartridge filters have the benefit of having simple maintenance requirements: when filters become clogged or saturated, they are simply removed and discarded. However, this process in itself may be a drawback in that the cartridges may need to be disposed of as a hazardous waste. Further, the cost of filters may be high and quickly reduce any benefit gained from reduced maintenance. Filters are typically made from fiberglass, nylon, polypropylene, and polyurethane foam; and are normally recommended as a secondary stage of treatment after gross solids and oil have been removed (Webb 1991).

Other problems exist with filter cartridges as well. Filters are easily clogged, even when pretreatment occurs. Also, if stable emulsions are present, surfactants will poison the filter by interfering with the surface-wetting properties of the filter (Tabakin, *et al.* 1978). Despite these problems, filters are known to remove oil to concentrations as low as 10 ppm, with all droplets greater than 20 μm being removed (Xerxes Corp).

Maintenance of Oil/Water Separators
Problems with oil/water separators can be attributed largely to poor maintenance by allowing waste materials to accumulate in the system to levels that hinder performance and to levels that can be readily scoured during intermittent high flows. When excess oil accumulates, it will be forced around the oil retention baffle and make its way into the discharge stream. Also, sludge buildup is a major reason for failure. As waste builds up, the volume in the chamber above the sludge layer is reduced and therefore the retention time is also reduced, allowing oil to be discharged. Therefore, the efficiency of oil/water separators in trapping and retaining solids and hydrocarbons depends largely upon how they are maintained. They must be designed for ease of maintenance and be frequently maintained. Apparently, few oil/water separators built for stormwater control are adequately maintained.

Manufacturers of prefabricated oil/water separators, as well as the American Petroleum Institute, all recommend periodic inspection and maintenance. Some manufacturers advise that these devices be cleaned twice per year, even if the device is apparently working properly. However, it is best if the devices are inspected after every rainfall to determine the rate of hydrocarbon and sludge buildup. The most effective maintenance schedule can then be obtained for each individual device. French researchers also advocate this approach, by developing individual maintenance schedules after intensive observations for six months (Aires 1995).

Ease of maintenance must be considered when designing separators, including providing easy access. Maintenance on these devices is accomplished by using suction equipment, such as a truck-mounted vacuum utilized by personnel trained to handle potentially hazardous waste. The vacuum is used to skim off the top oil layer and the device is then drained. In larger devices, the corrugated plates are left in place, but otherwise, they are lifted out along with any other filter devices that are present. The sludge is then vacuumed out or shoveled out and any remaining solids are loosened by spraying hot water at normal pressure.

Maintenance of parallel plate units and coalescing filters is similar. The separator is drained and the plates are washed by spraying. If there is inadequate space, then the plates will need to be lifted from the separator for effective cleaning. Cleaning should occur when coating of the plates is evident and before accumulations begin to clog the spaces. Cleaning of polypropylene coalescing tubes is also accomplished by lifting out the tube bundles and cleaning with a hose or high pressure water spray to remove accumulated oil and grit. Sludge is removed from underneath the coalescer supports and the coalescers are then replaced. No soaps or detergents are used in cleaning polypropylene components as they would destroy the oleophilic nature of the material.

Performance of Oil/Water Separators for Treating Stormwater

Manufacturers state that efficiencies observed during testing of oil/water separators are on the order of 97 – 99% for the removal of oil from wastewater. The test method typically applies oil to a paved washpad, with water added via a sprinkler system to simulate rainfall. Oil is of a specified density (typically 0.72 – 0.95). These synthetic events are necessary to evaluate the performance of a separator but do not necessarily reflect the processes which occur during actual rainfall conditions where rapidly changing flows rates, unknown oil mixtures, and other pollutants are present. Published research is difficult to find on how these units actually perform once placed in operation.

Interception of solid particles through settling, and flotation of oils and other floatables are processes occurring within an oil/water separator. French studies have shown that the average SS removal efficiency of separators is about 50% (Aires 1995). Oil/water separation requires an ascending speed of about 8 m/h, while the settling velocity of solids require descending velocities on the order of 1 to 3 m/h. At rates of 20% of the design flow rate, about 80% of the solids are removed; at 30% of the design flow rate, about 50% of the solids are removed. Negative removals also occur as the result of resuspension of previously settled material (Legrand, *et al.* 1994).

In many instances, pretreatment tanks are placed before the oil/water separator to remove settleable solids before stormwater enters the separator. A study in Velizy, France, found that the SS removal efficiency of a separator, placed downstream of a settling pond, was about 13%. This low value was attributed to the fact that solids had been allowed to settle during pretreatment, and therefore influent to the device had a low content of only the most difficult to remove solids (Ruperd 1993).

When the concentration of the oil in the wastewater is high, the oil removal efficiency increases. In Velizy, France, Ruperd (1993) found that oil/water separators fitted with cross current separators had removal efficiencies ranging from zero to 90%, with an average of 47%. Low efficiencies were associated with low influent levels and greater efficiencies were associated with higher influent levels. This finding supports those of Tramier (1983), stated earlier, that separators are effective in removing large amounts of oil when the oil concentrations are elevated.

The Metropolitan Washington Council of Governments (Washington, D.C.) has conducted a survey of 109 separator vaults in suburban Maryland and subsequently examined 17 in detail to determine their long-term effectiveness (Schueler and Shepp 1993). These separators were used for controlling runoff from areas associated with automobile usage. These separators were either pre-cast or poured in place concrete structures consisting of one, two or three chambers. The results of this study revealed that the amount of trapped sediments within separators varied from month to month and that the contained waters were commonly completely displaced during even minor storms (Shepp and Cole 1992).

Of the original 109 separators that were observed in the survey, devices less than one year old were effective in trapping sediments. Devices older than one year appeared to lose as much sediment as they retained (Shepp and Cole 1992). Not one of these separators had received maintenance since their installation. Survey observations suggested no net accumulation of sediment over time, in part because they received strong variations in flow. Of the 109 separators surveyed in this suburban Maryland study, 100% had received no maintenance, 1% needed structural repair, 6% were observed to have clogged trash racks, 84% contained high oil concentrations in the sediments trapped in their first chamber, 77% contained high oil concentrations in the sediments trapped in their second chambers, 27% contained high oil and floatables loading in their first chambers, and 23% contained high oil and floatables loading in their second chambers.

Numerous manufacturers have developed small prefabricated separators to remove oils and solids from runoff. These separators are rarely specifically designed and sized for stormwater discharges, but usually consist of modified oil/water separators. Solids are intended to settle and oils are intended to rise within these separators, either by free fall/rise or by counter-current or cross-current lamella separation. Many of these separators have been installed in France, especially along highways (Rupperd 1993). Despite the number of installations, few studies have been carried out in order to assess their efficiency (Aires and Tabuchi 1995).

The historical use of oil/water separators to treat stormwater has been shown to be ineffective for various reasons, especially lack of maintenance and poor design for the relatively low levels of oils present in most stormwaters (Schueler 1994). Stormwater treatment test results from Fourage (1992), Rupperd (1993) and Legrand, *et al*. (1994) show that these devices are usually greatly under-sized. They may possibly work reasonably well at flow rates between 20 and 30% of their published design hydraulic capacities. For higher flow rates, the flow is very turbulent (the Reynolds numbers can be higher than 6,000), and improvements in settling by using lamella plates is very poor. These devices need to be cleaned very frequently. If they are not cleaned, the deposits are scoured during storm events, with negative efficiencies. However, the cleaning is usually manually conducted, and expensive. In addition, the maintenance job is not very easy because the separators are very small. Some new devices are equipped with automatic sediment extraction pumps which should be a significant improvement. Currently, these researchers have found that the cleaning frequencies are very insufficient and the stormwater quality benefits from using oil/water separators are very limited.

Problems With Wet Detention Ponds

Wet detention ponds may experience various operating and nuisance problems. The following discussion attempts to describe these negative aspects of wet ponds, as reported in the literature, and to describe how they have been overcome through specific designs.

Safety of Wet Detention Ponds

The most important wet detention pond design guidelines are to maintain public safety. The following discussion briefly summarizes common suggestions to maintain and improve safety at wet detention facilities. Death by drowning is the most common safety concern associated with wet detention ponds. Marcy and Flack (1981) state that drownings in general most often occur because of slips and falls into water, unexpected depths, cold water temperatures, and fast currents. Four methods to minimize these problems include: eliminate or minimize the hazard, keep people away, make the onset of the hazard gradual, and provide escape routes. Many of the design suggestions and specifications contained in this discussion are intended to accomplish these objectives.

Jones and Jones (1982) consider safety and landscaping together because landscaping can be an effective safety element. They feel that appropriate slope grading and landscaping can provide a more desirable approach than wide-spread fencing around a wet detention pond. Fences are expensive to install and maintain and usually produce unsightly pond edges. They collect trash and litter, challenge some individuals who like to defy barriers, and impede emergency access if needed. Marcy and Flack (1981) state that limited fencing may be appropriate in special areas. When the pond side slopes cannot be made gradual (such as when against a railroad right-of-way or close to a roadway), steep sides having submerged retaining walls may be needed. A chain link fence located directly on the top of the retaining wall very close to the water's edge would be needed (to prevent human occupancy of the narrow ledge on the water side of the fence). Another area where fencing may be needed is at the inlet or outlet structures. However, fencing usually gives a false sense of security, as most can be easily crossed (Eccher 1991).

A following discussion on pond side slopes stresses gradual slopes near the water edge and a submerged ledge close to shore. Aquatic plants on the ledge would decrease the chance of continued movement to deeper water and thick vegetation on shore near the water edge would discourage access to the water edge and decrease the possibility of falling into the water accidentally. Pathways should not be located close to the water’s edge, or turn abruptly near the water.

Marcy and Flack (1981) also encourage the placement of escape routes in the water whenever possible. These could be floats on cables, ladders, hand-holds, safety nets, or ramps. However, they should not be placed to encourage entrance into the water.

The use of inlet and outlet trash racks and antivortex baffles is also needed to prevent access to locations having dangerous water velocities. Several types are recommended by the NRCS (SCS 1982), as shown on Figure 23. Racks need to have openings smaller than about 6 inches to prevent people from passing through them and need to be placed where water velocities are less than three feet per second to allow people to escape (Marcy and Flack 1981). Besides maintaining safe conditions, racks also help keep trash from interfering with the outlet structures operation.

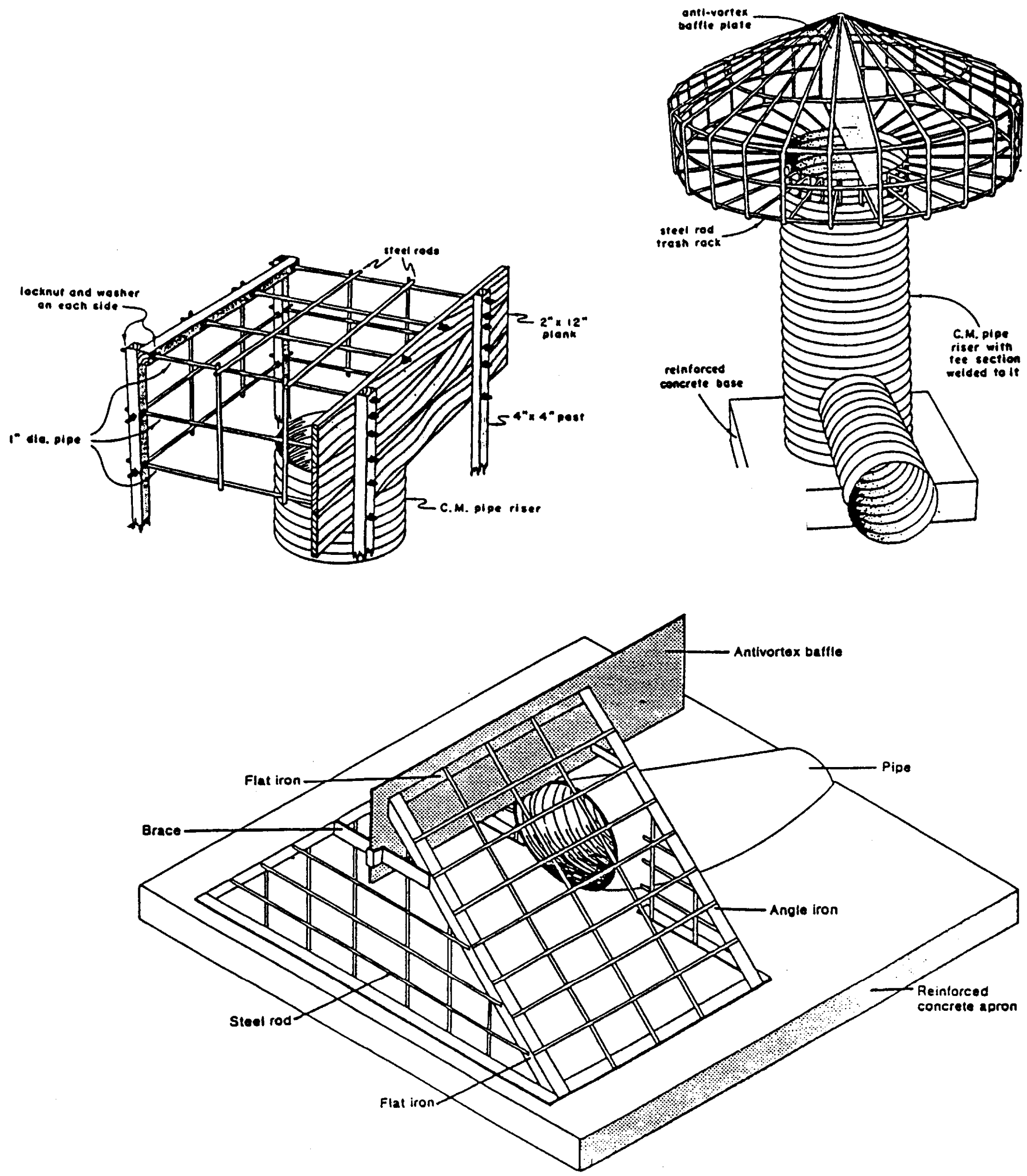


Figure 23. Various trash racks and baffles used by the SCS (NRCS). (SCS 1982).

Safety bars at Monroe St. detention pond, Madison, WI

Safety bars over pond outlet

Figure 24. Safety bars at detention pond inlets and outlets.

Eccher (1991) lists the following pond attributes to ensure maximum safety, while having good ecological control:

1) There should be no major abrupt changes in water depth in areas of uncontrolled access,
2) slopes should be controlled to insure good footing,
3) all slope areas should be designed and constructed to prevent or restrict weed and insect growth (generally requiring some form of hardened surface on the slopes), and
4) shoreline erosion needs to be controlled.

Attraction of nasty critters (State of Maryland photo)

Winter ice skating dangers near pond edges (Steve Auger photo)

Steep walkway leading to water (unknown Internet source)

Deep drop-off at pond edge
Figure 25. Safety problems associated with wet detention ponds.

Nuisance Conditions in Wet Detention Ponds and Degraded Water Quality
Most new detention ponds require from three to six years before an ecological balance is obtained (Ontario 1984). Excessive algal growths, fish kills, and associated nuisance odors may occur during this period, creating management problems for municipal officials and developers. Water quality is also generally poor in wet detention ponds, but unauthorized swimming can be common if alternative swimming facilities are not conveniently available. The poorest water and sediment quality in wet detention ponds usually occurs near the inlets and in depressions (Free and Mulamoottil 1983 and Wigington, *et al.* 1983). Some urban lakes have also been subjected to duck plagued disease which is a deadly virus that thrives in lakes having excessive algae growths (Ontario 1984).

Schueler (1986) and with Galli (1992) reported that water discharged from wet detention ponds may be warmed by as much as 10 to 15° F in the summer months, unless shaded or subsurface dischargers are used. Van Buren, *et al*. (2000) studied the thermal balance of an on-stream stormwater pond in Kingston, Ontario, Canada. During dry-weather periods, pond temperature increased as a result of solar heating, and thermal energy input exceeded output. Conversely, during wet-weather periods, pond temperature decreased as a result of limited solar radiation and replacement of warm pond water by cool inflow water from the upstream catchment, and thermal energy output exceeded input.

The haphazard installation of detention ponds can increase downstream flooding and erosion problems if a regional hydraulic analysis and careful plan is not developed and followed (Duru 1981, Jones and Jones 1982, and Hawley, *et al.* 1981). This can occur by increasing the duration of erosive flow velocities and by adding the delayed high discharge flows from a pond to the natural high flows from upstream areas. These problems can be substantially reduced with careful design and maintenance, as described in the following paragraphs.

Trash along shore from stormwater discharges

"Wet" pond built in area having Karst geology and sinkholes.

Figure 26. Aesthetic problems associated with wet detention ponds.

Attitudes of Nearby Residents and Property Values

Wet Detention ponds may create potential nuisance conditions if they are not properly designed or maintained. However, many people living near wet detention ponds do so because of the close presence of the wetlands, and their property values are typically greater than lots further from the ponds (Marsalek, *et al.* 1982). They also reported that small (well maintained) wet detention ponds are less subject to controversy than larger ponds (that are more commonly neglected). Debo and Ruby (1982) summarized a survey conducted in Atlanta of residents living near and downstream of 15 small detention ponds and found that almost half of the people surveyed who lived in the immediate areas of the ponds did not even know that they existed. Wiegand, *et al.* (1986) found that wet detention ponds, when properly maintained, are more preferred by residents than any other urban runoff control practice.

Emmerling-DiNovo (1995) reported on a survey of homeowners in the Champaign-Urbana area living in seven subdivisions having either dry or wet detention ponds. She reported that past studies have recognized that developers are well aware that proximity to water increases the appeal of a development. Detention ponds can create a sense of identity, distinguishing one development from another, and can be prominent design elements. Increased value is important because the added cost of the detention facility, including loss of developable land, must be recovered by increasing the housing costs. Others have also found that the higher costs of developments having stormwater detention facilities can also be offset by being able to sell the housing faster. In a prior survey in Columbia, MD, 73% of the respondents would be willing to pay more for property located in an area having a wet detention pond if designed to enhance fish and wildlife use. Although the residents were concerned about nuisances and hazards, they felt that these concerns were out-weighed by the benefits. In her survey, Emmerling-DiNovo (1995) received 143 completed surveys. Overall attractiveness of the neighborhood was the most important factor in purchasing their home. Resale value was the second most important factor, while proximity to water was slightly important. More than 74% of the respondents believed that wet detention ponds contributed positively to the image of the neighborhood and they were a positive factor in choosing that subdivision. In contrast, the respondents living in the subdivisions with the dry ponds felt that the dry ponds were not a positive factor in locating in their subdivision. Respondents living adjacent to the wet ponds felt that the presence of the pond was very positive in the selection of their specific lot. The lots adjacent to the wet ponds were reported to be worth about 22% more than lots that were not adjacent to the wet ponds. Lots adjacent to the dry ponds were actually worth less (by about 10%) than other lots; dry detention ponds actually decreased the assessed values of adjacent lots in two of the three dry basin subdivisions studied. The respondents favored living adjacent to wet ponds even more than next to golf courses. Living adjacent to dry ponds was the least preferred location.

Another example of increased land value occurred in Fairfax, VA (Land and Water 1996). A 1.6 acre wet detention pond was constructed using a modular concrete block retaining wall system. Total construction time was about six weeks and resulted in an attractive pond that added substantial value to the new housing development.

The Hennepin (MN) park district (John Barten, personal communication) reports that the park district is frequently asked by developers to be allowed to "improve" the parks by putting their wet detention ponds on park land that is adjacent to new developments. Needless to say, the park district cannot afford to convert their dry land to lakes that would dramatically decrease the utilization of the park by the park users. The park district is also frequently asked by residents of subdivisions to improve the water quality in the wet detention ponds located in their subdivisions, especially to allow fishing and swimming. The residents do not understand that their "lake" is actually a water treatment system and is not a natural lake or park and is not intended for water contact recreation or fishing. However, because many of these subdivisions are marketed by stressing the benefits of "lakeside" living, some of the residents expect the city to improve the wet detention ponds for recreational use. The park department, under a lot of citizen and political pressure, has actually had to construct new wet detention ponds upstream of some of these older wet detention ponds.

Auckland, NZ, sign explaining function of wet pond treatment

Sign advertising water quality treatment pond at new development in Austin, TX

Sign at percolation pond natural area in Madison, WI

Sign at dry pond site in Bellevue, WA

Figure 27. Signage near wet detention ponds.

Maintenance Requirements of Wet Detention Ponds

In order for detention ponds to perform as anticipated, they must be regularly maintained. Poor operation and maintenance not only reduces the pollutant and flow rate reduction effectiveness of detention ponds, but also can cause detention facilities to become eyesores, nuisances, and health hazards (Poertner 1974). If a pond does not "need" maintenance (such as sediment removal), then it is not providing significant water quality benefits. Ponds can be designed to minimize maintenance, however, a maintenance free detention facility (that is working properly) does not exist (SEMCOG 1981).

Institutional arrangements must be made to insure continued detention pond maintenance after construction. SEMCOG (1981) recommends that appropriate maintenance programs specifically identify the organization or person who will perform the maintenance and how the maintenance operations will be financed. They also found that major detention pond maintenance (dredging) is usually needed within about ten years after pond construction. More frequent (routine) maintenance may include: structural repairs (bank stabilization), removal of debris and litter from the water and surrounding land, grass cutting, fence repairing, algal control, mosquito control, and possible fish stocking. Wet detention ponds require a lot of attention.

As an extreme example of maintenance, it may be best to re-build a pond that was not originally designed for water quality benefits. As an example, the 30-year-old Expo Park regional stormwater detention facility in Aurora, Colorado, needed renewal (Hamilton, *et al*. 2001). Improvements to the multi-use 60-acre park facility were made to provide water quality benefits, improve site drainage, increase flood control detention, improve recreational usefulness and aesthetics, and upgrade the facility to meet jurisdictional State dam safety requirements. Dam safety related improvements included new outlet works, spillway improvements, and acceptance by the Engineer's Office for using irrigated turf grass as overtopping erosion protection for the emergency spillway.

Figure 28. Slope erosion due to seepage through pond dam.

Routine Maintenance Requirements

The following summary of routine maintenance requirements is based on a discussion by Schueler (1987).

Mowing

The most costly routine maintenance required of a detention facility is mowing the surrounding area. In residential areas, frequent mowing (up to 12 times a year) may be necessary to maintain a lawn surrounding the pond. Some native plants (such as in the small prairie surrounding the Monroe Street detention pond in Madison at the University of Wisconsin Arboretum) require much less maintenance. In all cases, the emergency spillway, side slopes, and pond embankments need to be mowed at least twice a year to control undesirable plants that may interfere with pond operation. Attractive landscaping and adequate landscaping maintenance are always needed. Careful plant selection (water and salt tolerant, disease and winter hardy, and slow growing) should be made in conjunction with a landscape architect or the Natural Resources Conservation Service.

Debris and Litter Removal

During the routine mowing operations and after each major storm, debris and litter should also be removed from the site, especially from the inlet and outlet grates and the water surface.

Inspections

Wet detention ponds need to be inspected at least once a year, and after each major storm. The inspection should include checking the pond embankments for subsidence, erosion, and tree growth. The conditions of the emergency spillway and inlets and outlets also need to be determined during the inspection. The adequacy of any channel erosion protection measures near the pond should also be investigated. Sediment accumulation in the pond (especially near, and in, the inlets and outlets) also needs to be examined.

Figure 29. Maintenance inspections and cleanout at outlet of a Birmingham, AL, pond.

Sediment Removal from Wet Detention Ponds

Large sediment accumulations in detention ponds can have significantly adverse affects on pond performance. Bedner and Fluke (1980) reported on the long term effects of detention ponds that received little maintenance. Lack of dredging actually caused the silted-in ponds to become a major sediment source to downstream areas. Poorly maintained ponds only delayed the eventual delivery of the sediment downstream, they did not prevent it.

Based on the NURP detention pond monitoring results (EPA 1983), a residential area pond having a surface area of about 0.6 percent of the contributing area should remove about 90 percent of the settleable solids (particulate residue) from the runoff. The Milwaukee NURP project (Bannerman, *et al.* 1983) estimated an annual sediment delivery of about 500 pounds per acre for medium density residential land uses and about 2,500 pounds per acre for commercial areas. Other land uses contribute sediment generally between these values. Assuming a density of about 120 pounds per cubic feet, about 3.6 and 18 cubic feet of sediment would be deposited in a well-designed detention pond for each medium density residential or commercial acre per year. With a pond 0.6 percent of the contributing area in size, this would only result in the deposition of between 0.2 and 0.9 inches per year. McComas and Sefton (1985) report two measured sediment accumulation rates in Chicago area wet detention ponds (about two and three percent of the drainage pond in size) of 0.24 and 1.3 inches per year. Kamedulski and McCuen (1979) report a much greater sedimentation rate of about three inches per year in another pond. When uncontrolled construction site erosion is allowed to enter a detention pond, the pond can literally fill up over night.

Most of the sedimentation would occur near the inlet and the resulting sediment accumulation would be very uneven throughout the pond. Sediment removal in a wet pond may therefore be needed about every five to ten years, depending on the variation in sediment deposition over the pond and the sacrificial storage volume designed. It is therefore necessary to plan for required maintenance during the design and construction of detention ponds. Ease of access of heavy equipment and the possible paving of a sediment trap near the inlet would ease maintenance problems. Deposited sediment can be heavily polluted and may require special disposal practices. Sediment concentrations of up to 100,000 mg organic carbon, several thousand mg lead, several hundred mg zinc, and more than ten mg arsenic per kg dry sediment are not uncommon for lakes receiving urban runoff (Pitt and Bozeman 1979). Dredged sediment is usually placed directly onto trucks, or is placed on the pond banks for dewatering before hauling to the disposal location. One common practice is to keep an area adjacent to the detention pond available for on-site sediment disposal. Small mounds can be created of the dried sediment and covered with topsoil and planted.

Poertner (1974) reviewed various sediment removal procedures. An underwater scoop can be pulled across the pond bottom and returned to the opposite side with guiding cables. If drains and underwater roads were built during the initial pond construction, the pond can be drained and front-end-loaders, draglines, and trucks can directly enter the pond area. Small hydraulic dredges can also be towed on trailers to ponds. The dredge pumps sediment to the shore through a floating line where the sediment is then dewatered and loaded into trucks or piled. A sediment trap (forebay) can also be constructed near the inlet of the pond. The entrances into the pond are widened and submerged dams are used to retain the heavier materials in a restricted area near the inlets. This smaller area can then be cleaned much easier and with less expense than the complete pond. Hey and Schaefer (1983) report the successful use of a submerged dam across the pond inlet in Lake Ellyn, near Chicago.

The estimated cost of removing sediment from a detention pond varies widely, depending on the amount to be removed and the disposal requirements. Costs as low as one dollar per cubic yard have been reported, but this low cost does not include any possible special disposal practices. Sediment removal costs are estimated to generally range from about $5 to $25 per cubic yard of sediment removed.

Sometimes the sediment wins (unknown Internet source photo)

Rebuilding pond and removing old sediment at apartment housing, Moscow, Russia

Central Park pond, NYC, being dredged and regraded

Figure 30. Mainenance dredging at wet detention ponds.

Problems with Contaminated Sediments in Wet Detention Ponds

Frequently, concern arises about the safety of disposing sediments from wet detention ponds. There have recently been several studies that have addressed this issue, as summarized in the following paragraphs.

Dewberry and Davis (1990) analyzed sediments from 21 ponds in northern Virginia. They found trace metals in many of the sediments, but the available forms of the metals were significantly less than applicable toxic thresholds. They concluded that the dredged materials could be safely disposed either on-site or at sanitary landfills without danger of health problems. However, they recommend that sediment samples from specific ponds be analyzed before dredging.

Yousef and Lin (1990) conducted extensive pond water quality and sediment quality analyses in six wet detention ponds in Florida as part of a Florida Dept. of Transportation study to develop pond maintenance procedures. The ponds had all been constructed from 4 to 13 years prior to analyses and received runoff from various urban watersheds that all contained different amounts of highway runoff. The dissolved oxygen levels in the ponds all dropped significantly with depth, in many cases being lower than 1 mg/L at the water-sediment interface. The pH of the pond water was also generally acidic in all of the ponds, being from 5.5 to 7.2 throughout the water columns. The temperature differences between the water surface and the bottom of the ponds was generally less than 1^oC. The sediment accumulation rates were found to be between 0.25 and 0.72 cm per year and correlated with pond age, size of drainage basin and size of pond. The bottom material was found to be poorly graded sand. Appreciable amounts of heavy metals (Cu: 7 to 73 µg/g, Ni: 12 to 82 µg/g, Pb: 84 to 1025 µg/g, and Zn: 13 to 538 µg/g), and nutrients (N: 1.1 to 5.2 mg/g, and P: 0.1 to 1.2 mg/g) were found in the surface layers of the sediments. However, the concentrations of the pollutants decreased rapidly with depth, generally being less than 10% of the surface sediment concentrations below 20 cm beneath the water-sediment interface. The bottom sediments were also analyzed to determine the TCLP extractable portions of the metals. These were found to be significantly less than the whole sediment metal concentrations (Cu: 0.13, Ni: 0.31, Pb: 0.27, and Zn: 0.33). They determined that the TCLP extractable fraction was lowest for sediments having higher clay and organic material. They concluded that the sediments could be removed during normal maintenance operations and disposed of on non-agricultural land.

Jones (1995) and Jones, *et al*. (1996) discuss the implications that the Resource Conservation and Recovery Act (RCRA) may have on sediments that need to be removed from stormwater management facilities, as summarized in the following discussion. The "mixture" (40 CFR Section 261.3(a)(2)(iv)) and "derived from" (40 CFR Sections 261.3(c)(2)(1) and 261.3(d)(2)) rules can cause sediments having very low concentrations of pollutants to be classified as "hazardous." These regulations are likely to be changed, with clearer definitions for non-hazardous operations and facilities. Sediments are evaluated as being hazardous when the wet detention pond is being dredged, not while they remain in-place. Many of the materials that are listed as hazardous under RCRA may enter stormwater, especially at vehicle service facilities, industrial facilities, and even golf courses and parks. These include solvents, degreasers, hydraulic fluids, herbicides, fungicides, and pesticides. For the sediments to be considered hazardous under the current RCRA mixture rule, the source of the specific material containing the listed hazardous material must contain more than 10% of the hazardous material. This is irrespective of how much of the material actually enters the stormwater. Therefore, site inventories become important tools in determining if a sediment would be classified as hazardous. If a listed material is used on the site, but it would not come in contact with rain (either through normal use or spills), the sediment would not likely be classified as hazardous. It is difficult to conduct detailed site surveys for a large drainage area having many separate owners, but it is feasible for small wet ponds serving single facilities.

MacDonald, *et al*. (1999) investigated the accumulation of heavy metals in a detention pond in Scotland. They found that the sediment in the pond can reach unacceptable concentrations of heavy metals within twenty years. Karouna-Renier and Sparling (2001) investigated the accumulation of Cu, Zn, and Pb by macroinvertebrates collected in Maryland stormwater treatment ponds serving commercial, highway, residential, and open-space watersheds to determine if land-use influenced metal concentrations in macroinvertebrates, sediments, and water. Composite Zn concentrations in odonates from ponds with commercial development (mean = 113.82 µg/g) were significantly higher than concentrations in the other land-use categories. Similarly, Cu levels in odonates from commercial ponds (mean = 27.12 µg/g) were significantly higher than from highway (mean = 20.23 µg/g) and open space (mean = 17.79 µg/g) ponds. However, metal concentrations in sediments and water did not differ significantly for the ponds in the different land-uses. The levels of Cu, Zn, and Pb in invertebrates from all ponds were less than dietary concentrations considered toxic to fish.

Jones (1995) and Jones, *et al*. (1996) also discuss other options to minimize the chance that wet pond sediment would be classified as hazardous under RCRA:

- Reduce the likelihood that listed substances would come in contact with precipitation or runoff.
- Inventory and track hazardous materials and encourage the use of less toxic replacement compounds.
- Install stormwater pre-treatment facilities to localize the problem.
- Reduce the accumulation rate, and increase the storage area for sediment in the pond.

Vegetation Removal from Wet Detention Ponds

In shallow detention ponds, excessive rooted aquatic plant (macrophyte) growths may occur over the entire pond surface. In deeper ponds, rooted aquatic plant growths are usually restricted close to the shoreline (Ontario 1984). Floating algae may create problems anywhere in a lake, irrespective of pond depth. As noted earlier, a narrow band of natural rooted aquatic plants along the narrow “safety” shelf is desirable as a barrier and to add habitat for pond wildlife.

Excessive algal growths create nuisance problems with strong odors, but more serious problems may also occur. Schimmenti (1980) reports that decaying vegetation, if not removed, promotes the breeding of mosquitoes. Certain types of algae (Anabaena, Aphanizomenon, and Anacystis) naturally produce toxins that can kill animals (including fish) which drink the water and can cause skin irritation and nausea in humans (Ontario 1984). Algae is usually mechanically controlled in detention ponds by using algae harvestors or by dewatering the pond. Certain fish also consume large amounts of algae, but the most common type of algae control is by using aquatic herbicides. Using a deep pond that restricts light penetration can significantly reduce many rooted aquatic plant growth problems.

Small weed harvestors can be delivered to a detention pond by trailer. The use of chemicals for algae control is popular, but must be carefully done to prevent contamination of the receiving water. Dead algae and rooted plants must also be removed to prevent odor and dissolved oxygen problems. Mechanical barriers can also be placed on the pond bottom to reduce rooted aquatic plant growth. AquaScreen is a fairly fine, dark mesh that is laid on the pond bottom that restricts sunlight from reaching the rooted aquatic plants. In tests conducted on Lake Washington, Perkins (1980) concluded that a two or three month use of the material resulted in about an 80 percent reduction of rooted aquatic plants where the material had been placed. Again, increased pond depth, possibly at less cost, can do the same thing.

Aquatic plant culture system harvester at stabilization pond (Lemna Corp. photo)


Engineered vegetation removal system at stabilization pond (Lemna Corp. photo)

Low-tech aquatic plant harvester, Singapore
Figure 31. Aquatic plant harvesting in treatment ponds.

Detention Pond Costs

Reported construction costs of detention facilities vary widely due to land value variations and special site or landscaping considerations. Even though the costs of detention facilities appear high, many benefits are available, besides just water quality that offset these costs. Some of these other benefits directly affect the cost of the development and may include using the wet pond as part of a fire protection system (as described below), and the obvious cost savings associated with reducing the size of parts of the downstream drainage system. In many cases, wet detention ponds have also significantly increased the value of the property due to increased landscaping and recreation benefits.

A series of nine inter-connected wet detention ponds at a hospital site in Southern California cost about $275,000 (about $30,000 per pond), including a pumping system for water recirculation (Rutherford 1977). This cost was about 25 percent of the total site grading, drainage, and paving costs. These ponds resulted in more than a million dollars in savings because the ponds were used as an emergency fire water supply instead of having to build conventional water storage tanks.

Chambers and Tottle (1980) compared the costs of ten detention pond systems. The total drainage system costs with detention ranged from about $1,200 to $11,500 per acre of land served, and averaged about $5,200 per acre of pond. Most of these detention systems produced significant peak runoff flow rate reductions, allowing substantial decreases in the sizes of the stormdrain pipes. Average savings were about $2,500 per acre of watershed served, or about 35 percent of the total drainage system costs. Cheng (1981) conducted a similar cost comparison analysis and estimated cost savings of about $1,800 per acre (1976 dollars). Although long-term maintenance costs of the detention ponds were not considered in these analyses, neither were additional benefits besides drainage system cost savings.

In a cost analysis conducted by the Ontario Ministry of the Environment (1984), on-site drainage systems containing detention facilities were generally found to have about the same costs as conventional systems. However, in almost all cases no additional off-site stormwater management measures were needed, in marked contrast to the conventional systems. Off-site increased pipe sizes and channels increased the total construction costs of the conventional systems by about 150 to 300 percent as compared to the alternatives containing on-site detention. On-site detention also substantially decreased the flood plain along the main channels, increasing the total area available for development, even when considering the land needed for on-site detention.

Poertner (1974) also presented several examples where on-site detention resulted in substantial savings to the site developers when compared to conventional drainage systems. In one example, providing on-site detention in a large residential development cost about $100 to $300 per lot, substantially less than providing conventional drainage systems.

The EPA (1983) analyzed costs associated with wet detention ponds construction for the NURP projects, as shown on Figure 32. A pond that covers 0.5 percent of a 150 acre watershed area would cost about $50 per watershed acre per year. This sized pond should remove between 80 and 90 percent of the annual suspended solids loading. These costs are for newly developed areas and are not applicable for estimating costs of retrofitting a pond in an established area.

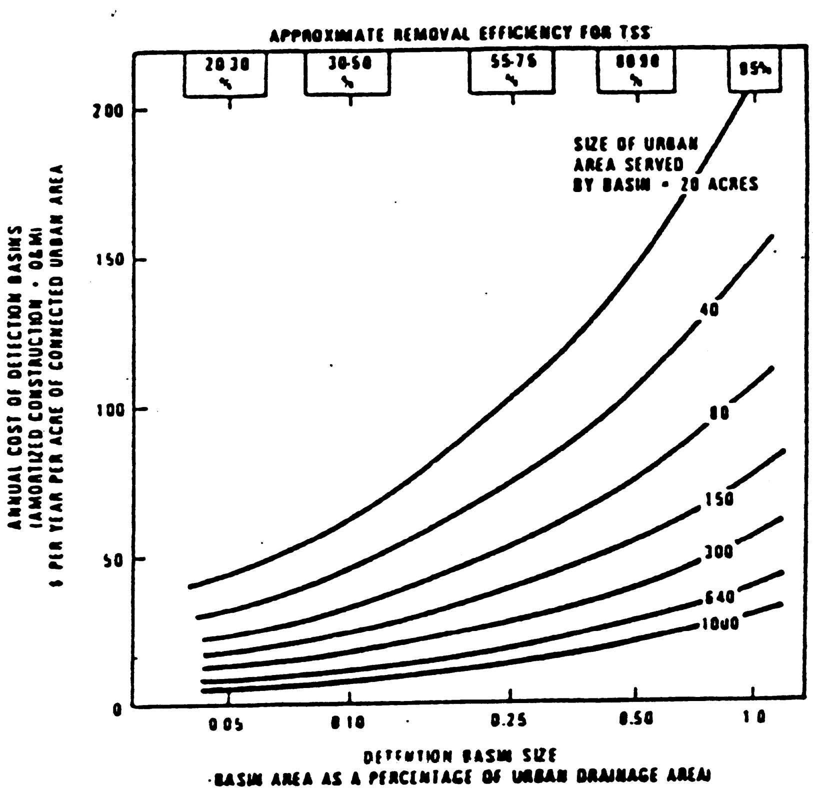


Figure 32. Detention pond costs (EPA 1983).

A detention pond and infiltration trench cost study in the Washington, D.C. area (Wiegand, *et al.* 1986) was based on a survey of engineering estimates and construction bids for 65 facilities constructed since 1982. They found that construction costs (excluding land purchase costs) varied mostly as a function of storage volume of the device (Vs). Their wet detention pond cost estimate equation was based on facilities having storage volumes (total storage in cubic feet, not just freeboard storage above the normal water level) greater than 100,000 cubic feet:

Cost = 34 Vs 0.64

This equation reflects a substantial cost savings with increasing size. As an example, a 0.5 acre pond (five feet deep) would cost about $50,000 (or $120,000 per pond acre), while a nine acre pond (also five feet deep) would cost about $400,000 (or about $40,000 per pond acre). In an interesting comparison, they did not find any significant differences in costs between large wet and dry detention ponds, probably because the wet ponds had greater economics of scale. However, smaller wet ponds were generally about 30 to 60 percent more expensive than small dry ponds (Schueler 1986). Schueler reexamined these detention pond costs in 1997and found that they have increased by about 15% since 1986 due to inflation (Schueler unpublished).

It is incorrect to directly compare the costs of wet ponds with dry ponds because of their very different objectives. When runoff water quality (of particulate pollutants) is the prime concern, then wet ponds are most appropriate, while dry ponds can be best used when peak flow rate reductions are desired. It is possible to design a wet pond to also achieve peak flow rate reduction objectives by increasing the freeboard pond storage and by careful design of the outlet structures. However, it may be best to construct a separate dry detention pond in series with a wet detention pond (or to use other upland source area controls, such as grass swales or infiltration devices) to achieve these multiple objectives.

Wiegand, *et al*. (1986) also examined the cost components of wet detention pond construction:

Cut and fill excavation	61%
Inlet and outlet works	18
Riprap	9
Land clearing	5
Sediment erosion control	5
Other	2

Excavation costs were the greatest wet pond cost component. Wet ponds required about 60 percent more excavation than dry ponds of comparable working volume. This extra excavation is often necessary to provide the needed permanent pool storage for wet ponds.

Maintenance is a necessary part of any stormwater management system, and the associated maintenance costs must be recognized along with the construction costs. Chambers and Tottle (1980) estimated that the annual maintenance costs for detention facilities to be about $35 (1978 dollars) per acre served per year, not considering sediment removal. About one-half of these annual costs are associated with maintaining the grassed embankments, about 25 percent is associated with weed and algae control, and the remaining 25 percent is associated with inspection and litter removal.

Sediment removal and disposal can be substantially greater than these other maintenance costs. Carr, *et al.* (1983) estimates that sediment removal and disposal for wet detention ponds in the Milwaukee area range from about $135 to $150 per acre of watershed served per year, depending on final disposal method (landfilling or land spreading). These costs ranged from about $5 to $25 per cubic yard (averaged $14). The differences in costs were associated with the sizes and accessibilities of the ponds. Small ponds (less than about 1/2 acre in size) had the lowest sediment removal costs of about $5 to $10 per cubic yard because front-end loaders could be used after pond de-watering. Larger ponds required the use of much more expensive draglines or hydraulic dredges. If on-site disposal was not available, hauling and final disposal costs substantially added to these removal costs. Hauling costs added another $5 to $10 per cubic yard, depending on the distance, and landfilling tipping fees could add another $15 to $25 per cubic yard to these costs. Therefore, in order to minimize sediment removal and disposal costs, Schueler (1986) stressed the need to provide adequate access to ponds, to provide small pre-sedimentation forebays near the inlets, to provide a drain in smaller ponds to allow complete de-watering, and to provide for on-site disposal of sediment near the pond (for at least two dredgings).

Typical wet detention pond construction costs, excluding land acquisition costs, are estimated to be about $40,000 per acre of pond. Maintenance costs (including periodic dredging) are estimated to be about four percent of this initial construction cost per year, or about $1,500 per acre of pond per year (1978 costs) (EPA 1983). Initial construction costs (excluding land costs) for a pond sized to achieve about 90 percent suspended solids reductions in a medium density residential area would be about $300 per watershed acre, with annual maintenance costs of about $12 per watershed acre. For a pond to achieve the same level of performance in an industrial area, the initial construction costs (again excluding land costs) would be about $800 per watershed acre, with annual maintenance costs of about $30 per watershed acre.

Guidelines To Enhance Pond Performance

The Natural Resources Conservation Service (NRCS, renamed from SCS, undated) has prepared a design manual that addresses specific requirements for such things as anti-seep collars around outlet pipes, embankment widths, type of fill required, foundations, emergency spillways, etc., for a variety of wet detention pond sizes and locations (SCS 1982). That manual must be followed for detailed engineering requirements.

The rest of this discussion presents some of the many design suggestions that have been made by researchers having many years of design and monitoring experience with detention ponds. Akeley (1980) listed several modifications that can be made to existing ponds to improve their performance. Gravel, or concrete, should be added along unstable banks and near the inlet and control structures. A baffle should be placed at the inlet to reduce turbulence, and barriers can be used to separate the pond into compartments to reduce short-circuiting. On-going maintenance is also needed to remove deposited sediment. Hawley, *et al.* (1981) also recommended similar design considerations. Hey and Schaefer (1983) found that a submerged dam near the pond inlets significantly reduced the area requiring maintenance dredging.

Lettenmaier and Dally (1983) stress the importance of source control of pollutants. As an example, for vehicle service areas, they suggest that reviews should be made of all maintenance operations that use detergents, oils and grease, solvent, and hydraulic fluid to minimize their discharge into the drainage system. Fuel storage and transfer operations need to be carefully conducted to minimize fuel spillage, and waste washwater should not be allowed to be discharged into the stormdrain system. Pitt and McLean (1986) also found large amounts of toxic pollutants in runoff flows from many source areas in an industrial area in Toronto. Most of these toxic pollutants were in soluble forms and would not be effectively removed by wet detention. It was obvious that much of these materials were being inappropriately discharged to the stormdrain system during both wet and dry weather. Careful investigations should therefore be made in areas discharging high concentrations of problem pollutants to identify their sources in order to eliminate their discharges at their source areas instead of assuming that outfall treatment is best or even possible.

Insect Control and Fish Stocking

Mosquito problems at wet detention ponds are increased when large water level fluctuations occur, especially when vast amounts of aquatic plants are wetted and available for egg laying. If ponds drain to normal water levels within several hours after a rain has ended, if aquatic vegetation is kept to a minimum (such as only along a narrow ledge close to shore), and if the pond shape allows adequate water movement and wind disturbance, then mosquito problems should be minimal.

Schimmenti (1980) made several recommendations to reduce the possibility of mosquito problems in detention ponds. Wet ponds should have adequate water quality to support surface feeding fish, such as sunfish, and various minnows, that feed on mosquitoes. Carp or crayfish also make adequate biological controls for midges, reducing the need for chemical controls (Ontario 1984).

Some developers have tried to stock trout, yellow perch, and northern pike in detention ponds, but no reproduction and poor wintering soon eliminates these less tolerant fish. Detention ponds receiving urban runoff are likely to contaminate fish, making them unsuitable for consumption. Brydges and Robinson (1986) have conducted extensive heavy metal and pesticide analyses in fish in two wet detention ponds near Toronto, Ontario and have found little problem accumulations of these substances. However, many other studies have reported problem toxic pollutant concentrations in fish from waters receiving urban runoff, so allowing fish consumption in wet detention faculties should only be allowed after careful study. Therefore, game fish should not generally be used in ponds, and consumptive fishing should be discouraged. Fathead minnows, stocked for mosquito control, have survived in detention ponds in Ontario.

Aquatic Plants for Detention Ponds

Aquatic plants are used in many ways in detention ponds, including providing increased nutrient and other soluble pollutant removals, competition with nuisance plants, aquatic life habitat, physical barriers, and decorative landscaping elements. Obviously, care needs to be taken when selecting aquatic plants to ensure that the plants will support the desired objectives and be compatible with multiple objectives and the local growing conditions. It is best to consult professional aquatic plant specialists to determine the best species for each project.

Rooted aquatic plants should be planted along much of the shallow perimeter shelf to deter small children, for aesthetics and to provide wildlife habitat. The use of native aquatic plants is to be encouraged to lessen maintenance costs and to prevent nuisance plants from becoming established in a waterway (such as purple loosestrife). Plants that could be established in wet detention ponds include arrowhead and cattails. Cattails sometimes interfere with the operation of a surface outlet because of large floating pieces clogging the weir. Subsurface weirs and trash racks (both recommended) would reduce this problem. Many rooted aquatic plants may be used in wet detention ponds, but their selection and planting should be done in consultation with landscape architects and wildlife biologists. Fuhr

(1996) warns against planting trees and brush on an impoundment because seepage problems may result by root action.

An interesting use of aquatic plants to enhance wet detention pond performance was described in the February 1991 *Lake Line*. Nutri-Pods, developed by the Limnion Corporation of Concord, CA, are two-meter diameter mesh balls, initially filled about 25% full with coontail (*Ceratophyllum demersum*). One to five Nutri-Pods are used per acre of pond surface, for ponds at least one acre in size. These reduce nutrient concentrations in the water and successfully compete with other aquatic plants, including planktonic algae. They were tested on a 27 acre lake near Sacramento, CA, which underwent periodic major increases in nutrients (phosphates as high as 50 mg/L) from fertilizing on the surrounding land. It took about two to four weeks for the Nutri-Pods to stabilize the lake after each major increase. Adding *Elodea* to the Nutri-Pods helped to keep nutrient concentrations very low (phosphorus at about 0.01 mg/L and nitrates less than 0.1 mg/L). The Nutri-Pods are inspected every few weeks and when they approach 100% capacity with the internal aquatic plants, they are removed from the water, and plants are removed, except for about 25% that are used as a starter. The Nutri-Pods therefore use aquatic plants to improve wet detention pond water quality, while enabling controlled harvesting with very little specialized equipment.

Planting wetland plants in artificial wetlands for stormwater control doesn't always determine the mixture of plants that will become established in the long term. Wind (1996) describes a site that was seeded with perennial rye, plus five wetland plants. After about three years, the site appeared to have a cattail monoculture, although no cattails were originally planted, nor were any apparent in the project vicinity. Upon surveying the site, a much greater diversity of healthy plants was found, though few were included in the initial seed mixture. Wind concluded that the inhabiting plants were successful because of their suitability to the site and natural invasion was perhaps the best end result. The initial seed mix should probably be considered a mechanism for erosion control and as "nursery" plants, giving invading natural species protection. However, invading nuisance plants should be controlled.

Tables 4 and 5 are examples of aquatic plants available from two different sources for upper Midwest ponds and extreme southeast ponds. Table 4, from J.P. Ludwig (Ecological Research Services, The Academy Center, Bay City, MI 48708), is a cold region native wet site plant list for a seed mixture that was available in 1987. This seed mixture was suited for saturated, moist, or flooded sites, (especially for clay or loamy organic soils) including pond edges.

Table 5 is a 1988 native plant list for extreme southeast wetlands from W. Miller (Aurora Incorporated, Florida). Aurora Inc. has assisted in the "aquascaping" of a number of freshwater Florida stormwater management ponds. Table 5 indicates specific plants for different water depths (such as for the subsurface ledge that would include upper and middle zone plants, and pond edges that would include the upper zone plants).

Figures 33 and 34 are maps showing the distribution of the growing season for common wetland plants used for water treatment. In much of the country, the growing season is 6 months, or less, for these plants. There remain serious questions concerning the ability of wetland plants to retain pollutants during their dormant season. Stormwater control with wetland vegetation is more restricted than sanitary sewage because it is not warm during winter months. Sanitary sewage is warmer than ambient temperatures which can significantly extend the growing season. The high chloride concentrations in snowmelt and early spring runoff may be especially harmful to wetland plants. Without deep pools of water (at least 3 feet), scour may also be a serious problem. It is recommended that wetland systems be used as polishing systems after wet detention ponds for use only during their active growing season. Most flows should be diverted around the wetlands during critical periods (especially dormant periods) to prevent scour. Moderate amounts of plant growth in wet detention ponds, especially along the edge on the shallow shelf, however, should be used. Tables 6 through 8 show the added benefits that biological systems can provide in ponds.

Table 4. Northern Native Seed Mixture for Wetlands

Agrimonia gryposepala	Agrimony
Amemone canadensis	Windflower
Apocynum cannibuim	Indian hemp
A. medium	Indian hemp
Asclepias incarnata	Swamp milkweed
Aster drummondii	Aster
A. novae-anglae	New England aster
A. pilosus	Aster
A. umbellatus	Aster
Bidens cernua	Begger tick
B. frondosa	Begger tick
Carex sparganioides	Sedge
C. Tenure	Sedge
Cephalanthus occidentalis	Buttonbush
Cirsium muticum	Swamp thistle
Convoloulus sepium	Bindweed
Cornus racemosa	Grey dogwood
C. stolonifera	Red-osier dogwood
Cyperus strigosus	Galingale
Epilobium angustifolium	Fireweed
E. hirsutum	Willow-herb
Eurpatorium maculatum	Joe-Pye weed
E. perfoliatum	Boneset
E. purpureum	Purple Joe-pyeweed
Gentiana andrewsii	Bottle gentian
G. crinita	Fringed gentian
G. procera	Gentian
Geum laniciatum	Avens
Glyceria canadensis	Mannagruss
Helianthus giganteus	Giant sunflower
H. grosseratus	Sawtooth sunflower
H. tuberosa	Jerusalem artichoke
Helinium antumnale	Sneezeweed
Iris versicolor	Iris
Jancus sp.	Rush
Leersia orizoides	Sawgrass
Liluim michiganese	Michigan lily
L. supurbum	Turk's-cap lily
Lobelia cardinalis	Cardinal flower
Lycopus americanus	Water horehound
Menaspermum canadensis	Moonseed
Onoclea sensibilis	Sensitive fern
Rosa palustrus	Swamp rose
Rudbeckia fulgida	Black-eyed Susan
R. hirta	Black-eyed Susan
R. subtomentosa	Black-eyed Susan
R. triloba	Black-eyed Susan
Saggitaria latifolia	Arrowhead
Scirpus americanus	Bulrush
Slphium terebinthinaceum	Prairie dock
Solidago graminifolia	Grass-leaved goldenrod
Sprirea tomentosa	Hardhack
Thelypteris palustris	Swamp fern
Verbena hastata	Vercain
Vernonia altissima	Tall ironweed

Source: Ecological Research Services, Bay City, MI

Table 5. Aquatic Plants Currently Utilized in Florida Aquascaping Projects

Upper Zone		
Sand cordgrass	*Spartina bakeri*	(+0.5' to 0')
Soft rush	*Juncus effusus*	(0' to -0.5')
Golden canna	*Canna flaccida*	(+0.5' to -0.5')
Blue flag iris	*Iris virginicus*	(+0.5' to -0.5')
Bulrush	*Scirpus validus*	(0' to -0.5')
Middle Zone		
Pickerelweed	*Pontederia cordata*	(-1' to -3')
Arrowhead	*Sagittaria lancifolia*	(-l' to -3')
Lower Zone		
Fragrant white water lily	*Nymphaea odorata*	(-3' to -5')
Strap leaf sagittaria	*Sagittsria subulata*	(-1' to -3')

Source: Aurora, Inc., FL

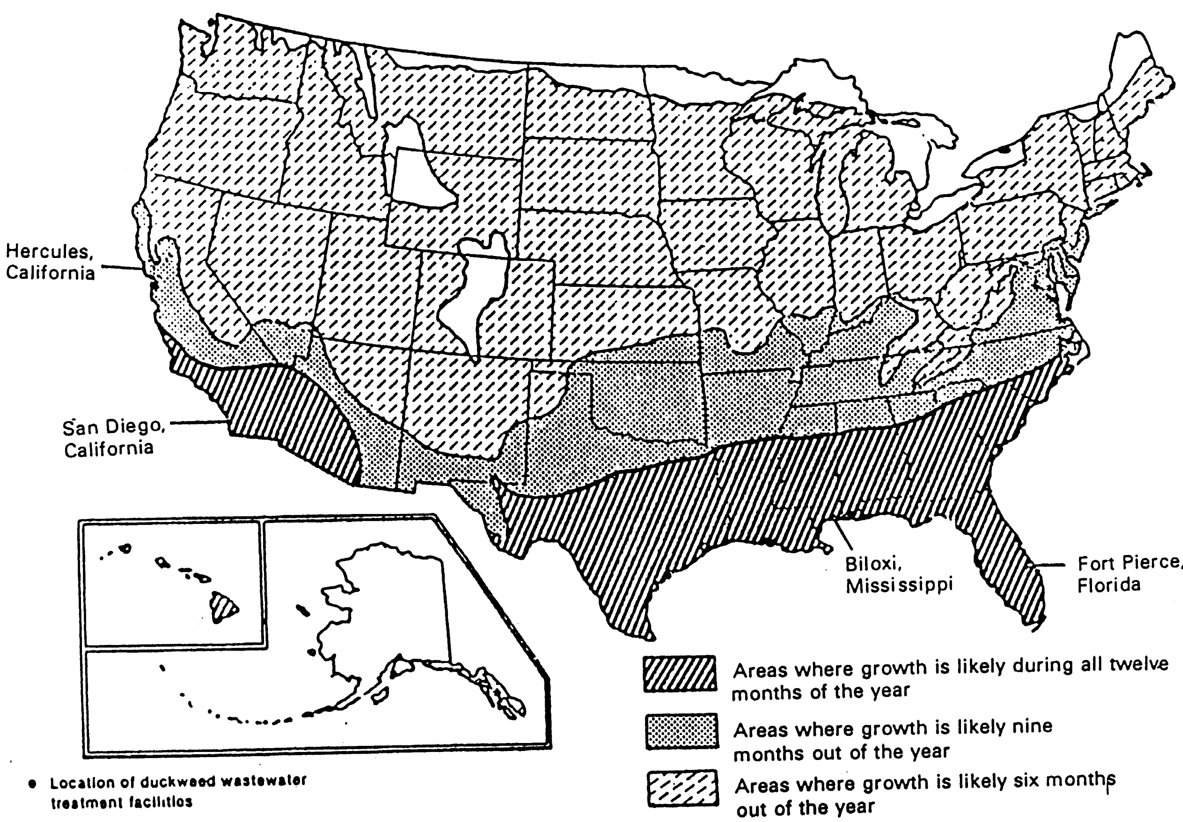


Figure 33. Potential growth distribution for duckweed in the U.S. (Reed, *et al*. 1988).

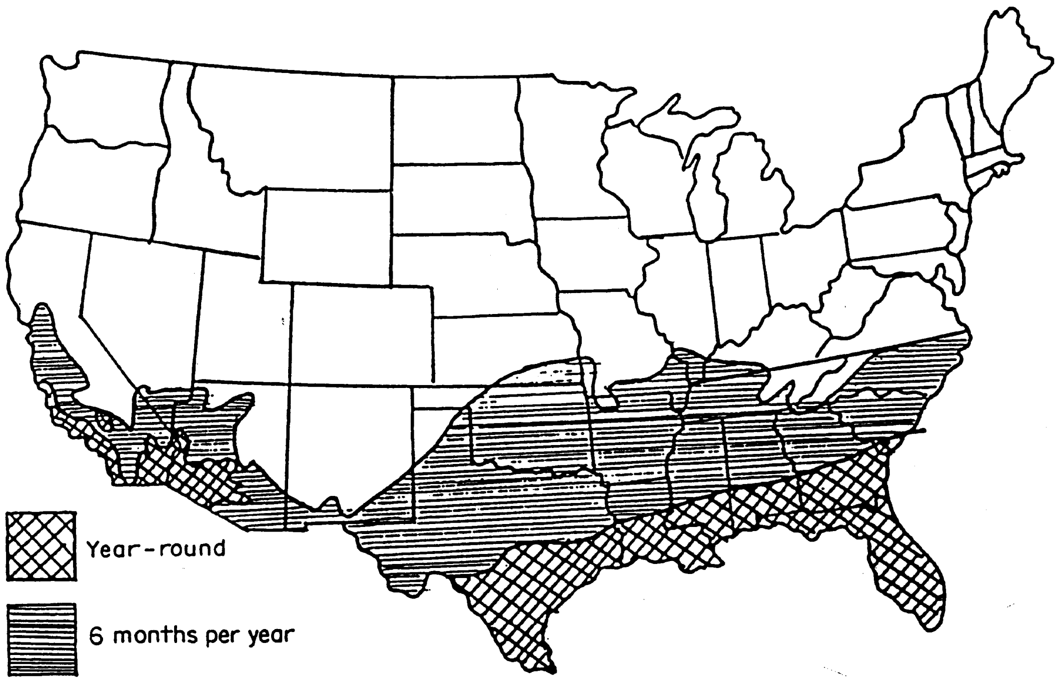


Figure 34. Suitable areas for hyacinth wetland systems (Reed, *et al*. 1988).

Table 6. Fish Species used in wastewater Treatment (Reed, *et al*. 1988)

Common name, *scientific name*	**Pond location**	**Feeding habits**
Silver carp, *Hypophthalmichthys molitrix*	Upper layers	Phytoplankton
Bighead carp, *Aristichthys nobilis*	Upper layers	Phytoplankton, zooplankton, and suspended solids
Black carp, *Mylopharyngodon piceus*	Bottom	Snails, crustaceans, and mussels
Grass carp, *Ctenopharyngodon idella*	Ubiquitous	Variable
Common carp, *Cyprinis carpio*	Bottom	Phytoplankton, zooplankton, and insect larvae
Tilapia, *Tilapia* spp., *Sarotherodon* spp.	Ubiquitous	Plants, plankton, detritus, and invertebrates
Catfish, *Ictalurus* spp.	Bottom	Crustaceans, algae, fish, and insect larvae
Fathead minnows, *Pinephales promelas*	Bottom	Phytoplankton, zooplankton, and invertebrates
Golden shiner, *Notemigonas crysoleucas*		
Mosquito fish, *Gambusia affinis*	Surface	Insect larvae, zooplankton, and algae
Buffalofish, *Ictiobus* spp.	Bottom	Crustaceans, detritus, and insect larvae

Table 7. Contaminant Removal Mechanisms Available in Wet Detention Ponds (Hammer 1989).

Mechanism	Contaminant Affected[a]	Description
Physical		
Sedimentation	P - Settleable solids S - Colloidal solids I - BOD, nitrogen, phosphorus, heavy metals, refractory organics, bacteria and virus	Gravity settling solids (and constituent contaminants) in pond/marsh settings.
Filtration	S - Settleable solids, colloidal solids	Particulates filtered mechanically as water passes through substrate, root masses, or fish.
Adsorption	S - Colloidal solids	Interparticle attractive force (van der Waals force).
Chemical		
Precipitation	P - Phosphorus, heavy metals	Formation of or coprecipitation with insoluble compounds.
Adsorption	P - Phosphorus, heavy metals S - Refractory organics	Adsorption on substrate and plant surface.
Decomposition	P - Refractory organics	Decomposition or alteration of less stable compounds by phenomena such as UV irradiation, oxidation, and reduction.
Biological		
Microbial metabolism[b]	P - Colloidal solids, BOD, nitrogen, refractory organics, heavy metals	Removal of colloidal solids and soluble organics by suspended, benthic, and plant-supported bacteria. Bacterial nitrification/denitrification. Microbially mediated oxidation of metals.
Plant metabolism[b]	S - Refractory organics, bacteria, and virus	Uptake and metabolism of organics by plants. Root excretions may be toxic to organisms of enteric origin.
Plant absorption	S - Nitrogen, phosphorus, heavy metals, refractory organics	Under proper conditions, significant quantities of these contaminants will be taken up by plants.
Natural dieoff	P - Bacteria and virus	Natural decay or organisms in an unfavorable environment.

Source: Stowell et al.[14]

[a]P = primary effect; S = secondary effect; I = incidental effect (effect occurring incidental to removal of another contaminant).

[b]Metabolism includes both biosynthesis and catabolic reactions.

Table 8. Potential Uptake Rates of Lemna System (Lemna System, undated)

Elements	Uptake Rate Lbs/acre/year
Phosphorus	700
Nitrogen	5,450
Iron	710
Chloride	940
Sulfur	580
Sodium	350
Potassium	2,250
Calcium	5,000
Copper	2
Zinc	6
Manganese	80
Magnesium	700
Chromium	5
Aluminum	2,300
Arsenic	5
Mercury	1

Auckland, NZ, heavily vegetated pond

Natural wetland plant growth occurring in FBM chamber, Sweden (Karl Dunkers photo)

Planting plants along wetland fringe at pond in Malmo, Sweden


Watch where you step Eric

Malmo, Sweden, wetland/pond system treating CSOs

Orlando, FL, heavily planted pond

Volunteer planting of wetland plants at Malmo,
Sweden, ponds

Figure 35. Aquatic plants in treatment wetlands and wet detention ponds.

Locating Ponds

Ponds that require limiting access, because of uncontrollable nuisance conditions, can be more easily located in industrial or commercial sites (Chambers and Tottle 1980). Ponds offering non-contact recreation and non-consumptive fishing (such as small boat use, ice skating, and aesthetic enjoyment) must be better maintained because of their visibility and need to be located for easy access. As noted in the following paragraphs, basin-wide hydraulic analyses must be used in developing watersheds to identify the best locations for detention ponds to be used for peak flow rate control.

Locating detention ponds close to the sources of the pollutants usually requires the use of many small ponds. Maintenance and cost considerations, however, usually dictate the use of a smaller number of larger detention ponds. In the Washington, D.C. area, detention ponds are discouraged for drainage areas less than 25 acres (Wiegand, *et al.* 1986). The largest drainage areas usually treated with wet detention ponds in the Washington, D.C. area are about 400 acres. This drainage size range (25 to 400 acres) translates to effective pond surface areas of about 1/4 to 12 acres.

Stormwater wet detention ponds for water quality benefits should be carefully located, considering critical source areas and the use of other control practices. Placement of stormwater detention ponds on the main stems of receiving waters is not recommended because of the large drainage area upstream that must be considered in the design and the difficulty of effectively using additional controls upstream. Retrofitting detention ponds in existing areas requires a different approach than for new construction. In retrofitting controls, detailed watershed analyses are needed to identify outfalls of drainages that contribute significant discharges and upland locations near critical sources (such as industrial and commercial areas), all in conjunction with other possible controls that can be applied simultaneously. They shouldn't be arbitrarily used at all outfalls.

For new construction, wet detention ponds are needed in areas that have large pollutant potentials and where infiltration controls cannot be used because of possible groundwater contamination. Large parking or storage areas (paved or unpaved) greater than one acre in size need on-site wet detention ponds to serve as pre-treatment devices before infiltration. Smaller areas may be better served with large catchbasins and oil and grease traps, or sand filters, as infiltration pretreatment. Shopping centers are an important example of these areas. Additionally, industrial areas greater than about three acres need to be served with on-site wet detention ponds, with no infiltration. Large residential areas, especially if having high-density single family or multi-family units, could also effectively use wet detention ponds as part of the landscaping plans to supplement an infiltration program.

Special consideration is needed for areas or developments that are likely to produce significant water volume or pollutant discharges. Large roofs produce substantial portions of the total runoff volumes from commercial and many industrial areas. Roof runoff is relatively unpolluted, however, except for high zinc concentrations from galvanized roof drainage systems. Paved parking and storage areas also produce large volumes of runoff, and this water can be heavily polluted, especially in manufacturing or heavy industrial areas. While infiltration of roof runoff from large roofs can produce significant water volume reductions, it cannot be used when roof runoff may be contaminated, as may occur in manufacturing industrial areas. Where groundwater contamination is likely (such as when the groundwater is close to the surface or in sandy soils) (Pitt, *et al.* 1994; 1996), wet detention basins (or grit chambers with oil and grease traps for small areas) may be the best control device.

The following list shows which specific controls should be considered for large source areas:

- Roofs should direct the roof runoff to infiltration devices, depending on groundwater conditions.
- Medium parking lots and storage areas, having areas between 5,000 to 500,000 square feet should direct this runoff to grit chambers and then to infiltration devices. If groundwater conditions prevent the use of infiltration devices, then wet detention ponds need to be used. The multi-chambered treatment tank (MCTT) was developed to be an effective control for these areas (Pitt, *et al*. 1999)
- Large parking lots and storage areas, having areas greater than 500,000 square feet, should use wet detention basins before infiltration devices (such as percolation ponds). Groundwater conditions may prevent the use of infiltration devices.
- Industrial sites greater than 100,000 square feet need to pre-treat their runoff in wet detention ponds before discharge. Additional treatment may be needed for all industrial areas.

It is usually easier to inspect (and maintain) a small number of relatively large facilities, and larger wet detention basins offer greater public use (such as non-contact recreation and non-consumptive fishing, for example). Industrial areas or large shopping areas pose an important exception to large, regional detention basins. Public water contact in industrial area wet detention basins should be discouraged because they can have very poor water quality. Industrial discharges should also be kept separated in their own detention basins to optimize any special controls that may be needed.

Stormwater control devices can be applied to storm drainage inlets and storm sewerage, besides at critical areas. These may include infiltration devices, perforated underground storm drainage systems, roadside grass swales, or catchbasin cleaning. Outfall controls also may include many options, but the two most efficient are infiltration devices (percolation ponds) and wet detention basins.

Industrial areas have been found to produce very portions of the total urban runoff wasteload in cities, especially of heavy metals and toxic organics. Unfortunately, much of this material is discharged during dry weather, possibly as part of wash operations or minor spills. Wet detention basins at the outfalls of industrial developments are needed to control runoff from the industrial sites and to offer an opportunity to remove any dry weather industrial spills and discharges. Reported spills that enter the stormwater drainage system in industrial areas may also be contained for cleanup in outfall wet detention basins. Installation of detention basins during the early phases of a construction project (before the drainage system is installed) can significantly reduce sediment transport from a construction site to receiving waters.

Many stormwater control options can be used together very well. Infiltration trenches, for example, can treat runoff from rains having relatively low intensities but long durations (and therefore large rain volumes). Infiltration devices also remove most pollutants and flow volume from the runoff. However, they discharge these pollutants to the soil and groundwater systems, requiring careful consideration. In all cases, local groundwater contamination potential must be evaluated to reduce the probability of contaminating groundwater with stormwater infiltration (Pitt, *et al.* 1994; 1996). Detention basins, on the other hand, work well with high intensity, low volume rains, but do not reduce soluble forms of the pollutants or flow quantities. These two devices can be used together to treat many runoff pollutants for a wide range of rain conditions.

Rosmiller (1987) notes that the location and amount of detention pond storage in relation to the size of the watershed is important in determining the peak flow rate reduction potential of a pond. He found that large ponds on the main stem of a stream and on its major tributaries result in greater reductions in peak flow rates than numerous smaller ponds spread throughout the watershed. Unfortunately, this can conflict with water quality and biological objectives in areas upstream of a main stem detention pond. He concludes that the best peak flow rate reductions in downstream portions of a watershed are associated with detention ponds located in the middle portions of a watershed. Detention ponds located on tributaries in the downstream portions of watersheds can increase peak flows in the main stem because of the superposition of peak flows from upper portions of the watershed and the peak flows from delayed hydrographs from the downstream detention ponds.

Figures 36 through 38, from Rosmiller (1987), illustrate how detention pond locations can greatly influence the resultant peak flow rates. Figure 36 shows a watershed with a downstream urbanizing tributary. Figure 37 shows the predevelopment (and pre-detention) tributary, main stem, and combined hydrographs for this watershed. Figure 38 shows how a tributary detention pond located downstream of the urbanizing area maintains the predevelopment peak runoff rate for the tributary, but results in substantially greater combined flows downstream after combining with the main stem hydrograph.

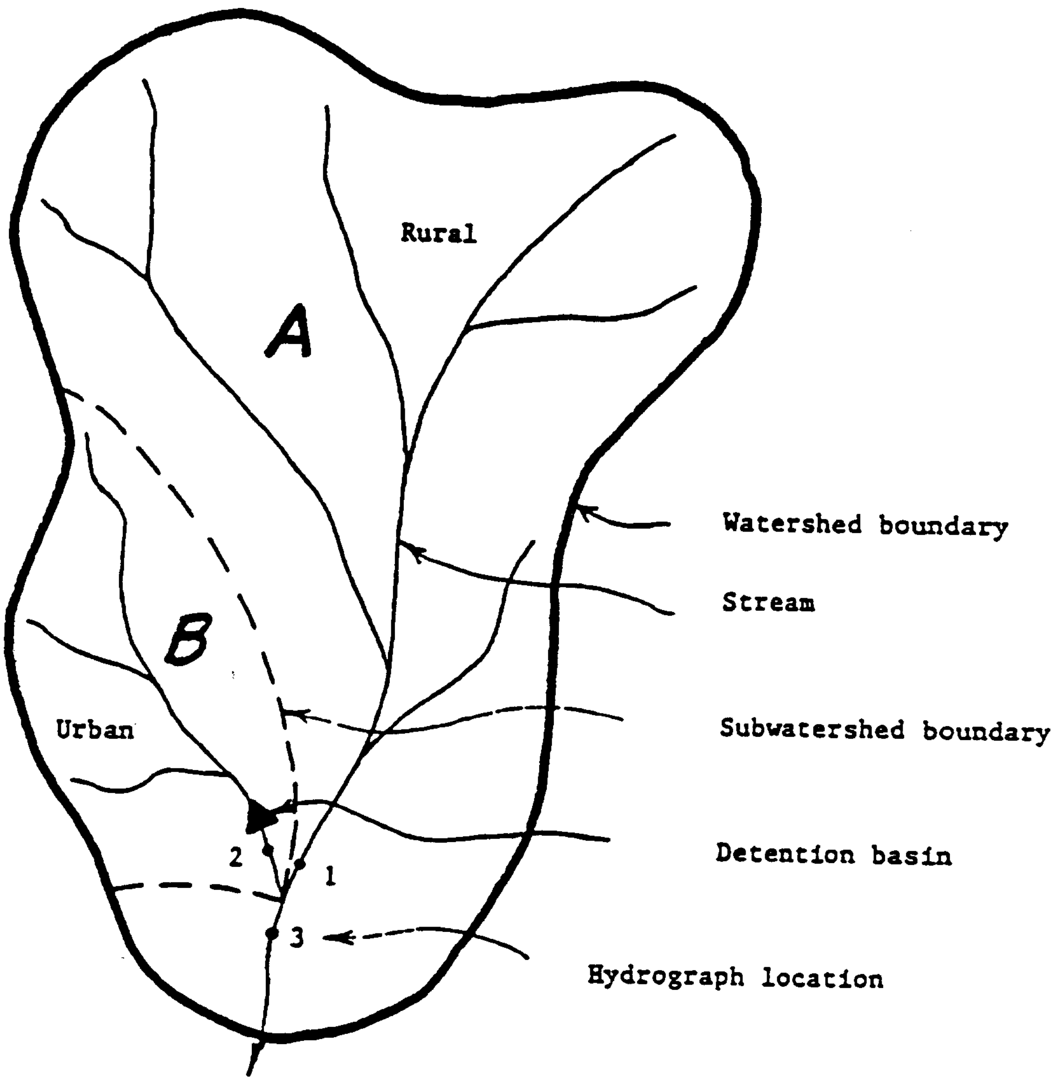


Figure 36. Detention pond located in downstream portion of watershed (Rosmiller 1987).

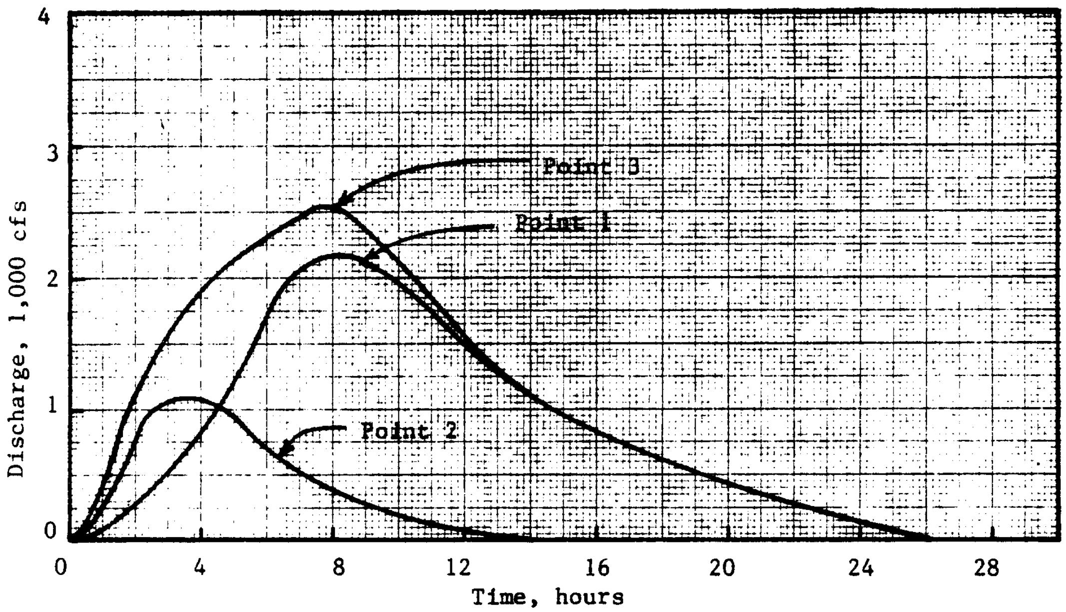


Figure 37. Hydrographs before urbanization without detention (Rosmiller 1987).

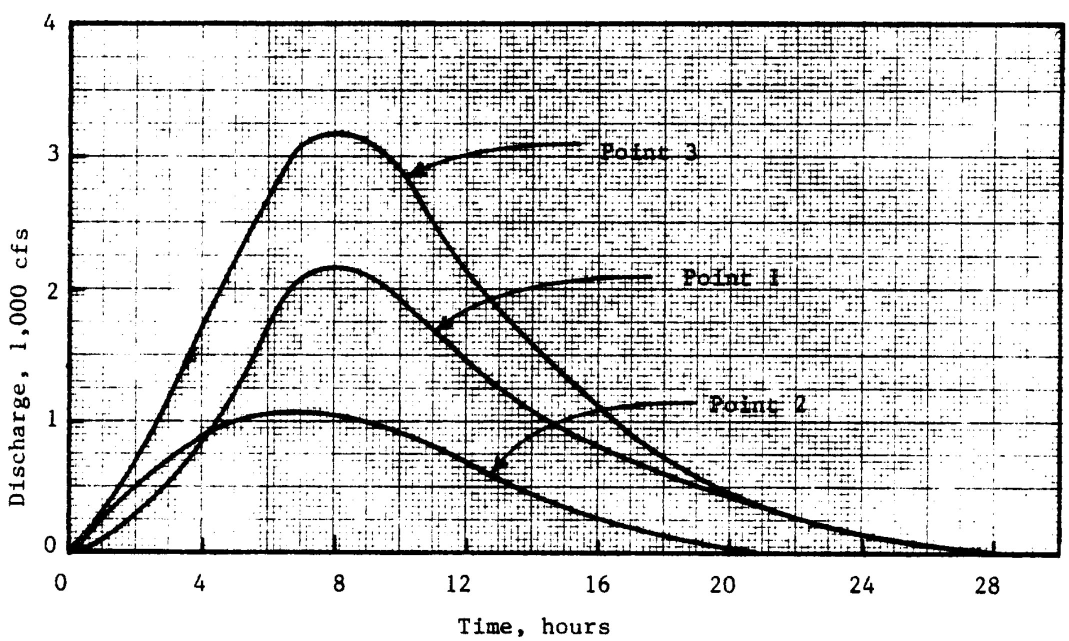


Figure 38. Hydrographs after urbanization with downstream detention (Rosmiller 1987).

A detention pond does not reduce the runoff volume (in the absence of evaporation or seepage), but can only delay the discharge of the runoff. Urbanization results in both increased peak runoff rates and runoff volume. Detention can radically alter the shape of a hydrograph (and therefore the peak runoff rate) but it cannot reduce the runoff volume. If the peak runoff rate is reduced, and no volume reduction occurs (such as from infiltration practices) then the hydrograph base must be expanded. This expanded base hydrograph, if from a downstream area, can interact with the naturally delayed portions of upstream hydrographs (assuming the rain duration was less than the total watershed time of concentration).

Rosmiller (1987) also states that similar problems may occur with detention facilities randomly located throughout a watershed. This can be caused by stormwater ordinances requiring detention facilities located at each development site that are to preserve pre-development peak runoff rates. He points out that detention ponds for peak flow rate objectives must be carefully located to minimize these interferences. He explains that effective stormwater management to obtain peak flow rate objectives must be met using a combination of regional ponds on the main stem and major tributaries for main stem protection and smaller on- and off-site ponds for local area protection. Rosmiller's (1987) three steps to minimize peak flow increases with interfering hydrographs from multiple ponds are as follows:

"1. Locate the regional ponds first and determine the volume of storage needed to obtain the attenuation needed to reduce future peak flows to pre-development peaks.
2. Address each watershed upstream of each regional basin in turn to determine where supplemental ponds are needed to give protection to the inhabitants and property in each watershed.
3. Design these localized on- and off-site ponds plus the regional pond for that watershed in concert with each other so that the overall effect is achieved."

Pond Surface Area and Shape

Surface area is one of the most important design considerations for particle removal. Surface area is also important if the pond is to be used for recreational purposes. A minimum pond size of about five acres is necessary for a pond to have much recreation value for anything but ice skating (Ontario 1984). Large pond volumes also reduce the chance of a rain displacing all of the pond volume and increase the residence times of the water for further water quality improvement (Hey and Schaefer 1983).

Hittman (1976) reports that pond length to width ratios of about five have produced maximum pond efficiencies (decreased short-circuiting) during dye tests. If a long and narrow pond cannot be constructed, Schueler (1986) suggests that baffles or gabions be placed within the pond to lengthen the flow path between the inlets and outlets. Bondurat, *et al.* (1975) has also suggested that the idealized pond shape would be triangular: narrow near the inlet and wider near the outlet. This triangular configuration would allow more efficient particle settling by having a continually decreasing forward velocity. Very irregular pond shapes may decrease circulation and cause localized nuisance problems. The pond shape should be irregular for aesthetic considerations, but with minimal opportunities for water stagnation.

Pond Water Depth

Chambers and Tottle (1980) state that pond water depth affects algae growth, aquifer contamination, water stratification, fish survival, sedimentation, and flood control. A storage volume above the permanent pool elevation of the pond affects the pond's ability to absorb excess flows for flood control. Harrington (1986) found that increasing the wet pool depth increases sedimentation efficiency (due to flocculation), but that surface area increases were much more effective in enhancing the water quality performance of wet ponds. A minimum wet pool depth is very critical in wet ponds to decrease scour losses of previously settled material. Without an adequate permanent pool depth, very little water quality benefits can be expected from wet ponds.

To reduce widespread attached aquatic plant growth problems, a pond depth of at least four feet is recommended. This depth will generally prevent the growth of attached aquatic plants in clean ponds. Similarly, shallower pond depths are needed in areas where attached aquatic plants are wanted, such as along much of the recommended perimeter shelf of wet ponds. Schueler (1986) reports that many emergent plants require water depths of less than six inches, while submerged plants typically require water one to two feet deep. Deep ponds will therefore restrict plant growth. A water depth of about six feet over the major portion of the pond will also increase winter survival of fish.

Extra pond depth needs to be considered for sediment storage between removal operations (Schimmenti 1980). Wiegand, *et al.* (1986) state that it costs about five times as much to removal sediment during pond dredging operations (about $14 per cubic yard) as it does to provide extra sediment storage capacity (sacrificial volume) during initial pond construction (about $3 per cubic yard). This sacrificial storage should be provided as deeper forebays near the pond inlets (Driscoll 1986). These forebays, or the use of underwater dams, need to be designed as pre-sedimentation traps to encourage the deposition of sediment in a relatively restricted area. This would result in more frequent sediment removal operations, but at a much lower cost.

Sufficient water depth (at least three feet over the maximum deposited sediment thickness) is also needed to decrease the potential of sediment scour caused by increased flows during large storms (EPA 1983). Hey and Schaefer (1983) found that a depth of five feet was sufficient to protect the unconsolidated sediment from resuspension in Lake Ellyn. Deep isolated pools should also be discouraged, as they will tend to accumulate poor sediment and poor water quality (Free and Mulamoottil 1983 and Wigington, *et al*. 1983). Schueler (1986) also recommends against ponds with average depths greater than six to eight feet to prevent water stratification and associated water quality and fish survival problems.

Pond Side Slopes

Reported recommended side slopes of detention ponds have ranged from 1:4 (one vertical unit to four horizontal units) to 1:10. Steeper slopes will cause problems with grass cutting and may erode. Steep slopes are not as aesthetically pleasing and are more dangerous than gentle slopes (Chambers and Tottle 1980). Schueler (1986) also recommends a minimum slope of 1:20 for land near the pond to provide for adequate drainage.

The slope near the waterline, and for about one foot below, should be relatively steep (1:4) to reduce mosquito problems (by reducing the amount of frequently wetted land surface), and to provide relatively fast pond drawdown after common storms. However, a flat underwater shelf several feet wide and about one foot below the normal pond surface is needed as a safety measure to make it easier for anyone who happens to fall into the pond to regain their footing and climb out. This shelf should also be planted with native rooted aquatic plants (macrophytes) to increase the aesthetics and habitat benefits of a pond and to create a barrier making unwanted access to deep water difficult.

Another method of treating pond edges is placing gravel along the pond edge to decrease erosion and to make mowing easier (Chambers and Tottle 1980). This method requires placing a layer of gravel about one foot deep and 15 feet wide along the pond edge, from about ten feet above the normal waterline edge and extending about five into the water.

Internal Baffles

The use of baffles within ponds has been shown to significantly increase detention pond performance (Hittman 1976). Baffles increase the travel distance of the water (increase the length to width ratio) and reduce short-circuiting. Particle removal is therefore closer to what is theoretically predicted.

Outlet Structures

Most of the effort given to alternative outlet structure designs has been for dry detention ponds. Wet ponds usually only have a surface weir, outlet pipe, or other simple overflow device to allow the passage of displaced pond water during rains. With the use of a more sophisticated outlet device, located at the normal wet pond surface elevation, more efficient particulate removals and flood control benefits may occur.

Hittman (1976) recommends that wide outflow (and inflow) channels be used to decrease erosion. If wide flow channels are not possible, then energy dissipaters to reduce the water velocity should be used. The Natural Resources Conservation Service (was SCS 1982) has prepared design guidelines for outlet structures for wet detention ponds. These guidelines include a turf-covered embankment having a trapezoidal cross section, a pipe passing through the embankment as the major outlet with a metal riser and upstream trash rack, and an emergency spillway.

Controlled emptying of a detention pond at low outlet flow rates is desirable for effective sediment removal and flood control. A small diameter outlet pipe, or a small orifice on a plate, is usually used to achieve low outflows. The rate of discharge varies for these outlets because of varying overlying water levels. High flow rates occur with higher water levels and the outlet flows decrease with falling water levels. Selecting an appropriate outlet structure has significant effects on pond performance. To have a constant pond performance for all events (if desired), the shape of the outlet must allow a constant upflow velocity (pond outflow rate divided by pond surface area).

If water temperature increases are expected to be a problem, then subsurface outlets may be needed. Subsurface outlets also minimize trash fouling of the outlet. One method of achieving subsurface discharges is to use a submerged large diameter pipe (the pipe bottom must still be at least three feet off of the pond bottom to minimize sediment scour) discharging to a control box that contains the outlet weir (such as a v-notch weir) whose invert is above the top of the pipe.

Mason (1981) states that the benefits of regulating runoff from the frequent less intense storms are usually overlooked. Smaller storms produce less runoff per event, but may be heavily contaminated and occur frequently. Outlets having variable opening sizes with depth can be designed to provide some detention of small rains while allowing flood control benefits from the larger storms. V-notch weirs and multi-stage outlets can control both low and high flows and are recommended for general use. These devices need to be located with their lowest openings at the permanent pool water elevation in wet ponds to provide both desired water quality and flood control benefits.

Orlando, FL, pond modifications to capture litter and other floating debris in pond

Typical perforated riser vertical stand-pipe at temporary pond, with no bar screens

Retro-fitted v-notch weir in place of large rectangular weir at Monroe St. pond in Madison, WI

Temporary outlet control at construction site pond using a plastic membrane over a timber weir, Auckland, NZ

Figure 39. Outlet structures at wet detention ponds.

Floating, multi-stage risers at Auckland, NZ, construction site pond

Debris dome over Auckland, NZ, pond riser outlet


Box outlet emergency spillway at Gulf Port, MS, pond


Vertical riser with safety bars, but culvert outlet should normally be placed near pond bottom to allow drainage for maintenance (State of Maryland photo)


Birmingham, AL pond outlet structure, showing cattails partially blocking 22 in orifices through concrete wall (John Easton photo)


Rock overflow weir

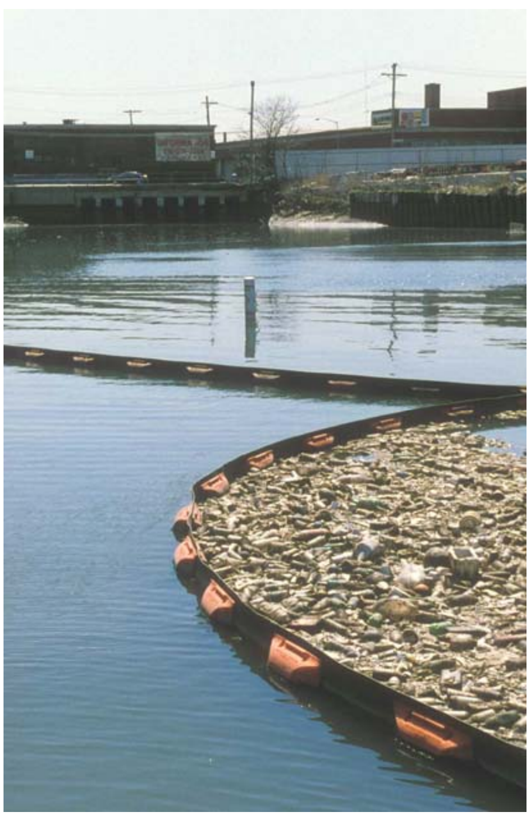

Slot energy dissipitators at pond outlet

Adjustable spillway using boards in grooved weir

Figure 40. Outlet structures at wet detention ponds.

Bar screen at Boston CSO facility for trash control

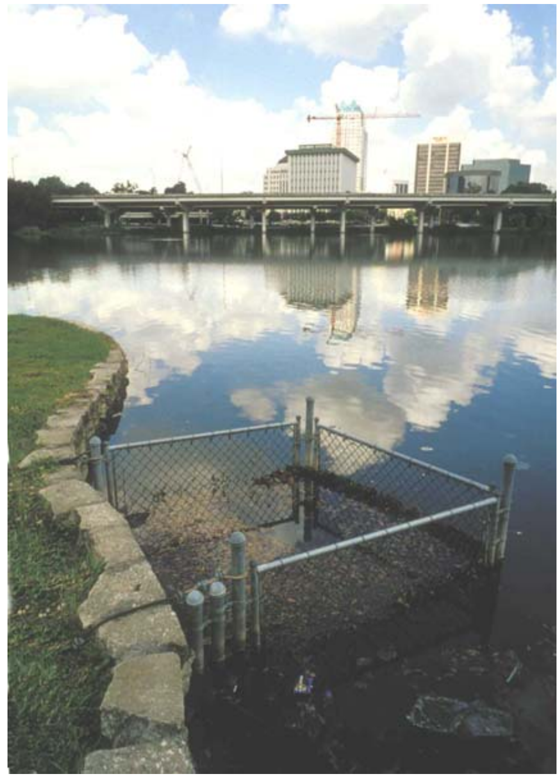
NYC Dept. of Envir. Protection trash barge



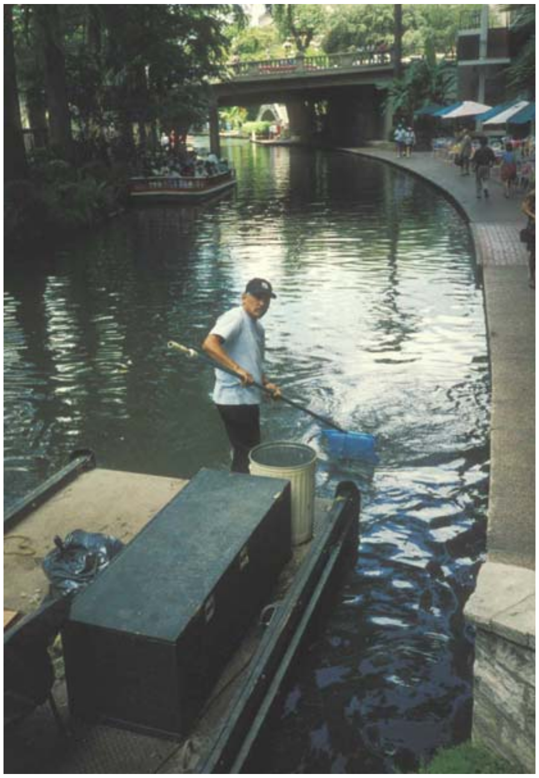
Garbage boom

Trash Net at Fresh Creek Brooklyn, NY


Trash Net system at Fresh Creek, Brooklyn, NY


Trash fence at pond inlet at Orlando pond, FL


Trash fence at pond inlet at Orlando pond, FL



Manual garbage removal along Riverwalk, San Antonio, TX

Figure 41. Trash collection and traps at ponds and urban waterways.

Emergency Spillways

All detention ponds must also be equipped with emergency spillways. Mason (1982) states that the preferred location of an emergency spillway is on undisturbed ground rather than over a prepared embankment to reduce the erosion potential. Detention ponds treating runoff from small contributing areas can safely handle overflows as sheetflows through well designed swales.

The Natural Resources Conservation Service (SCS 1982) guidelines for designing runoff control measures must be followed when designing emergency spillways for wet detention ponds. In addition, if the detention pond is large, special regulations of the state and the Army Corps of Engineers must be followed.

Large concrete emergency spillway

Large concrete emergency spillway with energy dissipater blocks

Broken concrete spillway due to undercutting (photo by Mark Burford)

Gravel emergency spillway over riser and culvert, Madison, WI

Figure 42. Emergency spillways at wet detention ponds.

Multiple Detention Ponds and their use With Other Control Devices

Two or more wet detention ponds in series have been used to increase the removal of fine-grained sediment (Hittman 1976). Multiple ponds usually have better removals than a single large pond having the same surface area. Reduced short-circuiting and scouring of sediment usually occurs and maintenance dredging is restricted to the first pond. It is important however that the downstream pond be significantly larger than the upstream pond for improved performance.

Detention ponds can also be appropriately used in conjunction with other control measures. Because detention ponds only affect particulate pollutants, source area infiltration of relatively unpolluted waters may be needed to reduce soluble pollutant discharges. Source area infiltration also reduces the flow volumes that need to be treated by outfall wet detention ponds, allowing size reductions for the ponds or increased performance.

Wet detention ponds can be used as pretreatment devices before infiltration to reduce the potential contamination of groundwater. However, very little soluble pollutants (the pollutants that have greater potential for affecting groundwater) are typically removed by wet detention ponds. They can, however, remove most of the particulates that are likely to clog infiltration devices, greatly extending the life of the infiltration device.

Enhancing Pond Performance During Severe Winter Conditions

Oberts (1990 and 1994) monitored four urban wet detention ponds during both warm and cold weather in Minnesota. The ponds performed as expected during warm weather, providing typical removals of suspended solids (80%), lead (68%), and TP (52%). However, he found that the ponds did a much worse job of removing suspended solids (39%), organic matter (12% for COD), nutrients (4 % for TKN to 17% for TP) and lead (20%) in the winter. He found that thick ice, which can form as much as 1 m in thickness, effectively eliminated much of the detention volume for incoming snowmelt water. In addition, the first melting water was forced under the ice, causing scour of the previously sediments. Later snowmelt water flowed across the surface of the ice, with very little sedimentation opportunities. Any sediment that was accumulated on top of the underlying ice was later discharged when the ice melted. Similar research in Minnesota wetlands also showed similar dismal performance during winter conditions, for much the same reasons.

Oberts (1990 and 1994) proposed several improvements in stormwater management during winter conditions. His initial recommendation is to utilize infiltration and grass filtering in waterways before any detention facilities. He found that substantial infiltration can occur, even in clayey soils, underlying the snow. The ground under snowpacks is rarely frozen and infiltration can be significant until the soil becomes saturated. If the snowmelt is originating from areas having automobile activity (streets and parking areas) or sidewalks, care must be taken because the snowmelt likely would have high concentrations of salts that would adversely affect the local groundwater (Pitt, *et al*. 1996). Figure 43 shows a layout of a stormwater treatment facility for northern areas, using grass swales, infiltration areas, and a wetland/detention facility (Oberts 1994). The design of the detention pond should be modified for winter operations. A low flow channel leading to and through the pond will discourage the formation of ice. The pond can also be aerated to prevent ice formation, however, if it gets extremely cold, ice formation could then be very thick and rapid. The most important suggestion by Oberts is to use a special riser for the outlet of the pond that can be used to draw down the water elevation during the winter. Ice would then form near the bottom of the pond and seal off the sediments. As the snowmelt occurs, the bottom outlets on the riser should be closed, forming a deeper pond for better sedimentation. Figure 44 shows a schematic of this pond.

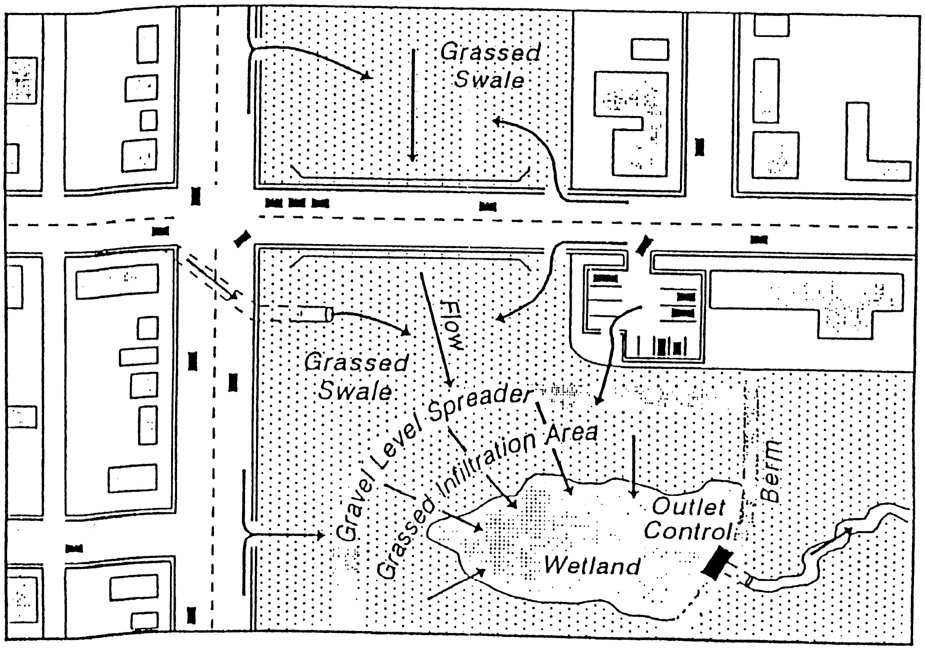


Figure 43. Treatment park concept for severe weather runoff and snowmelt treatment (Oberts 1994).

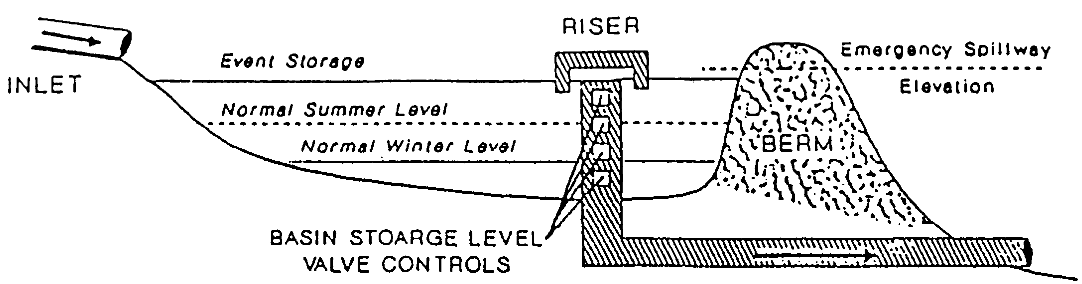


Figure 44. Wet detention pond outfall risers for winter conditions (Oberts 1994).

Droste and Johnston (1993) examined snowmelt quality from snow disposal areas in Ottawa and conducted treatability tests to examine the benefits of different settlement times in 1 L test columns. They found that 2 to 6 hour settling times in these columns produced suspended solids and metal removals approaching 90%. These tests were conducted in controlled laboratory conditions and were not subjected to the actual site problems identified by Oberts. These tests do indicate that sedimentation treatment of snowmelt is likely beneficial, especially if the unique problems of scour and ice formation can be overcome.

Mayer, *et al.* (1996) examined the performance of four wet detention ponds in Toronto during different seasons and during non-storm conditions. The thick ice cover on the ponds during the winter severely affected the pond water quality. In addition, snowmelt and runoff from rainfall occurring on an existing snowpack, were poorly treated by the ponds. Few of the biochemical processes that normally enhance pollutant removal in wet detention ponds during warm weather are available during the winter, plus the ice pack decreases the efficiency of the physical processes, as noted by Oberts. Water beneath the winter ice was typically devoid of oxygen, causing the release of ammonia from sediments and increasing the water column concentrations to about 0.5 mg/L. High grit concentrations in snowmelt, associated with winter sanding of streets, were effectively removed in the detention ponds. However, the high chloride concentrations, from salting of the streets, were not affected by the ponds, as expected.

Marsalek, *et al.* (2000) examined the hydrodynamics of a frozen in-stream stormwater management pond located in Kingston, Ontario, Canada. Measurements of the velocity field under the ice cover agreed well with that simulated by a CFD model (PHOENICS^{TM}). During a snowmelt event, the near-bottom velocities reached up to 0.05 m/s, but were not sufficient to scour the bottom sediment.

Detention Pond Design Fundamentals

The basic design approaches for wet detention ponds consider either slug flow or completely mixed flow. Martin (1989) reviews these flow regimes and conducted five tracer studies in a wet detention pond/wetland in Orlando, FL, to determine the actual flow patterns under several storm conditions. Completely mixed flow conditions assumes that the influent is completely and instantaneously mixed with the contents of the pond. The concentrations are therefore uniform throughout the pond. Under plug flow conditions, the flow proceeds through the pond in an orderly manner, following streamlines and with equal velocity. The concentrations vary in the direction of flow and are uniform in cross section. The steady state resident time for both flow conditions is the same for both flow patterns, namely the pond volume divided by the discharge rate. Historically, wet detention ponds have been designed using the plug flow concept, probably because it had been used in conventional clarifier designs for water and wastewater treatment. In reality, detention ponds exhibit a combination flow pattern that Martin terms moderately mixed flow. He found that the type of mixing that actually occurs is dependent on the ratio of the storm volume to the pond storage volume. If the ratio is less than one, plug flow likely predominates. If the ratio is greater than one, the flow type is not as obvious. With faster flows in the pond, short-circuiting effectively reduces the available pond storage volume (and therefore the resident time), with less effective treatment.

The stormwater management system that Martin (1989) monitored was comprised of a 0.2 acre wet detention pond followed by a 0.7 acre wetland. The drainage area was 41.6 acres, with 33% roadway, 28% forest, 27% high density residential, and 13% low density residential land uses. The system was therefore about 2% of the drainage area, with the wet detention pond portion about 0.5% of the drainage area. The pond's maximum available live storage volume was 18,500 ft^3. The system produces moderate to high pollutant reductions of solids, lead, and zinc (between 50 and 80%) and smaller reductions for nitrogen and phosphorus (between 30 and 40%). At low discharges and with large storage volumes, the pond was found to be moderately well mixed with residence times not much less than the maximum expected if operating under ideal mixing conditions, with little short-circuiting apparent. At higher discharges and with less storage volume, significant short-circuiting occurred.

Detention facilities designed for flood control differ greatly from those designed for water quality improvements (Jones and Jones 1982 and Dally, *et al*. 1983). However, it is still possible to design dual-purpose detention facilities to meet both water quality and flood control benefits. Flood control facilities are designed to affect large, but infrequent, storms and "ignore" smaller, but common, storms. Water quality facilities need to address the opposite set of conditions. Stormwater quality concerns are most commonly associated with frequent events that cause chronic long-term receiving water effects. As an example, very few fish kills have been related to specific storms, but many urban receiving waters have very poor fisheries due to continually poor quality urban runoff discharges (Pitt 1986). Retention basins should be cost-effectively sized based on analysis of local-hydrological characteristics and their impact on stormwater-runoff capture. Urbonas, *et al*. (1996) suggested that fine-tuning a pond design to site characteristics and local conditions can save significant costs over recommending oversized ponds. For more than a decade, the Queen's University/National Water Research Institute Stormwater Quality Enhancement Group studied stormwater ponds with a fully instrumented on-line system in Kingston, Ontario, Canada as a representative field installation (Anderson, *et al.* 2001). The research group concluded that a number of identifiable factors would significantly influence the success, failure and sustainability of these ponds. These factors included initial design, operation and maintenance, performance and adaptive design.

Detention facilities can be designed to suppress the flows from small events and provide significant water quality benefits by using small primary outlets, such as stacked orifices or V-notch weirs. If adequate free-board storage is provided, significant flood control benefits from the same detention facilities are also possible. Alternately, wet detention ponds designed for water quality benefits can discharge to downstream dry detention facilities (through small primary outlets and emergency spillways) designed for flood control benefits alone.

Design considerations based on watershed scale is also important, especially for flood control purposes. Local flooding can be addressed by a relatively small detention facility that provides little, if any, downstream flood control benefit. From a water quality viewpoint, a detention facility can also be designed to protect a local sensitive water body that would produce very little downstream water quality benefits. These local objectives are legitimate, as long as downstream problems are not increased (as can occur with flood control facilities). Alternative local controls may also be available to alleviate both local problems and larger scale watershed problems.

Upflow Velocity
Linsley and Franzini (1964) stated that in order to get a fairly high percentage removal of particulates, it is necessary that a sedimentation pond be properly designed. In an ideal system, particles that do not settle below the bottom of the outlet will pass through the sedimentation pond, while particles that do settle below/before the outlet will be retained. The path of any particle is the vector sum of the water velocity (V) passing through the pond and the particle settling velocity (v). Therefore, if the water velocity is slow, slowly falling particles can be retained. If the water velocity is fast, then only the heaviest (fastest falling) particles are likely to be retained. The critical ratio of water velocity to particle settling velocity must therefore be equal to the ratio of the sedimentation pond length (L) to depth to the bottom of the outlet (D):

$$\frac{V}{v} = \frac{L}{D}$$

as shown on Figure 45.

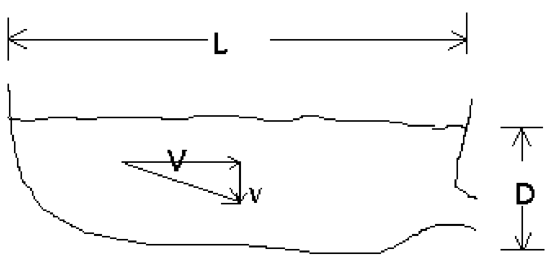


Figure 45. Critical Velocity and Pond Dimensions

The water velocity is equal to the water volume rate (Q, such as measured by cubic feet per second) divided by the pond cross-sectional area (a, or depth times width: DW):

$$V = \frac{Q}{a}$$

or

$$V = \frac{Q}{DW}$$

The pond outflow rate equals the pond inflow rate under steady state conditions. The critical time period for steady state conditions is the time of travel from the inlet to the outlet. During critical portions of a storm, the inflow rate (Q_{in}) will be greater than the outflow rate (Q_{out}) due to freeboard storage. Therefore, the outflow rate controls the water velocity through the pond. By substituting this definition of water velocity into the critical ratio:

$$\frac{Q_{out}}{WDv} = \frac{L}{D}$$

The water depth to the outlet bottom (D) cancels out, leaving:

$$\frac{Q_{out}}{Wv} = L$$

Or

$$\frac{Q_{out}}{v} = LW$$

However, pond length (L) times pond width (W) equals pond surface area (A). Substituting leaves:

$$\frac{Q_{out}}{v} = A$$

and the definition of upflow velocity:

$$v = \frac{Q_{out}}{A}$$

where Q_{out} = pond outflow rate (cubic feet per second),
A = pond surface area (square feet: pond length times pond width), and
v = upflow velocity, or critical particle settling velocity (feet per second).

Therefore, for an ideal sedimentation pond, particles having settling velocities less than this upflow velocity will be removed. Only increasing the surface area, or decreasing the pond outflow rate, will increase pond settling efficiency. Increasing the pond depth does lessen the possibility of bottom scour, decreases the amount of attached aquatic plants, and decreases the chance of winter kill of fish. Deeper ponds may also be needed to provide sacrificial storage volumes for sediment between dredging operations.

The EPA (1986) detention pond water quality analysis procedure includes a partial credit for the removal of particles having settling velocities less than the critical upflow velocity. This is based on the assumption of full depth and well-mixed inlet zones that are used in conventional water treatment clarifiers, but are not likely for stormwater detention ponds which mostly have surface (or near surface) inlets. For stormwater detention ponds, it should be assumed that inlet zones are restricted to the pond surface and that the outlet zones are full depth, providing a worst-case situation.

For continuous flow conditions (such as for water or wastewater treatment), the following relationships can be shown:

$$t = \frac{Volume}{Flow\ rate}$$

and

$$Flow\ rate\ (Q_{out}) = \frac{Volume}{t}$$

where t = detention (residence) time. With

$$v = \frac{Q_{out}}{A}$$

and substituting:

$$v = \frac{Volume}{(t)(A)}$$

but

$$Volume = (A)(depth)$$

therefore,

$$v = \frac{(A)(depth)}{(t)(A)}$$

leaving:

$$v = \frac{depth}{t}$$

It is seen that the overflow rate (Q/A) is equivalent to the ratio of depth to detention time. It is therefore not possible to predict pond performance by only specifying detention time. If pond depth was also specified (or kept within a typical and narrow range), then detention time could be used as a performance specification for a continuous or slug flow condition. However, it is not possible to hold all of the water in a detention pond for the specified detention time. Outlet devices typically release water at a high rate of flow when the pond stage is increased (resulting in minimal detention times during peak flow conditions) and lower flow rates at lower stages, after most of the detained water has already been released. The average detention time is therefore difficult to determine and is likely very short for most of the water during a moderate to large storm. It is much easier to design and predict pond performance using the upflow relationships for variable flow stormwater conditions.

The upflow ratio of outflow rate to pond surface area can be kept constant (or less than a critical value) for all pond stages. This results in a much more direct method in designing or evaluating pond performance. Pond performance curves can therefore be easily prepared relating upflow velocity (and therefore critical particle control) for all stages at a pond site.

Effects of Short-Circuiting on Particulate Removals in Wet Detention Ponds

Under dynamic conditions, particle trapping can be predicted using the basic Hazen theory presented by Fair and Geyer (1954) that considers short-circuiting effects:

$$\frac{y}{y_0} = 1 - \left[1 + \frac{v_o}{n(Q/A)}\right]^{-n}$$

where y_o = initial quantity of solids having settling velocity of v_o
y = quantity of these particles removed
y/y_o = proportion of particles removed having this settling velocity
Q = wet pond discharge
A = wet pond surface area
n = short-circuiting factor (number of hypothetical basins in series)

This equation is closely related to the basic upflow velocity equation developed previously and is also included in WinDETPOND. The short-circuiting factor is typically given a value of 1 for very poor conditions, 3 for good conditions, and 8 for very good conditions. Short-circuiting allows some large particles to be discharged that theoretically would be completely trapped in the pond. However, field monitoring of particle size distributions of detention pond effluent shows that this has a very small detrimental effect on the suspended solids (and pollutant) removal rate of a pond. Figure 46 shows the effects of different n values on the removal of particles having different settling rates (v) compared to the critical settling rate (Q/A). For a particle having a settling rate equal to the critical values (v = Q/A), the ideal settling indicates 100% removal, while for "best performance" ($n = \infty$), the actual removal would be only about 65%. If the pond had an n of 1 (very poor performance), the removal of this critical particle would be only 50%.

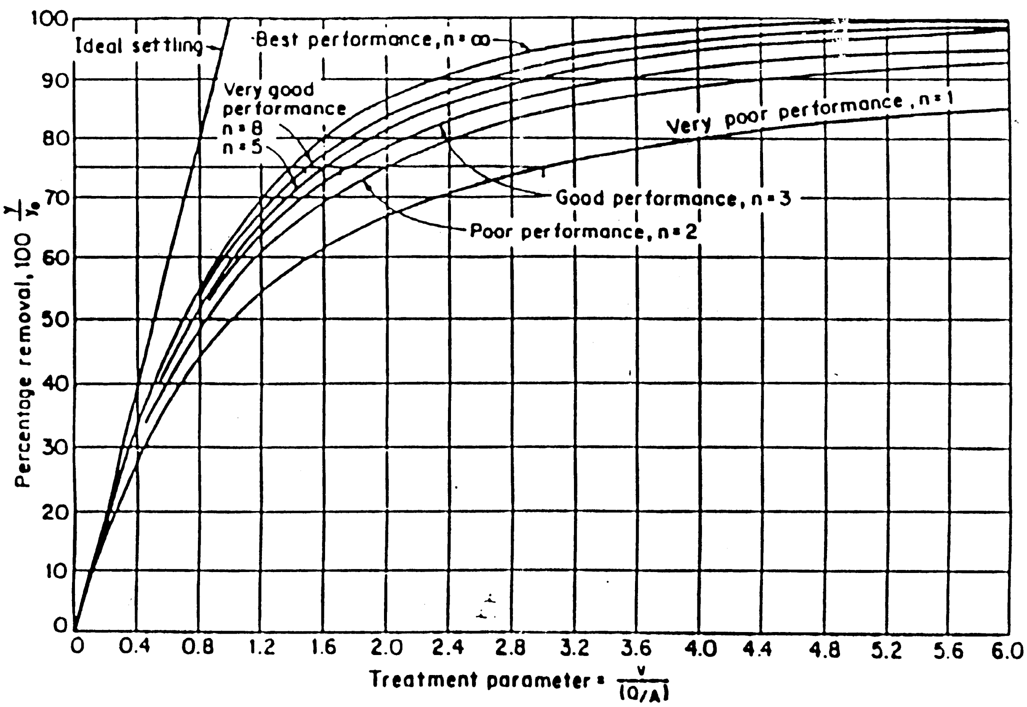


Figure 46. Performance curves for settling basins of varying effectiveness (AWWA 1971).

The degradation of performance is much worse for particles having settling rates much larger than the critical rate. However, most wet detention ponds are greatly over-sized according to their ability to remove large particles, so this degraded performance has minimal effect on the overall suspended solids removal. The suggested detention pond design presented in this discussion only operates at the "design" stage (where the critical particle size is being removed) a few times a year. At all other times, the smallest particles being removed in stormwater wet detention ponds are much smaller than the critical size used in the pond design. Most larger particles are effectively trapped because they are much larger than the design particle size (the pond is over-sized for these large particles), even if they are not being removed at their highest possible rate. In most cases, a few relatively large particles (much larger than the critical design particle size) will be observed in the pond effluent, but they have little effect on the overall SS removal.

Figure 47 shows example particle settling distributions for a pond, comparing effluent conditions using the short-circuiting effects of Hazen's theory. The most common particle size (the mode) changes very little for the different effluent conditions. However, there are more larger-sized particles present in the effluent using Hazen's theory compared to the ideal theory, and the median size obviously increases as the value for n decreases.

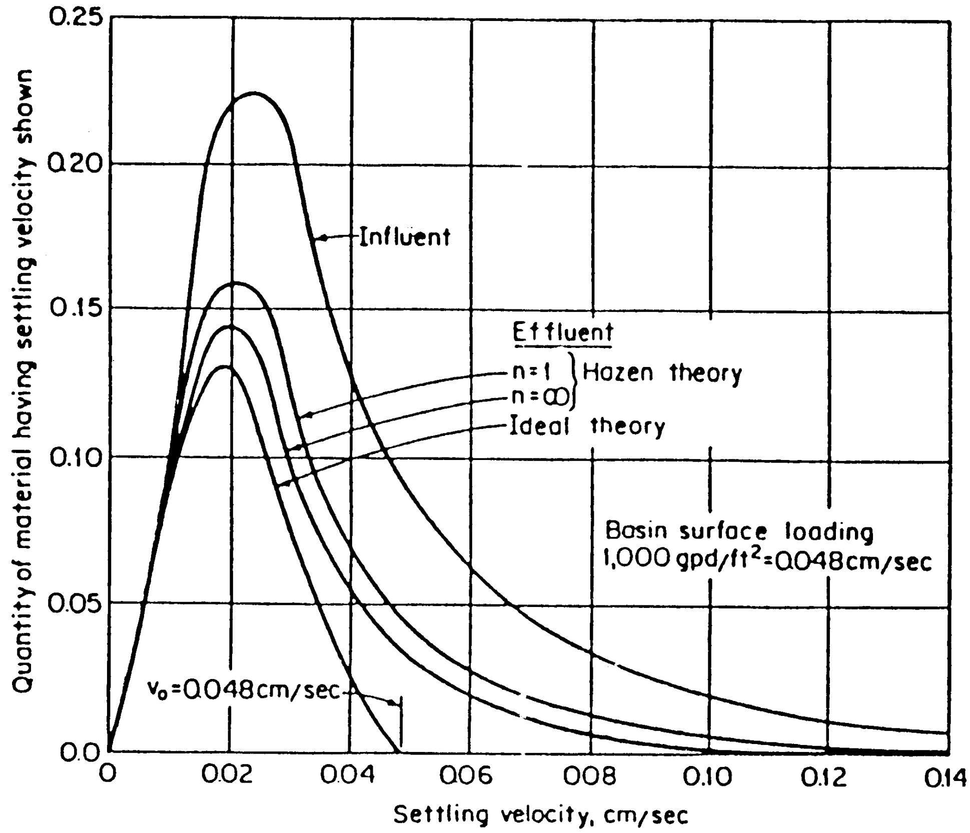


Figure 47. Influent and effluent particle settling rate distributions for settling basins of varying effectiveness (AWWA 1971).

Very little degraded performance was observed at a pond monitored during NURP (EPA 1983) in Lansing, MI, that was expected to have significant short-circuiting. A golf course pond located across the street from a commercial strip was converted into a stormwater pond, but the inlets and outlets were adjacent to each other in order to reduce construction costs. It was assumed that severe short circuiting would occur because of the close proximity of the inlet and outlet, but the pond produced suspended solids removals close to what was theoretically predicted, and similar to other ponds having much similar pond area to watershed area ratios. Actually, the close inlet and outlet may have resulted in less short-circuiting because the momentum of the inflowing waters may have forced the water to travel in a general circular pattern around the pond, instead of directly flowing across the pond (and "missing" some edge area) if the outlet was located at the opposite side of the pond.

In another example, the USGS and the Wisconsin Department of Natural Resources have been monitoring the Monroe St. wet detention pond in Madison for a number of years. Particle size distributions of influent (including bedload) and effluent have been monitored for about 50 storms. The actual particle size distributions and suspended solids removals have been compared to calculated pond performance, using the WinDETPOND computer program (Pitt and Voorhees 1989; Pitt 1993a and 1993b), for different short-circuiting factors. The calculated values of n (based on matching measured effluent particle size distributions with distributions calculated using different values of n) ranged from about 0.2 to 1, indicating “very poor performance”, or worse. However, the pond is producing very good suspended solids removals (85 to 90% reductions) as designed, but the particle size distributions of the effluent indicate some short circuiting (some large particles are escaping from the pond). The short circuiting has not significantly reduced the effectiveness of the pond (measured as the percentage of suspended solids captured). Therefore, care should be taken in locating and shaping ponds to minimize short circuiting problems, but not at the expense of other more important factors (especially size, or constructing the pond at all). Poor pond shapes probably cause greater problems by producing stagnant areas where severe aesthetic and nuisance problems originate.

Several recent urban wet pond studies have investigated various aspects of short-circuiting, and how to reduce its effects on pond performance. The results of dye-tracing studies performed in a stormwater pond in Kingston Township, Ont., Canada, demonstrated an increase in retention times with a reduction in the velocity and volume of short-circuiting flow, and a decrease in wind-generated-flow patterns, due to the installation of retrofitted baffles (Matthews, *et al*. 1997). Shaw, *et al*. (1997) also concluded that regardless of the magnitude of inflow, the length to width ratio (3:2) of the urban-stormwater-detention pond studied in Kingston, and inflow momentum, promoted short-circuiting of the flow and limited settling efficiency. Dewey, *et al*. (2000) used a two-dimensional, vertically averaged hydrodynamic model to compute the circulation and sedimentation patterns in stormwater detention ponds or other water impoundment facilities. The Circulation and Water Quality Model (CWQM) identified areas in the pond where short-circuiting and dead zones occurred. Sedimentation, based on first-order decay, also could be predicted. Field-testing verified that the predicted suspended solids concentration at the outlet and monitored outlet concentrations agreed.

Baffle fence constructed in pond to separate inlet (to right) from pond outlet (to left), forcing water to travel further down pond length.

Railroad on one side and Interstate highway on adjacent side of property restricted pond placement and layout options

Figure 48. Pond site constraints and baffles to reduce short-circuiting.

Residence Time and Extended Detention Ponds

During quiescent conditions, simple column sedimentation occurs, with very little flow through a wet pond. Lateral flow would be caused by a baseflow from the watershed, supplemental water pumped from wells, or groundwater intrusion. Urban area baseflows of about 0.001 cfs per acre of contributing watershed have been observed (Pitt and McLean 1986), but can vary widely. The corresponding lateral flow for most ponds would be very small during dry weather. A 200 acre watershed may only have a baseflow of about 0.2 cfs and a two acre wet pond adequate to serve this watershed may be about 200 feet wide and three feet deep. The dry weather lateral flow would therefore be about 3×10^{-4} ft/sec. It would therefore require very large baseflows and very small ponds to result in significant lateral flows during dry periods. Therefore, interevent settling mainly occurs as a quiescent process, similar to what would be observed during typical settling column experiments (water depth divided by the residence time equaling the critical particle settling rate).

Residence time is defined as the ratio of volume to average flow rate, resulting in a time dimension. It can be assumed to be the average length of time any parcel of water remains in the pond. As in any pond performance measure or design criteria, residence time values are very dependent on good pond configurations. Harrington (1986) stresses the need to subtract pond "dead zones" from pond volume when calculating residence times. Dead zones (and associated short-circuiting) can significantly reduce pond effectiveness.

Designing a wet pond for the treatment of stormwater runoff based on residence time is usually not recommended. Barfield (1986) states that residence (detention) time is not a good criteria for pond performance, but the ratio of peak discharge rate to pond surface area (the peak upflow velocity) is a good criteria of performance. The state of Maryland uses a residence time standard as part of their design criteria for "extended detention" ponds. These ponds are normally dry between events, or have a small and shallow wet pond area near the outlet, and greatly extend in surface area during storms. For these types of ponds, Harrington (1986) found, through computer modeling studies, that a residence time of about nine days is needed to achieve a 70 percent reduction of particulate residue. Nine days is longer than the inter-event period for most rains in the Midwest and the southeast, which is about three to five days. These types of ponds are therefore not expected to be very useful for locations where the interevent periods of rains is short, or the drain-down time of the pond is rapid.

Extended detention ponds may be a suitable retrofitting alternative for existing dry detention ponds to achieve some water quality benefits in situations where it may not be cost-effective, or it may be excessively disruptive, to convert a dry detention pond into a standard wet detention pond. Most dry detention ponds are designed for flow rate reduction benefits and need large amounts of storage volume, or are used as athletic fields during dry weather. Complete re-grading of the site could be very expensive. The use of a relatively small wet pond near the outlet area could achieve some water quality benefits in addition to the existing water flow benefits, be a cost-effective retrofit control measure, and still allow multiple use of the site. For new ponds, much more cost-effective solutions meeting water quality, flood control, and recreation benefits could be achieved with the use of a conventional wet pond located above a dry pond that has an infiltration trench along the dry pond invert.

Figure 49 can be used to estimate the residence time needed in an extended detention pond to achieve specific particle size reduction goals. For a six feet deep pond, a detention time of about three hours would allow particles greater than about 10 µm to settle to the pond bottom. A detention period of 200 hours (about nine days) in this pond, would settle particles greater than about 1-1/2 µm, which would produce very good SS control.

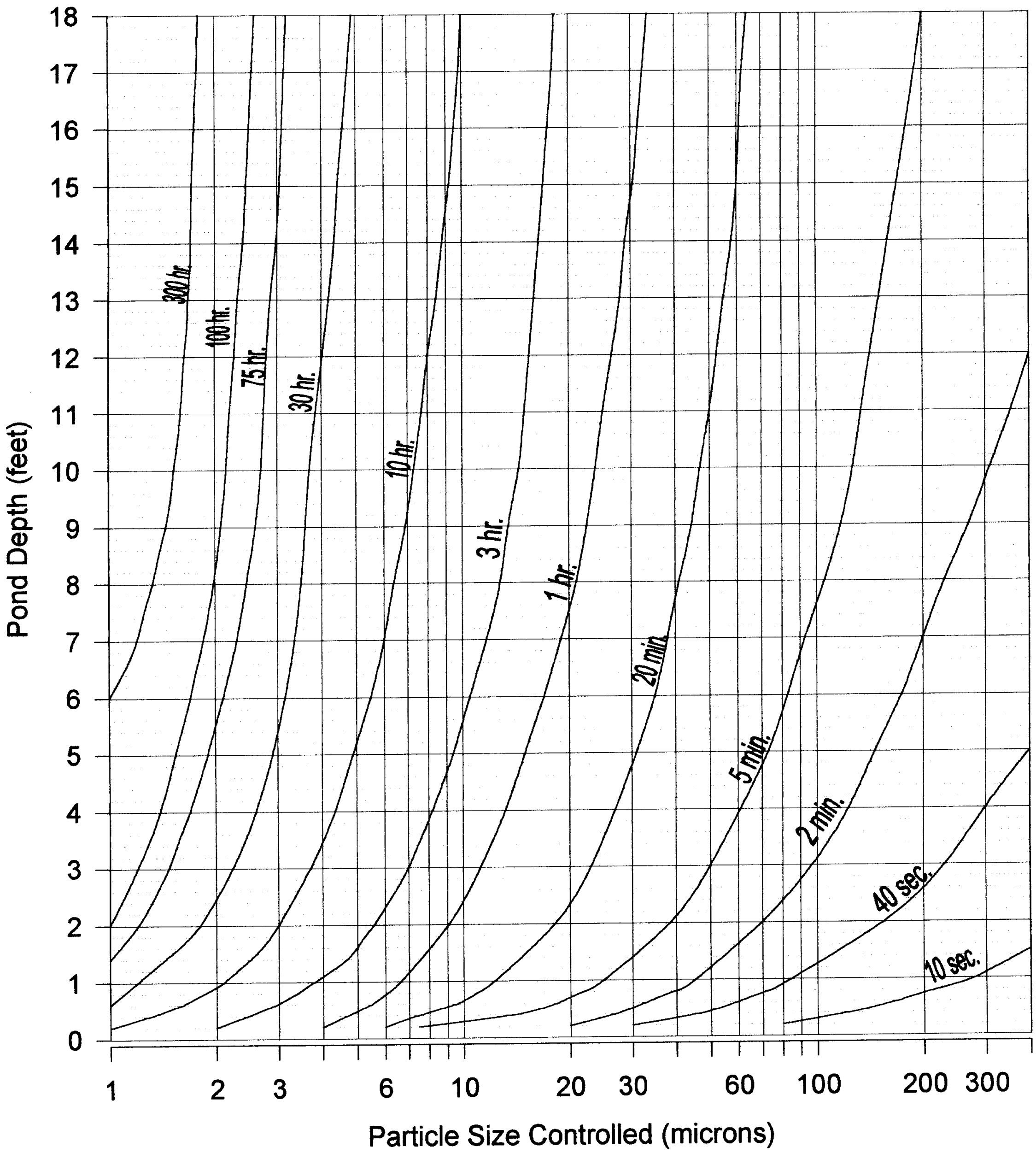


Figure 49. Required residence time for complete settling (hours).

Unfortunately, dry ponds usually do not allow permanent retention of the settled particles. Subsequent storms usually scour the fine particles previously settled to the pond bottom. As stated previously, dry detention ponds have not been shown to be consistently effective water quality control devices. The use of a small permanently wet detention pond or wetland at the downstream end of a dry detention pond could help recapture some of these scoured particles. As noted above, a wet detention pond above a dry pond is usually a much better solution, as the wet pond would then act as a pre-treatment pond, keeping particles and debris out of the dry pond. This would reduce dry pond maintenance and increase its safety by eliminating the deposition of toxic pollutants associated with polluted dust and dirt particles. This is very important if the dry pond is to be used for recreation.

One must be careful not to use Figure 49 to assume that shallow wet detention ponds are more effective than moderately deep ponds. In some cases, shallow forebays (about one foot deep) have been recommended for wet detention ponds, based on this residence time relationship. It appears that shallow detention ponds would require less residence time to control particles. The particles would strike the pond bottom sooner for a shallow pond, but increased turbulence (because of the shallow flow) would not allow the particles to remain in place, scouring and washing them into the main body of the pond, or out the pond outlet.

The discussion on pond depth summarizes many recommendations that wet ponds be at least three feet deep (and preferably five feet deep) over much of their area to reduce particle resuspension from flow turbulence. The discussion of pond configuration also recommends that a deep forebay be used at each pond inlet to provide extra sacrificial sediment storage volume and to concentrate the area of needed sediment removal. These design practices would significantly reduce pond maintenance dredging costs, as compared to dredging the entire pond.

The discussion on upflow velocity as a design criteria showed the relationship between particle settling rates and upflow velocity, while this discussion showed the relationship between particle settling rates and residence times. There must therefore be a relationship between residence time and upflow velocity. Residence time is dependent on pond volume and outlet rate, while upflow velocity is dependent on pond surface area and outflow rate. The relationship between residence time and upflow velocity is therefore equal to the relationship between pond volume and pond surface area, or the pond depth. When a pond depth of five feet is used, the residence times of ponds designed using the upflow velocity method are generally the same residence times needed for similar control levels using the residence time criteria. Even though the two procedures result in the same basic design, it is still recommended that the upflow procedure be used for evaluating wet detention ponds during continuous records of storm events. The depth and configuration design criteria are very critical for the other pond uses (aquatic life, aesthetics, and safety, besides scour prevention) and they should not be varied as part of the major design elements.

The upflow velocity design procedure requires knowing the same stage-surface area and stage-discharge relationships that are also needed when designing ponds for flood control. These relationships also allow specific guidance in the selection of an outlet control device. The residence time design method should be used when designing extended detention ponds or for evaluating pond performance during dry intervals between rains when very little flow occurs.

Stormwater Characteristics and Particle Size

The treatability of stormwater depends a lot on various characteristics of the stormwater. The most important characteristics are the pollutant concentrations, the associations of the pollutants with different particle sizes (or at least the particulate and filterable fractions), and the particle size (and settling velocity) distributions. The following discussion summarizes these stormwater attributes.

Stormwater Pollutant Characteristics for Different Land Uses

The University of Alabama and the Center for Watershed Protection were awarded an EPA Office of Water 104(b)3 grant in 2001 to collect and evaluate stormwater data from a representative number of NPDES (National Pollutant Discharge Elimination System) MS4 (municipal separate storm sewer system) stormwater permit holders. The initial version of this database, the National Stormwater Quality Database (NSQD, version 1.0) is currently being completed. These data are being collected and reviewed to describe the characteristics of this data, to provide guidance for future sampling needs, and to enhance local stormwater management activities in areas having limited data. The cumulative value of the monitoring data collected over nearly a ten-year period from more than 200 municipalities throughout the country has a great potential in characterizing the quality of stormwater runoff and comparing it against historical benchmarks.

As of mid-summer 2003, 3,770 separate events from 66 agencies and municipalities from 17 states have been collected and entered into NSQD. Figure 50 shows the locations of these municipalities on a national map. The current database (NSQD, Version 1.0) covers areas mostly in the southern, Atlantic, central, and western parts of the US. Anticipated future project phases will help us extend our national coverage. The database is located at:

http://www.eng.ua.edu/~rpitt/Research/ms4/mainms4.shtml

Table 9 is a summary of the Phase 1 data collected and entered into the database as of mid-summer 2003. The data are separated into 11 land use categories: residential, commercial, industrial, institutional, freeways, and open space. In addition, summaries are also shown for mixed land use areas (indicating the most prominent land use), and for the total data set combined. The total number of individual events included in the database is 3,770, with most in the residential category (1069 events). For most common constituents, we have detectable values for almost all monitored events. However, filtered heavy metal observations, and especially organic analyses, have many fewer detected values. This table shows the percentage of analyzed samples that had detected values. The median and coefficient of variation (COV) values are only for those data having detectable concentrations.

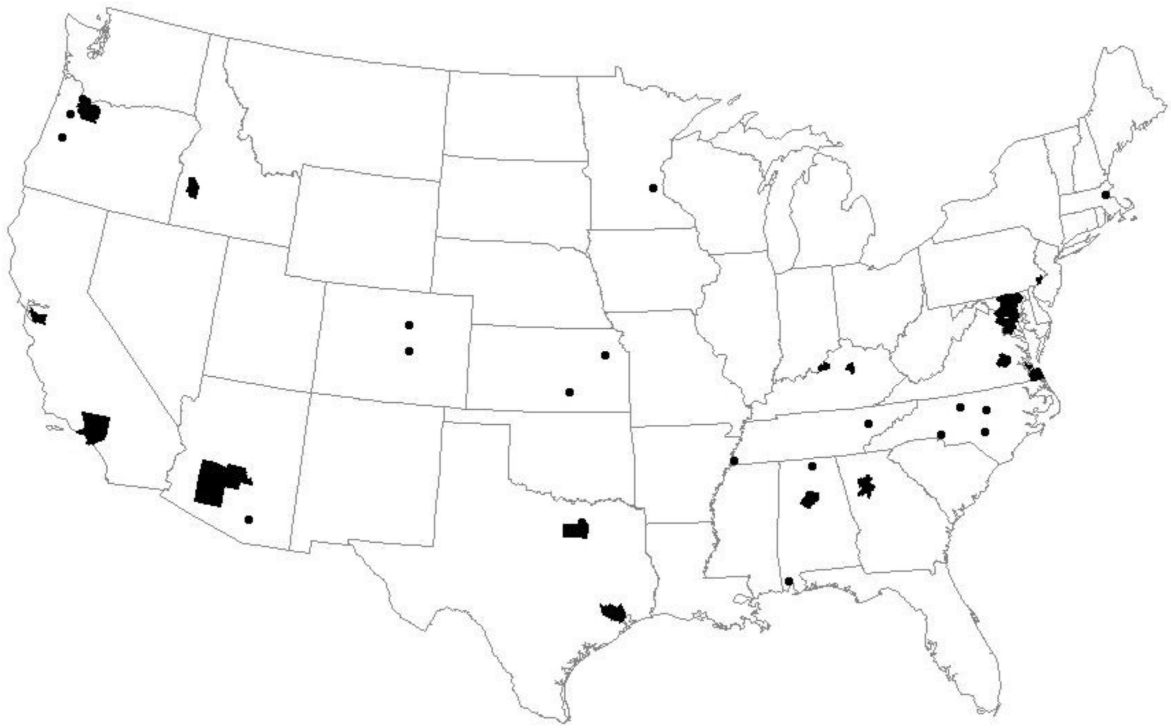


Figure 50. Communities from which data has been obtained and entered in NSQD.

Figures 51 and 52 contain examples of grouped box and whisker plots for several constituents for different major land use categories. The TKN, plus copper, lead, and zinc observations are lowest for open space areas, while the freeway locations generally had the highest median values, except for phosphorus, nitrates, fecal coliforms, and zinc. The industrial sites had the highest reported zinc concentrations. ANOVA analyses for all land use categories (using SYSTAT) found significant differences for land use categories for all pollutants.

Table 9. Summary of Available Stormwater Data Included in NSQD, version 1.0

	Area (acres)	% Imperv.	Precip. Depth (in)	Runoff Depth (in)	Cond. (uS/cm @25ºC)	Hardness (mg/L CaCO3)	Oil and Grease (mg/L)	pH	Temp. (C)	TDS (mg/L)	TSS (mg/L)	BOD_5 (mg/L)	COD (mg/L)
Overall Summary (3770)													
Number of observations	3756	2185	3187	1446	688	1083	1835	1668	864	2994	3396	3110	2758
% of samples above detection	100	100	100	100	100	98.6	71.8	100	100	99.5	99.6	96.2	98.3
Median	56.0	54.3	0.47	0.18	120	38.0	4.0	7.50	16.4	80	58	8.6	53
Coefficient of variation	3.6	0.4	1.0	2.0	1.8	1.4	10.1	0.1	0.4	2.5	1.8	1.6	1.2
Residential (1069)													
Number of observations	1066	647	906	418	107	250	524	319	205	861	985	935	791
% of samples above detection	100	100	100	100	100	100	64.5	100	100	99.2	99.7	97.6	98.6
Median	57.3	37.0	0.46	0.12	96	32.0	3.1	7.3	16.4	70.7	48	9	55
Coefficient of variation	4.7	0.4	1.0	1.9	1.5	1.0	8.0	0.1	0.4	2.0	1.8	1.5	1.1
Mixed Residential (615)													
Number of observations	612	277	438	217	106	157	255	324	143	477	578	561	448
% of samples above detection	100	100	100	100	100	98.1	74.2	100	100	99.4	99.8	93.9	99.3
Median	150.8	44.9	0.55	0.18	115	39.7	4.0	7.50	16.0	86	68	7.7	42
Coefficient of variation	2.1	0.3	0.8	1.4	1.2	1.2	2.5	0.1	0.3	2.2	1.6	1.3	1.4
Commercial (497)													
Number of observations	497	258	415	134	66	139	302	165	79	407	452	426	367
% of samples above detection	100	100	100	100	100	100	72.2	100	100	99.5	98.9	97.4	98.4
Median	38.8	83.0	0.39	0.23	119	38.9	4.7	7.30	16.0	77	43	11.9	63
Coefficient of variation	1.2	0.1	1.0	1.2	1.0	1.1	3.2	0.1	0.4	1.8	2.0	1.1	1.0
Mixed Commercial (303)													
Number of observations	303	237	276	106	40	80	116	137	79	250	280	261	250
% of samples above detection	100	100	100	100	100	98.8	88.8	100	100	100	100	99.2	99.6
Median	75.0	60.0	0.47	0.35	103	35.0	4.0	7.60	15.0	69	54	9	60
Coefficient of variation	2.1	0.3	0.9	1.1	0.6	1.8	2.9	0.1	0.4	1.9	1.4	1.7	1.0
Industrial (524)													
Number of observations	524	317	436	202	108	138	324	234	140	422	431	407	364
% of samples above detection	100	100	100	100	100	96.4	70.7	100	100	99.8	99.8	95.3	98.6
Median	39.0	75.0	0.49	0.14	136	39.0	4.0	7.50	17.9	92	77	9	60
Coefficient of variation	1.6	0.3	1.0	2.7	1.3	1.5	12.4	0.1	0.3	3.5	1.5	1.7	1.2

Table 9. Summary of Available Stormwater Data Included in NSQD, version 1.0 (continued)

	Area (acres)	% Imperv.	Precip. Depth (in)	Runoff Depth (in)	Cond. (uS/cm @25ºC)	Hardness (mg/L CaCO3)	Oil and Grease (mg/L)	pH	Temp. (C)	TDS (mg/L)	TSS (mg/L)	BOD_5 (mg/L)	COD (mg/L)
Mixed Industrial (252)													
Number of observations	252	133	226	117	58	83	80	180	71	224	244	220	218
% of samples above detection	100	100	100	100	100	94.0	96.3	100	100	100	100	95.0	98.6
Median	127.7	44.0	0.45	0.29	111	33.0	3.3	7.69	18.0	80	82	7.2	40
Coefficient of variation	2.0	0.3	0.8	1.2	0.8	0.5	2.2	0.1	0.4	2.4	1.4	1.7	1.1
Institutional (18)													
Number of observations	18	18	17	14						18	18	18	18
% of samples above detection	100	100	100	100						100	94.4	88.9	88.9
Median	36.0	45.0	0.18	0.00						53	17	8.5	50
Coefficient of variation	<0.1	0.2	0.9	2.1						0.7	0.83	0.7	0.9
Freeways (185)													
Number of observations	185	154	182	144	86	128	60	111	31	97	134	26	67
% of samples above detection	100	100	100	100	100	99.2	71.7	100	100	99.0	99.3	84.6	98.5
Median	1.6	80.0	0.54	0.41	99	34.0	8.0	7.10	14.0	78	99	8	100
Coefficient of variation	1.4	0.13	1.0	1.7	1.0	1.9	0.6	0.1	0.4	0.8	2.5	1.3	1.1
Mixed Freeways (20)													
Number of observations	20		20		13	12	15	19	19	17	17	17	17
% of samples above detection	100		100		100	100	100	100	100	100	100	100.0	100.0
Median	63.1		0.68		418	83	4.0	7.80	16.0	174	81	7.4	48
Coefficient of variation	<0.1		0.6		0.6	0.3	1.6	0.06	0.3	0.4	1.2	0.7	0.5
Open Space (68)													
Number of observations	68	34	60	22	23	28	33	34	23	62	61	62	59
% of samples above detection	100	100	100	100	100	100	75.8	100	100	98.4	96.7	85.5	76.4
Median	73.5	2.0	0.48	0.17	155	117	11.0	7.70	15.5	113	51	4.2	21
Coefficient of variation	1.8	1.3	1.1	1.3	0.7	0.6	1.4	0.08	0.24	0.7	1.9	0.7	1.8
Mixed Open Space (159)													
Number of observations	159	89	158	61	62	50	73	107	55	125	151	142	123
% of samples above detection	100	100	100	100	100	100	82.2	100	100	100	100	99.3	99.2
Median	115.4	34.0	0.43	0.12	215	55.0	2.0	8.00	16.0	106	78	6.6	39
Coefficient of variation	0.9	0.14	0.9	1.2	1.8	1.5	2.5	0.07	0.3	2.3	1.4	2.4	1.5

Table 9. Summary of Available Stormwater Data Included in NSQD, version 1.0 (continued)

	Fecal Coliform (mpn/100 mL)	Fecal Strep. (mpn/100 mL)	Total Coliform (mpn/100 mL)	Total E. Coli (mpn/100 mL)	NH3 (mg/L)	N02+NO3 (mg/L)	Nitrogen, Total Kjeldahl (mg/L)	Phos., filtered (mg/L)	Phos., total (mg/L)	Sb, total (ug/L)	As, total (ug/L)	As, filtered (ug/L)	Be, total (ug/L)
Overall Summary (3770)													
Number of observations	1707	1143	85	67	1914	3087	3199	2480	3283	883	1506	209	948
% of samples above detection	91.2	94.0	90.6	95.5	71.7	97.4	96.5	86.3	96.7	8.0	57.2	27.3	7.7
Median	5081	17000	11000	1750	0.44	0.6	1.4	0.12	0.27	3.2	3.0	1.5	0.4
Coefficient of variation	4.61	3.8	2.4	2.3	3.6	1.1	1.4	1.6	1.5	2.6	2.4	1.0	2.5
Residential (1069)													
Number of observations	440	300		14	595	923	951	732	957	283	417	32	292
% of samples above detection	88.2	89.0		100	81.3	97.6	97.1	84.4	96.9	4.2	46.0	6.3	7.5
Median	7750	24000		700	0.31	0.6	1.4	0.17	0.30	28.0	3.0	1.5	0.5
Coefficient of variation	5.1	1.8		1.6	1.1	1.3	1.3	1.0	1.1	1.5	2.1	0.5	2.5
Mixed Residential (615)													
Number of observations	314	158	27	11	263	540	529	411	557	91	176	18	88
% of samples above detection	94.9	98.1	85.2	90.9	58.6	98.1	95.8	83.5	96.2	8.8	79.0	27.8	12.5
Median	11000	26000	5467	1050	0.39	0.6	1.3	0.12	0.27	1.0	3.1	2.0	0.3
Coefficient of variation	3.3	2.2	1.4	2.1	4.4	1.0	1.9	1.1	1.7	2.1	3.9	0.8	2.7
Commercial (497)													
Number of observations	228	176			299	419	443	317	440	138	207	21	157
% of samples above detection	87.7	91.5			83.3	98.1	97.5	81.7	95.9	2.2	33.3	9.5	4.5
Median	4500	10800			0.50	0.6	1.6	0.11	0.22	69.0	2.3	1.5	0.5
Coefficient of variation	2.8	2.7			1.2	1.1	0.9	1.3	1.2	0.8	3.2	0.5	2.0
Mixed Commercial (303)													
Number of observations	104	87			163	273	261	222	273	80	123	20	80
% of samples above detection	94.2	98.9			66.9	97.1	96.9	95.1	98.6	12.5	64.2	10.0	10.0
Median	4990	11000			0.60	0.6	1.4	0.11	0.25	15.0	2.2	1.8	0.4
Coefficient of variation	3.2	2.8			1.0	0.7	0.9	2.1	1.5	1.0	1.0	0.2	1.6
Industrial (524)													
Number of observations	299	195			255	415	442	327	437	159	264	23	202
% of samples above detection	88.1	93.8			85.9	96.1	96.4	87.2	96.3	17.0	58.0	13.0	10.9
Median	2500	13000			0.50	0.7	1.4	0.11	0.26	4.0	4.0	1.0	0.4
Coefficient of variation	5.6	6.9			4.0	1.0	1.2	1.2	1.4	3.0	1.4	0.4	2.5

Table 9. Summary of Available Stormwater Data Included in NSQD Database, version 1.0 (continued)

	Fecal Coliform (mpn/100 mL)	Fecal Strep. (mpn/100 mL)	Total Coliform (mpn/100 mL)	Total E. Coli (mpn/100 mL)	NH3 (mg/L)	N02+NO3 (mg/L)	Nitrogen, Total Kjeldahl (mg/L)	Phos., filtered (mg/L)	Phos., total (mg/L)	Sb, total (ug/L)	As, total (ug/L)	As, filtered (ug/L)	Be, total (ug/L)
Mixed Industrial (252)													
Number of observations	115	70	40		125	214	197	215	217	33	101	15	33
% of samples above detection	95.7	97.1	90.0		31.2	98.6	98.0	89.3	96.3	3.0	90.1	40.0	0
Median	3033	10000	12500		0.43	0.56	1.0	0.08	0.20	1.0	3.0	2.0	-
Coefficient of variation	2.5	2.6	2.4		0.7	0.7	1.5	2.2	1.5	<0.1	1.0	0.4	-
Institutional (18)													
Number of observations					18	18	18	17	17				
% of samples above detection					88.9	100	100	82.4	94.1				
Median					0.31	0.6	1.4	0.13	0.18				
Coefficient of variation					0.5	0.6	0.5	0.5	1.0				
Freeways (185)													
Number of observations	49	25	16	13	79	25	125	22	128	14	61	72	12
% of samples above detection	100	100	100	100	87.3	96.0	96.8	95.5	99.2	50.0	55.7	50.0	16.7
Median	1700	17000	50000	1900	1.07	0.3	2.0	0.20	0.25	3.0	2.4	1.4	0.3
Coefficient of variation	1.9	1.2	1.5	2.2	1.3	1.2	1.4	2.1	1.8	0.3	0.7	1.1	0.5
Mixed Freeways (20)													
Number of observations	16	12				14	16	13	14		15		
% of samples above detection	81.3	93.8				100	100	100	100		80		
Median	730	19000				0.6	1.6	0.04	0.26		3.0		
Coefficient of variation	2.0	1.1				0.7	0.9	0.8	0.8		0.7		
Open Space (68)													
Number of observations	37	37			35	58	62	61	63		33		13
% of samples above detection	94.6	94.6			22.9	88.5	79	82	88.9		75.8		0
Median	3100	24000			0.30	0.6	0.6	0.08	0.25		5.0		-
Coefficient of variation	2.9	2.6			1.1	0.86	1.0	1.2	3.6		1.2		-
Mixed Open Space (159)													
Number of observations	75	55			65	150	121	125	150	29	65		26
% of samples above detection	97.3	100			20.6	97.3	95.8	94.4	99.3	6.9	84.6		0
Median	3249	21000			0.51	0.7	1.2	0.09	0.27	1.0	4.0		-
Coefficient of variation	2.1	2.3			1.17	0.94	1.3	1.08	1.0	<0.1	0.8		-

Table 9. Summary of Available Stormwater Data Included in NSQD, version 1.0 (continued)

	Cd, total (ug/L)	Cd, filtered (ug/L)	Cr, total (ug/L)	Cr, filtered (ug/L)	Cu, total (ug/L)	Cu, filtered (ug/L)	Pb, total (ug/L)	Pb, filtered (ug/L)	Hg, total (ug/L)	Ni, total (ug/l)	Ni, filtered (ug/L)	Zn, total (ug/L)	Zn, filtererd (ug/L)
Overall Summary (3770)													
Number of observations	2582	388	1599	260	2728	411	2955	446	1026	1435	246	3018	382
% of samples above detection	49.6	30.4	71.6	60.8	87.9	83	79.2	50.2	11.2	60.1	64.2	96.5	96.3
Median	1.0	0.50	7.0	2.1	16	8.0	16.0	3.0	0.20	8.0	4.0	116	52
Coefficient of variation	28.2	1.1	1.5	0.7	2.2	1.6	1.8	2.0	2.7	2.1	1.5	3.4	3.9
Residential (1069)													
Number of observations	707	85	426	33	790	90	779	108	295	410	25	803	89
% of samples above detection	35.5	5.9	56.1	27.3	83.7	63.3	72.7	33.3	9.8	46.3	44.0	96.4	89.9
Median	0.5	0.70	4.5	1.3	12	7.0	12.0	3.0	0.20	5.4	2.0	73	33
Coefficient of variation	1.7	0.5	1.4	0.4	1.8	2.0	1.9	1.8	1.2	1.2	0.5	1.3	0.8
Mixed Residential (615)													
Number of observations	434	30	184	21	448	29	515	30	105	133	25	532	28
% of samples above detection	49.3	40.0	83.2	52.4	84.3	72.4	80.4	46.7	16.2	63.9	72.0	92.5	100
Median	0.8	0.30	7.0	2.0	17	5.5	18.0	3.0	0.20	7.9	5.5	100	48
Coefficient of variation	3.8	0.6	1.6	0.8	1.3	0.9	1.4	0.7	1.0	0.8	0.9	1.0	0.9
Commercial (497)													
Number of observations	353	47	230	27	381	48	371	59	154	227	23	386	49
% of samples above detection	46.7	23.4	60.9	40.7	93.2	79.2	87.1	54.2	7.1	61.2	47.8	99.0	100
Median	0.9	0.30	6.0	2.0	17	7.6	18.0	5.0	0.20	7.0	3.0	150	59
Coefficient of variation	1.6	1.34	1.4	0.6	1.5	0.8	1.6	1.6	0.8	3.8	0.8	1.2	1.4
Mixed Commercial (303)													
Number of observations	169	24	121	20	173	24	226	24	79	91	16	225	23
% of samples above detection	63.9	41.7	90.1	75.0	94.8	91.7	89.8	79.2	8.9	80.2	81.3	98.7	100
Median	0.9	0.40	4.5	2.0	17	9.5	17.0	6.0	0.10	5.0	3.0	132	94
Coefficient of variation	1.1	0.9	1.2	0.7	3.0	0.6	1.5	0.6	1.1	1.3	0.6	1.7	0.7
Industrial (524)													
Number of observations	394	42	253	36	415	42	411	51	208	248	36	432	42
% of samples above detection	53.8	54.8	72.7	55.6	89.9	90.5	78.9	52.9	13.0	63.3	58.3	98.8	95.2
Median	2.0	0.60	14.5	3.0	22	8.0	25.0	5.0	0.20	16.0	5.0	210	112
Coefficient of variation	2.4	1.1	1.2	0.7	2.0	0.7	1.8	1.6	2.7	1.2	1.4	2.3	3.6

Table 9. Summary of Available Stormwater Data Included in NSQD, version 1.0 (continued)

	Cd, total (ug/L)	Cd, filtered (ug/L)	Cr, total (ug/L)	Cr, filtered (ug/L)	Cu, total (ug/L)	Cu, filtered (ug/L)	Pb, total (ug/L)	Pb, filtered (ug/L)	Hg, total (ug/L)	Ni, total (ug/l)	Ni, filtered (ug/L)	Zn, total (ug/L)	Zn, filtererd (ug/L)
Mixed Industrial (252)													
Number of observations	183	25	124	15	183	24	247	25	65	82	15	247	24
% of samples above detection	66.7	92.0	93.5	66.7	88.0	100.0	82.6	92.0	21.5	85.4	100.0	98.4	95.8
Median	1.0	0.60	8.0	2.0	17	6.0	18.5	5.0	0.25	9.0	5.0	160	2100
Coefficient of variation	10.9	0.6	1.7	0.7	0.9	0.6	1.5	1.0	0.6	0.9	0.6	3.3	1.2
Institutional (18)													
Number of observations	18				17		18			15		18	
% of samples above detection	16.7				41.2		77.8			0		100	
Median	0.5				17		5.8			-		305	
Coefficient of variation	0.7				0.6		0.8			-		0.8	
Freeways (185)													
Number of observations	95	114	76	101	97	130	107	126	34	99	95	93	105
% of samples above detection	71.6	26.3	98.7	78.2	99.0	99.2	100	50.0	5.9	89.9	67.4	96.8	99.0
Median	1.0	0.68	8.3	2.3	35	10.9	25	1.8	0.19	9.0	4.0	200	51
Coefficient of variation	0.9	1.0	0.7	0.7	1.0	1.5	1.5	1.7	0.8	0.9	1.4	1.0	1.9
Mixed Freeways (20)													
Number of observations	15		15		17		17					17	
% of samples above detection	80		100		94		82					100	
Median	0.5		6.0		8.5		10.0					90	
Coefficient of variation	0.7		1.1		1.1		0.9					0.9	
Open Space (68)													
Number of observations	55		50		56		62			32		62	
% of samples above detection	65.4		60		84		62.9			18.8		79	
Median	0.5		5.0		5.3		5.0			27.0		39	
Coefficient of variation	1.7		2.1		2.2		2.0			0.9		1.3	
Mixed Open Space (159)													
Number of observations	102		65		100		150	27		50		152	
% of samples above detection	51		87.7		93		76	26		84		97.9	
Median	1.0		5.0		11		10	0.1		7		100	
Coefficient of variation	1.9		1.5		1.5		2.3	1.1		1.2		1.0	

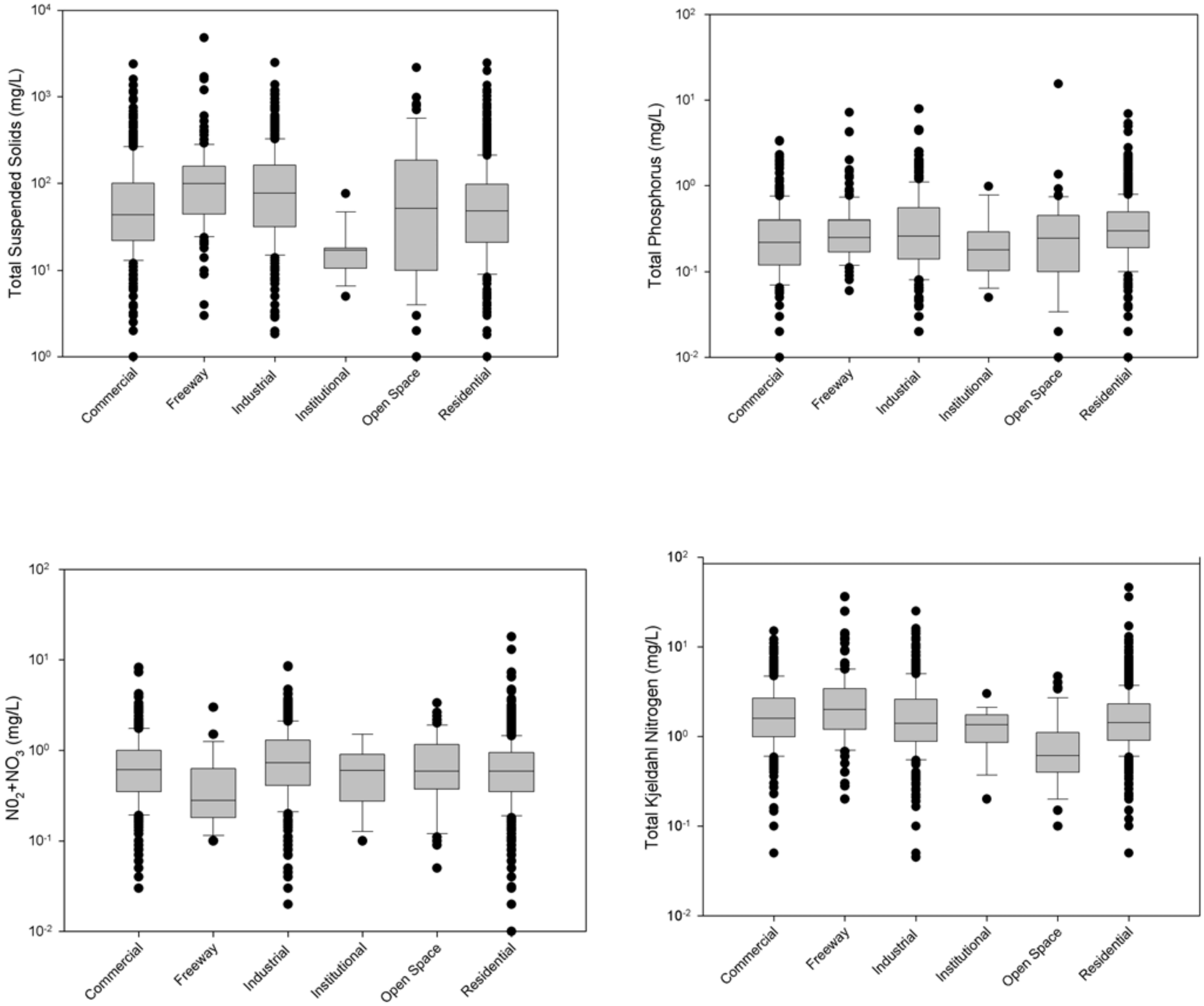


Figure 51. Example stormwater data sorted by land use (no mixed land use data included in plots).

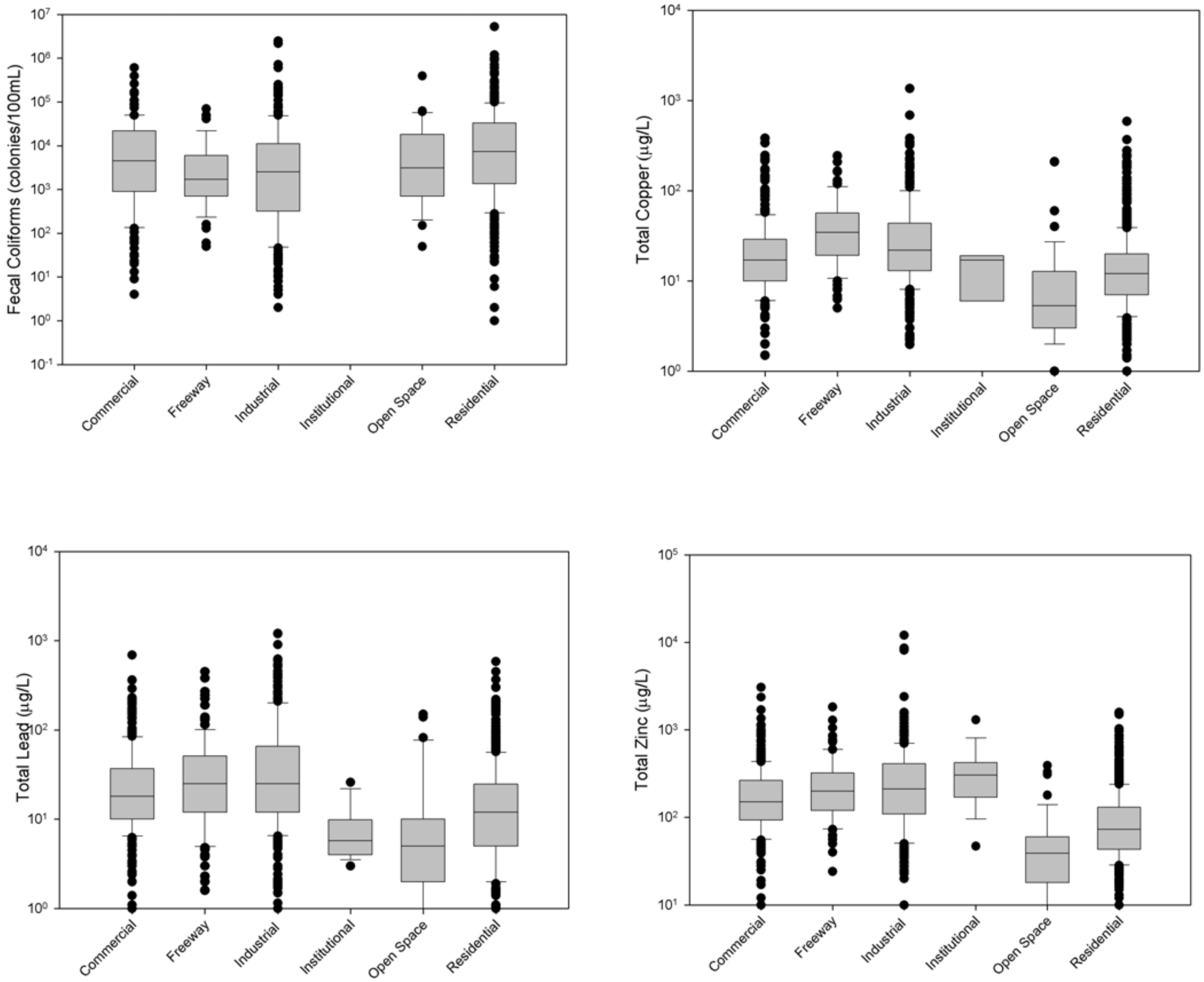


Figure 52. Example stormwater data sorted by land use (no mixed land use data included in plots).

Burton and Pitt (2001) present extensive examples and procedures showing the importance of a balanced monitoring program. Stormwater quality monitoring is a crucial component of local programs. Specific objectives for monitoring programs can include:

• Receiving water assessments to understand local problems. Receiving water monitoring is needed to identify local problems, especially when identifying beneficial use impairments. Assimilative capacity calculations (TMDLs) require knowledge of local source discharges. Obviously, the NSQD data and information should be used for preliminary designs and cost estimates, but it is important to invest a small amount of resources to accurately determine local discharge conditions before expensive controls are designed.

• Source area monitoring to identify critical sources. In many cases, source area controls may be more cost-effective that regional controls. The identification of critical source areas is therefore needed as part of a comprehensive stormwater management program. Monitoring within a critical drainage area should be conducted to identify the sources of pollutants, while simultaneous outfall monitoring is needed to verify these source area measurements.

• Treatability tests to verify performance of stormwater controls for local conditions. In areas where stormwater controls are being installed, local measurements of performance are a good investment. Before and after monitoring, or parallel monitoring, is usually needed to measure the performance of many types of stormwater controls. The ASCE National Stormwater BMP database (http://www.bmpdatabase.org/) is a good place to start in predicting the performance of controls, but site-specific validations in an area where the controls have not been previously used should be conducted.

• Assessment monitoring to verify success of stormwater management approach. Stormwater quality monitoring is a critical component of an assessment monitoring effort. Receiving water monitoring needs to focus on beneficial use impairments, and associated chemical, physical, and biological monitoring. In many cases, source area or outfall controls are being used as part of comprehensive management programs. Therefore, outfall monitoring may also be needed.

Dissolved and Particulate Forms of Pollutants
Table 10 summarizes the filterable fraction of heavy metals found in stormwater runoff sheet flows from many urban areas (Pitt, *et al*. 1995). Constituents that are mostly in filterable forms have a greater potential of affecting groundwater and are more difficult to control using conventional stormwater control practices that mostly rely on sedimentation and filtration principles. In most cases, most of the metals are associated with the non-filterable (suspended solids) fraction of the stormwaters. Likely exceptions include zinc that may be mostly found in the filtered sample portions. However, dry-weather flows in storm drainage tend to have much more of the toxicants associated with filtered sample fractions.

Coulter Counter used to measure particle size distributions

Graphite furnace atomic absorption spectrophotometer for low level heavy metal analyses (detection limits of 1 to 10 µg/L commonly needed, especially for filterable forms)

Polarized light microscope to identify sources and characteristics of stormwater particulates

Microtox unit to conduct relative toxicity tests on sample portions

Gas chromatograph with mass spectrograph for the analyses of stormwater organic compounds (mostly PAHs and phthalate esters)

Figure 53. Laboratory equipment used to characterize stormwater pollutants.

Table 10. Reported Filterable Fractions of Stormwater Toxicants from Source Areas

Constituent	Filterable Fraction (%)
Cadmium	20 to 50
Chromium	<10
Copper	<20
Iron	small amount
Lead	<20
Nickel	small amount
Zinc	>50

source: Pitt, *et al*. 1995

Pitt, *et al.* (1998) analyzed 550 samples for a broad list of constituents, including the total and filtered observations shown in Table 11. The samples were collected from telecommunication manhole vaults that were mostly affected by stormwater. However, some other contaminating water, and groundwater, sources likely also influenced these samples. These data are very similar to cold and warm season stormwater data collected during other projects. This is the largest data base available that contains both total and filtered metal analyses. These samples were obtained throughout the US and represent all seasons.

Table 11. Average Particulate Fraction of Selected Constituents from 550 Nationwide Samples (mg/L, unless otherwise noted)

Constituents	Total Concentration	Filtered Concentration (after a 0.45 μm membrane filter)	Percent Associated with Filterable Fraction	Percent Associated with Particulates
Turbidity (NTU)	13	1.2	8%	91%
COD	25	22	86%	14%
Color (HACH)	34	20	59%	41%
Copper (μg/L)	29	9.5	33%	67%
Lead (μg/L)	14	3	21%	79%
Zinc (μg/L)	230	160	70%	30%

Pitt, *et al.* (1998)

Sansalone (1996) investigated the forms of heavy metals in stormwater and snowmelt. It was found that Zn, Cd, and Cu were mainly dissolved in stormwater, while only Cd was mainly dissolved in snowmelt. Pb was associated with the finer particulate fractions in both stormwater and snowmelt. The dissolved fraction of the metals should be immobilized by sorption, while the particulate bound metals should be immobilized by filtration in a partial exfiltration trench. Another study by Sansalone and Buchberger (1997) analyzed stormwater runoff at five sites on a heavily traveled roadway in Cincinnati, Ohio. They found that the event-mean concentrations (EMC) of Zn, Cd, and Cu exceeded surface-water-quality-discharge standards. Further, it was noted that Zn, Cd, and Cu are mainly in the dissolved form while other metals, i.e., Pb, Fe, and Al are mainly bound to particles.

Sedlak, *et al.* (1997) used analytical techniques to determine the speciation of Cu and Ni in point and non-point source (NPS) discharges. They found that the existence of a strong metal-complexing ligand in wastewater effluent, and to a lesser degree, surface runoff must be accounted for when evaluating metal treatability.

Grout, *et al*. (1999) studied the colloidal phases in urban stormwater runoff entering Brays Bayou (Houston, Texas). Colloids in the filtrate (after 0.45 μm filtering) and further separation by ultracentrifuging, accounted for 79% of the Al, 85% of the Fe, 52% of the Cr, 43% of the Mn, and 29% of the Zn present in the filtrates. Changes in the colloidal composition were caused by changes in colloidal morphologies, varying from organic aggregates to diffuse gel-like structures rich in Si, Al, and Fe. Colloids were mostly composed of silica during periods of dry weather flow and at the maximum of the stormwater flow, while carbon dominated the colloidal fraction at the beginning and declining stages of the storm events. Garnaud, *et al.* (1999) examined the geochemical speciation of particulate metals using sequential extraction procedures for different runoff sources in Paris, France. They found that most metals were bound to acid soluble particulates in the runoff but that Cu was almost entirely bound to oxidizable and residual fractions.

Barry, *et al*. (1999) identified salinity effects on the partitioning of heavy metals in the stormwater canals entering Port Jackson (Sydney), Australia. Cu, Pb, and Zn were found increasingly in dissolved phases as the salinity increased in the lower sections of the canals. During high flows, most of the metals seemed to be rapidly exported from the estuary as a discrete surface layer, while low flows contributed most of the metals to the estuary.

Water quality and particle-size distribution were characterized from urban-stormwater runoff from two storms that indicated potential relationships between Zn/organic carbon and Fe/macrocolloid (0.45 μm to 20 μm) pairs. Results also indicated that concentrations of particle ion number, organic carbon, suspended solids, Fe, and Zn increased during storms but showed no evidence of a “first flush” (Characklis and Wiesner 1997).

Shafer, *et al*. (1999) investigated the partitioning of trace metal levels (Al, Cd, Cu, Pb, and Zn) in Wisconsin rivers and found that the concentrations in the rivers were comparable to recent data collected in the Great Lakes and other river systems where ‘modern’ clean methods were used for sampling and analysis. They also found that the variation in the partitioning coefficients of each metal between sampling locations could be explained by the amount of anthropogenic disturbance in the watershed and by the concentration of dissolved organic carbon (DOC) in the water.

Parker, *et al*. (2000) analyzed the particulates found in urban stormwater runoff in the Phoenix, Arizona, metropolitan area. They found that the inorganic content of the particulates was similar to that in soils that were not impacted by urban runoff. The metals concentrations (Cd, Cu, Pb, and Zn) were higher, but below levels that would recommend remediation. Arsenic concentrations were above recommended levels; however, this contribution likely was geologic not anthropogenic. Sediment toxicity was seen, but could not be explained based on their chemical results.

Krein and Schorer (2000) investigated heavy metals and PAHs in road runoff and found that, as expected, an inverse relationship existed between particle size and particle-bound heavy metals concentration existed. Sutherland, *et al*. (2000) investigated the potential for road-deposited sediments in Oahu, Hawaii, to bind contaminants, and thus transporting these bound contaminants to the receiving water as part of the runoff. In the sediment fractions less than 2 mm in diameter, the origins of the Al, Co, Fe, Mn and Ni were determined to be geologic. Three of the metals concentrations (Cu, Pb and Zn) were found to be enhanced by anthropogenic activities. Sequential extraction of the sediment determined the associations of the metals with the following fractions: acid extractable, reducible, oxidizable, and residual.

The fate and transport of metallic pollutants through a watershed were related to the characteristics of the solid particles to which they are bound (Magnuson, *et al*. 2001). Because the particles most often associated with metal pollution have nominal diameters of < 50 µm, split-flow thin-cell (SPLITT) fractionation was investigated as a means to study the metal loading as a function of particle settling rate. Sansalone, *et al*. (2001) showed that urban stormwater levels of Zn, Cu, Cd, Pb, Cr, and Ni can be significantly above ambient background levels, and for many urban and transportation land uses, often exceed surface water discharge criteria for both dissolved and particulate-bound fractions. They advocated a multiple-unit-operation approach to stormwater treatment.

Glenn, *et al.* (2001a and 2001b) described their research at highway test sites in Cincinnati, OH, investigating the effects of traffic activities and winter maintenance on the behavior of particulates in the runoff. They found that urban snow has a much greater capacity to accumulate traffic-related pollutants, as compared to stormwater, due to longer residence times before melting, and the snow's porous matrix. Parameters such as residence time, solids loadings, alkalinity, hardness and pH influence the heavy metal partitioning in the snow. They found that Pb, Cu, Cd, Zn, Al, Mg, and Fe were mostly particulate bound, while Na and Ca were mostly dissolved. Partition coefficients for most heavy metals in snowmelt water ranged from 103 to 106 L/kg.

Significant amounts of non-point source runoff were shown to enter the Santa Monica Bay (CA) from the Ballona Creek Watershed during wet weather flow during monitoring by Buffleben, *et al.* (2001). The watershed is developed mostly with residential, commercial and light industrial land uses. They found that the suspended solids phase primarily transported the mass for five of the six metals studied: Cd, Cr, Cu, Pb, and Ni. Arsenic was found primarily in the aqueous phase.

Mosley and Peake (2001) characterized urban runoff from a catchment in Dunedin, New Zealand, during base flows and storm flows from five rainfall events. Fe and Pb were found to be predominantly particle-associated (>0.4 µm) with concentrations increasing significantly at the beginning of storm run-off. In contrast, the majority of Cu and Zn was found in the <0.4 µm fraction prior to rain but a significant proportion was present in the > 0.4 µm fraction during the initial period of storm flows. The results indicate that Cu and Zn may be more bioavailable, and more difficult to remove by stormwater treatment, than Pb. The pH level and the concentration of major ions (Ca^{+2}, Na^{+}, Mg^{+2}, K^{+}), dissolved PO_4-P, and NO_3 generally decreased during storm flows due to rainwater dilution. Concentrations of total N and P often increased during the initial period of storm runoff, likely because of washoff of particulate plant material.

Fan, *et al.* (2001b) reviewed the transport of toxic pollutants through multiple media and drainage systems in the urban watershed during wet-weather periods. Field studies have identified that a major portion of hazardous waste priority pollutants including benzene, polynuclear aromatic hydrocarbons (PAHs), polychlorinated biphenyls (PCBs), pesticides, and heavy metals (e.g., arsenic, cadmium, chromium, copper, lead, mercury, and zinc) contained in urban stormwater runoff are in particulate form, or sorbed onto particles.

Table 12 summarizes inlet and outlet concentrations for total and dissolved metals (Cd, Cr, Cu, Pb, Ni, and Zn), TSS, hardness and total oil and grease for a retrofitted pond in Sunnyvale, CA. The metals removal data indicated that the removal of total Cr, Cu, Pb, Ni and Zn were well correlated with TSS removal. Tables 13 through 15 also summarize the particulate and filterable fraction of stormwater heavy metals from a number of studies. In almost all cases, the heavy metals are mostly associated with particulates, except for Zn that is mostly associated with the filterable fraction. Interesting exceptions are noted, however. Zinc stormwater concentrations from Birmingham industrial storage areas were found to be almost completely associated with the particulate fraction. These samples were apparently not affected by runoff from areas having galvanized metals, but were affected by heavy truck traffic, where the particulate forms of Zn would be mostly from tire wear.

Table 12. Average Inlet and Outlet Observed Concentrations and Pollutant Removals (Sunnyvale, CA)

	Cadmium			Chromium			Copper			Lead			Nickel			Zinc			TSS (mg/L)
	nf	F	% filt	nf	f	% filt	nf	f	% filt	nf	f	% filt	nf	f	% filt	nf	f	% filt	
SE 17																			
Inlet	0.4	<0.2	<50%	3.6	1.8	50%	8.7	5.4	62	6.4	2.2	34	1.7	<2	<100	46	28	61	12
Outlet	0.2	<0.2	<100	2.7	1.1	41	6.8	4.7	69	3.4	1	29	1.7	<2	<100	26	19	73	73
Reduction	--	--		25%	--		22%	--		47%	--		0%	--		43%	--		39%
SE 20																			
Inlet	6.6	1.3	20	12	1	8.3	24	3	13	45	1	2.2	16	1	6.3	180	19	11	90
Outlet	4.8	2.5	52	6	1	17	9	3	33	10	1	10	4	1	25	73	22	30	24
Reduction	--	--		50%	--		63%	--		78%	--		75%	--		59%	--		73%
SE 21b																			
Inlet	1.1	0.2	18	18	1	5.6	24	2	8.3	53	<1	<2	25	<1	<4	180	5	2.8	140
Outlet	1.5	<0.2	<13	14	1	7.1	16	2	13	35	<1	<3	19	<1	<5.3	120	7	5.8	93
Reduction	--	--		22%	--		33%	--		34%	--		24%	--		33%	--		34%
SE 23																			
Inlet	1	0.2	20	11	<1	<9.1	27	5	19	30	1	3.3	13	3.9	30	190	41	22	74
Outlet	0.6	<0.2	<33	8.3	1.4	17	12	4.7	39	12	<1	<8	5.8	2.2	38	82	45	55	31
Reduction	--	--		25%	--		56%	--		60%	--		55%	--		57%	--		58%
SE 24																			
Inlet	1.6	<0.2	<13	21	1.1	5.2	40	2.1	5.3	76	<1	<2	42	9.6	23	270	22	8.1	180
Outlet	1.3	0.2	15	15	8.6	57	24	5	21	40	1.4	3.5	29	15	52	160	31	19	96
Reduction	--	--		29%	--		40%	--		47%	--		31%	--		41%	--		47%
SE 27																			
Inlet	1	0.5	50	6.3	1.4	22	14	5.4	39	13	<1	<8	83	63	76	70	35	50	30
Outlet	0.6	0.4	67	4.9	1.7	35	8.9	4.5	51	6.6	<1	<15	25	20	80	47	26	55	15
Reduction	--	--		22%	--		36%	--		49%	--		70%	--		33%	--		50%
Average Reduction	--	--		29%	--		42%	--		53%	--		51%	--		44%	--		50%
Median % filterable at inlet			19%			8%			16%			3%			27%			17%	
Median % associated with particulates at inlet			81%			92%			84%			97%			73%			83%	

nf: non-filtered (total)
f: filtered ("dissolved")
removals are only given if most observations were >PQL

Table 13. Filterable Fraction of Heavy Metals Observed at the Inlet to the Monroe St. Wet Detention Pond, Madison, WI (average and standard deviation)

	Copper	Lead	Zinc
Number of observations	60 to 64	59 to 64	57 to 64
Average total concentration	50 (14)	85 (52)	152 (136)
Average filtered concentration	6.4 (3.3)	3.5 (1.7)	51 (34)
Average percentage filterable	13%	4.1%	34%
Average percentage associated with particulates	87%	96%	66%

Data from: House, *et al*. 1993.

Table 14. Milwaukee and Long Island NURP Source Area Heavy Metal Associations (based on mean concentrations observed)

	Residential roof runoff		Commercial parking runoff	
	% filt	% part	% filt	% part
Arsenic[b]			25	75
Cadmium[b]			18	82
Chromium[b]			24	76
Lead[a]	8	92	3	97
Lead[b]			16	84

[a] Bannerman, *et al*. 1983 (Milwaukee) (NURP)
[b] STORET Site #59629A6-2954843 (Huntington-Long Island, NY) (NURP)

Table 15. Birmingham, AL, Source Area Heavy Metal Particulate Associations (based on mean concentrations observed)

	Roof areas (12 samples)		Parking areas (16 samples)		Storage areas (8 samples)		Street runoff (6 samples)		Loading docks (3 samples)		Vehicle service areas (5 samples)		Landscaped areas (6 samples)		Urban creeks (19 samples)		Detention ponds (12 samples)	
	% filt	% part	% filt	% part	% filt	% part	% filt	% part	% filt	% part	% filt	% part	% filt	% part	% filt	% part	% filt	% part
Aluminum	3.4	97	13	87	7.8	92	29	71	na[a]	na	25	75	52	48	31	69	47	53
Cadmium	12	88	9.5	90	36	64	1	99	29	71	3.2	97	na	na	2.4	98	25	75
Copper	2.6	97	9.5	90	86	14	1.4	99	40	60	6.2	94	5.1	95	2.8	97	47	53
Chromium	2.1	98	4.1	96	15	85	18	82	na	na	na	na	2.5	97	2.5	97	5.4	95
Lead	2.7	97	4.6	95	2.5	97	4.6	95	na	na	3.8	96	na	na	7.0	93	5.3	95
Nickel	na	na	11	89	na	na	na	na	na	na	na	na	na	na	7.9	92	13	87
Zinc	88	12	78	22	1.3	99	53	47	60	40	70	30	61	39	100	0	100	0

[a] na: not available, too few detectable observations for calculation

Pitt, *et al.* 1999

Johnson, *et al.* (2003) recently conducted tests to examine the treatability of stormwater pollutants. The objectives of these tests were to determine the associations of heavy metals (along with some major constituents and nutrients) with different-sized particulates in stormwater. The binding strengths of these pollutants to the particulates were also examined by using a sequential extraction procedure using different acids and bases under a wide range of pH and Eh conditions. Experiments examining the characteristics of the filterable (<0.45 μm) portion of the heavy metals were also conducted.

In order to obtain the most meaningful data on either bioavailability or toxicity, it is important that chemical speciation techniques be applied (Florence and Bately 1980). Chemical speciation may be defined as the determination of the individual concentrations of the various chemical forms of an element when together make up the total concentration of that element in a sample. Speciation of metals is dependent upon chemical and physical parameters such as pH, temperature, and the presence of ligands and particulates. Depending upon the chemical form of the metal, a water with a high total metal concentration may be less toxic than another water with a lower total metal concentration (Florence and Bately 1980).

The threat from metals to humans and aquatic life is due to their toxicity, persistence and bioaccumulation. It is important to determine the speciation of a metal because the toxicity of many metals is related to their speciation and valence state. Most metals are essential nutrients for living cells, but only in small quantities. When metals are present in excess, they can become cumulative toxins. Other metals, such as lead and mercury, have no nutritional value and are always considered dangerous, even at relatively small concentrations. Associations with fine particulates also affects heavy metal toxicity to many forms of aquatic life.

The development of analytical techniques which can reliably measure the concentration of the various chemical forms of a trace metal in a water sample is a challenging problem. The analytical schemes developed by Florence and Batley (1980), Florence (1977) and Bott (1995) enable associations of metals to different categories to be measured. Their schemes do not provide a detailed species distribution, but rather an estimation of the fraction of the metal present in toxic forms. Florence and Bately (1980) used Anodic Stripping Voltammetry (ASV) with a chelating resin separation and UV irradiation to examine the dissolved (<0.45 micron) metal fractions, as shown in Figure 54. A chelating resin (Chelex-100) was used to separate ionic metals from metals strongly bound in metal-ligand complexes, or strongly adsorbed to colloidal particles. The UV-irradiation was applied to the water at a pH of 4.8 to cause dissociation of the metals from organic complexes, or colloids. The use of a chelating resin such as Chelex-100, in combination with ASV, has been utilized by many researchers, and various forms of the resin have been characterized (Figura and McDuffle 1977, 1979, 1980; Yousef, *et al.* 1985). The methods used during this project were modified, based on the Florence and Bately scheme, and are shown on Figure 55.

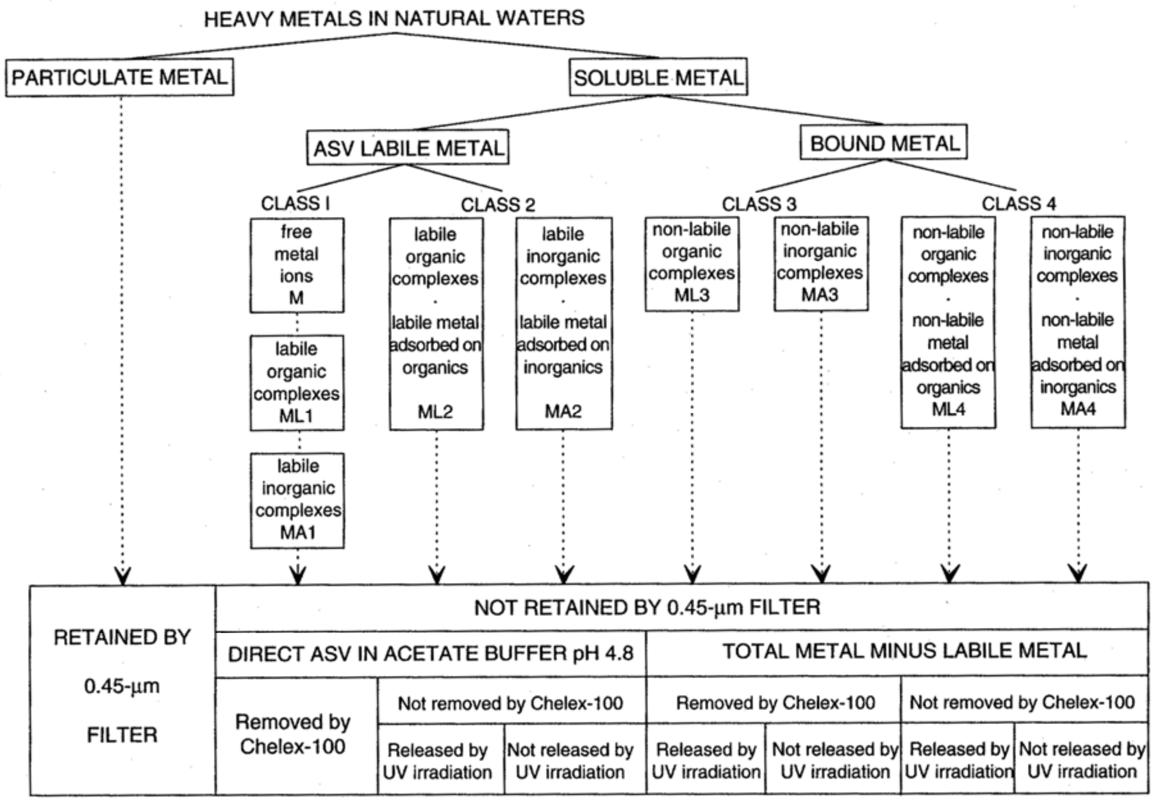


Figure 54. Diagram of sequential extraction (Florence and Batley 1980), as presented by Bott (1995).

Each stormwater sample was analyzed in four separate experiments, according to the flowchart shown in Figure 55:

- The initial characterization of the stormwater sample.
- The sample is sieved/filtered and a portion of each filtrate is preserved for further analysis.
- The metal-binding strength of the particles in the sample is investigated by exposing the water to various pH levels using a weak acid, strong acid, and a strong base.
- Ionic and colloidal metal fractions in the water samples are separated by exposure to both ultraviolet irradiation and a chelating resin.

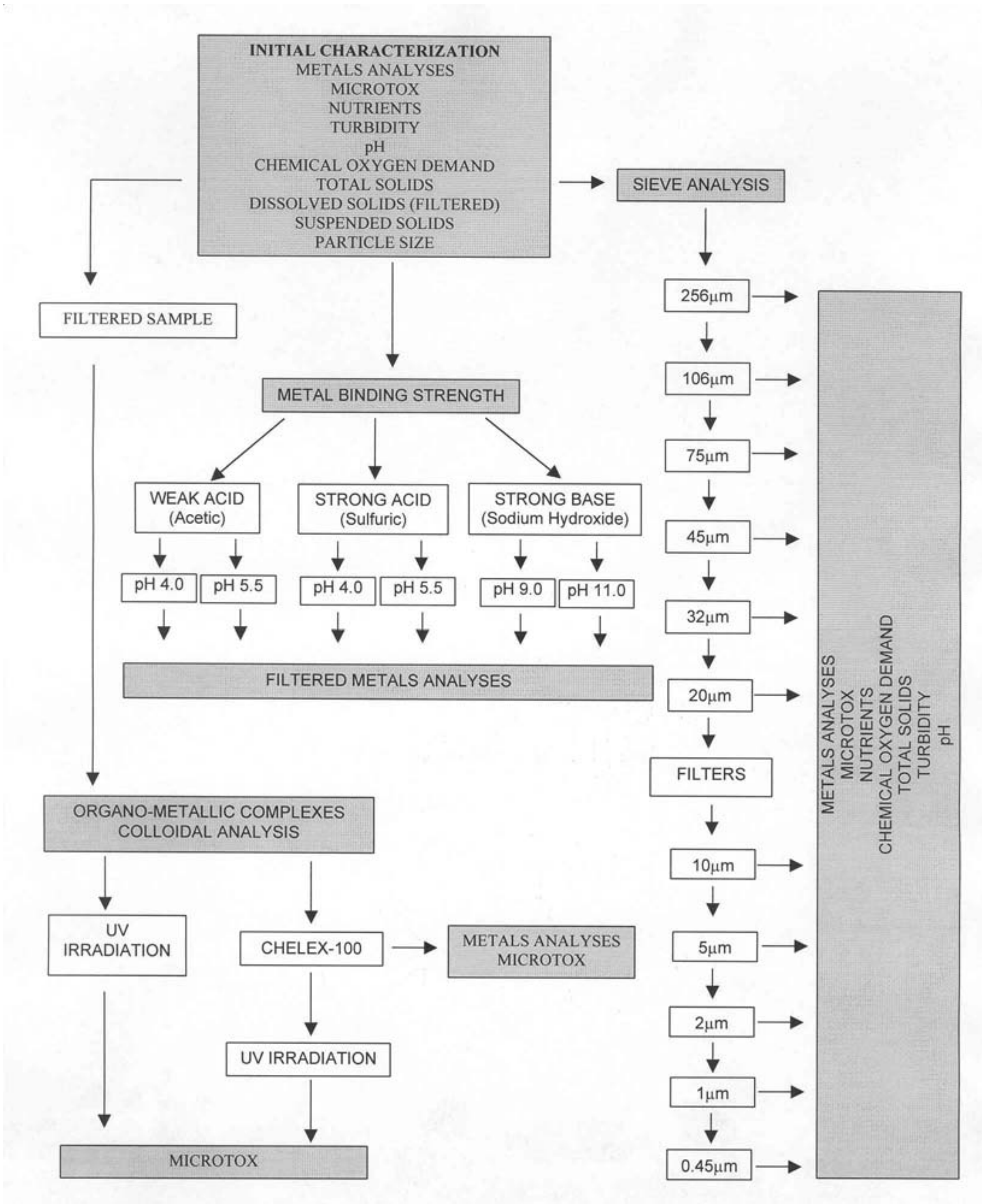


Figure 55. Flow chart of original analysis scheme.

Associations of Stormwater Pollutants with Different Sized Particulates

All samples were divided into three portions for particle size analyses: >106 µm, <0.45 µm, and the intermediate range from 0.45 to 106 µm. The Coulter Counter (Multi-Sizer 3) results for the intermediate size range were compiled with results from the other two size ranges to obtain the complete particle size distribution.

As expected, the general trend is a lower concentration of total solids as the particle size decreases. Constituents that did not change significantly with filtration included nitrate and sodium, as expected. Other analytes (COD and cadmium) also had little change, except for a single sample in each case. Most of the analytes had large decreases with filtration, especially for the more contaminated samples (turbidity, phosphorus, phosphate, magnesium, chromium, copper, iron, lead, and zinc). Total solids and COD had much smaller changes with filtration, with substantial fractions associated with the filterable (<0.45 µm) fraction.

Table 16 summarizes the calculated “potency” factors for several size ranges for the heavy metals during these tests. These are the masses of these constituents associated with the particulates, expressed as mg constituent per kg of suspended solids. As expected, these values increase with decreasing particle size, except for iron that is seen to generally remain constant. The iron measurements show that these stormwater particulates were about 1.5 to 3% iron, by mass. There were also large zinc components for the smallest sizes. Typical soil values for iron are about 50,000 mg Fe per kg solids, somewhat higher than these observations, while observed copper, lead, and zinc are all higher than typical soil values of about 100 mg/kg (Lindsay 1979).

Typical street dirt (mostly for the smallest <43 µm size particles) contents of these heavy metals are about 100 to 500 mg/kg for copper, about 500 to 7,500 mg/kg for lead, and about 250 to 1,200 mg/kg for zinc (Bannerman, *et al*. 1983, Pitt 1979, Pitt 1985, Pitt and McLean 1986, Pitt and Sutherland 1982, and Terstrip, *et al*. 1982). The lead values found were much less than the earlier street dirt values, due to the decreased use of lead in gasoline. However, the copper and zinc values observed are much greater than the street dirt values.

In Toronto, Pitt and McLean (1986) found copper values of about 30 to 200 mg/kg for most residential and commercial source area particulates (<125 µm), but from 100 to over 1,000 mg/kg at industrial areas. Similar values for residential and commercial area lead soil concentrations were generally from 200 to 1,000 mg/kg and slightly higher in industrial areas. Zinc ranged from 50 to 2,000 mg/kg in all land use areas. Again, the observed copper and zinc values were much greater than these previous measurements, while the lead values were much less.

Table 16. Summary Table Showing Heavy Metal Associations for Different Particle Sizes

	Copper		Iron		Lead		Zinc	
particle size (µm)	mg Cu/kg SS	COV	mg Fe/kg SS	COV	mg Pb/kg SS	COV	mg Zn/kg SS	COV
>250	50	na	28,604	1.50	117	0.58	266	0.88
106 to 250	2,137	1.45	21,730	0.85	375	1.03	3,486	0.79
45 to 106	1,312	1.16	14,615	0.72	226	0.85	2,076	0.88
10 to 45	735	0.97	26,221	0.54	229	0.50	1,559	0.74
2 to 10	4,668	1.60	18,508	1.16	868	0.78	13,641	1.88
0.45 to 2	2,894	1.21	29,267	1.31	199	1.40	13,540	1.56

Figure 56. Sieving samples and collecting sample portion for chemical analyses.

Randall, *et al*. (1982), recognized the strong correlation between pollutant removal effectiveness in wet detention ponds and pollutant associations with suspended solids. High lead removals were related to lead's affinity for suspended solids, while much smaller removals of BOD_5 and phosphorus were usually obtained because of their significant soluble fractions.

Wet detention ponds also are biological and chemical reactors. Changes in many pollutants can take place in the water column or in the sediments of ponds. Dally, *et al.* (1983) monitored heavy metal forms in runoff entering and leaving a wet detention pond serving a bus maintenance area. They found that metals entering the monitored pond were generally in particulate (nonfilterable) forms and underwent transformations into filterable (smaller than 0.2 µm in size) forms. The observed total metal removals by the pond were generally favorable, but the filterable metal outflows were much greater than the filterable metal inflows. This effect was most pronounced for cadmium and lead. Very little changes in zinc were found, probably because most of the zinc entering the pond was already in filterable forms. These metal transformations may be more pronounced in wet detention ponds that in natural waters because of potentially more favorable (for metal dissolution) pH and ORP conditions in wet pond sediments. Other studies have found similar transformations in the forms and availability of nutrients in wet detention ponds, usually depending on the extent of algal growth and algal removal operations.

The previously presented information can be used to estimate the design configuration of detention ponds based on many site conditions and objectives, for suspended solids. Table 17 can be used to estimate the approximate controls for other pollutants. These ratios of pollutant removals to suspended solids removals are based on many field observations (mostly from the NURP studies, EPA 1983) of detention pond performance and can vary significantly. Three general groupings were identified: total lead and total copper were most efficiently removed, while organic nitrogen was the least efficiently removed. Many of the nutrients showed “negative” removals during monitoring, possibly because of biological cycling of the nutrients in the ponds. Wet detention ponds should not be expected to provide significant removals of any pollutants in “soluble” forms (associated with very small particles, colloids, or truly dissolved).

Table 17. Approximate Control of Stormwater Pollutants in Wet Detention Ponds

Constituent Group	Percentage Control as a Fraction of Suspended Solids Control
Lead and copper	0.75 to 1.00+
COD, BOD_5, soluble and total phosphorus, nitrates, and zinc	0.6
Organic nitrogen	0.4

Example: If 85% control of suspended solids, then:
Lead and copper: 0.75 to 1.0+ of 85% = 64 to 85+%
COD, etc.: 0.6 of 85% = 51%
Organic nitrogen: 0.4 of 85% = 34%

The relationship between solids retention and pollution retention is important for wet detention ponds. Becker, *et al.* (1995) used settling column tests to measure the settling characteristics of different pollutants in sanitary sewage. They found that the majority of the particulate fractions of COD, copper, TKN, and total phosphorus was associated with particles having settling velocities of 0.04 to 0.9 cm/sec. Figure 57 is an example plot showing the relationship of particulate COD and different settling velocity fractions.

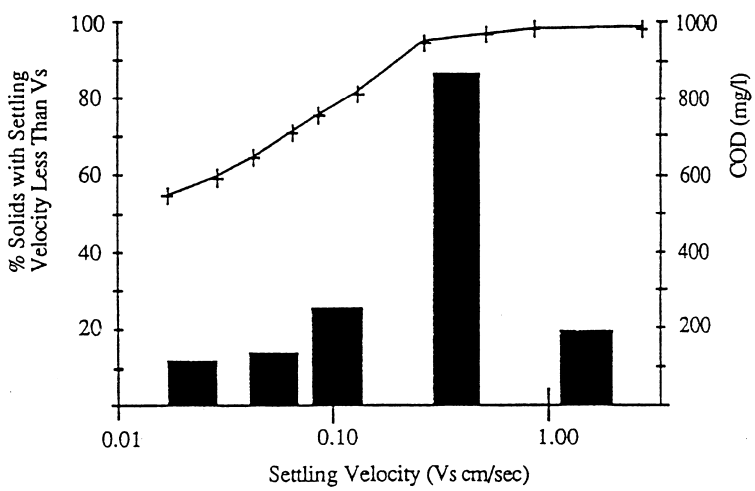


Figure 57. COD and particulate settling velocity (Butler, *et al*. 1993).

Particulate pollutant strength (or potency factor) is the ratio of a particulate pollutant concentration to the suspended solid concentration, expressed in mg/kg. The strengths of stormwater particulates were calculated for each pollutant with a particulate form and plotted on a probability versus strength chart for the Madison, WI, data from House, *et al*. (1993) (Figure 58 for Zn). All pollutants had higher outlet than inlet strength values due to preferential removals of large particles in the detention pond, leaving relatively more small particles in the discharge water. The small particles in stormwater have higher pollutant strengths than the large particles.

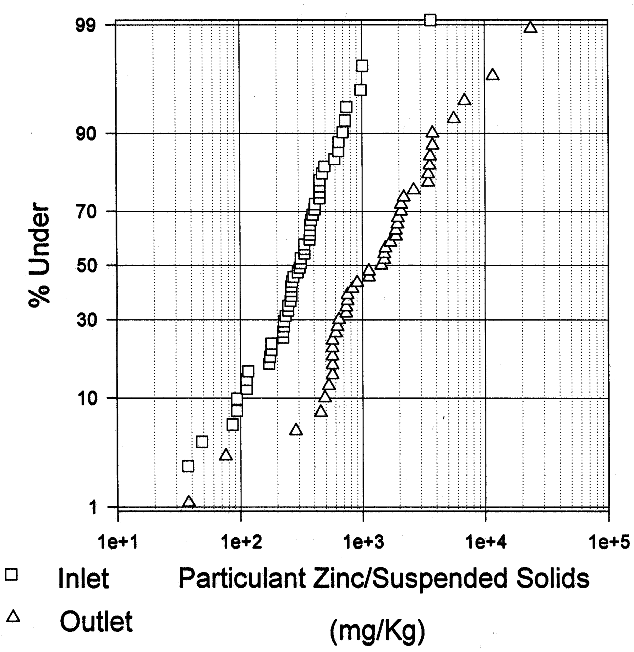


Figure 58. Particulate pollutant strengths for zinc (data from House, *et al*. 1993).

Vignoles and Herremans (1995) also examined the heavy metal associations with different particles sizes in stormwater samples from Toulouse, France. They found that the vast majority of the heavy metal loadings in stormwater were associated with particles less than 10 μm in size, as shown on Table 18. They concluded that stormwater control practices must be able to capture the very small particles.

Table 18. Percentages of Suspended Solids and Distribution of Heavy Metal Loadings Associated with Various Stormwater Particulate Sizes (Toulouse, France) (Percentage associated with size class, concentration in mg/kg).

	>100 μm	50 to 100 μm	40 to 50 μm	32 to 40 μm	20 to 32 μm	10 to 20 μm	<10 μm
Suspended solids	15%	11%	6%	9%	10%	14%	35%
Cadmium	18 (13)	11 (11)	6 (11)	5 (6)	5 (5)	9 (6)	46 (14)
Cobalt	9 (18)	5 (16)	4 (25)	6 (20)	6 (18)	10 (22)	60 (53)
Chromium	5 (21)	4 (25)	2 (26)	6 (50)	3 (23)	9 (39)	71 (134)
Copper	7 (42)	8 (62)	3 (57)	4 (46)	4 (42)	11 (81)	63 (171)
Manganese	8 (86)	4 (59)	3 (70)	3 (53)	4 (54)	7 (85)	71 (320)
Nickel	8 (31)	5 (27)	4 (31)	5 (31)	5 (27)	10 (39)	63 (99)
Lead	4 (104)	4 (129)	2 (181)	4 (163)	5 (158)	8 (247)	73 (822)
Zinc	5 (272)	6 (419)	3 (469)	5 (398)	5 (331)	16 (801)	60 (1,232)

Source: Vignoles and Herremans (1995)

Associations with Particle Sizes

Most stormwater treatment efforts involve the physical removal of particulates. In order to better design sedimentation stormwater treatment devices, it is important to understand which pollutants are associated with different sized particulates and how they may be controlled during the removal of the particulates. Table 19 lists the remaining concentrations for several monitored pollutants after controlling for particle sizes ranging from 20 to 0.45 μm. The corresponding percentage reductions are shown in Table 20. Some pollutants can be reduced by a reduction in particulates, such as suspended solids, total phosphorus and most heavy metals. Other pollutants, such as nitrates, are reduced much less, even after filtration down to 0.45 μm.

Table 19. Average Remaining Concentrations after Controlling for Different Sized Particles

	Residual Concentration after Removing all Particulates Greater than Size Shown (mg/L, unless otherwise noted)				
	Unfiltered	**20 μm**	**5 μm**	**1 μm**	**0.45 μm**
Total Solids	178	106	102	86	84
Suspended Solids	94	22	18	2	0
Turbidity (NTU)	53	30	24	4	2
Total-P	0.38	0.12	0.07	0.04	0.03
Total-N	2.13	1.50	1.25	1.38	1.63
Nitrate	0.6	0.60	0.60	0.53	0.50
Phosphate	0.8	0.23	0.18	0.15	0.10
COD	99	51	48	47	52
Ammonia	0.26	0.17	0.14	0.12	0.11
pH	7.35	7.45	7.66	7.39	7.53
Cadmium (μg/L)	4.9	3.9	3.8	3.8	3.8
Chromium (μg/L)	15.4	4.7	3.8	2.8	2.5
Copper (μg/L)	24.6	18.3	16.2	17.8	15.6
Iron (μg/L)	2830	1350	1050	140	80
Lead (μg/L)	15.7	9.2	6.0	3.7	2.8
Zinc (μg/L)	179	64	53	53	51

Table 20. Average Percentage Reduction in Pollutants after Controlling for Different Particle Sizes

	Percent Pollutant Reduction after Removing all Particulates Greater than Size Shown			
	20 μm	**5 μm**	**1 μm**	**0.45 μm**
Total Solids	40%	43%	52%	53%
Suspended Solids	76	81	98	100
Turbidity	43	55	92	96
Total-P	68	82	89	92
Total-N	30	41	35	23
Nitrate	0	0	12	17
Phosphate	71	78	81	88
COD	48	52	52	47
Ammonia	35	46	54	58
Cadmium	20	22	22	22
Chromium	69	81	82	84
Copper	26	34	34	37
Iron	52	63	95	97
Lead	41	62	76	82
Zinc	64	70	70	72

Most well designed wet detention ponds remove most particulates down to about 1 to 5 μm, depending on the rain conditions and drainage area. Smaller ponds may only be able to remove the particulates down to about 20 μm, while no pond can remove the filterable fraction by physical removal processes alone. The following list shows which constituents can be controlled to different levels and the associated particulate removal targets:

>90%

- suspended solids (0.45 to 1 μm)
- turbidity (0.45 to 1 μm)
- total phosphorus (0.45 μm)
- iron (0.45 to 1 μm)

80 to 89%

- suspended solids (5 µm)
- turbidity (2 µm?)
- total phosphorus (1 to 5 µm)
- phosphate (0.45 to 1 µm)
- chromium (0.45 to 5 µm)
- lead (0.45 µm)

70 to 79%

- suspended solids (20 µm)
- turbidity (3 µm?)
- phosphate (5 to 20 µm)
- lead (1 µm)
- zinc (0.45 to 5 µm)

all <50%

- total nitrogen
- nitrate
- COD
- cadmium
- copper

Of course, samples for other locations and situations would likely result in different levels of control. However, the predicted level of control associated with particulate control at least to 1 to 5 µm are close to what is expected for a well-designed wet detention pond designed for the control of these particulates, with the exception of copper which would normally be better controlled than indicated with this data. These data enable us to specify the level of treatment of particulates to obtain specific pollutant treatment goals.

Colloidal Analysis. Chelex-100 resin was added to portions of the filtered (0.45 µm filter) stormwater samples collected in Birmingham and Tuscaloosa, AL (Johnson, *et al,* 2003). After shaking for one hour, the samples were again filtered (again using a 0.45 µm filter). Ionic forms of the metals will bond to the Chelex, while colloidal bound metals and metals strongly bound to ligands will remain in solution and be filtered. Therefore, the "after" Chelex exposed concentrations reflects these bound metals, while the differences in the "before" and "after" exposure concentrations reflects the portions associated with free ionic forms.

The Chelex-100 tests indicated that almost all of the filterable (through 0.45 µm filters) heavy metals (and major ions) were not associated with colloids and organo-metallic complexes, but occurred as free ions. The main exception was cadmium, with only about 30% of the filterable cadmium in ionic forms, and copper with about half of the filterable copper in ionic forms. The UV exposure tests used to indicate likely organo-metallic complexes were quite uncertain due to the low concentrations of all complexed metals in the filterable sample fraction and the imprecision of the toxicity tests at such low values.

Strengths of Major Constituent and Heavy Metal Associations with Particulates. Most of the heavy metals in stormwater are associated with the particulates larger than 0.45 µm in size, although some exceptions exist (usually zinc, and sometimes copper, can predominantly be associated with the filterable fraction of stormwater). The fate of these particulate-bound heavy metals is of interest when designing stormwater controls. Specifically, do these metals stay bound to the particulates under typical environmental conditions of pH and Eh? This is of special interest when considering sediment in urban streams, and captured sediment in stormwater controls. Additional tests were therefore conducted by Johnson, *et al.* (2003) to examine the strengths of association between the pollutants and the particulates. It was expected that the filtered concentrations after exposure to varying pH conditions would be intermediate between the original filterable (filtered through 0.45 µm filters and digested) and the initial total (unfiltered and digested) concentrations. If the concentrations under all pH conditions were similar to the concentrations of the original filtered samples, then the particulate bound metals are strongly associated with the particulates and are unlikely to enter the overlying water column in a pond. However, if the concentrations approached the total metal concentrations, and were much higher than the filtered metal concentrations, then the metals are weakly associated with the particulates and my become disassociated during normal environmental conditions, with potentially adverse water quality consequences.

Experiments were conducted to examine the likelihood of the metals disassociating from the particulates under pH conditions ranging from about 4 to 11. In addition, tests were conducted with both weak and strong acids. Related tests were conducted to measure the disassociation potential of heavy metals (and nutrients) under aerobic and anaerobic conditions having extreme Eh values. Figure 59 is an example plot for the copper tests.

Table 21 summarizes the results of these tests, indicating if the major constituents and heavy metals can be disassociated from the particulates under different pH conditions. The major constituents (calcium, potassium and sodium) had very little change for any pH exposure condition; all concentrations were very similar. Magnesium also had several samples that had better retention at high pH, compared to low pH conditions. There was also no apparent difference between exposure to the strong or the weak acids at the two low pH conditions for any constituent.

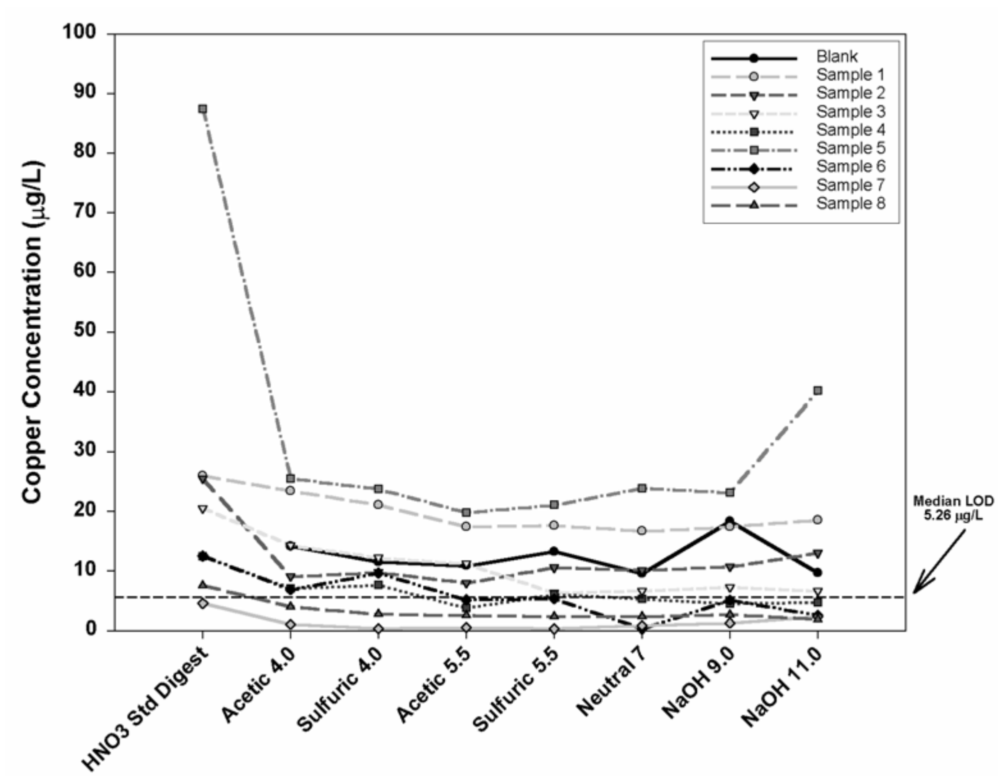


Figure 59. Leachability of copper under different pH conditions.

Table 21. Summary of Strengths of Associations of Major Constituents and Heavy Metals with Particulates under Varying pH Exposure Conditions

	pH exposure conditions (after 3 days on shaking table)				
	pH = 4	pH = 5.5	neutral	pH = 9	pH = 11
Calcium	dissolved[1]	dissolved	dissolved	dissolved	dissolved
Magnesium	weak[2]	moderate[3]	moderate	moderate	strong[4]
Potassium	strong	strong	strong	strong	strong
Sodium	dissolved	dissolved	dissolved	dissolved	dissolved
Cadmium	precipitated[5]	precipitated	precipitated	precipitated	precipitated
Chromium	strong	strong	strong	strong	strong
Copper	strong	strong	strong	strong	strong
Iron	strong	strong	strong	strong	strong
Lead	strong	strong	strong	strong	strong
Zinc	weak	strong	strong	precipitated	precipitated

[1] dissolved constituents occur then the exposed concentrations were close to the original filterable concentration, and most of the constituent was in the dissolved (filterable) fraction.
[2] weak associations occur when the exposed concentrations are close to the original total concentration which was substantially larger than the dissolved concentration.
[3] moderate associations occur when the exposed concentrations are intermediate between the original total and filterable concentrations.
[4] strong associations occur when the exposed concentrations remain close to the original filterable concentration which is substantially smaller than the total concentration.
[5] precipitation may have occurred when the exposed concentrations are less than the original filterable concentration.

These tests indicate that the heavy metals of concern remain strongly bound to the particulates during long exposures at the extreme pH conditions likely to occur in receiving water sediments. They will also likely remain strongly bound to the particulates in stormwater control device sumps or detention pond sediments where particulate-bound metals are captured. The associated tests examining metal binding to filtration media under aerobic and anaerobic conditions also found that the heavy metals likely remain strongly associated with a variety of organic and inorganic media under varying Eh conditions. However, those other tests did indicate that captured nutrients are likely to come off the media under anaerobic conditions.

Particle Sizes. Knowing the settling velocity characteristics associated with stormwater particulates is necessary when designing wet detention ponds. Particle size is directly related to settling velocity (using Stokes law, for example, and using appropriate shape factors, specific gravity and viscosity values) and is usually used in the design of detention facilities. Particle size can also be much more rapidly measured in the laboratory than settling velocities. Settling tests for stormwater particulates need to be conducted for about three days in order to quantify the smallest particles that are of interest in the design of wet detention ponds. If designing rapid treatment systems (such as grit chambers or vortex separators for CSO treatment), then much more rapid settling tests can be conducted. Probably the earliest description of conventional particle settling tests for stormwater samples was made by Whipple and Hunter (1981).

Whipple and Hunter (1981) contradict the assumption sometimes used in modeling detention pond performance that pollutants generally settle out in proportion to their concentrations. However, Grizzard and Randall (1986) have shown a relationship between particulate concentrations and particle size distributions. High particulate concentrations were found to be associated with particle size distributions that had relatively high quantities of larger particulates, in contrast to waters having low particulate concentrations. The high particulate concentration water would therefore have increased particulate removals in detention ponds. This relationship is expected to be applicable for pollutants found mostly in particulate forms (such as suspended solids and most heavy metals), but the relationship between concentration and settling would be much poorer for pollutants that are mostly in soluble forms (such as filterable residue, chlorides and most nutrients). Therefore, the partitioning of specific pollutants between the "particulate" and "dissolved" forms, and eventually for different particulate size fractions, is needed.

Smith (1982) also states that settleability characteristics of the pollutants, especially their particle size distribution, is needed before detention pond analyses can be made. Kamedulski and McCuen (1979) report that as the fraction of larger particles increase, the fraction of the pollutant load that settles also increases. Randall, *et al.* (1982), in settleability tests of urban runoff, found that non-filterable residue (suspended solids) behaves liked a mixture of discrete and flocculant particles. The discrete particles settled out rapidly, while the flocculant particles were very slow to settle out. Therefore, simple particle size information may not be sufficient when flocculant particles are also present. Particle size analyses should include identification of the particle by microscopic examination to predict the extent of potential flocculation.

Recent detailed pond studies have provided additional in-sight to the particle sizes in stormwater and their behavior in wet ponds. Greb and Bannerman (1997) reported a long-term pond study for Madison, WI. They found that the influent particle-size distribution affected the efficiency of the wet pond's performance on sediment and associated pollutant removal. Krishnappan, *et al*. (1999) monitored in-pond particle sizes using a submersible laser particle size analyzer, reducing potential changes in particle characteristics that may occur during sampling. The suspended solids were mostly composed of flocs, and were about 30 μm in size during winter surveys and about 210 μm during the summer surveys. Three seasonal surveys of suspended solids were carried out in an on-stream stormwater management pond, by means of a submersible laser particle size analyzer. Size distributions were measured at up to 17 points in the pond, and water samples collected at the same locations were analyzed for primary particles aggregated in flocs. Using a relationship defining the flee density as a function of flee size and Stokes' equation for settling, an empirical relationship expressing the free fall velocity as a function of floc size was produced (Krishnappan, *et al*. 1999). It is possible that the flocs formed were associated with in-pond chemical and biological processes and were not representative of influent particle size conditions. Pettersson (2002) investigated the characteristics of suspended particles in a small stormwater pond. The results showed the particle volume in the stormwater (particle volume per stormwater volume) predominately consisted of very fine particles and that the smallest particles comprised most of the surface area. The stormwater pollutants exhibited strong correlations with particle characteristics. Petterson (1999) examined the partitioning of heavy metals in particulate-bound and dissolved phases in a stormwater pond in Goteborg, Sweden. The results showed a clear variation in lead partitioning affected by specific conductivity.

Figure 60 shows approximate stormwater particle size distributions derived from several upper Midwest and Ontario analyses, from all of the NURP data (Driscoll 1986), and for several eastern sites that reflect various residue concentrations (Grizzard and Randall 1986). Pitt and McLean (1986) microscopically measured the particles in selected stormwater samples collected during the Humber River Pilot Watershed Study in Toronto. The upper Midwest data sources were two NURP projects: Terstriep, *et al*. (1982), in Champaign/Urbana Ill. and Akeley (1980) in Washtenaw County, Michigan.

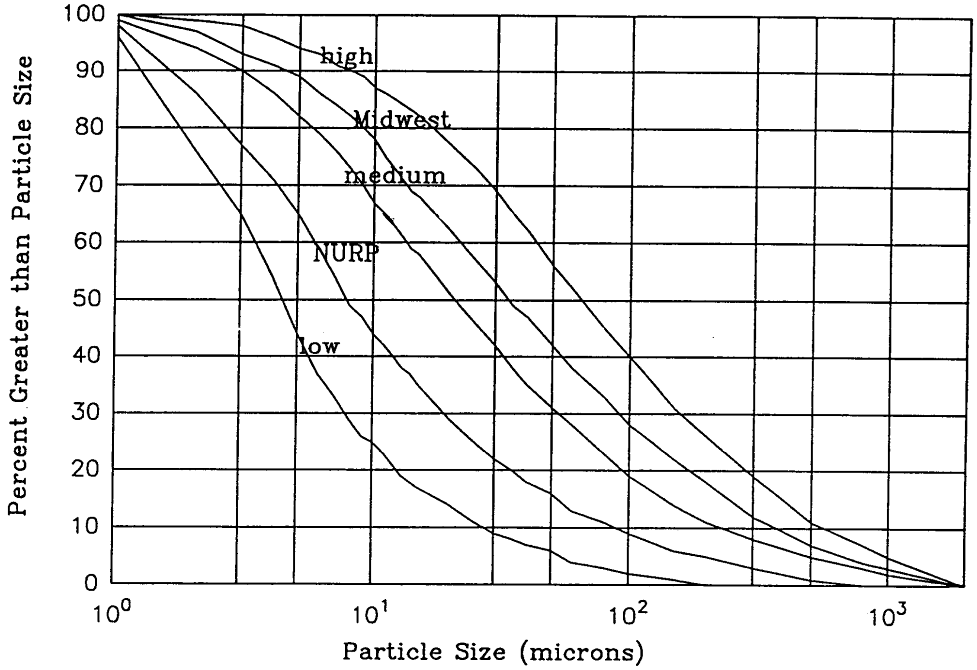


Figure 60. Particle size distributions for various stormwater sample groups.

Relatively few samples have been analyzed for stormwater particle sizes and no significant trends have been identified relating the particle size distribution to land use or storm condition. However, the work by Grizzard and Randall (1986) does indicate significantly different particle size distributions for stormwaters from the same site having different suspended solids concentrations. The highest suspended solids concentrations were associated with waters having relatively few small particles, while the low suspended solids concentration waters had few large particles. The particle size distribution for the upper Midwest urban runoff samples falls between the medium and high particulate concentration particle size distributions.

For many urban runoff conditions, the median stormwater particle size is estimated to be about 30 µm, (which can be much smaller than the median particle size of some source area particulates). Very few particles larger than 1000 µm are found in stormwater, but particles smaller than 10 µm are expected to make up more than 20 percent of the stormwater total residue weight. Similar observations of the predominance of very small particles have been made in other urban runoff detention pond studies (Ferrara 1982).

Specific conditions (such as source area type, rain conditions and upstream controls) have been shown to have dramatic effects on particle size distributions. Randall, *et al.* (1982) monitored particle size distributions in runoff from a shopping mall that was cleaned daily by street cleaning. Their data (only collected during the rising limb of the hydrographs) showed that about 80 percent of the particles were smaller than 25 µm, in contrast to about 40 percent that were smaller than 25 µm during the outfall studies. They also only found about two percent of the runoff particles in sizes greater than 65 µm, while the outfall studies found about 35 percent of the particles in sizes greater than 65 µm. This shopping mall runoff would therefore be less effectively treated by wet detention facilities because of the relatively smaller number of large particles present.

Limited data is also available concerning the particle size distribution of erosion runoff from construction sites. Hittman (1976) reported erosion runoff having about 70 percent of the particles (by weight) in the clay fraction (less than four µm), while the exposed soil being eroded only had about 15 to 25 percent of the particles (by weight) in the clay fraction. When the available data is examined, it is apparent that many factors affect runoff particle sizes. Rain characteristics, soil type, and on-site erosion controls are all important.

Tests have also been conducted to examine the routing of particles through the Monroe St. detention pond in Madison, Wisconsin (Roger Bannerman, Wisconsin Department of Natural Resources, personal communication). This detention pond serves an area that is mostly comprised of medium residential, with some strip commercial areas. This joint project of the Wisconsin Department of Natural Resources and the U.S. Geological Survey has obtained a number of inlet and outlet particle size distributions for a wide variety of storms. The observed median particle sizes ranged from about 2 to 26 µm, with an average of 9 µm. The following list shows the average particle sizes corresponding to various distribution percentages for the Monroe St. outfall:

Percent larger than size	Particle Size (µm)
10 %	450
25	97
50	9.1
75	2.3
90	0.8

These distributions included bedload material that was also sampled and analyzed during these tests. This distribution is generally comparable to the "all NURP" particle size distribution presented previously. The critical particle sizes corresponding to the 50 and 90 percent control values are as follows for the different data groups:

	90 %	50%
Monroe St.	0.8	9.1 µm
All NURP	1	8
Midwest	3.2	34
Low solids conc.	1.4	4.4
Medium solids conc.	3.1	21
High solids conc.	8	66

The particle size distributions of stormwater at different locations in an urban area greatly affect the ability of different source area and inlet controls in reducing the discharge of stormwater pollutants. A series of U.S. Environmental Protection Agency (USEPA) funded research projects has examined the sources and treatability of urban stormwater pollutants (Pitt, *et al.* 1995). This research has included particle size analyses of 121 stormwater inlet samples from three states (southern New Jersey; Birmingham, Alabama; and at several cities in Wisconsin) in the U.S. that were not affected by stormwater controls. Particle sizes were measured using a Coulter Counter Multi-Sizer IIe and verified with microscopic, sieve, and settling column tests. Figures 61 through 63 are grouped box and whisker plots showing the particle sizes (in µm) corresponding to the 10^{th}, 50^{th} (median) and 90^{th} percentiles of the cumulative distributions. If 90 percent control of suspended solids (by mass) was desired, then the particles larger than the 90^{th} percentile would have to be removed, for example. In all cases, the New Jersey samples had the smallest particle sizes (even though they were collected using manual "dipper" samplers and not automatic samplers that may miss the largest particles), followed by Wisconsin, and then Birmingham, Alabama, which had the largest particles (which were collected using automatic samplers and had the largest rain intensities). The New Jersey samples were obtained from gutter flows in a residential neighborhood that was xeroscaped, the Wisconsin samples were obtained from several source areas, including parking areas and gutter flows mostly from residential, but from some commercial areas, and the Birmingham samples were collected from a long-term parking area on the UAB campus. Generally, source area samples have larger particles, while outfall samples do not have the largest particles that washoff from the source areas. The largest particles accumulate in the drainage system and the runoff water does not have the power to transport these materials to the outfall.

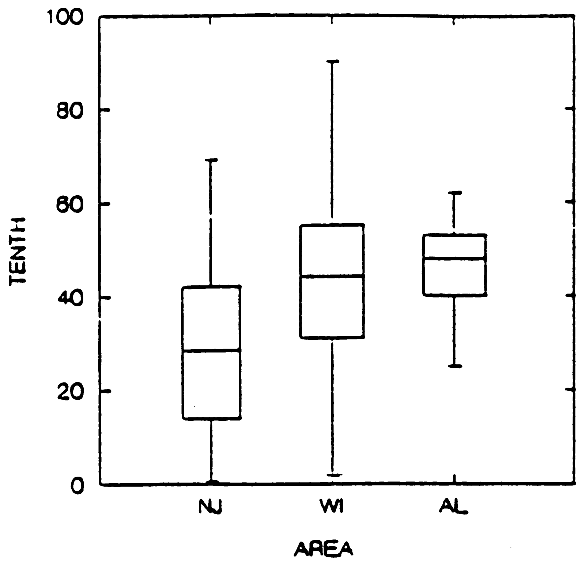


Figure 61. Tenth percentile particle sizes for stormwater inlet flows (Pitt, *et al*. 1997).

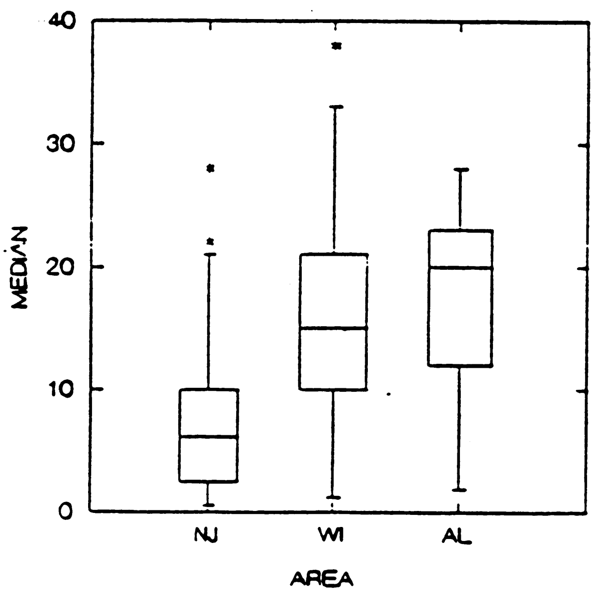


Figure 62. Fiftieth percentile particle sizes for stormwater inlet flows (Pitt, *et al*. 1997).

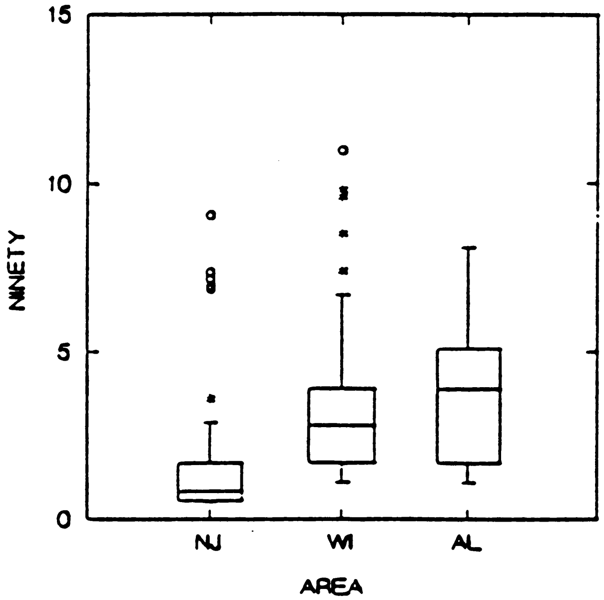


Figure 63. Ninetieth percentile particle sizes for stormwater inlet flows (Pitt, *et al*. 1997).

The median particle sizes ranged from 0.6 to 38µm and averaged 14µm. The 90^{th} percentile sizes ranged from 0.5 to 11µm and averaged 3µm. These particle sizes are all substantially smaller than have been typically assumed for stormwater.

Stormwater particle size distributions typically do not include bed load components because automatic sampler intakes are usually located above the bottom of the pipe where the bed load occurs. During the Monroe St. (Madison, WI) detention pond monitoring, the USGS and WI DNR installed special bed load samplers that trapped the bed load material for analysis. The bedload samplers were liter-sized wide mouth containers which were placed in bored holes in the bottom of the enclosed flat bottomed concrete small box channels right before the pond. Three units were placed in the channel bottom, each having different width slots cut in their lids. The mass of material trapped was directly related to the ratio of the width of the slot to the width of the channel. The material was removed, dried and weighed, and sieved. This particle size distribution was combined with the flow-weighted particle distributions obtained for the runoff events monitored during the same exposure period. Practically all events were monitored, with little flow not represented in these analyses. Figure 64 shows the measured particle size distributions for 16 seasonal samples (each having several runoff events), also including the bedload particle size distributions. The bedload samplers were in place for several weeks at a time in order to accumulate sufficient sample for analyses. The bedload material was comprised of the largest material represented on this figure (generally about 300 or 400 µm and larger particulates) and comprised about 10 percent of the annual total solids loading, but ranged from about 2 to 25 percent for individual periods. The bed load component in Madison was most significant during the early spring rains when much of the traction control sand that could be removed by rains was being washed from the streets. This is not a large fraction of the solids, but it represents the largest particle sizes flowing in the stormwater and it can be easily trapped in most detention ponds or catchbasins.

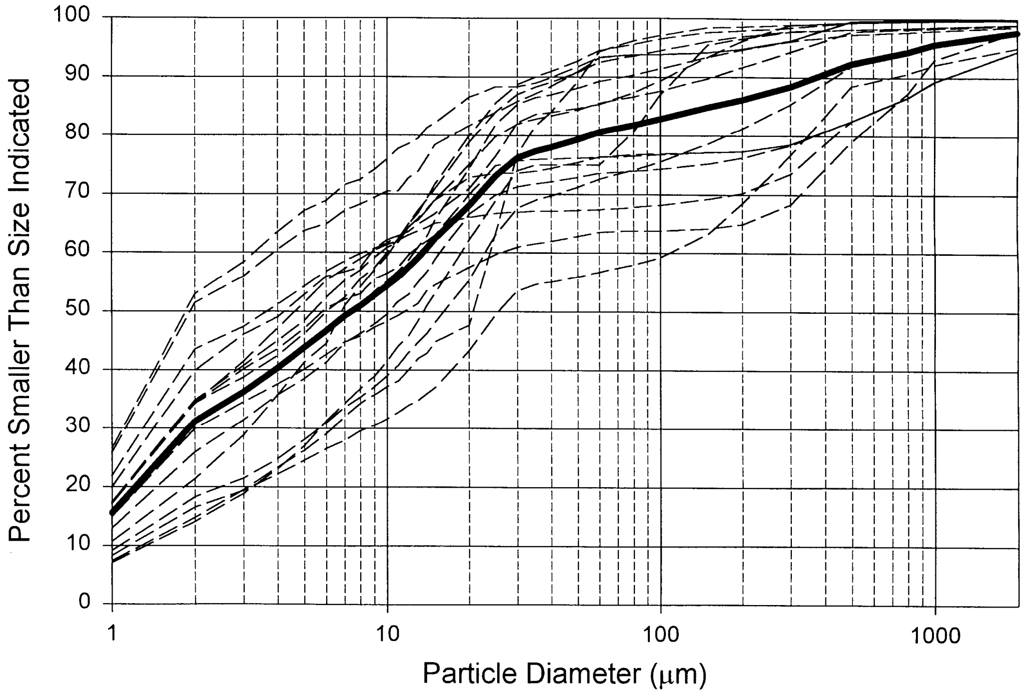


Figure 64. Inlet particle size distributions observed at the Monroe St. wet detention pond.

Figure 65 shows a typical “delta” of large material immediately near the influent to a wet detention pond, along with an accumulation of material along the invert of the corregated steel drainage pipe. This photo was taken at a pond in Snowmass, CO, in an area of heavy sand applications for traction control in the winter. The bedload sediment material in this photo is quite large, several mm in diameter, and near the upper range of the particle size distribution shown previously. This drainage pipe is relatively short, connected to an adjacent parking area. It is rare for this large material to be transported great distances in drainage systems. If it does enter a pond, or any type of sediment device, it is easily trapped near the inlet. However, moderate sized particulates can easily be transported quite some distance in the drainage system and be deposited well away from the inlet when discharged into wet detention ponds. The use of forebays, or small pre-settlement ponds, will trap much of the moderate-sized particulates (down to about 25μm) within a relatively small area for easier removal during maintenance operations. The finer particulates (down to just a few μm), containing most of the pollutants, would be trapped in the main pond area, although the sediment mass is relatively small.

Figure 65. Bedload sediment accumulation in sewerage and near outlet in pond (Snowmass, CO).

Additional data obtained by Pitt, *et al.* (1997) for the USEPA described particle sizes from many different source flows in the Birmingham, Alabama, area. These data did not indicate any significant differences in particle size distributions for different source areas or land uses, except that the roof runoff had substantially smaller particle sizes than the other areas sampled. Also, the source area particle size distributions indicated that larger particles were much more likely to be present at source areas than at outfalls. The larger particles appear to be trapped in the flow paths and drainage system before they reach the outfalls.

Particle Settling Velocities

The settling velocities of discrete particles are shown in Figure 66, based on Stoke's and Newton's settling relationships. Probably more than 90% of all stormwater particulates are in the 1 to 100 µm range, corresponding to laminar flow conditions, and appropriate for using Stoke's law. This figure also illustrates the effects of different specific gravities on the settling rates. In most cases, stormwater particulates have specific gravities in the range of 1.5 to 2.5. This corresponds to a relatively narrow range of settling rates for a specific particle size. Particle size is much easier to measure than settling rates and it is generally recommended to measure particle sizes using automated particle sizing equipment (such as a Coulter Counter Multi-Sizer IIe) and to conduct periodic settling column tests to determine the corresponding specific gravities. If the particle counting equipment is not available, then small scale settling column tests (using 50 cm diameter Teflon™ columns about 0.7 m long) can be easily used.

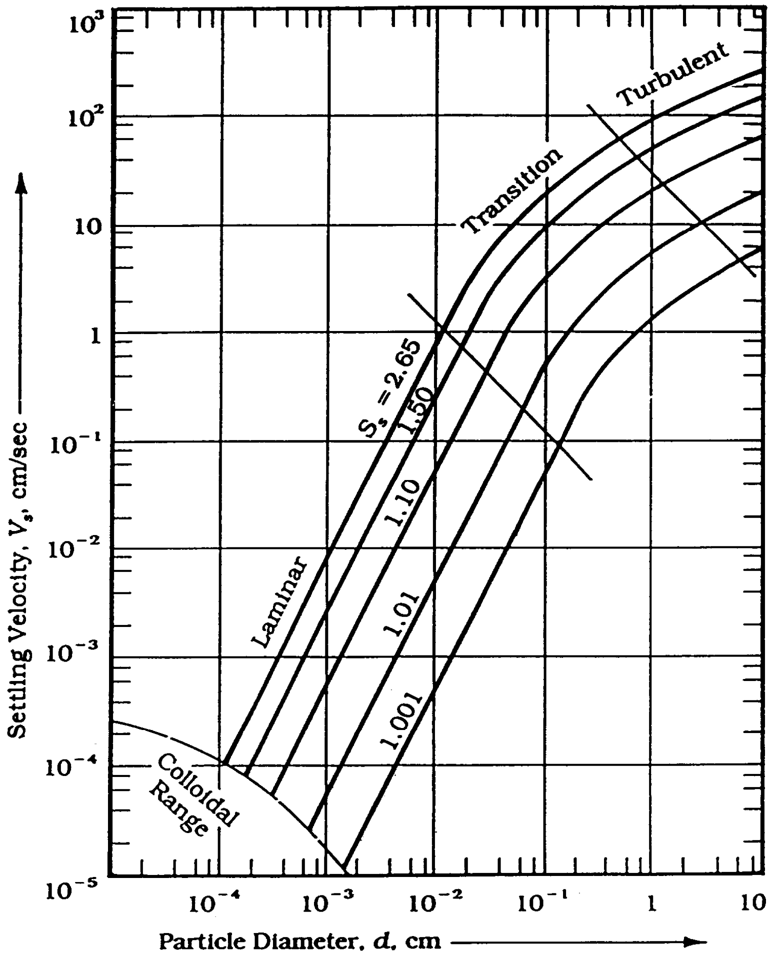


Figure 66. Type 1 (discrete) settling of spheres in water at 10° C (Reynolds 1982).

These settling velocities (or particle sizes) are used with the pond outflow rate to determine the required pond surface area. Figure 67 shows the minimum pond surface area needed to capture particles of a specific size (and larger) for different pond outflow rates.

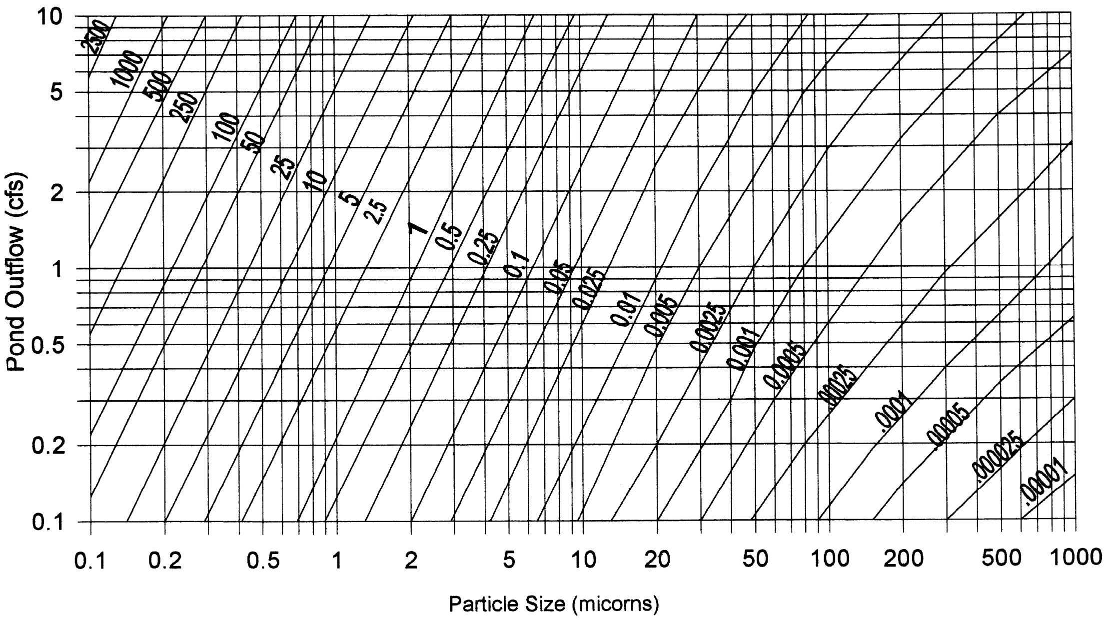


Figure 67. Minimum pond size (acres) needed for complete settling of specific sized particles at various pond overflow rates.

Particle settling observations in actual detention ponds have generally confirmed the ability of well designed and operated detention ponds to capture the "design" particles. Gietz (1983) found that particles smaller than 20 μm were predominate (comprised between 50 to 70 percent of the sediment) at the outlet end of a "long" monitored pond, while they only made up about ten to 15 percent of the sediment at the inlet end. Particles between 20 and 40 μm were generally uniformly distributed throughout the pond length, and particles greater than 40 μm were only found in the upper (inlet) areas of the pond. The smaller particles were also found to be resuspended during certain events.

Pisano and Brombach (1996) summarized numerous solids settling curves for stormwater and CSO samples. They are concerned that many of the samples analyzed for particle size are not representative of the true particle size distribution in the sample. As an example, it is well known that automatic samplers do not sample the largest particles that are found in the bedload portion of the flows. Particles having settling velocities in the 1 to 15 cm/sec range are found in grit chambers and catchbasins, but are not seen in stormwater samples obtained by automatic samplers, for example. It is recommended that bedload samplers be used to supplement automatic water samplers in order to obtain more accurate particle size distributions (Burton and Pitt 2002). Selected US and Canadian settling velocity data are shown in Table 22. The CSO particulates have much greater settling velocities than the other samples, while the stormwater has the smallest settling velocities. The corresponding "Stoke's" particle sizes for the geometric means are about 100 μm for the CSOs, about 50 μm for the sanitary sewage, and about 15 μm for the stormwater.

Table 22. Settling Velocities for Wastewater, Stormwater, and CSO (Pisano and Bromback 1996)

Samples	Geometric Means of Settling Velocities Observed (cm/sec)	Range of Medians of Settling Velocities Observed (cm/sec)
CSO	0.22	0.01 to 5.5
dry weather wastewater (sanitary sewage)	0.045	0.030 to 0.066
stormwater	0.011	0.0015 to 0.15

More than 13,000 CSO control tanks have been built in Germany using the ATV 128 rule (Pisano and Bromback 1996). This rule states that clarifier tanks (about 1/3 of these CSO tanks) are to retain all particles having settling velocities greater than 10 m/hr (0.7 cm/sec), with a goal of capturing 80% of the settleable solids. Their recent measurements of overflows from some of these tanks indicate that the 80% capture was average for these tanks and that the ATV 128 rule appears to be reasonable.

Pond Water Losses and Liners

Evaporation and infiltration losses can have very important aesthetic, recreation, and wildlife effects. In some cases, the pond may totally dry-up if little supplemental inflows (baseflows from the drainage area, groundwater infiltration to the pond, or supplemental groundwater pumping to the pond) occur for ponds over permeable soils, and/or if long dry periods occur between rains. Maryland does not allow wet detention ponds to be located in areas of highly permeable NRCS hydrologic class A soils (those having saturated soil infiltration rates greater than 0.5 inches per hour) (Harrington 1986). Harrington also reports that Maryland requires pond sealing if underlying soils have infiltration rates between 0.1 and 0.5 inches per hour (generally includes all class C and class B soils, and even some class D soils). NRCS hydrologic soil types in urban areas or for small areas should not be determined using the soil maps for undisturbed areas alone. Typical soil disturbance in urban areas can greatly affect the infiltration and percolation characteristics of native soils. In addition, it is common for excavation and fill processes to expose a completely different soil at a site.

As part of the Wisconsin Priority Watershed Program (Pitt 1986), a series of infiltration tests were conducted in the city of Oconomowoc, Wisconsin. These tests were conducted to calibrate the Source Loading and Management Model (SLAMM) (Pitt 1988) being used by the Wisconsin Department of Natural Resources. Oconomowoc is characterized as having mostly sandy soils (NRCS hydrologic soil types A and B predominated before development). Observed infiltration rates varied greatly, ranging from 0 to 25 inches per hour. The only relationship found between the type of area tested and the infiltration rates was the amount of site disturbance (compaction). Even though the soils were mostly sandy, high foot traffic areas (such as at schools and in many front lawns) had very low infiltration rates. Many swale linings also had very low infiltration rates (many had 0 inches per hour), probably because of construction techniques that used clayey soils as ditch linings or were clogged with fines from construction erosion. These areas could not be considered as “pervious.” It would be very misleading to assume that they had similar infiltration rates as native A or B soils. In addition, many of the final infiltration rates observed were substantially greater than the initial infiltration rates, in contrast to typical infiltration rate theory. The median infiltration rate for these “sandy” soils was about 5 inches per hour, corresponding to rates for type A soils, but many sandy soil areas tested had rates that were much less than this value (corresponding to class D soils). It is therefore imperative that percolation and infiltration tests, along with soil surveys, be conducted at all potential pond locations before final design. If the pond will be excavated, a percolation test should be used, while double ring infiltration tests should be conducted for areas that will use the natural surface for the pond lining.

A series of 153 double ring infiltrometer tests were conducted in disturbed urban soils in the Birmingham, and Mobile, Alabama, areas (Pitt, *et al.* 1999). The tests were organized in a complete 2^3 factorial design (Box, *et al.* 1978) to examine the effects of soil-water, soil texture, and soil compactness on water infiltration through historically disturbed urban soils. Turf age was also examined, but insufficient sites were found to thoroughly examine these effects. Ten sites were selected representing a variety of desired conditions (compaction and texture) and numerous tests were conducted at each test site area. Soil-water content and soil texture conditions were determined by standard laboratory soil analyses. Compaction was measured in the field using a cone penetrometer and confirmed by the site history. Soil-water levels were increased using long-duration surface irrigation before most of the saturated soil tests. From 12 to 27 replicate tests were conducted in each of the eight experimental categories in order to measure the variations within each category for comparison to the variation between the categories.

The data was plotted on 3D plots (Figure 68 and 69), showing the infiltration observations affected by soil-water levels and compaction, for both sand and clay. Four general conditions were observed to be statistically unique, as listed on Table 23. Compaction has the greatest effect on infiltration rates in sandy soils, with little detrimental effects associated with higher soil-water content conditions. Clay soils, however, are affected by both compaction and soil-water content. Compaction was seen to have about the same effect as saturation on these soils, with saturated and compacted clayey soils having very little effective infiltration.

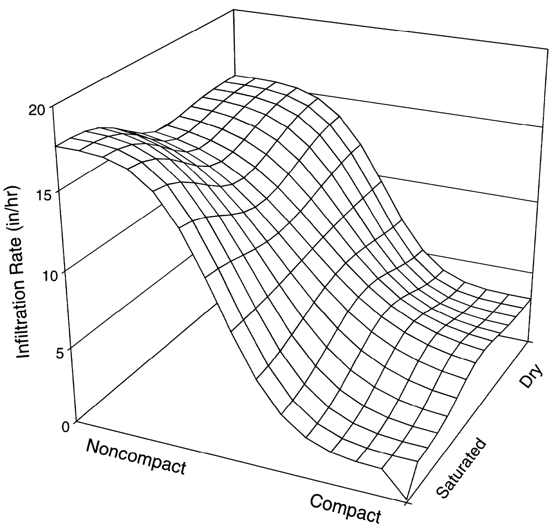


Figure 68. Three-dimensional plot of infiltration rates for sandy soil conditions (Pitt, *et al.* 1999).

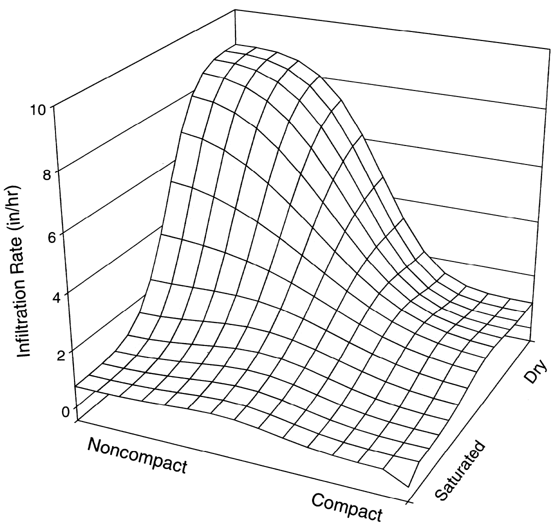


Figure 69. Three-dimensional plot of infiltration rates for clayey soil conditions (Pitt, *et al.* 1999).

Table 23. Infiltration Rates for Significant Groupings of Soil Texture, Soil-Water Content, and Compaction Conditions

Group	Number of tests	Average infiltration rate (in/hr)	COV
noncompacted sandy soils	36	13	0.4
compact sandy soils	39	1.4	1.3
noncompacted and dry clayey soils	18	9.8	1.5
all other clayey soils (compacted and dry, plus all wetter conditions)	60	0.2	2.4

Figure 70 illustrates how much the pond surface elevation could decrease for various pond loss rates and dry periods. The total pond loss rates include both infiltration losses through underlying soils plus evaporation. Table 24 presents approximate infiltration rates for different soil texture classes and NRCS hydrologic soil groups (from Harrington 1986 and SCS 1986). If pan evaporation losses average about 30 inches per year (not unusual), it may possibly reach as high as 0.03 inches per hour during the hottest summer afternoons. Only clay soils probably have infiltration losses less than this evaporation loss rate.

Table 24. Approximate Saturated Infiltration Rates for Different Soil Texture Classes

Soil Texture Class	SCS Hydrologic Soil Group	Saturated Soil Infiltration Rates (in/hr)	(min/in)
Sand	A	8	7.5
Loamy Sand	A	2.5	24
Sandy Loam	A	1	60
Loam	B	0.5	120
Silt Loam	B	0.3	200
Sandy Clay Loam	C	0.2	300
Clay Loam	D	0.1	600
Silty C1ay Loam	D	0.05	1200
Sandy Clay	D	0.05	1200
Silty Clay	D	0.04	1500
Clay	D	0.02	3000

Source: Harrington 1986 and SCS 1986

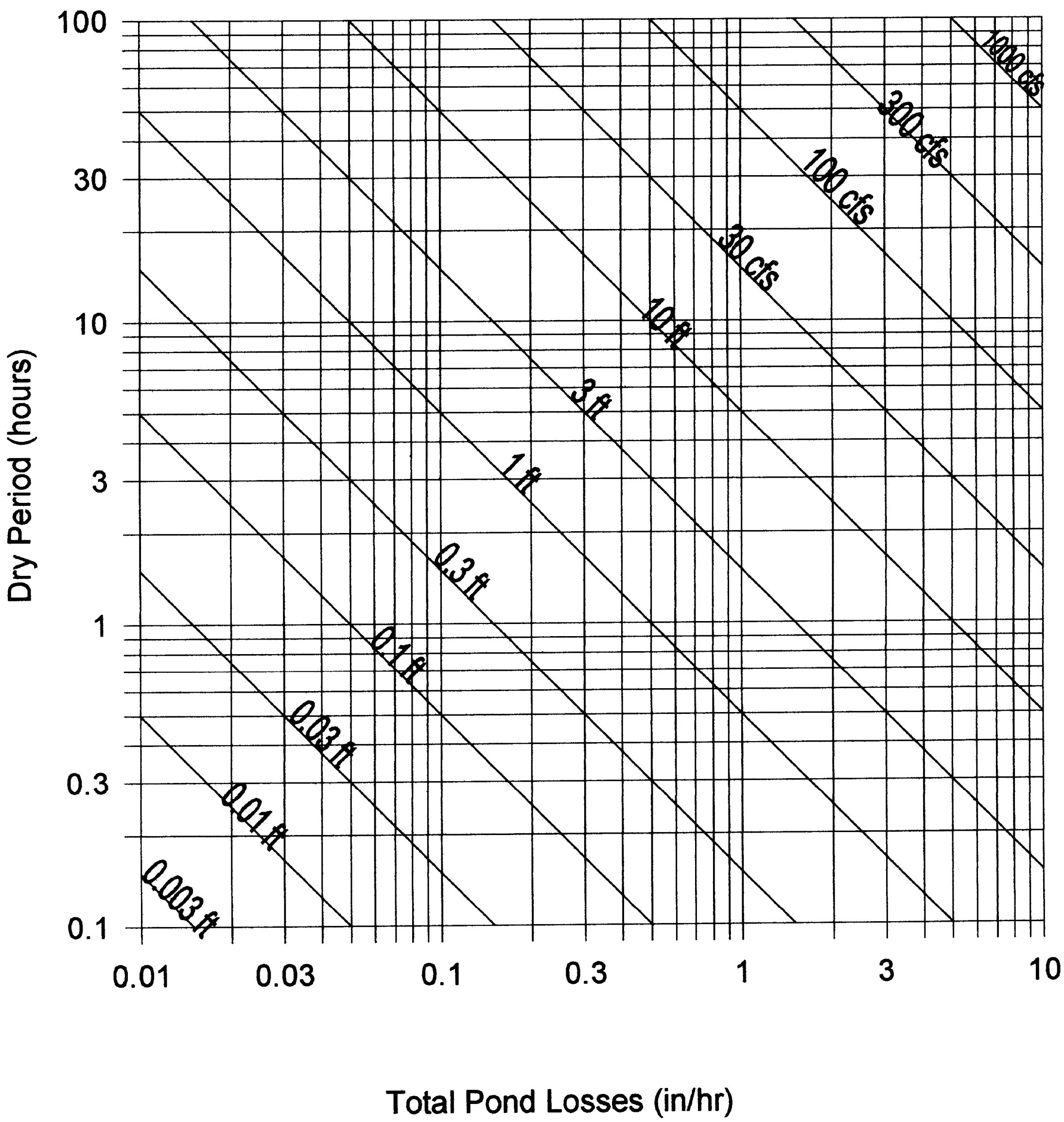


Figure 70. Pond water surface elevation drop (ft.) if no pond inflow during dry periods.

Figure 71 shows that unlined ponds in class A soils could lose about one foot of water elevation to infiltration during a two to eight hour runoff event and about ten feet of water surface elevation between the three to five days between events. Clearly, a wet pond over class A soils, without a liner and/or supplemental inflow, would not remain wet for long.

Figure 71 shows that a two acre pond over class A soils (having a three inch per hour infiltration rate) would need an inflow of about five cfs in order to maintain a constant water surface elevation. A two acre pond may adequately serve a residential area of about 250 acres, or a shopping center of about 75 acres. This inflow requirement could therefore vary from about 0.02 to 0.07 cfs per acre of watershed. Dry weather urban runoff baseflows may be less than five percent of the inflow requirement (Pitt and McLean 1986). Therefore, in order to maintain a constant water surface elevation for typical watershed and pond sizes, maximum infiltration rates from a residential area would need to be less than about 0.15 inches per hour (a class C soil), while a shopping center would require a maximum soil infiltration rate of about 0.04 inches per hour (a class D soil). In most cases, pond percolation losses will decrease with time as sediments accumulate.

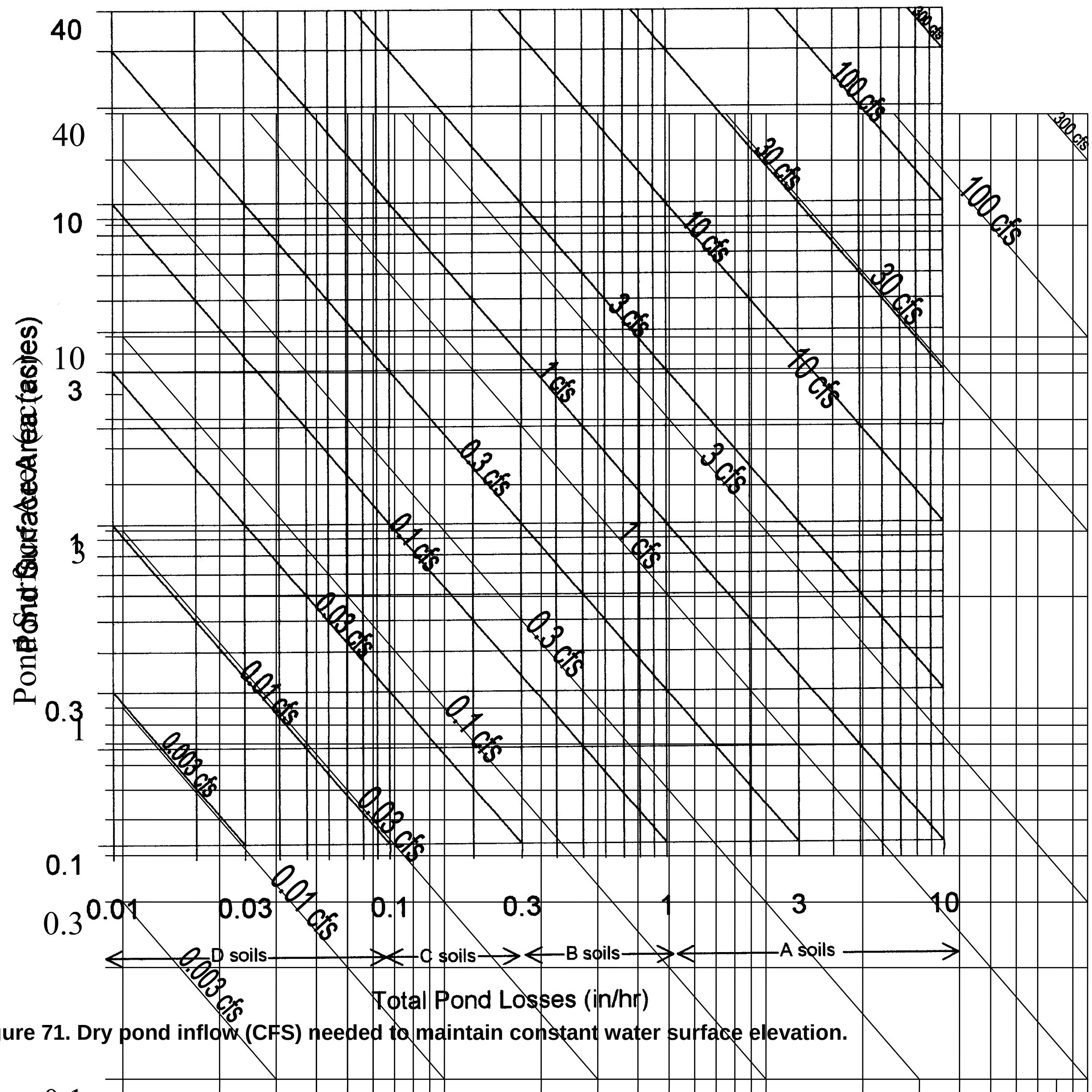


Figure 71. Dry pond inflow (CFS) needed to maintain constant water surface elevation.

Harrington (1986) has summarized different wet pond linings used in Maryland:

"**Compacted**

Some pond areas can be made relatively impervious by compaction alone if the underlying soil material contains sand, silt, and at least ten percent clay. However, the reduction of seepage losses may be difficult to determine without taking an infiltration test of the soil. This method is the least expensive method to reduce the soil permeability.

Figure 28. Dry pond inflow (cfs) needed to maintain constant water surface elevation.

Clay Blankets

Pond areas containing high percentages of coarse-grained soils can be sealed by blanketing them with clay layers. The best clay blanket would consist of a material containing 40 percent or more of clay, but no less than 20 percent. The thickness of the blanket is a function of the depth of water to be impounded. Use a minimum thickness of 12 inches for all depths of water up to ten feet. Increase this thickness by two inches for each foot of water over ten feet.

Waterproof Linings

Waterproof linings are another way of reducing excessive seepage. Polyethylene, vinyl, butyl-rubber membranes, and asphalt-sealed fabric liners are being used because they virtually eliminate seepage if properly installed.

Thin films of these materials are structurally weak, but if not broken or punctured, they are almost completely watertight. Black polyethylene films are less expensive and have better aging properties than vinyl. Vinyl, on the other hand, is more resistant to impact damage and is readily seamed and patched with a solvent cement.

All plastic membranes should have a cover of earth and gravel not less than six inches thick to protect against punctures. The bottom three inches of cover should be no coarser than silty sand."

Flow Rate Reductions In Water Quality Ponds

Most flood control ponds are dry ponds so the maximum storage volume is available to attenuate excess inflows. As stated previously, dry ponds do not effectively retain sediment because of bottom scour of the deposited sediments. A wet pond only slightly reduces peak flows during large storms if substantial amounts of extra storage are not provided above the permanent pond water surface elevation. Inflowing peak flows would be slightly moderated because of backwater profile and surface wave effects providing some temporary additional storage volumes. Peak flow rates from small storms can be substantially reduced with the freeboard storage normally provided in wet detention ponds, however. If additional land area and pond depth is available, then wet ponds can be designed to provide both significant flood control and water quality improvements. NRCS (SCS 1986) methods can be used to estimate the additional storage volume above the permanent wet pond water surface to provide desired flood control benefits. The use of multiple outlet devices can be effectively used to help provide these dual benefits.

An emergency spillway is always needed, even for temporary detention ponds at construction sites. Most local regulatory agencies will require an emergency spillway that is capable of discharging a specific design storm, typically in the range of 25 to 100-yr events, depending on the size of the pond. The typical procedure is to use the SCS (now NRCS) (1986) version of TR-55. The graphical peak discharge method in TR-55 is commonly used to estimate the peak flow associated with the design storm, and the TR-55 "structure" methods are then used to estimate the emergency spillway design. This spillway design should consider the outlet device selected for water quality benefits also. Figure 72 shows that for type II and III rains, the storage volume would have to be about 0.55 of the runoff volume, if the peak runoff rate is to be reduced to 0.1 of its influent peak flow rate.

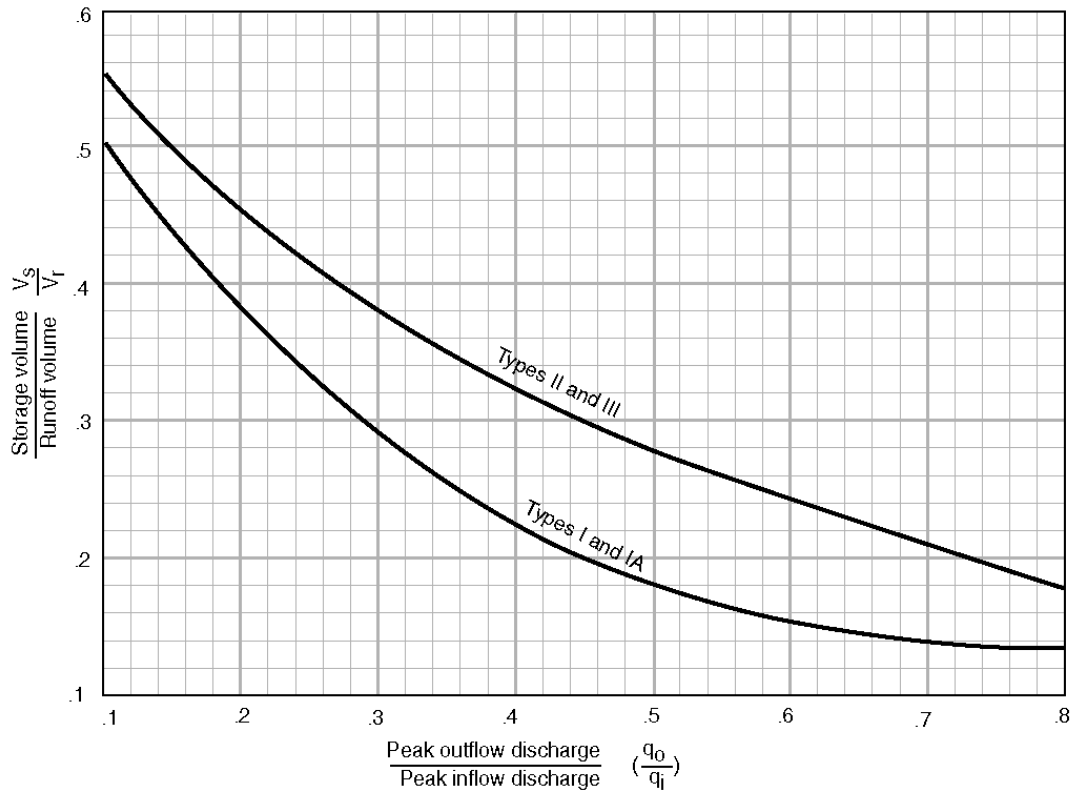


Figure 72. SCS TR-55 plot used to size additional freeboard needed for emergency spillway (NRCS 1986).

The SCS methods can be used indirectly to size an emergency spillway. The pond is sized to provide the water quality benefits, and this storage volume is taken as Vs in Figure 72. The design storm volume that must safely be accommodated by the emergency spillway is taken as Vr. The ratio of these values can be used with this figure to estimate the peak flow attenuation that the pond will provide only using the primary water quality pond components. The peak inflow discharge rate, qi, can be estimated using the SCS graphical peak discharge method (or the tabular hydrograph method, or WinTR-55). The peak outfall discharge, qo, is then calculated based on the measured attenuation factor. As an example, consider:

Vs = 1.53 acre-ft
Vr = 7.5 acre-ft
and Vs/Vr = 0.20

Therefore, for type II or III rain categories (generally covers most of the US, except the northwest):

qo/qi = 0.72

if the calculated peak discharge rate entering the pond (qi) = 8.7 cfs, the resulting peak discharge rate leaving the pond, qo, (through the water quality primary outlet plus the emergency spillway) is therefore: 0.72 (8.7) = 6.3 cfs. TR-55 shows how to calculate the needed emergency spillway for a specific discharge goal, considering multiple outlet structures. This method will help determine the size of the spillway, plus the additional freeboard that must be added to the pond design to accommodate the emergency spillway and desired outlet flow rate.

McCuen (1980) has defined a peak flow reduction factor to describe the ability of a detention pond to reduce flow rates. This factor is:

$$PRF = 1 - Q_o / Q_i$$

where Q_o is the outflow rate from the pond, and Q_i is the inflow rate to the pond. A 90% reduction in peak flow (say form 10 cfs inflow to 1 cfs outflow) would therefore have a PRF of 0.9. This value approaches 1.0 for very large flow reductions and 0.0 for very small flow changes.

Natural Bacterial Dieoff in Detention Ponds

Chick's law can be used to predict the dieoff of bacteria (Chick 1908). It is usually expressed as:

percent of bacteria remaining = e^{-Ket}, therefore

the fraction of bacteria removed (in time t) = $1 - e^{-Ket}$

where K_e is the dieoff rate (units per day) and t is the time (days). K_e is 2.3 times larger than the commonly reported K_{10} values. Since detention ponds can hold runoff water for a substantial period of time, significant bacteria reductions may be possible because of natural dieoff. However, during most storms, most of the water passes through the pond with little delay, and only relatively small portions of the annual discharges are actually held in the pond during extended interevent periods.

The average detention time of a lake is determined by dividing the lake volume by the average flow rate. In a stormwater detention pond, the flow rate is highly variable, being very high for short periods of time and very low for relatively long interevent periods. It is not uncommon for most rains to only last for several hours, while the interevent durations may last for several days. Therefore, the detention time is difficult to analyze. The overall reductions in bacteria populations would therefore be dependent on the relative quantities of runoff that pass through a pond during an event (with a relatively short detention time) and the amount that would be stored before the next event (with a relatively long detention time). WinDETPOND calculates and tracks the pond "flushing ratio," the storm volume compared to the amount of water in the pond at the beginning of the event, and is useful for these determinations.

The Long Island NURP project (Lawler, Matusky and Skelly Engineers, *et al.* 1982) investigated the dieoff of bacteria in detention ponds. They summarized *in situ* coliform bacteria dieoff rates from other locations and measured dieoff rates for their local conditions. They summarized K_{10} rates ranging from 0.18 to 11.4 per day (corresponding to K_e rates of 0.41 to 26.2 per day). They concluded that coliform bacteria dieoff rates (K_e) of about 2.3 per day are reasonable for stormwater.

Figure 73 indicates the percentage dieoff of bacteria, based on differing K_e rate constants and detention times. This figure indicates that coliform dieoff should be quite complete after about two days of detention (assuming a typical K_e value of 2.3/day). However, most urban runoff receiving waters (including ponds) probably seldom experience fecal coliform levels less than several hundred counts per 100 mL, compared to discharge concentrations of many thousand counts per 100 mL (EPA 1983). This indicates maximum reductions of about 90 percent, which is certainly significant, but the resulting fecal coliform populations are still high compared to most water quality standards. Bacteria reductions of about 20 percent may also be expected during runoff events that may last several hours. Sustained high fecal coliform populations may be caused by continued discharges of contaminated baseflows into detention ponds (Pitt and McLean 1986). It is not unusual for baseflows to have fecal coliform levels of several thousand counts per 100 mL (Pitt, *et al.* 1993).

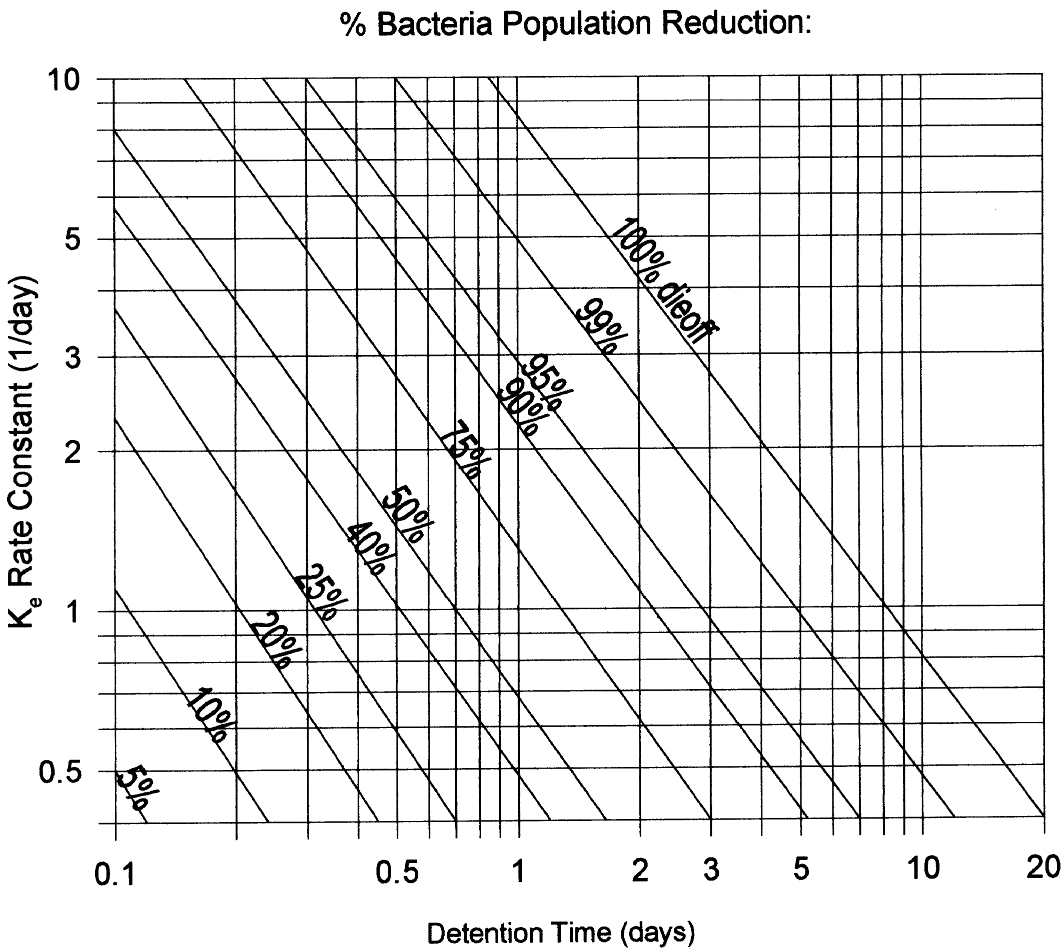


Figure 73. Chick's law for bacterial dieoff.

Public swimming area near CSO outfall on Navisink River, NJ

Public swimming area near CSO outfall on Navisink River, NJ (closeup)

Children playing in Lincoln Creek, Milwaukee, WI

Private swimming area at detention pond, WI

Goose poop at private swimming area at detention pond, WI

Geese frequenting detention pond

Figure 74. Exposure to and sources of pathogens in urban waters.

Design Based on NURP Detention Pond Monitoring Results

As summarized earlier, several NURP projects investigated the performance of different types of detention ponds. About 150 rain events were monitored at nine ponds located throughout the U.S. The EPA (1983) determined that long-term detention pond performance could be estimated based on geographical location and the ratio of the pond surface area to contributing source area.

Driscoll (1989; and EPA 1986) presented a basic methodology for the design and analysis of wet detention ponds. A pond operates under dynamic conditions when the storage of the pond is increasing with runoff entering the pond and with the stage rising, and when the storage is decreasing when the pond stage is lowering. Quiescent settling occurs during the dry period between storms when storage is constant and when the previous flows are trapped in the pond, before they will be partially or completely displaced by the next storm. The relative importance of the two settling periods depends on the size of the pond, the volume of each runoff event, and the inter-event time between the rains.

Driscoll (1989) produced a summary curve, shown as Figure 75, that relates wet pond performance to the ratio of the surface area of the pond to the drainage area, based on the numerous NURP wet detention pond observations. The NURP ponds were in predominately residential areas and were drained with conventional curb and gutters. This figure indicates that wet ponds from about 0.3 to 0.8 percent of the drainage area should produce about 90% reductions in suspended solids. Southeastern ponds need to be larger than ponds in the Rocky Mountain region because of the much greater amounts of rain and the increased size of the individual events in the southeast. Also, wet ponds intending to remove 90% of the suspended solids need to be about twice as large as ponds with only a 75% suspended solids removal objective.

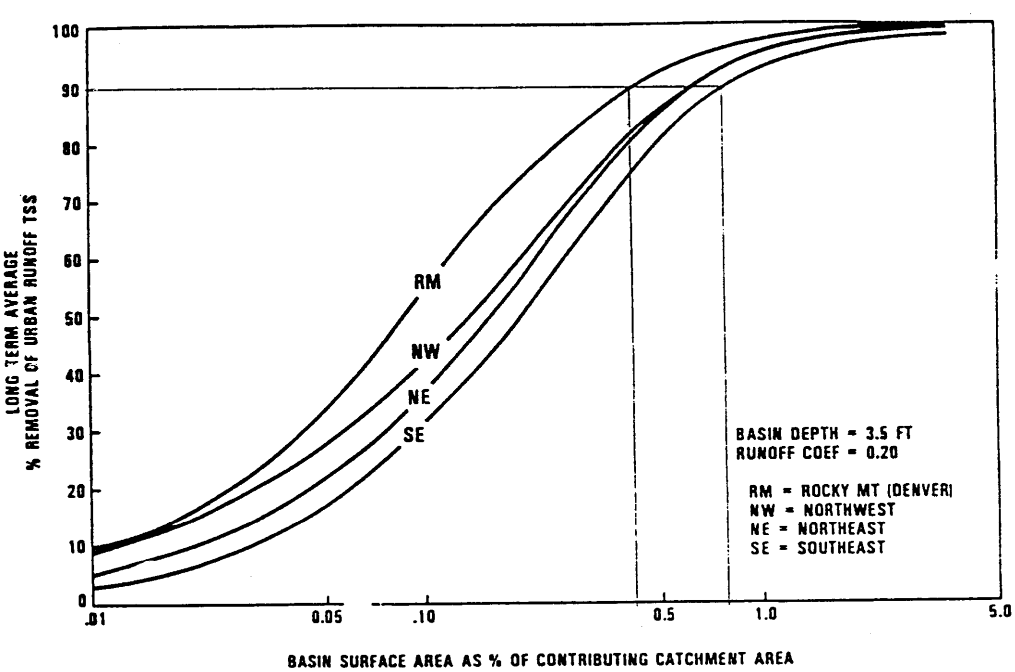


Figure 75. Regional differences in detention pond performance (EPA 1983).

The NURP detention pond monitoring results mostly included residential areas and therefore could not effectively examine the effects of land use on pond performance. Hey and Schaefer (1983), during the West Chicago NURP project in Glen Ellyn, Illinois, prepared Table 25 showing how land uses with large fractions of impervious areas require about twice the pond surface area as suburban residential areas. These ratios are all substantially greater than shown on Figure 75 to provide an extra margin of safety for a broader range of expected rain conditions.

Table 25. Area Required for Wet Detention Ponds for Different Land Uses

Land Use	Percent Impervious	Storage Needed (inches)	Percent of Drainage Area Needed for Detention Storage[1]
Parking Lot	100%	1.0	2.8%
Suburban and Commercial	25	0.6	1.7
Suburban	10	0.5	1.3
Undeveloped	0	0.4	1.1

[1] Assuming an average depth of three feet.
Source: Hey and Schaefer (1983)

Importance of Reservoir Routing

The discharged water from a detention pond is simply displaced pond water. In some cases, observed outlet water characteristics during a specific storm cannot be related to the inlet water characteristics. If the storm is small, the volume of water coming into the pond can be substantially less than the resident water in the pond. In these cases, the outlet water is mostly "left-over" water from a previous event or from relatively low volume (but long duration) baseflows that had previously entered the pond since the last storm. However, if the storm is large, then the water being discharged from the pond is mostly related to the specific event. Therefore, analyses of detention pond behavior must consider the relative displacement of pond water. Long-term continuous analyses comparing many adjacent storms resulting in seasonal inlet and outlet discharges of pollutants may be more appropriate than monitoring simple paired samples.

The following discussion on routing includes a procedure to examine these pond water displacement considerations and their effects on particulate trapping. The Source Loading and Management Model (WinSLAMM) and the Detention Pond Analysis model (WinDETPOND) include a computerized version of the storage-indication method.

Introduction To Storage-Indication Method

The pond routing calculation procedure presented in the remainder of this section is based on the Natural Resources Conservation Service Technical Release-20 (TR-20) procedures (SCS 1982), as presented by McCuen (1982). The reservoir routing subroutine in TR-20 (RESVOR) is based on the storage equation:

$$I - O = \frac{\Delta S}{\Delta T}$$

where I is the pond inflow and O is the pond outflow. The difference between the inflow and outflow must be equal to $\Delta S/\Delta T$, the change in pond storage per unit of time. McCuen presents a series of equations and their solutions that require the preparation of a "storage-indication" curve to produce the pond outflow hydrograph. The storage-indication curve is a plot of pond outflow (O) against the corresponding pond storage at that outflow (S) plus 1/2 of the outflow times the time increment. When the pond outflow hydrograph is developed, the upflow velocity procedure described earlier can be used to estimate pond pollutant removal and peak flow rate reduction performance.

Outflow Rates From Discharge Control Devices

The first step in using the storage-indication method is to determine the stage-discharge relationship for the pond under study. This relationship (the rating curve) is the pond outflow rate (expressed in cubic feet per second, or cfs) for different pond water surface elevations (expressed in feet). Figures 76 through 78 are approximate rating curves for several common outlet control weir types for water surface elevation ranges up to six feet above the weir inverts. As an example, Figure 76 shows six separate curves for different lengths of rectangular weirs (from two to 18 feet wide). At a water surface elevation of 2.5 feet above the bottom of the weir (stage), not the bottom of the pond, a three foot wide rectangular weir would discharge about 34 cfs, while a 12 foot wide rectangular weir at this same stage would discharge about 150 cfs. For most applications, other stage-discharge rating curves will need to be developed and used, especially for commonly used broad crested weirs or culverts.

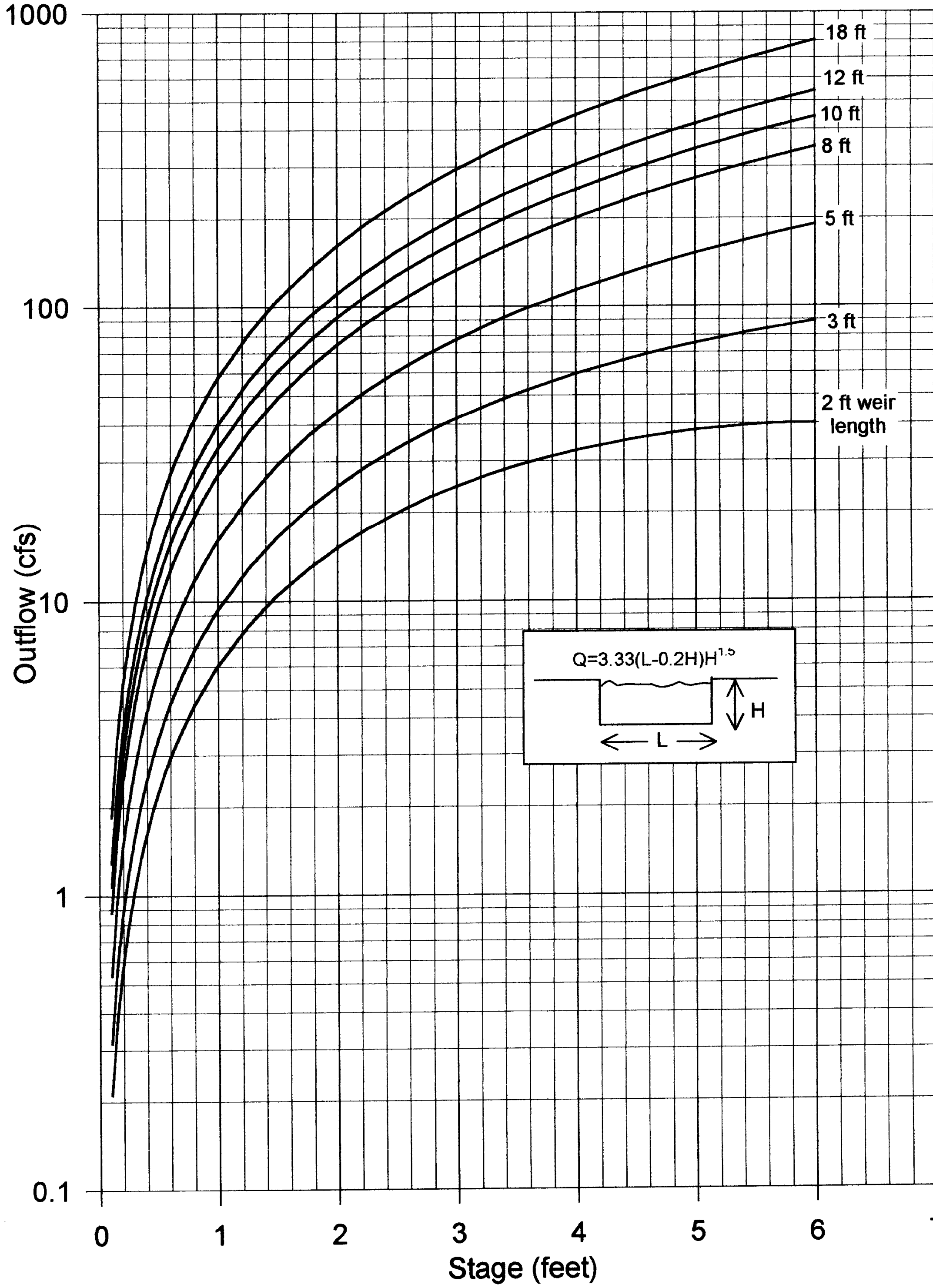


Figure 76. Approximate rating curves for rectangular weirs.

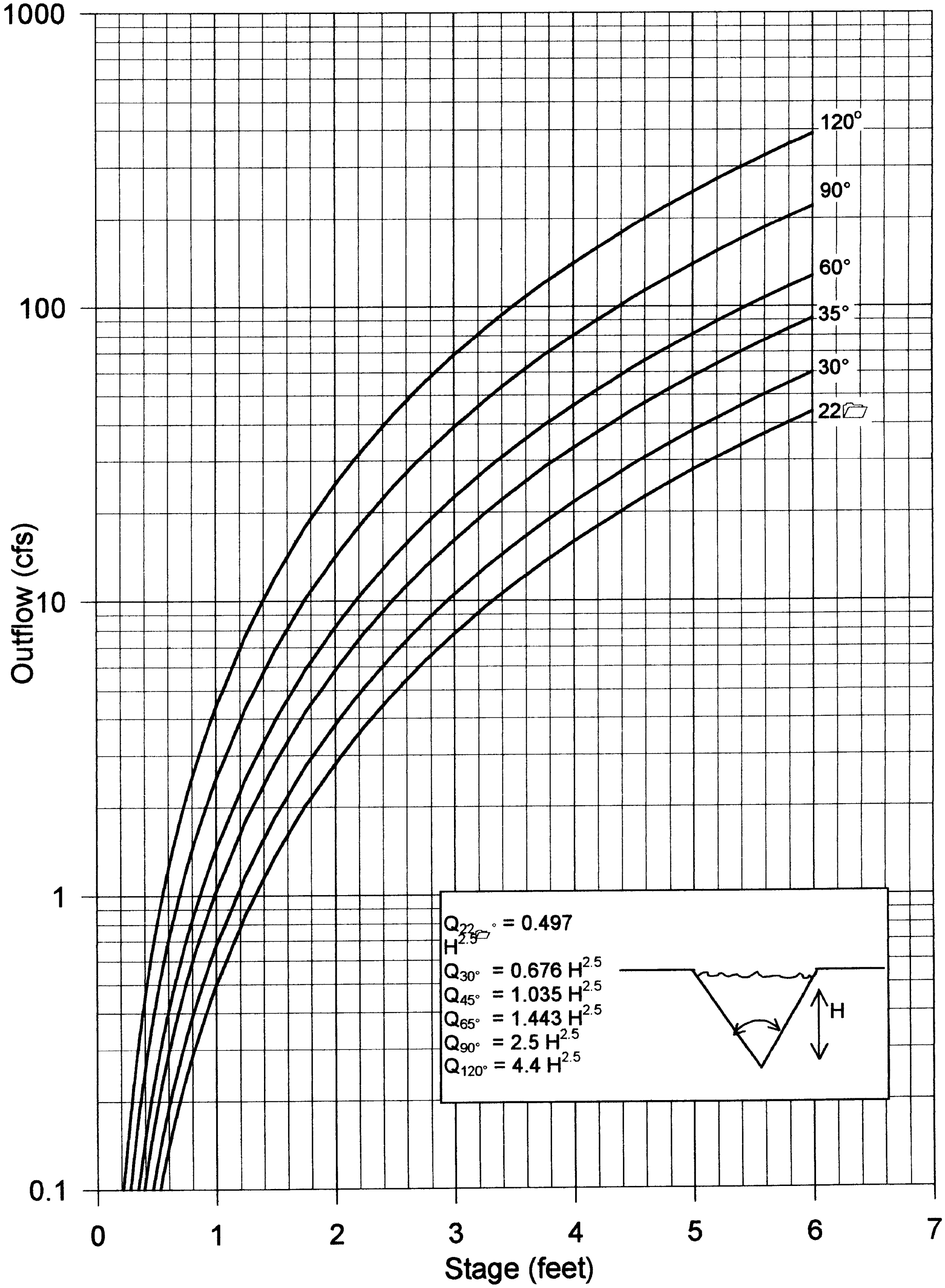


Figure 77. Approximate rating curves for V-notch weirs.

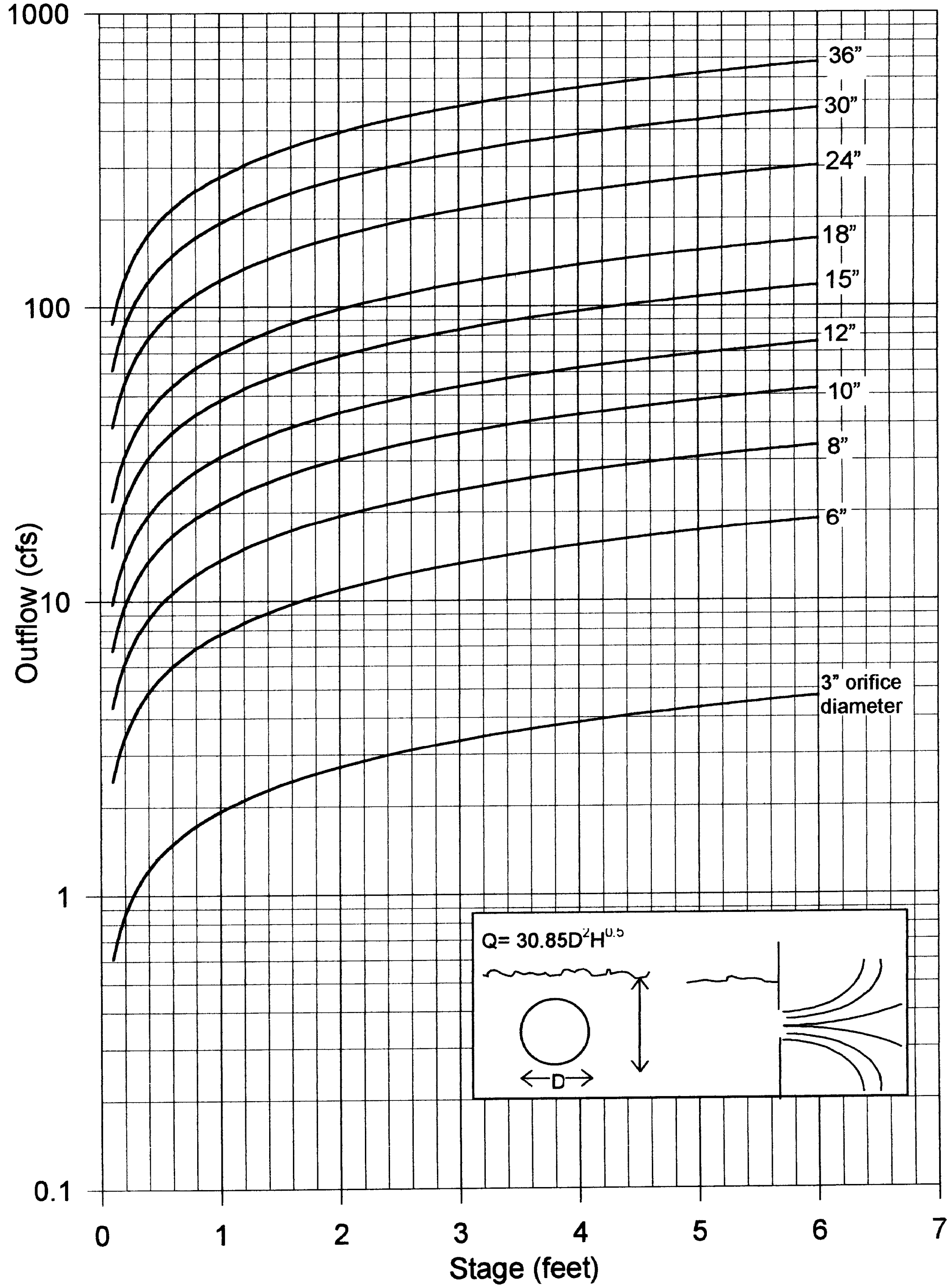
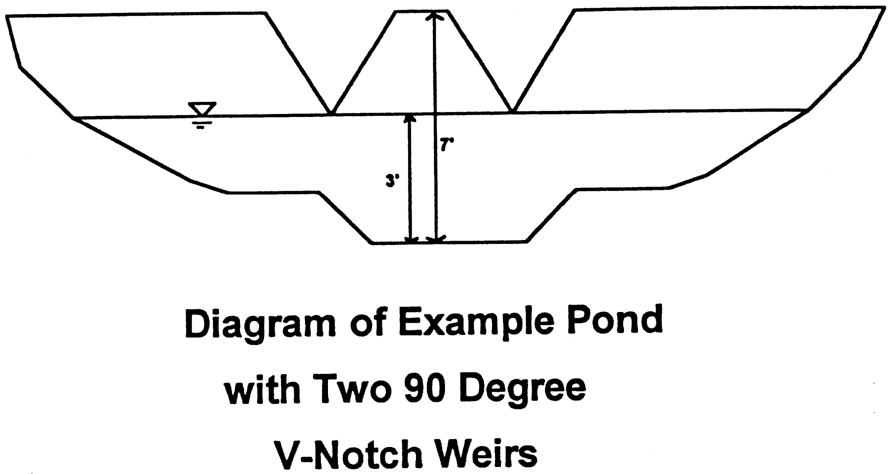


Figure 78. Approximate rating curves for orifice discharges.

Stage-Area and Storage-Indication Curve Development

The relationship between the pond stage and the surface area for the pond under study is also needed in order to calculate the storage volume available for specific pond stages. Figure 79 is an example stage-area curve developed from topographic maps of the Monroe Street detention pond in Madison, Wisconsin. The normal pond wet surface is at 13 feet (arbitrary datum) and the emergency spillway is located at 16 feet, for a resultant useable stage range of three feet.



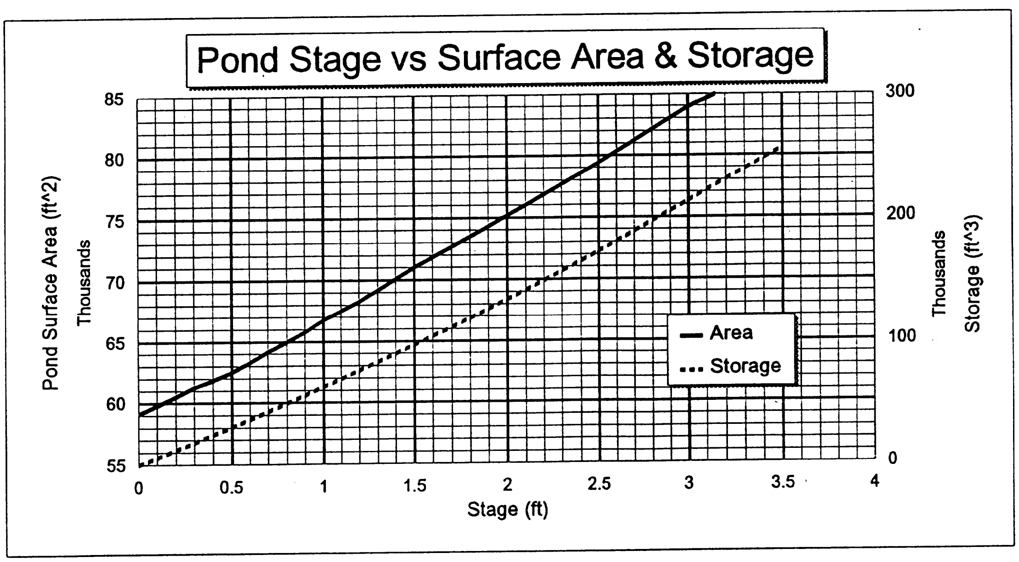


Figure 79. Pond-stage surface area relationship for example problem.

Table 26 shows the calculations used to produce the storage-indication figure (Figure 80) for the Monroe St. pond. This example assumes some pond modifications: two 90^{o} V-notch weirs, with a maximum stage range increased to 3.5 feet available before the emergency spillway is activated. The storage calculations assume an initial storage value of zero at the bottom of the V-notch weirs (13.0 feet). The time increment used in these calculations is ten minutes, or 600 seconds. The storage-indication curve shown as Figure 80 is therefore a plot of pond outflow (cfs) verses pond storage plus 300 (1/2 of 600 seconds) times the outflow rate. The storage-indication figure must also include the stage verses outflow and storage verses outflow curves (also from Table 26).

Table 26. Calculation of Storage-Indication Relationships for Example Pond and 1.5-Inch, 3-Hour Rain.

Datum Stage (H) (ft)	Discharge Rate[1] (O) (ft^3/sec)	Surface Area (ft^2)	Storage (S) (ft^2)	S + ½ OΔt (see footnote 2)
0	0	59,100	0	0
0.1	0.016	59,800	5,980	5,985
0.2	0.09	60,500	12,100	12,130
0.3	0.25	61,250	18,375	18,450
0.4	0.51	61,850	24,740	24,890
0.5	0.88	62,520	31,260	31,520
0.6	1.4	63,300	37,980	38,400
0.7	2.1	64,200	44,940	45,570
0.8	2.9	65,000	52,000	52,870
0.9	3.8	65,800	59,200	60,340
1.0	5.0	66,767	66,770	68,270
1.2	7.9	68,300	82,000	84,370
1.5	14	71,000	107,000	111,200
1.8	22	73,500	130,000	136,600
2.0	28	75,148	150,300	158,700
2.5	49	79,400	200,000	214,700
3.0	78	83,928	251,800	275,200
3.5	115	87,500	306,300	340,800

[1] Using two 90° V-notch weirs:

$Q = 2(2.5H^{2.5})$

[2] $S + ½ O \Delta t = S + O (½ \Delta t) = S + 300 (O)$

$\Delta t = 600$ seconds

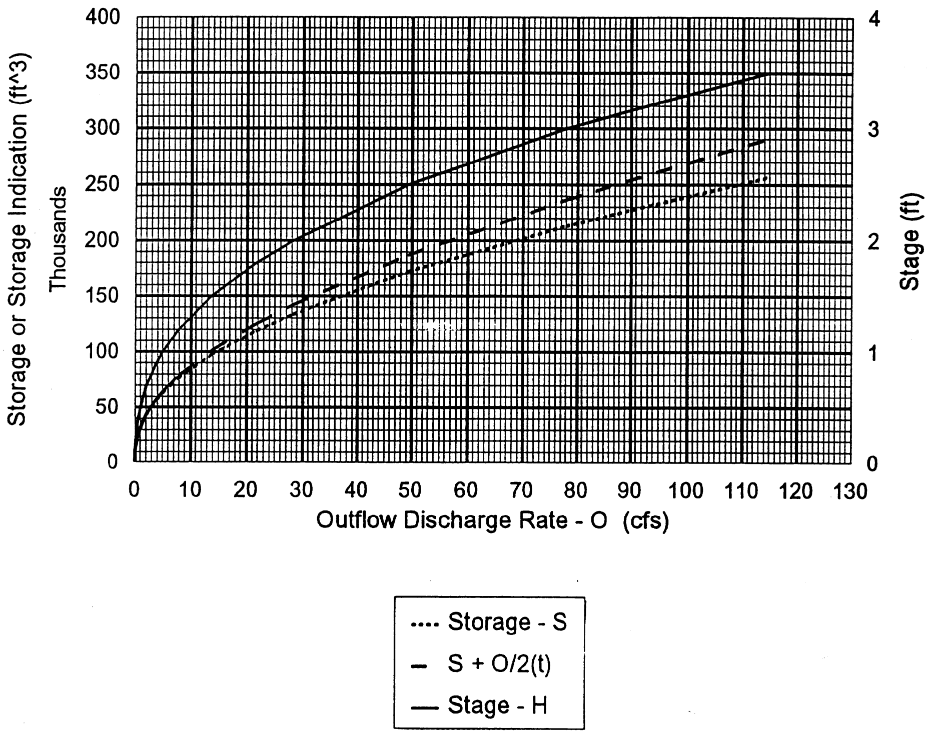


Figure 80. Pond-stage/storage indication curve for example problem.

Storage-Indication Calculation Procedure

Table 27 shows the calculations necessary to develop the pond outflow hydrograph and the plot of particle removal, for a triangular inflow hydrograph resulting from a 1.5 inch, 3-hour rain. Columns A through J of this table (to develop the outflow hydrograph and pond surface area) need to be calculated by rows (horizontally), while columns K through O (to calculate the upflow velocity and associated particulate removals) can be calculated vertically, based on the previously calculated column values. It should be noted that columns C through F are offset between the indicated time values and not for the specific times shown in column A. All of the starting values (time zero) in columns B (the beginning inflow rate), G (the beginning outflow rate), H (the pond storage volume above the normal wet pond water surface elevation), and I (the pond stage) are zero for this example.

Table 27a. Pond Performance Calculations for Example 1.5-Inch, 3-Hour Rain

A Time (min)	B Inflow (cfs)	C Average inflow for increment	D Average inflow volume (avg. inflow x time period)	E Previous storage minus incremental outflow S-0.5(O)Δt	F Previous storage plus incremental outflow S+0.5(O)Δt	G Outflow (O) (cfs)	H Storage (S) (ft^2)	I Pond stage (ft)	J Pond surface area (ft^2)
0	0					0	0	0	59,000
		4.5	2,700	0	2,700				
10	9					0.01	3,000	0.1	60,000
		13.5	8,100	2,997	11,100				
20	18					0.09	12,100	0.2	60,400
		22.5	13,500	12,073	25,600				
30	27					0.51	24,740	0.4	62,000
		31.5	18,900	24,590	43,490				
40	36					1.0	44,000	0.7	64,100
		40.5	24,300	43,700	68,000				
50	45					5.1	66,770	1.0	66,800
		50.0	30,000	65,240	95,240				
60	55					10	95,000	1.4	70,000
		59.5	35,700	93,500	129,200				
70	64					19	125,000	1.8	73,500
		68.5	41,100	119,300	160,400				
80	73					30	155,000	2.1	76,000
		77.5	46,500	146,000	192,500				
90	82					41	180,000	2.3	77,800
		86.5	51,900	167,700	219,600				
100	91					52	205,000	2.6	80,200
		95.5	57,300	189,400	246,700				
110	100					63	225,000	2.8	81,800
		95.5	57,300	206,100	263,400				
120	91					71	240,000	2.9	82,700
		86.5	51,900	218,700	270,600				
130	82					77	250,000	3.0	83,700
		77.5	46,500	226,900	273,400				
140	73					78	250,000	3.0	83,800
		68.5	46,100	226,600	267,700				
150	64					73	245,000	2.9	82,700
		59.5	35,700	223,100	258,800				
160	55					69	240,000	2.8	81,800
		50.0	30,000	219,300	249,300				
170	45					65	230,000	2.7	81,800
		40.5	24,300	210,500	234,800				
180	36					58	220,000	2.6	80,200
		31.5	18,900	202,600	221,500				
190	27					52	205,000	2.5	79,400
		22.5	13,500	189,400	202,900				
200	18					44	185,000	2.4	78,600
		13.5	8,100	171,800	180,000				
210	9					36	170,000	2.2	76,900
		4.5	2,700	159,200	162,000				
220	0					29	152,000	2.0	75,200
		0	0	143,300	143,300				
230	0					22	135,000	1.8	73,500
		0	0	128,400	128,400				
240	0					18	125,000	1.7	72,700
		0	0	119,600	119,600				

Table 27a. Pond Performance Calculations for Example 1.5-Inch, 3-Hour Rain (Continued).

A Time (min)	B Inflow (cfs)	C Average inflow for increment	D Average inflow volume (avg. inflow x time period)	E Previous storage minus incremental outflow S-0.5(O)Δt	F Previous storage plus incremental outflow S+0.5(O)Δt	G Outflow (O) (cfs)	H Storage (S) (ft^2)	I Pond stage (ft)	J Pond surface area (ft^2)
250	0					16	115,000	1.6	71,900
		0	0	110,200	110,200				
260	0					13	105,000	1.5	71,000
		0	0	101,100	101,100				
270	0					11	100,000	1.4	70,000
		0	0	96,700	96,700				
280	0					10	95,000	1.3	69,200
		0	0	92,000	92,000				
290	0					9	90,000	1.3	69,200
		0	0	87,300	87,300				
300	0					8	85,000	1.2	68,500
	Maximum = 100 cfs		Total = 660,000			Max. = 78 Total = 981			

Table 27b. Particle Removal Performance Calculations for Example 1.5-inch, 3-hr Rain

A Time (min.)	B Inflow (cfs)	G Outflow (O) (cfs)	J Pond surface area (ft^2)	K Upflow velocity (ft/sec)	L Critical particle size (μm)	M Weighted particle size (outflow x size)	N Percent suspended solids control	O Weighted control (outflow x control)
0	0	0	59,000	0	-	0	100	0
10	9	0.01	60,000	1.7×10^{-7}	0.3	0.003	100	1
20	18	0.09	60,400	1.5×10^{-6}	0.6	0.05	100	9
30	27	0.51	62,000	8.2×10^{-6}	1.3	0.66	99	50
40	36	1.0	64,000	1.6×10^{-5}	1.8	1.8	98	98
50	45	5.1	66,800	7.6×10^{-5}	3.8	19.4	91	464
60	55	10	70,000	1.4×10^{-4}	5.1	51	88	880
70	64	19	73,500	2.6×10^{-4}	7	133	84	1,596
80	73	30	76,000	4.0×10^{-4}	8	240	82	2,460
90	82	41	77,800	5.3×10^{-4}	10	410	78	3,200
100	91	52	80,200	6.5×10^{-4}	11	572	75	3,900
110	100	63	81,800	7.7×10^{-4}	12	756	73	4,600
120	91	71	82,700	8.6×10^{-4}	12	852	73	5,180
130	82	77	83,700	9.2×10^{-4}	13	1,000	71	5,470
140	73	78	83,800	9.3×10^{-4}	13	1,010	71	5,540
150	64	73	82,700	8.8×10^{-4}	13	949	71	5,180
160	55	69	81,800	8.4×10^{-4}	12	830	73	5,040
170	45	65	81,800	8.0×10^{-4}	12	780	73	4,750
180	36	58	80,200	7.2×10^{-4}	11	638	75	4,350
190	27	52	79,400	6.6×10^{-4}	11	572	75	3,900
200	18	44	78,600	5.6×10^{-4}	10	440	78	3,430
210	9	36	76,900	4.7×10^{-4}	9	320	80	2,880
220	0	29	75,200	3.9×10^{-4}	8	232	82	2,380
230	0	22	73,500	3.0×10^{-4}	7.5	170	83	1,830
240	0	18	72,700	2.5×10^{-4}	6.5	120	85	1,530
250	0	16	71,900	2.2×10^{-4}	6	96	86	1,380
260	0	13	71,000	1.8×10^{-4}	6	78	86	1,120
270	0	11	70,000	1.6×10^{-4}	5.5	61	88	968
280	0	10	69,200	1.5×10^{-4}	5.3	53	88	880
290	0	9	69,200	1.3×10^{-4}	5	45	89	800
300	0	8	68,500	1.2×10^{-4}	4.7	38	89	710
						Total = 10,468		Total = 74,576

Peak reduction factor: PRF = 1- [($Q_{o\ max}$)/($Q_{i\ max}$)] = 1 – [(78)/(100)] = 0.22
Weighted average critical particle size =[total (outflow x size)]/[total (outflow)] = 10,468/981 = 10.7 μm
Weighted average suspended solids control =[total (outflow x control)]/[total (outflow)] = 74,576/981 = 76%

Column A shows the times at ten minute increments for five hours (300 minutes) since the start of the runoff. Column B is the pond inflow hydrograph (instantaneous flow rates at each time increment). The calculation of the inflow hydrograph is shown on Table 28. The inflow runoff rates can be estimated using WinTR-55 for a design storm, or by any other method, or from an observed hydrograph. Table 28 shows how the example Monroe Street detention pond watershed is divided into these three major land surfaces and how the average runoff rates are calculated for the storms under consideration.

Table 28. Rain and Inlet Hydrograph Characteristics for Example

				Average cfs/acre			Average cfs (for total area)[1]								
Rain depth (in.)	Rain intensity (in/hr)	Rain duration (hrs)	Return frequency (years)	Imperv.	Pervious	Imper. To Pervious	Imperv.	Pervious	Imper. To Pervious	Total avg. flow (cfs)	Total flow volume (10^3 ft^3)	Runoff duration (hrs)	Peak 5-min. flow (cfs)	Time to peak flow (hrs)	Total storm volume as a fraction of pond base storage[2]
0.1	0.03	3	<1	0.013	0.0001	0.006	0.8	0.01	0.4	1.2	16	3.6	2.4	1.8	0.1
0.5	0.17	3	<1	0.16	0.0035	0.08	10	0.4	3	13	180	3.6	26	1.8	1.0
0.9	1.8	0.5	1	1.8	0.06	0.45	113	8	28	150	320	0.6	300	0.3	1.8
1.1	1.1	1	1	1.0	0.045	0.5	63	6	32	100	430	1.2	200	0.6	2.4
1.3	0.7	2	1	0.7	0.03	0.35	44	4	22	70	610	2.4	140	1.2	3.4
1.5	0.5	3	1	0.5	0.025	0.25	32	3	16	50	640	3.6	100	1.8	3.6
1.7	0.3	6	1	0.3	0.015	0.15	19	2	9	30	780	7.2	60	3.6	4.3
2.0	0.2	12	1	0.2	0.012	0.10	13	2	6	20	1,100	14	40	7	6.1
2.3	0.1	24	1	0.1	0.006	0.05	6	1	3	10	1,100	29	20	14.5	6.1
5.5	0.23	24	100	0.23	0.05	0.12	14	6	8	28	2,900	29	56	14.5	16

Areas for different land cover types:

Impervious area: 63 acres
Pervious area: 126 acres
Impervious area draining to pervious area: 63 acres

Pond base storage (normally wet volume) is about 180,000 ft^3.

Urban hydrographs can be represented with a simple triangular shape (as shown on Figure 81), with a peak runoff rate equal to about twice the average runoff rate and with the runoff duration about 20 percent longer than the rain duration (Pitt and McLean 1986). This simplification is reasonable for most small to intermediate rains, especially when the effects of a relatively large series of individual rains on a pond are to be evaluated statistically, instead of describing the pond performance associated with a single “design” storm. For larger rains, the ratio of the peak to average flow usually increases to about 3.4. This higher ratio can be represented using a multiple triangular hydrograph, similar to that used by the SCS. The peak flow rate in this example (1.5 inch, 3 hour rain) is assumed to be about 100 cfs and occurs at 1.8 hours into the runoff period. Of course, any hydrograph shape can be used in these calculations. This triangular shape is used in SLAMM as a simplification when evaluating very large numbers of storms. However, WinDETPOND is a more detailed detention pond program that allows any runoff hydrograph to be evaluated (if manually entered). Pond leakage, groundwater intrusion, evaporation, or any other additional water losses or inflows can be added or subtracted from the pond inflow hydrograph, if desired, and are included in the computer programs.

Column C shows the average runoff rates (cfs) for the two adjacent time increments. Column D shows the incremental incoming runoff volume (cubic feet) for each time increment (average inflow runoff rate, from column C, times the increment time, or 600 seconds). Column E shows the previous storage volume minus one-half of the outflow rate times the time increment (one-half of the outflow volume). The first value shown in this column (for the increment 0 to 10 minutes) is zero because the previous storage and outflow rate values (for time 0) are both 0: 0 - 1/2 (0) (600) = 0 - 0 = 0. The second value in column E (for the time increment 10 to 20 minutes) is: 3,000 - 1/2 (0.01) (600) = 3,000 - 3 = 2,997. Before this second value in column E can be calculated, the previous outflow rate (O) and pond storage (S) values (for time 10 minutes) must be calculated.

Column F is the Column E value plus the Column D value (increment inflow). The first value shown in Column F is therefore equal to the first value shown in Column D (2700 for this example). The second value in column F (for the time increment 10 to 20 minutes) is 8,100 + 2,997 = 11,100.

Column G (pond outflow rate, O) and column H (pond storage, S) also start as 0 values at time 0. Later values in these columns are obtained from the storage-indication curve, using the column F value for the previous time increment. The 2,700 value in column F (representing S + 1/2 (O) (dt)) is used in Figure 80 to obtain a corresponding pond outflow rate of about 0.01 cfs and a pond storage volume of about 3,000 cubic feet.

The stage values in column I are obtained from the stage-discharge curve (shown in tabular form on Table 26 for this example), using the corresponding outflow rates from column G. The pond surface area values are obtained from the stage-area curve (Figure 79), using the corresponding stage values from column I.

The particle removal calculations are based on the previously described upflow velocity method, using the “instantaneous” pond surface area values (from column J) and outflow rate values (from column G). Column K shows the upflow velocities (in feet per second) calculated by dividing the outflow rate values (column G) by the corresponding pond surface area values (from column J). Column L shows the sizes of the critical particles (the smallest particles that would settle below the bottom of the outfall structure and therefore be “retained”) and are estimated from Figure 80 based on these upflow velocities. Column M shows the outflow rate weighting of these particle sizes (critical particle size times the outflow rate). In this example, the "flow-weighted" critical particle size is about 11 μm.

Column N shows the estimated particulate residue percentage removals, based on a particle size distribution from Figure 80. Column O shows the flow-weighted calculations. For this example, a particulate residue reduction of about 75 percent may be expected.

The results of these calculations can be effectively presented on several graphs. Figure 81 compares the inlet and outlet hydrographs, Figure 82 shows the stage elevations above the permanent pool and the upflow velocities, and Figure 83 shows the critical particle sizes controlled and the estimated percentage control of particulate residue for this example.

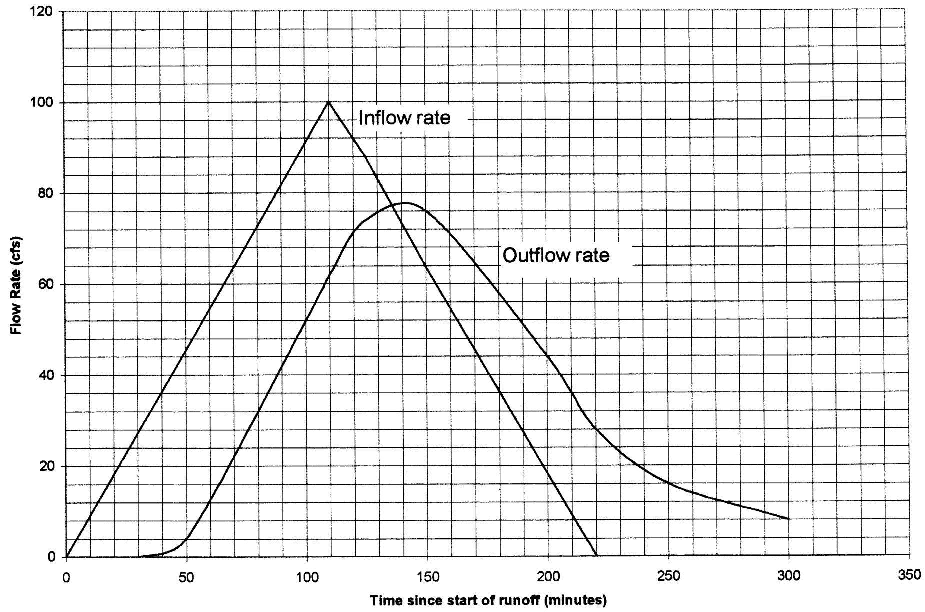


Figure 81. Inflow and outflow hydrographs for example problem.

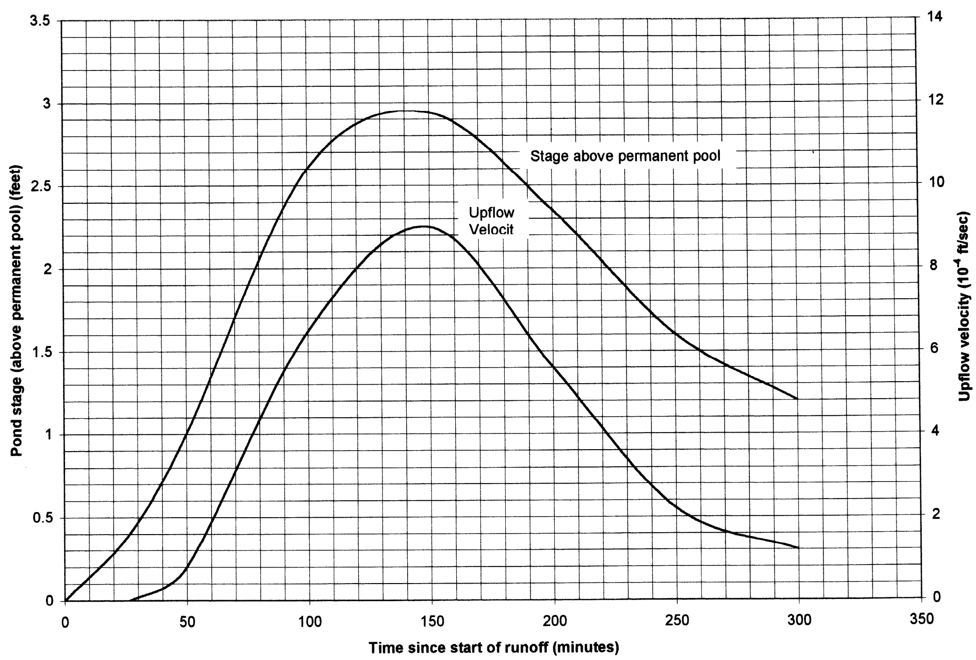


Figure 82. Stage and upflow velocity plots for example problem.

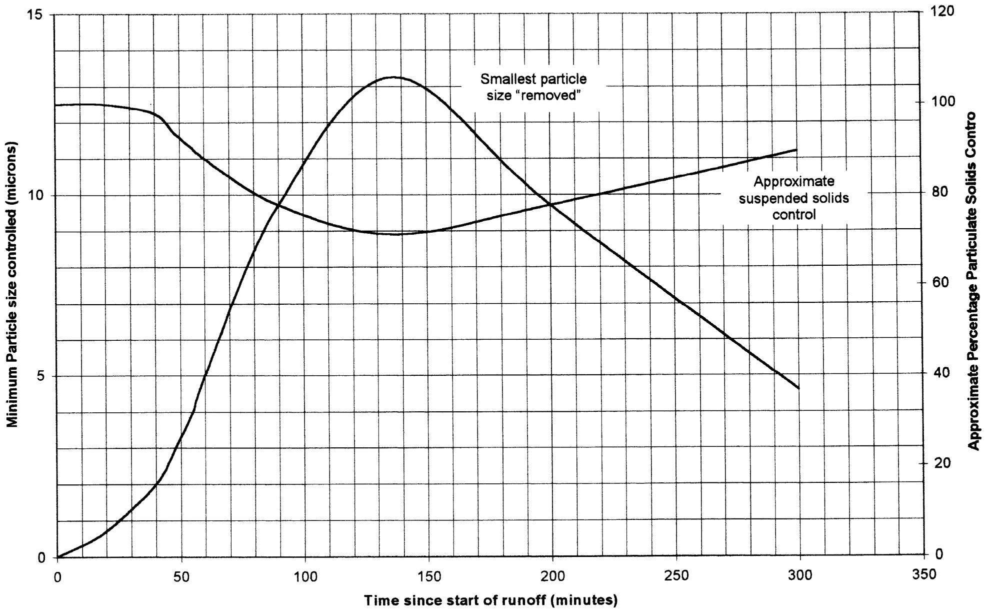


Figure 83. Particle sizes and percentage suspended solids removed for example problem.

Selecting Outflow Control Devices To Meet Water Quality Objectives

A simple analysis procedure can be used to guide the selection of an outflow control device for a given stage-surface area relationship for a potential pond location and desired particle size control objective. The definition of upflow velocity (outflow rate divided by surface area) allows the simple evaluation of detention pond performance for any pond stage. Similarly, if the pond stage-surface area relationship is known for a potential pond location, an outfall device can be selected to obtain control of critical particle sizes.

Tables 29 through 32 provide a quick method of selecting appropriate outfall devices for a pond. These tables indicate the minimum amount of pond surface area needed at each stage to provide a five µm critical control for a variety of conventional outfall devices. Table 32 presents multipliers to adjust the minimum areas for other critical particle sizes. In order to improve the pond performance by selecting a two µm critical particle size instead of five µm, the pond surface area would have to be increased by about 6.7 times. If the critical particle size was increased to ten µm, then the required pond surface would be reduced by about 0.27 compared to the pond surface areas needed for five µm control.

Table 29. Surface Area Requirements for 5-μm Particle Size Control for Various V-notch Weirs.

Head (ft)	Flow (cfs)	22.5° Storage (ac-ft)	Reqd. area (acres)	Flow (cfs)	30° Storage (ac-ft)	Reqd. area (acres)	Flow (cfs)	45° Storage (ac-ft)	Reqd. area (acres)
0.5	0.1	<0.01	0.01	0.1	<0.01	0.02	0.2	<0.01	0.03
1	0.5	0.03	0.1	0.7	0.05	0.1	1.0	0.05	0.2
1.5	1.4	0.1	0.2	1.9	0.2	0.3	2.9	0.2	0.5
2	2.8	0.3	0.5	3.8	0.3	0.7	5.9	0.6	1.0
3	7.8	1.2	1.4	11	1.6	1.8	16	1.6	2.8
4	16	3.3	2.8	22	4.4	3.8	33	5.9	5.8
5	28	7.2	4.9	38	9.6	6.6	58	14	10
6	44	14	7.7	60	18	10	91	27	16
	Flow (cfs)	60° Storage (ac-ft)	Reqd. area (acres)	Flow (cfs)	90° Storage (ac-ft)	Reqd. area (acres)	Flow (cfs)	120° Storage (ac-ft)	Reqd. area (acres)
0.5	0.3	<0.01	0.05	0.4	0.02	0.08	0.8	0.04	0.1
1	1.4	0.07	0.3	2.5	0.2	0.4	4.4	0.3	0.8
1.5	4.0	0.3	0.7	6.9	0.6	1.2	12	1.7	2.1
2	8.2	0.8	1.4	14	1.5	2.5	25	3.3	4.4
3	28	3.5	3.9	39	6.2	6.8	69	12	12
4	46	9.5	8.1	80	17	14	140	30	25
5	81	21	14	140	36	25	250	69	43
6	130	39	22	220	67	39	390	120	68

Table 30. Surface Area Requirements for 5-μm Particle Size Control for Various Rectangular Weirs.

Head (ft)	Flow (cfs)	2 ft. Storage (ac-ft)	Reqd. area (acres)	Flow (cfs)	5 ft. Storage (ac-ft)	Reqd. area (acres)	Flow (cfs)	10 ft. Storage (ac-ft)	Reqd. area (acres)
0.5	2.1	0.10	0.4	5.7	0.3	1.0	12	0.5	2.0
1	6	0.5	1.1	16	1.2	2.8	33	2.4	5.7
1.5	10	1.2	1.8	29	3.2	5.0	59	6.3	10
2	15	2.3	2.6	43	6.4	7.6	90	13	16
3	24	5.7	4.2	80	17	14	160	35	29
4	32	11	5.6	110	34	20	250	71	43
5	37	17	6.5	150	47	26	340	120	59
6	39	23	6.9	190	77	33	430	190	75
	Flow (cfs)	15 ft. Storage (ac-ft)	Reqd. area (acres)	Flow (cfs)	20 ft. Storage (ac-ft)	Reqd. area (acres)	Flow (cfs)	30 ft. Storage (ac-ft)	Reqd. area (acres)
0.5	17	0.8	3.0	23	1.0	4.1	35	1.5	6.1
1	49	3.7	8.6	66	5.1	12	99	7.3	17
1.5	90	9.9	16	120	13	21	180	20	32
2	140	20	24	190	27	32	280	40	49
3	250	54	44	340	72	59	510	110	89
4	380	110	66	510	150	89	780	220	140
5	520	190	91	710	250	120	1100	390	190
6	680	290	120	920	390	160	1400	610	250

Table 31. Surface Area Requirements for 5-μm Particle Size Control for Various Drop-tube Structures.

Head (ft)	Flow (cfs)	8" Storage (ac-ft)	Reqd. area (acres)	Flow (cfs)	12" Storage (ac-ft)	Reqd. area (acres)	Flow (cfs)	18" Storage (ac-ft)	Reqd. area (acres)
0.5	0.5	0.02	0.09	0.9	0.04	0.2	1.6	0.07	0.3
1	0.7	0.07	0.1	2.2	0.2	0.4	4.4	0.3	0.8
1.5	0.7	0.1	0.1	2.2	0.4	0.4	6.5	0.8	1.1
2	0.7	0.2	0.1	2.2	0.6	0.4	6.5	1.4	1.1
3	0.7	0.3	0.1	2.2	0.9	0.4	6.5	2.5	1.1
4	0.7	0.4	0.1	2.2	1.3	0.4	6.5	3.6	1.1
5	0.7	0.6	0.1	2.2	1.7	0.4	6.5	4.7	1.1
6	0.7	0.7	0.1	2.2	2.1	0.4	6.5	5.8	1.1
	Flow (cfs)	24" Storage (ac-ft)	Reqd. area (acres)	Flow (cfs)	30" Storage (ac-ft)	Reqd. area (acres)	Flow (cfs)	36" Storage (ac-ft)	Reqd. area (acres)
0.5	1.6	0.07	0.3	1.9	0.08	0.3	2.0	0.09	0.4
1	5.6	0.4	1.0	6.3	0.4	1.1	7.2	0.5	1.3
1.5	11	1.1	1.8	13	1.3	2.3	16	1.5	2.8
2	14	2.1	2.4	21	2.8	3.7	27	3.4	4.7
3	14	4.5	2.4	25	6.9	4.4	42	9.4	7.3
4	14	6.9	2.4	25	11	4.4	42	17	7.3
5	14	9.3	2.4	25	16	4.4	42	24	7.3
6	14	12	2.4	25	20	4.4	42	31	7.3

Table 32. Corrections for Needed Surface Areas for Particle Size Controls other than 5 μm.

Particle size for control (μm)	Typical percentage of particles larger than indicated size	Particle settling rate (cm/sec)	Required area multiplier, compared to 5 μm
1	100	1.5×10^{-4}	27
2	94	6×10^{-4}	6.7
5	88	4×10^{-3}	1.0
10	78	1.5×10^{-2}	0.27
20	62	6×10^{-2}	0.067
40	47	2×10^{-1}	0.02
100	28	8×10^{-1}	0.005

If a site had a surface area of 3 acres at two feet above the lowest invert level, a number of outlet devices could be used to provide at least five µm critical control:

- all V-notch weirs from 22.5^{o} through 90^{o} (but not 120^{o})
- only a 2 foot long rectangular weir
- all pipes from 8" to 24"

All stage levels therefore have to be examined and the most critical device selected that provides the desired level of control. In a similar manner, it would be possible to specify the shape of a pond (area versus stage) to closely match the natural topography with minimal required grading by selecting an outfall structure that provides close to the required outfall rates.

Wet Pond Design Criteria for Water Quality

A wet detention pond performance specification for water quality control needs to result in a consistent level of protection for a variety of conditions, and to allow a developer a large range of options to best fit the needs of the site. It must also be easily evaluated by the reviewing agency and be capable of being integrated into the complete stormwater management program for the watershed. It should have minimal effects on the hydraulic routing of stormwater flows, unless a watershed-wide hydraulic analysis is available that specifies the specific hydraulic effects needed at the specific location.

The following suggested specifications should meet these objectives under most conditions. However, the specific pond sizes should be confirmed through continuous long-term simulations using many years of actual rainfall records for the area of interest (such as possible by using WinDETPOND). These guidelines should therefore be considered as a starting point and modified for specific local conditions. As an example, it may be possible to provide less treatment than suggested by the following guidelines (Vignoles and Herremans 1996). The following guidelines were developed by Pitt (1993a and 1993b), based on literature information and on his personal experience.

1) The wet pond should have a minimum water surface area corresponding to land use, and desired pollutant control. The following values were extrapolated from extensive wet detention pond monitoring, mainly the EPA's NURP (EPA 1983) studies:

Percent of Drainage Area Required as Pond for:

Land Use	5 μm control	20 μm control
Totally paved areas	3.0 percent	1.1 percent
Freeways	2.8	1.0
Industrial areas	2.0	0.8
Commercial areas	1.7	0.6
Institutional areas	1.7	0.6
Residential areas	0.8	0.3
Open space areas	0.6	0.2
Construction sites	1.5	0.5

Two levels of control are shown, corresponding to the worst-case control of particles greater than 5 μm and greater than 20 μm. For most stormwater facilities, these would correspond to annual suspended solids controls of about 90 percent for the 5 μm particle size, and about 65 percent for the 20 μm particle size. These values are based upon early work done by Gene Driscoll for NURP (EPA 1983). During NURP, the use of stormwater detention ponds in residential areas was investigated. Ponds having surface areas between 0.5 and 1 percent of the drainage areas were found to provide about 90 percent control. As the runoff changes because of other land uses besides residential areas, the size of the wet pond must correspondingly change. These values are based on expected runoff volumes for typical development conditions and would therefore vary for different development practices (especially if drained using grass swales, or if have or extensive infiltration controls).

2) The pond freeboard storage should be equal to the runoff associated with a 1.25 inch rain for the land use and development type. It should be noted that this storage volume is associated with the runoff volume from a specific type of rain and not for a set runoff volume. This has the benefit of providing the same level of control for all land uses. As an example, many ordinances require capture and treatment of the first 0.5 inch, or 1 inch, of runoff for an area. Unfortunately, this has the effect of providing very uneven levels of control because of different rainfall-runoff characteristics for different land uses. As an example, a residential area may require a rain of about 1.50 inches to produce 0.5 inches of runoff. However, a commercial area, such as a strip commercial development, would only require a rain of about 0.6 inches to produce 0.5 inches of runoff. It is obvious that the residential area is providing treatment for a much more severe rain, with a correspondingly greater level of annual control, compared to the commercial area. Most would agree that this is the opposite of what may be desired. By requiring a set amount of control associated with a rain having the same re-occurrence interval, a more consistent effort and benefit is obtained throughout the community.

The following table summarizes the approximate runoff depths associated with 1.25 inches of rain for several curb and gutter drained land uses, based on Pitt's (1987) small storm hydrology procedures:

Land Use	Sandy Soil	Clayey Soil
Freeways	0.35	0.40
Totally paved area	1.1	1.1
Industrial	0.85	0.9
Commercial	0.75	0.85
Schools	0.2	0.4
Low density residential	0.1	0.3
Medium density residential	0.15	0.35
High density residential	0.2	0.4
Developed parks	0.5	0.6
Construction sites	0.5	0.6

Pitt (1987) found that currently used urban runoff volume prediction methods commonly result in inaccurate runoff volumes for the common small storms that are most responsible for annual pollutant discharges in urban areas. For sandy soil areas, this table shows that the runoff volume associated with 1.25 inches of rain can vary from a low of 0.1 inch for low density residential areas to a high of 1.1 inch for totally paved areas, such as a parking lot. The difference in runoff volumes for different land uses having sandy or clay soil conditions varies much more for land uses having larger amounts of pervious surfaces. For areas having less amounts of pervious surfaces, the runoff differences produced by similar land use areas for these different soil conditions varies less. If an area is drained with grass swales, has an unusual amount of disconnected roofs, or has extensive upland infiltration controls, then the runoff volume associated with a 1.25 inch rain would be much less than shown in the above table.

3) The selection of the outlet device for the wet detention pond. This outlet device must be selected based upon the desired pollutant control at every specific pond stage in the wet detention pond. This specification regulates the detention time periods and the "draining" period to produce consistent removals for all rains. The ratio of outlet flow rate to pond surface area for each stage value needs to be at the most 0.00013 ft^3 /sec/ft^2 for 5 μm (about 90 percent annual) control and 0.002 (ft^3/sec/ft^2) for 20 μm (about 65 percent annual) control. In practice, the desired pond surface area to stage relationship (simply the "shape" of the hole) is compared to the minimum surface areas needed at each stage for various candidate outlet structures. As an example, the following list summarizes the minimum surface areas needed for 5 μm particle control for different stage values. Also shown are the freeboard storage values below each elevation:

	45° V-notch		90° V-notch		24" pipe	
stage feet	storage acre-ft	surface acres	storage acre-ft	surface acres	storage acre-ft	surface acres
0.5	<0. 01	0.032	0.02	0.08	0.07	0.28
1.0	0.05	0.18	0.15	0.44	0.39	0.98
1.5	0.22	0.5	0.56	1.2	1.1	1.8
2.0	0.60	1.0	1.5	2.5	2.1	2.4
3.0	1.6	2.8	6.2	6.8	4.5	2.4
4.0	5.9	5.8	17	14	6.9	2.4
5.0	14	10	36	25	9.3	2.4
6.0	27	16	67	39	12	2.4

The largest stages above the normal wet pond depth may result in unsafe conditions for most wet detention ponds. A maximum depth of about 3 feet above the normal wet pond depth is recommended.

The selection of the outlet control device is based upon the concept of surface overflow rate. The surface overflow rate is equivalent to the settling velocity of a critical particle size. Particles that have greater settling velocities than the surface overflow rate will theoretically be retained in the detention pond. The surface overflow rate is defined as the ratio between the instantaneous discharge and the pond surface area. The advantage of using the surface overflow rate as a design criteria for detention ponds arises from the fact that flows to a detention pond are very irregular. The surface overflow rate is equivalent to the ratio of detention time to pond depth. Unfortunately, the use of detention time alone, as commonly used in many ordinances and design guidelines, is not adequate to describe theoretical settling. In addition, detention time is very difficult to define for a stormwater detention pond because of the highly variable flow rates. However, the use of surface overflow rate works well because the ratio of discharge to surface area is known, or can be selected, for every pond stage. At any depth in a detention pond, the surface area is known, based upon the shape of the pond. The selection of a discharge device is therefore made simple because it must provide less than the critical discharge rate for each stage, and corresponding surface area.

Figure 84 is a schematic showing a cross section of the pond. The area below the invert of the major control device is the dead storage and is provided to minimize scour of the retained particulates. The water quality storage volume in the detention pond is the volume associated with the runoff associated with a 1.25 inch rain. The topmost layer in the detention pond is additional storage that is provided for drainage benefits. This storage would be provided (with the appropriate additional outlet structure) only if a basin-wide hydraulic analysis has been conducted to insure that inappropriate interferences of the different flood hydrographs would not occur. Also, it is important to note that an emergency spillway must also be provided above the water quality storage area. Therefore, the additional storage for drainage benefits as shown in this figure would at least be provided to cover the range of stage of the emergency spillway.

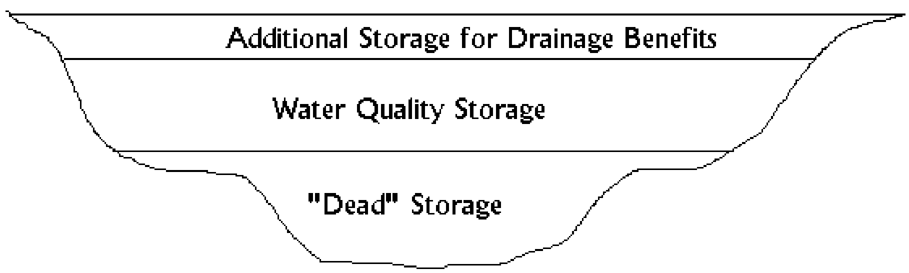


Figure 84. Cross-section of pond showing water quality storage portion

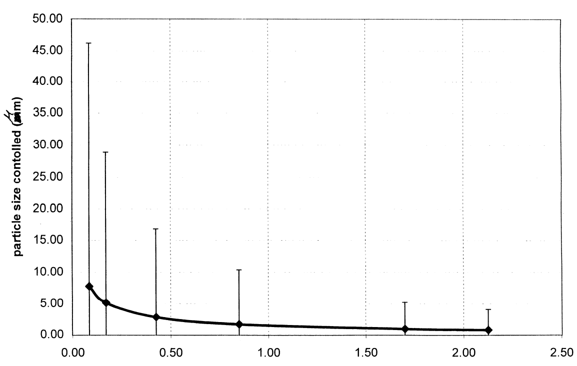
4) The ponds must also be constructed according to specific design guidelines to insure the expected performance and adequate safety. The guidelines need to specify such things as pond depth, side slopes, vegetation, and shape.

These procedures will result in the largest storms that do not enter the secondary spillway to have treatment levels equal to the critical particle size specified. As an example, the above calculations focus on the 5 μm particle, at least, being controlled at all stage depths of the primary outfall structures in order to provide 90 percent annual control of suspended solids. The outfall device is selected to provide an outfall rate no greater than a critical value, that when divided by the pond surface area at that stage, will be no larger than the settling rate of the critical particle size. In almost all cases, the critical stage will be at the top of the primary outfall device, and all stages below that will more than meet the critical objective, and will therefore be controlling particles much smaller than the critical size specified in the objective. It may seem that the pond is therefore over-designed and that the pond is larger than needed. However, the 5 μm critical particle size is typically substantially larger than the 90^{th} percentile particle size, and the added control provided at the lower stages in the pond is generally needed to provide this level of control on an annual basis. As indicated previously, the 90^{th} percentile particle size is typically only 3 μm, or smaller.

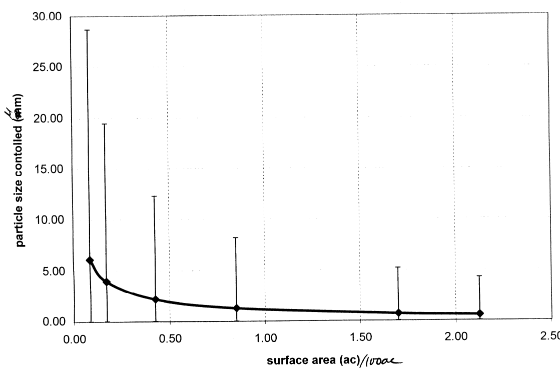
To check pond sizing criteria, a sensitivity analysis can be conducted using WinDETPOND, with varying pond sizes. WinDETPOND allows easy modifications of the pond surface areas by applying a multiplier to all surface area values. The model can then be re-run for each condition (after modifying the outlet structure to provide the critical flow rate at the pond stages). A typical set of plots are shown as Figures 85 and 86, for Austin, TX, and

Minneapolis, MN (prepared by John Easton, an UAB graduate student as part of a class project). These basic pond designs were based on the design criteria presented earlier and evaluated for several decades of recorded rain events. The ponds were then modified (making them smaller and larger than the basic design) to observe the effect on the overall performance. Figure 85 indicates the effect of different surface areas on the critical particle size controlled for commercial and residential areas for each of these cities. If the annual average control objective was 5 µm (indicated by the solid line), then the pond can be substantially smaller than if 5 µm was the worst-case control objective. The basic commercial ponds in both cities were sized to be 1.7 surface acres per 100 acres of drainage area (1.7% of the contributing areas), while the medium density residential area basic ponds were sized to be about 0.8 surface acres per 100 acres of drainage area (0.8% of the contributing areas). These plots show that all particles smaller than 5 µm particles would be controlled at these pond sizes in both cities for all rain events. However, the annual average particle size removals would be much smaller than five µm for these sized ponds (about 1 to 3 µm for these examples). This results in suspended solids controls of about 90%. The 90th percentile particle size (by weight) in stormwater was previously shown to be from about 0.8 to 8 µm, but more typically it is in the narrow range of about 2 or 3 µm. If the average control objective was for 5 µm particles and larger, then the ponds could be substantially smaller, but the suspended solids removals would then be much worse. An average control objective (instead of a worst-case control objective) of 5 µm would likely only provide about 50 to 75% suspended solids control.

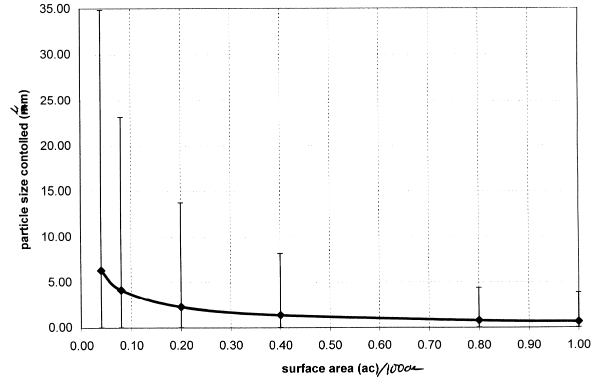
Figure 86 shows the number of events per year that would likely enter the secondary spillway (exceeding the storage capacity of the pond, based solely on the "water quality volume" of the pond). A reasonable goal for the use of the secondary spillway would be about twice a year. In these cases, the ponds in Austin exceed the base storage capacity much more frequently than the Minneapolis ponds for under-sized ponds, due to the differences in the rain characteristics at the two cities.



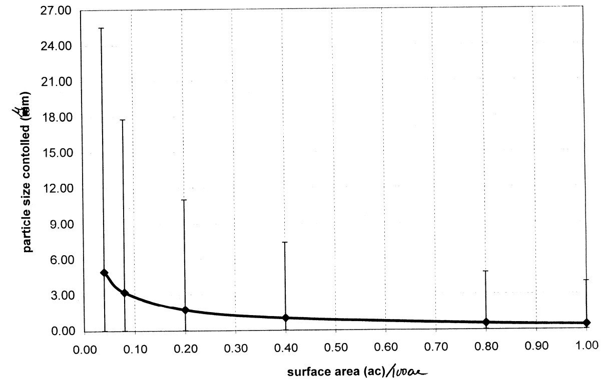
Austin, TX, commercial site sensitivity analysis based on surface area of pond.



Minneapolis, MN, commercial area sensitivity analysis based on surface area of pond.

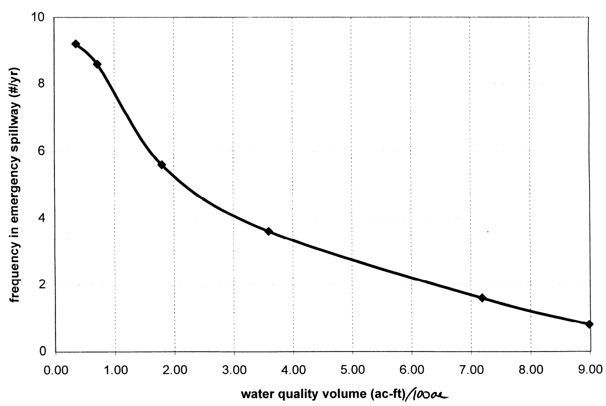


Austin, TX, medium density residential area sensitivity analysis based on surface area of pond.

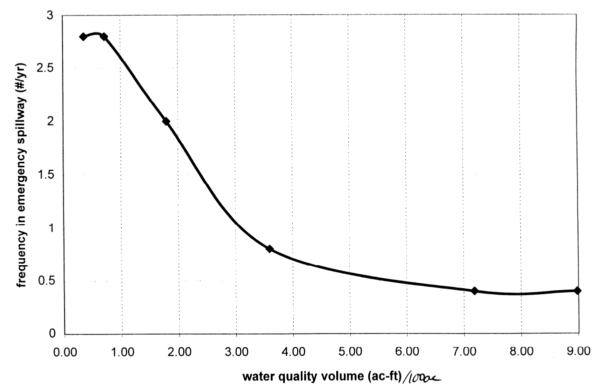


Minneapolis, MN, medium density residential area sensitivity analysis based on surface area of pond.

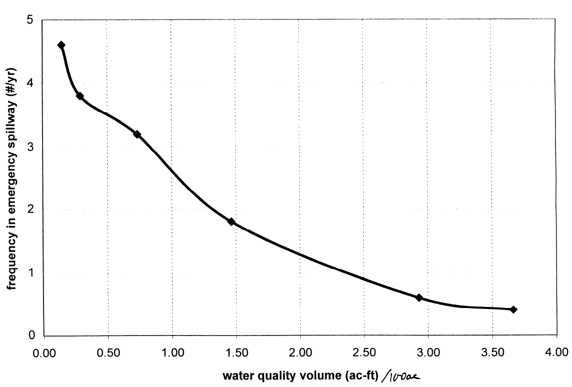
Figure 85. Sensitivity of pond area and level of treatment provided in wet pond.



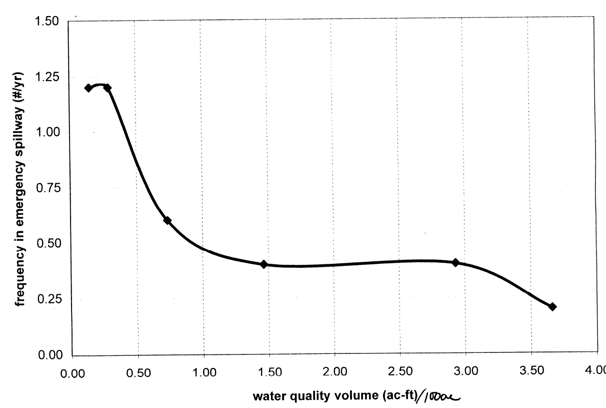
Austin, TX, commercial area sensitivity analysis indicating number of spillway events per year.



Minneapolis, MN, commercial area sensitivity analysis indicating number of spillway events per year.



Austin, TX, medium density residential area sensitivity analysis indicating number of spillway events per year.



Minneapolis, MN, medium density residential area sensitivity analysis indicating number of spillway events per year.

Figure 86. Sensitivity of pond volume and overlow into emergency spillway.

The Use of the WinDETPOND Program to Statistically Evaluate Wet Pond Performance

WinDETPOND was developed by Bob Pitt and John Voorhees to enable a continuous simulation of wet stormwater detention ponds. This continuous simulation is important to understand the storm to storm variation and long-term performance for typical rain conditions. The basic analysis procedures in WinDETPOND are similar to the detention pond analysis procedures provided in WinSLAMM, the Source Loading and Management Model, but offers additional model output choices to enable more detailed evaluations of detention facilities. Appendix A is a user's guide for WinDETPOND which also includes a simple design example. Additional assistance is provided in the "Help" components of the model (either the drop-down help menu, or one can use the F1 key for context sensitive help at any screen).

WinDETPOND uses conventional procedures to predict hydraulic conditions (pond storage-indication routing) and the behavior of particulates in stormwater as it passes through a detention pond (surface overflow rates described by the Hazen equation and quiescent settling using Stoke's and Newton's laws), as described in previous discussions. WinDETPOND was specifically designed for continuous long-term evaluations, using lengthy rain series. In its current Windows configuration, it is limited only by computer resources (and available time) in the number of rains that it can evaluate. It is also currently quite fast, requiring only a few minutes on most computers to complete a single run using several decades of rainfall data. Whereas most computer-based pond models require time increment direction from the user and frequently crash due to unstable algorithms, WinDETPOND predicts reasonable calculation increments based on the duration of each rain and interevent period. If the calculation appears to approach unstable conditions, it automatically starts over with a reduced calculation increment. In addition, if the pond design is too small or if the outfall is inadequate, causing catastrophic overflow conditions, the program doesn't crash, but continues using the last known outfall or surface area value, and notes that the pond overflowed. The tabular output of the model can also be easily imported into spreadsheets and graphing programs to produce statistical summaries of the pond performance.

WinDETPOND can therefore be easily used to evaluate an existing design or pond under a wide variety of rain conditions. It can be used with a single event (most commonly used when observed influent hydrograph data is available) or with a lengthy rain series (when the program predicts runoff and hydrograph characteristics).

Example Pond Performance Using Suggested Design Specifications and WinDETPOND

An evaluation of the performance of a pond was conducted using the above specifications for a wide range of Birmingham, Alabama, rains. This example illustrates how the pond performed for these varying conditions. The following list shows the pond dimensions used:

- 100 acre medium density residential area watershed
- 0.8 acre (35,850) pond (0.8 percent of 100 acres to result in a 5 µm, or 90 percent control of suspended solids).
- 5 feet wet pond depth during dry weather (to minimize scour and to provide sacrificial storage for sediments between pond dredging). This results in a storage volume of about 175,000 cubic feet below the invert.
- 0.5 inch of runoff freeboard storage, corresponding to 1.25 inch of rain.
- pond surface area and stage relationship, above the normal pond elevation:

stage (ft)	surface area (ft^2)
0	35,850
0.8	50,600
1.6	65,340
2.4	81,680
3.2	98,010

- 90° V-notch weir from 0 to 3.2 feet of stage (above normal wet pond depth), and a 20 foot long emergency spillway from 1.6 to 3.2 feet of stage.

WinDETPOND was used to investigate the performance of this pond for many local rains. Analyses showed that the pond stage barely reached the emergency spillway and the hydraulic effects of the pond were not significant for a typical Birmingham design storm (4.1 inch rain). The peak runoff flow rate for this event was not changed, and the assumed triangular inlet hydrograph shape changed very little (Figure 87). However, the pond had significant suspended solids reductions (Figure 88), even for this moderately large rain. The flow-weighted average performance of the pond was better than 90 percent removal of suspended solids, and the worst performance, occurring at peak flow rates, was only reduced to about 85 percent. The pond could have been designed to also provide appreciable peak runoff flow rate reductions, but that was not desired due to the lack of a basin-wide hydraulic analysis. Peak flow rate reductions in detention ponds are only obtained through extending the period of flow. If not carefully done, this extended flow period can easily increase downstream peak flow rates to greater values than if no detention was used.

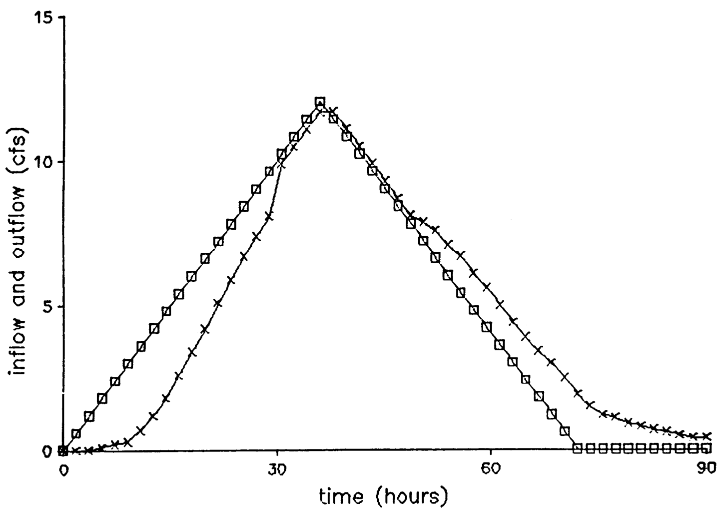


Figure 87. Modeled detention pond outflow hydrograph for 4.1 inch, 24-hour rain example.

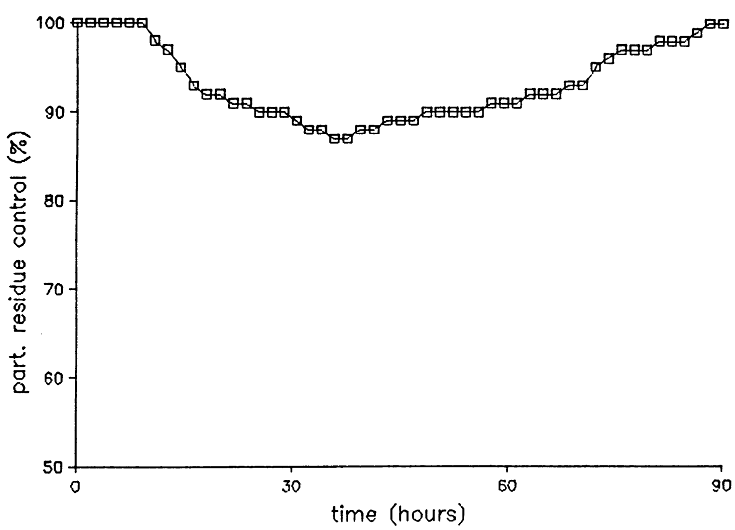


Figure 88. Modeled detention pond suspended solids removal performance for 4.1 inch, 24-hour rain example.

Pond performance was also modeled for many typical rain conditions (the 112 rains occurring during the 1975 Birmingham rain year) and for all major storms having 1 to l00-year frequencies and 1 to 24 hour durations. The pond achieved suspended solids reductions of greater than 86 percent for all typical events and achieved greater than 65 percent removals of suspended solids, even for the extremely intense 1 hour, 100-year event. Many of the drainage and flooding design storms had suspended solids removal rates of greater than 80 percent.

Figure 89 shows that the particle size control levels were closely related to rain intensity for the large storms, but were better related to rain depth for the less intense, typical rains. The typical rains all had similar rain intensities, narrowing the data scatter. Only two of the 112 storms in the 1975 rain year failed the 5 µm design criterion, and only by small amounts. The smaller rains all have much better removals than the 5 µm criterion. The median performance of the pond was greater than 95 percent control of suspended solids. Even for the extreme events, the detention pond should provide greater than a 65 percent control of suspended solids. Analyzing the extreme drainage and flooding rains is needed to check the adequacy of the emergency spillway. As noted, the initial designs for spillway capacity can be made using the procedures given in TR55, chapter 6 (SCS 1986) and briefly presented earlier.

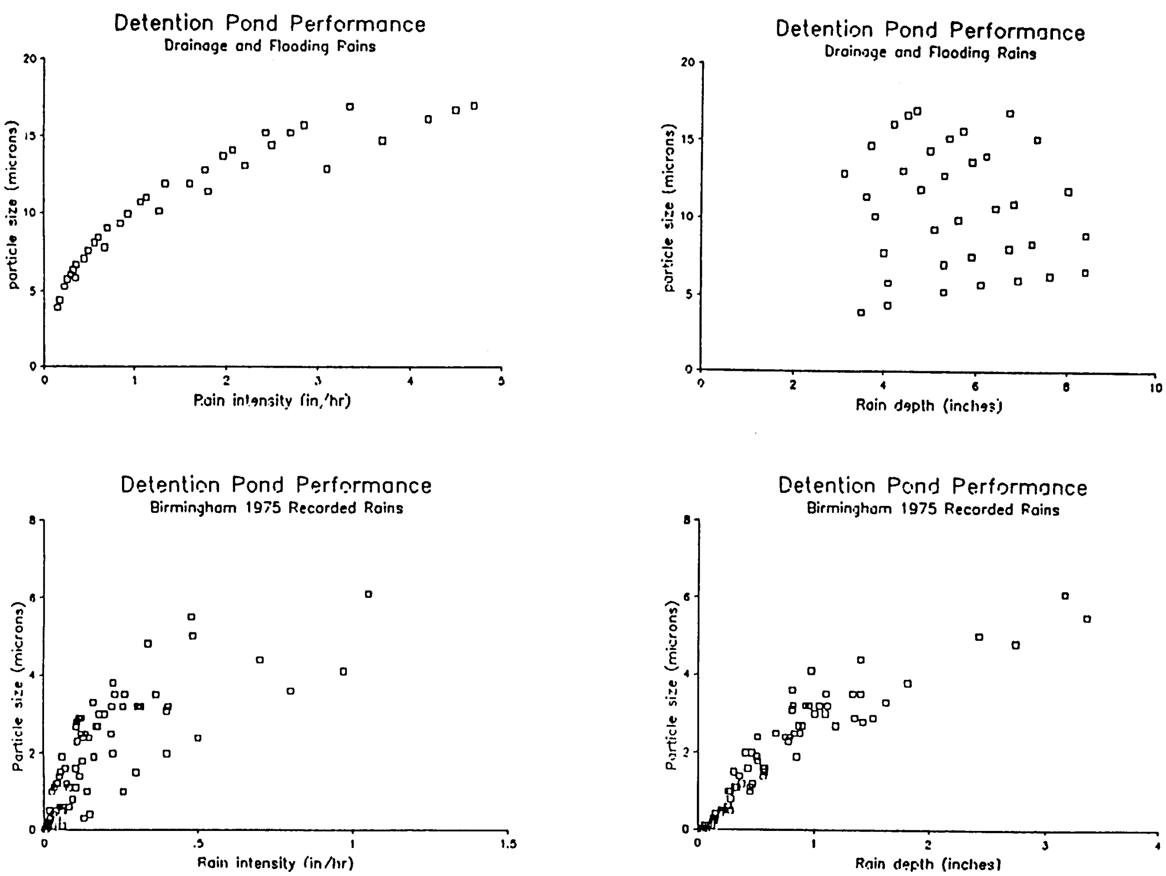


Figure 89. WinDETPOND modeled particle size removals by standard pond for various rain depths and intensities.

Figure 90 contains plots of the flushing ratios for the different rains. The flushing ratio is the ratio of the storm runoff volume to the pond storage volume below the lowest invert. A low flushing ratio indicates that much of the effluent from the pond is from the preceding dry period, while a high flushing ratio indicates that the pond may have been "blown out" during the event. Rain depth is the best indicator of flushing. Rains of about 1.5 inch in depth had runoff volumes about equal to the dry period storage volume. It is important to know the flushing ratio for a pond that is being monitored in order to understand the mixture of waters captured at the pond discharge. Consistently having low flushing ratios during most storms may indicate an over-sized pond, with unnecessary warming of the pond waters.

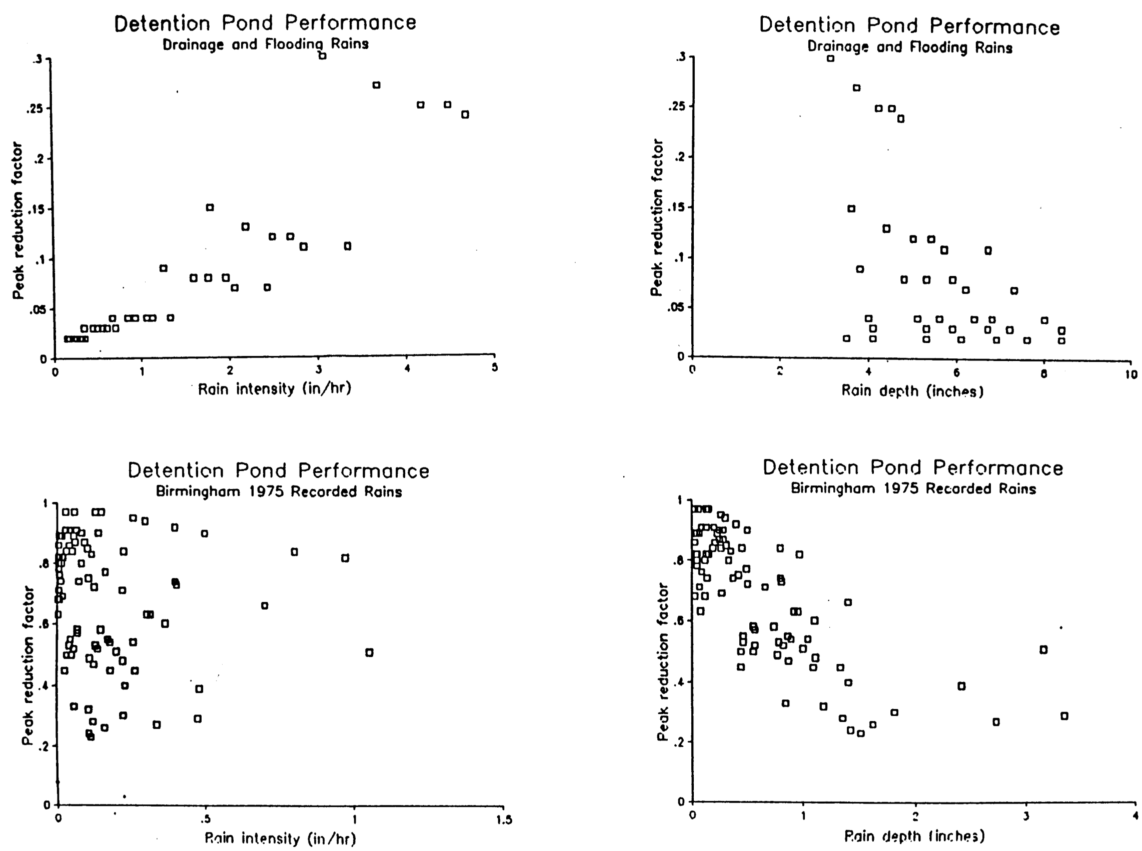


Figure 90. WinDETPOND modeled peak reduction factors by standard pond for various rain depths and intensities.

Figure 91 illustrates the relationships of maximum pond stage with rain. Like particle control, rain intensity was most important for larger rains, but rain depth was the better indicator of maximum stage attained during typical rains.

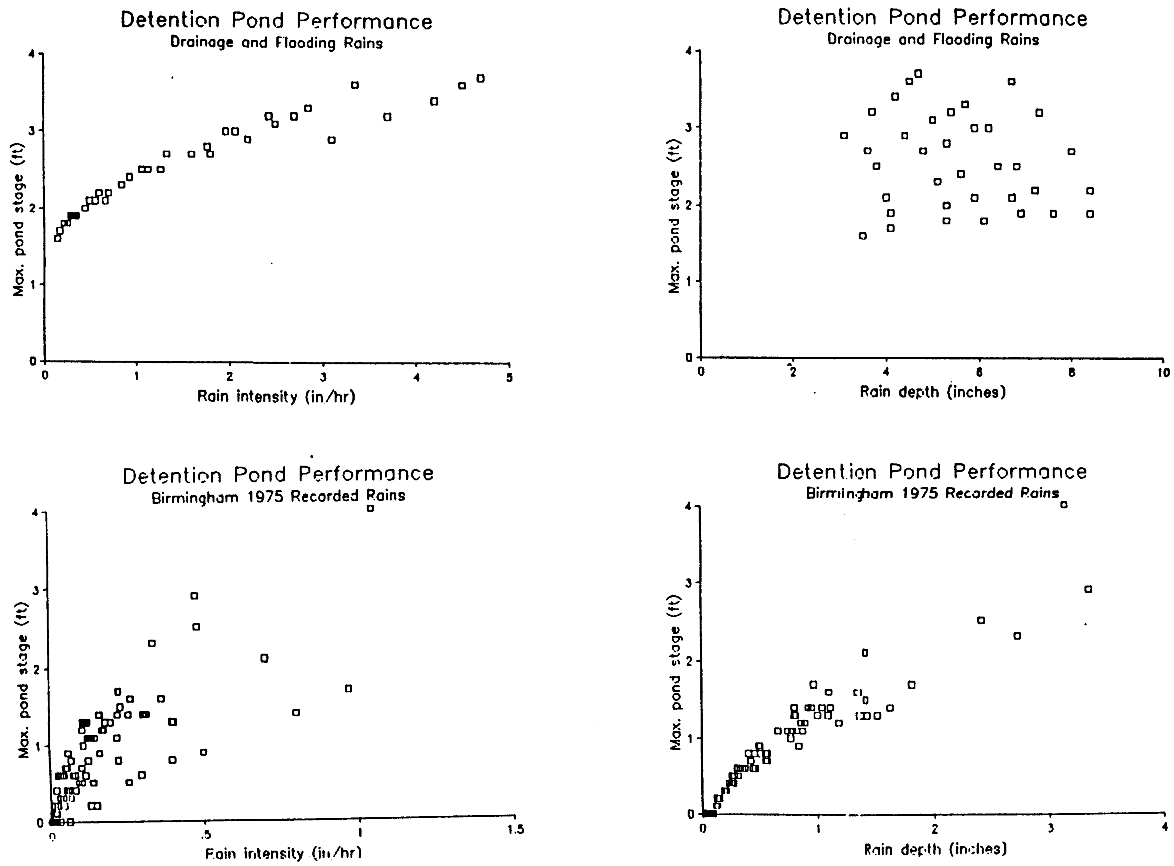


Figure 91. WinDETPOND modeled maximum stage of standard pond for various rain depths and intensities.

Figure 92 shows plots of the peak reduction factors (PRF) for the pond. Peak reduction factor is a measure of the peak flow rate reduction, comparing the effluent to influent peak flow rates. A PRF value of 0.5 indicates a 50 percent flow rate reduction, while a PRF of 0.9 indicates a 90 percent reduction in flow rates. PRF values are usually of most concern during major storms. These values were quite low during these events. The most intense rains only achieved PRF values of about 0.3. Water quality ponds should have minimal effects on flow rate, unless actual flow rate reduction objectives are available, based on basin-wide hydraulic analyses.

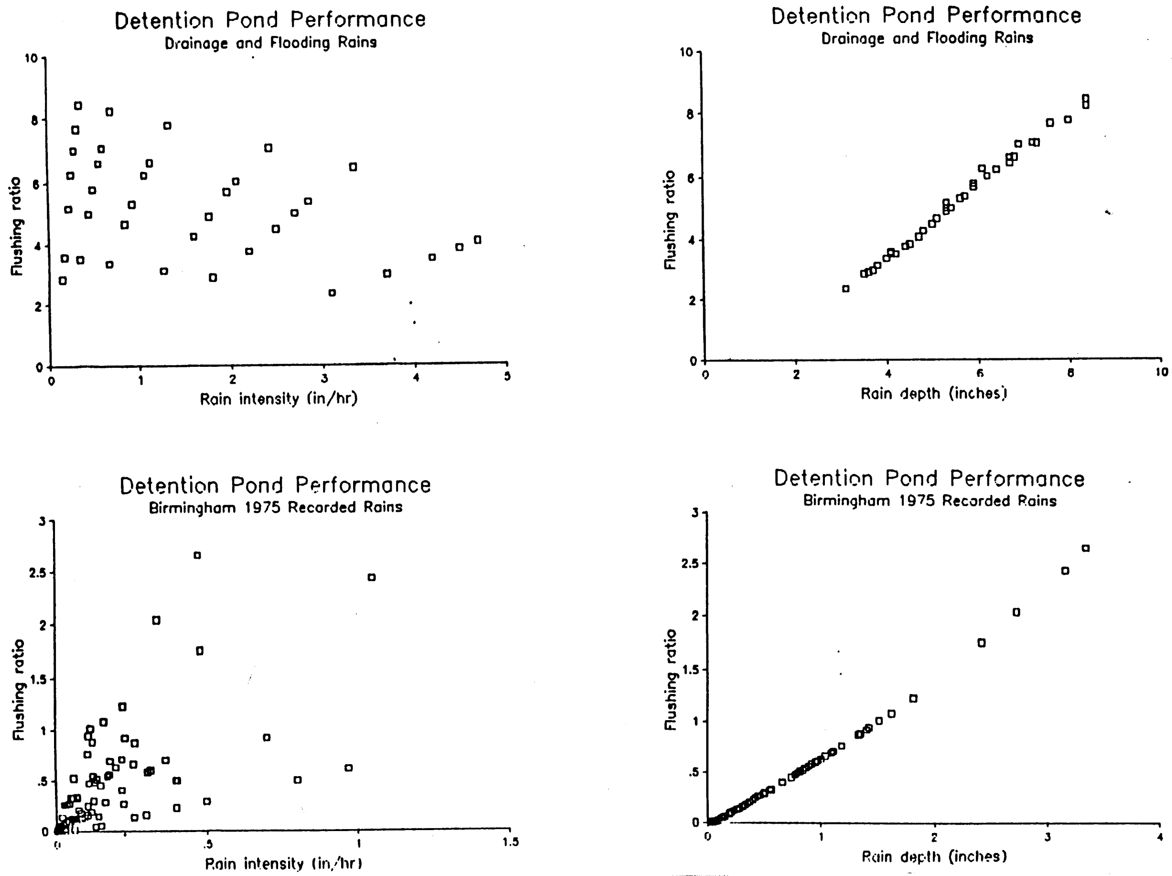


Figure 92. WinDETPOND modeled flushing rations of standard pond for various rain depths and intensities.

WinDETPOND Verification using Data Collected at the Monroe St. Detention Pond, Madison, WI

The USGS and the Wisconsin Department of Natural Resources have been monitoring the Monroe St. wet detention pond in Madison for a number of year (House, *et al.* 1993). The University of Wisconsin Arboretum originally constructed the pond to protect the water quality and ecology of Lake Wingra and surrounding wetlands from stormwater. The original pond was creating severe downstream erosion in the channels between the pond and the receiving water, and the pond storage volume was not effectively being used for either flood control or water quality benefits. The outlets were modified and the pond has undergone extensive monitoring to confirm the water quality benefits of the retrofit. Particle size distributions of influent (including bedload) and effluent have been monitored for about 50 storms. The actual particle size distributions and suspended solids removals have been compared to calculated pond performance, using WinDETPOND.

Figure 93 shows the location of the pond and the watershed. The pond is located on the downstream side of Monroe street at the outlet of a storm sewer that drains a 0.96-square km (237 acre) urbanized area. Land use in the watershed area consists mostly of single-family residences and commercial strip development, with some institutional uses (schools and churches). The average basin slope is 2.2 percent.

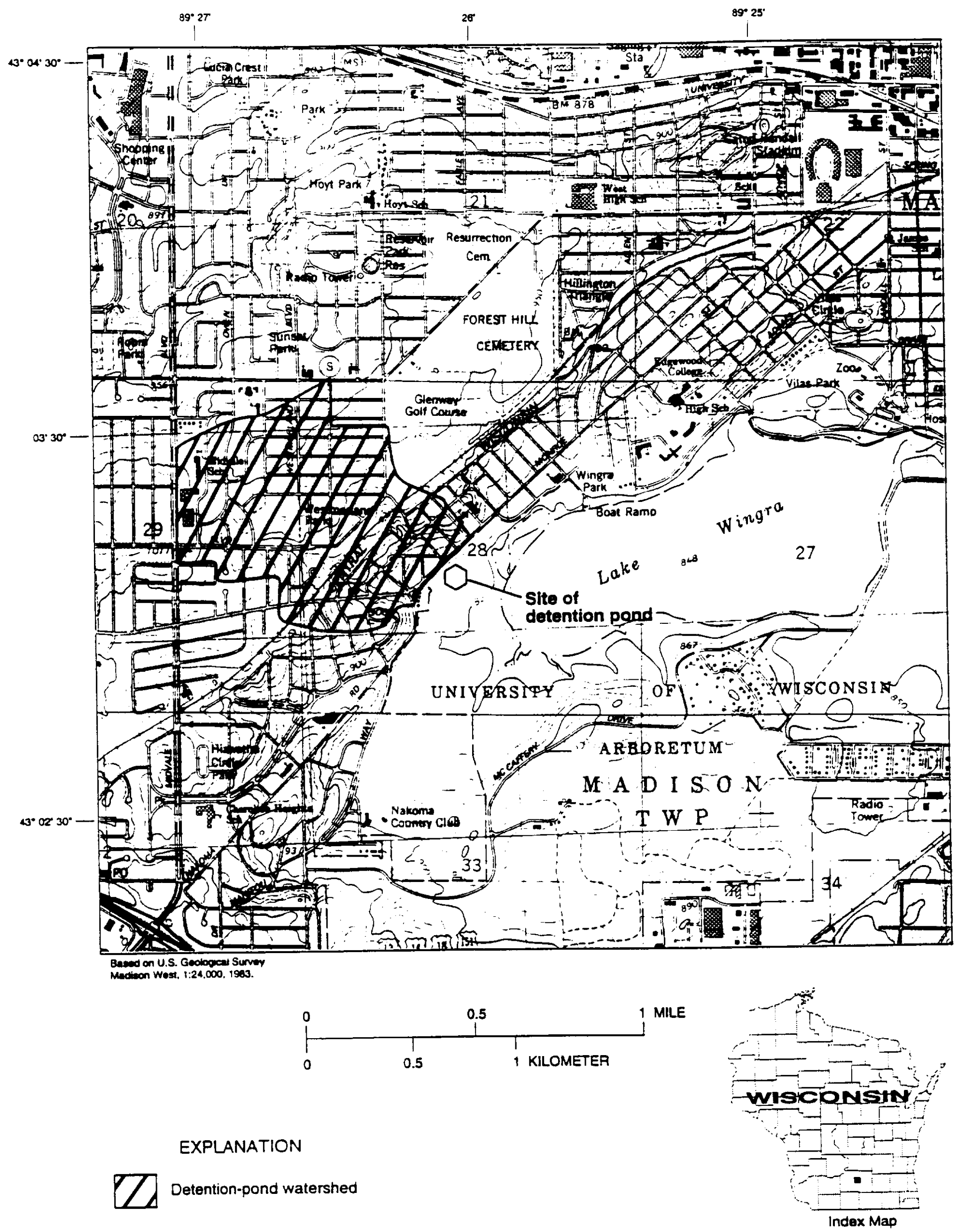


Figure 93. Monroe St. watershed area, Madison, WI.

The Monroe Street pond has a surface area of 5,670 m^2 (1.42 Acre), a maximum depth of 2.3 m (7.5 ft) and an average depth of 1.1 m (3.6 ft) at normal pool elevation. The shape of the pond is basically round to oval with a small island. The inlet side is nearest to Monroe Street and the two outlets are on the far side away from Monroe Street. Figure 94 shown the bottom contours of the pond. The pond has a surcharge storage volume above the normal pool elevation that is capable of holding the 10-year, 24-hour storm-runoff volume without overtopping the containment berm around the pond. Figure 94 is the pond stage-surface area curve. The pond has two outlets, each controlled by 90-degree V-notch weirs that drain to channels leading to Lake Wingra. The weirs are located in 8 ft. diameter concrete vaults, with 30 in. concrete pipes leading to the pond. The outlets in the pond are therefore submerged. Figure 96 is the pond composite outlet discharge curve. The bottom of the pond consists of a clay layer that inhibits infiltration of water from or into the pond.

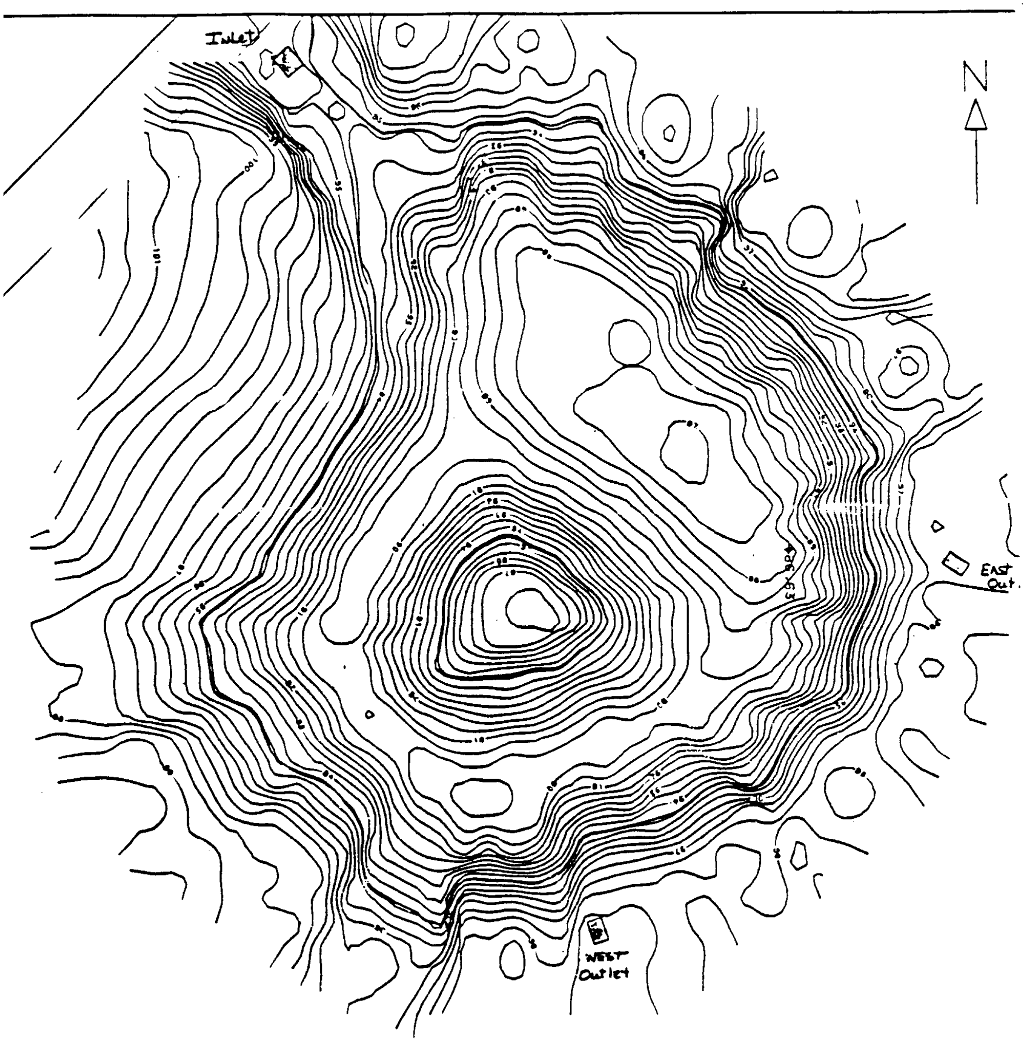


Figure 94. Monroe St. pond contour map.

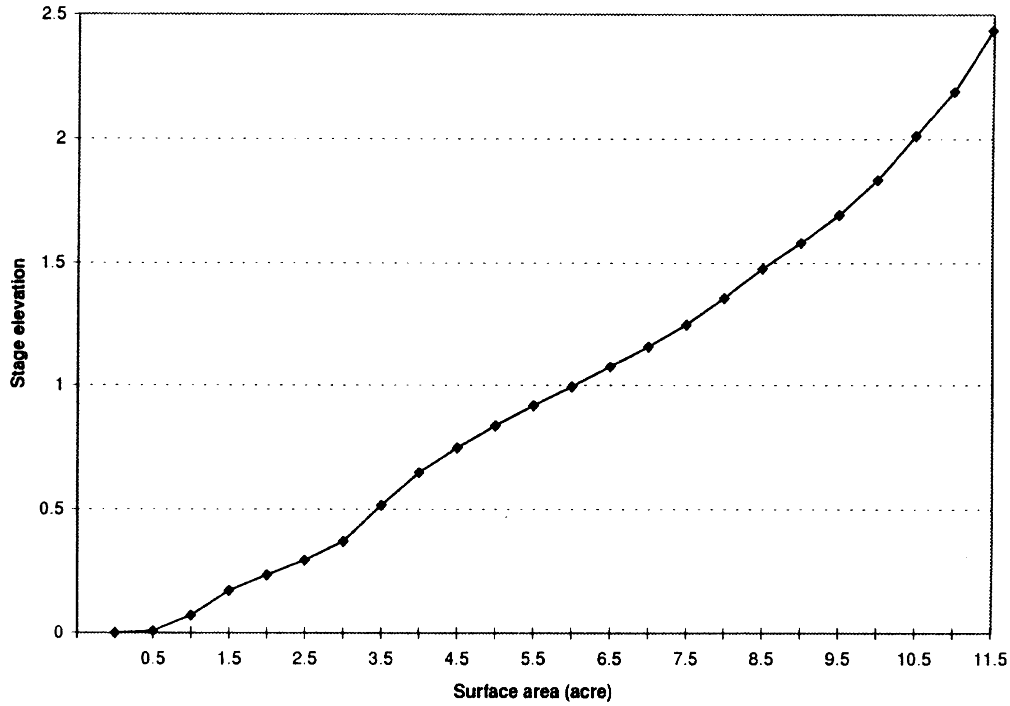


Figure 95. Pond-surface area curve for Monroe St. pond.

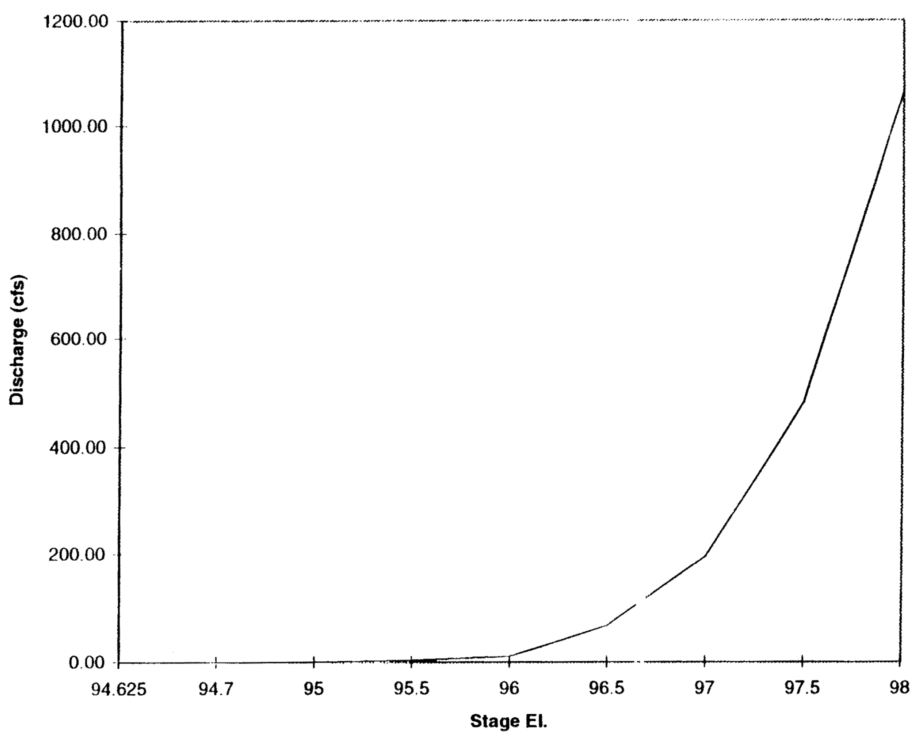


Figure 96. Stage-discharge curve for Monroe St. pond.

The initial primary outlet configuration consisted of two 8 ft. long rectangular weirs located in the vaults, made with concrete block walls. The original flow capacity of these two weirs was enormous, being about 50 cfs at 1 ft. head and 250 cfs at 3 ft. head. As noted above, the effluent peak flows from the pond were little attenuated from the influent flows and severe channel erosion was occurring in the wetlands, negating the sediment trapping benefits of the pond. The cleaner effluent also increased the sediment carrying capacity and channel erosion capabilities of the discharged water, further exasperating the problem. There was also no evidence that the emergency spillway was ever used since construction, even with several massive storms. In fact, the pond elevation barely fluctuated.

The outlets were therefore modified to reduce the downstream erosion problems by removing several courses of concrete blocks and installing 90-degree V-notch weirs made of plate steel in each vault. The pond normal water level was dropped about 6 inches with the lowered inverts. The new primary outlets have total flow capacities of about 5 cfs at 1 ft. head and 80 cfs at 3 ft. head. The pond surface fluctuates more now, and the emergency spillway has been active every few years. Most significantly, the downstream channels are now stable.

The pond was designed for an expected average 90% removal for suspended solids (particulate residue). The ratio of pond to drainage area is 0.6 percent. This percentage is close to the value (0.4% to 0.8%) required for 5 μm control for the land uses in the watershed, which generally corresponds to a 90 percent reduction of suspended solids.

A total of 64 events were extensively monitored between February 1987 and April 1988. The monitored rains varied from 2 to more than 82 mm during this period. Periodic water quality and flow monitoring has also continued at this pond since 1988.

Method of Investigation

The U.S. Geological Survey (House, *et al.* 1993) collected water-quality data by using programmable automatic water samplers (refrigerated), installed at the inflow and outflow sites of the pond. The outflow data was collected at two locations, east and west. The samplers were programmed to obtain flow-proportional samples for each storm. These samples represent the flow-averaged constituent concentrations during a runoff event. These samples were removed from the samplers, preserved, and shipped to the Denver USGS laboratory for analysis within 24 hours of being collected. The samples were analyzed for suspended solids, volatile total solids, total and dissolved chemical oxygen demand (COD), total chloride, total and dissolved phosphorus, phosphate total and dissolved forms of total Kjeldahl nitrogen (TKN), nitrates, and total and dissolved forms of copper, zinc, and lead. Particle size analyses were also conducted. The Wisconsin Department of Natural resources also installed bed load samplers to measure the amount of coarser material entering the pond. Most of the copper and lead data were too low for the analytical method used and are not reported here.

Precipitation data were also recorded at 5-minute intervals during the storm events using a recording rain gage located at the pond site. Storm runoff (pond inflow) was monitored at the box culvert that was the terminus of the 0.96-km^2 drainage area. Discharge rates and flow volumes passing through the culvert were determined by use of a flow velocity sensor and water level indicator installed inside the culvert. The velocity and depth sensors were connected to a data logger that recorded the water level and velocity data and computed discharge rates based on the culvert geometry.

Data Analysis and Observations

The pond inlet and outlet pollutant concentrations were analyzed to determine the pollutant reduction within the pond. Statistical analyses were used to investigate various relationships between inlet and outlet concentrations. Statistical analyses were also used to describe particulate pollutant strengths and percent controls. Each statistical process are described in the following paragraphs. The basic data are contained in the USGS report. (House, *et al.* 1993).

Hydrograph/Flow Calibration

An important part of the Monroe St. project was validating the WinDETPOND wet detention pond water quality model that was used to design the retrofit of the outlet structures (Pitt and Voorhees 1995). The first step in the validation was to check flow volumes and peak flow rates, and the complete hydrographs.

Fifteen storm events were used to validate the flow portions of the WinDETPOND program. WinDETPOND predicted outflow flow values from the inflow hydrographs using the storage-indication routing method. The outfall predictions (at 5 minute intervals) were compared to the observed outfall flow values. The predicted outflow hydrographs very closely matched the corresponding observed outflow hydrographs. In addition to comparing the general shape of the discharge hydrographs, the outflow total discharge volume, peak discharge flow rate, suspended solids removal, and outflow particle size distributions were also compared for validation. The predicted outflow volumes and peak discharges also very closely matched the observed outflow conditions. These comparisons are summarized on Figure 97 that compares the predicted and observed outflow volumes and the outflow peak flow rates. Figure 98 contains four of the actual outflow hydrographs, illustrating the close fits between the observed and modeled flows for highly different rain conditions.

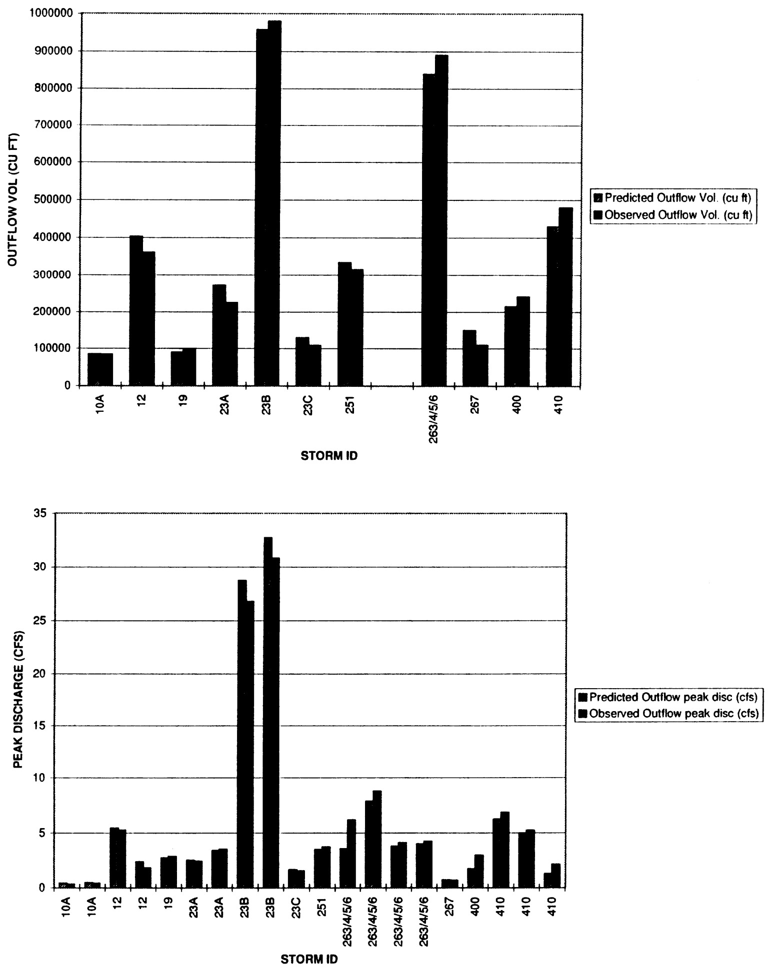


Figure 97. Predicted and observed flow volumes and peak flow rates.

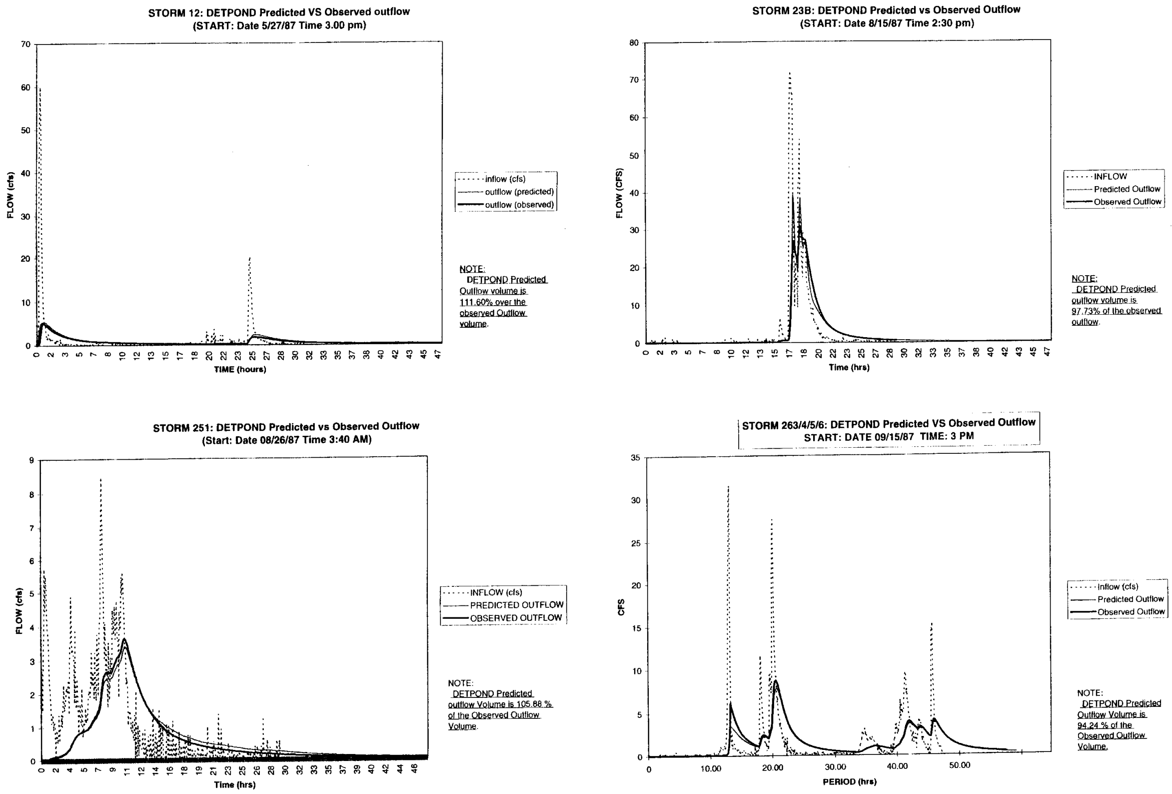
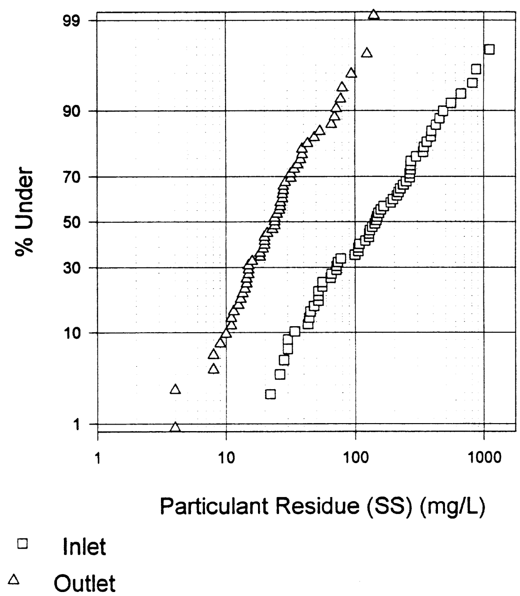


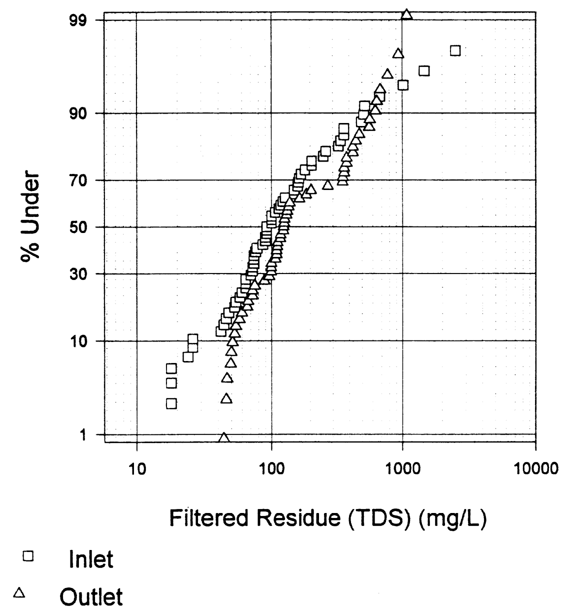
Figure 98. Predicted and observed hydrographs.

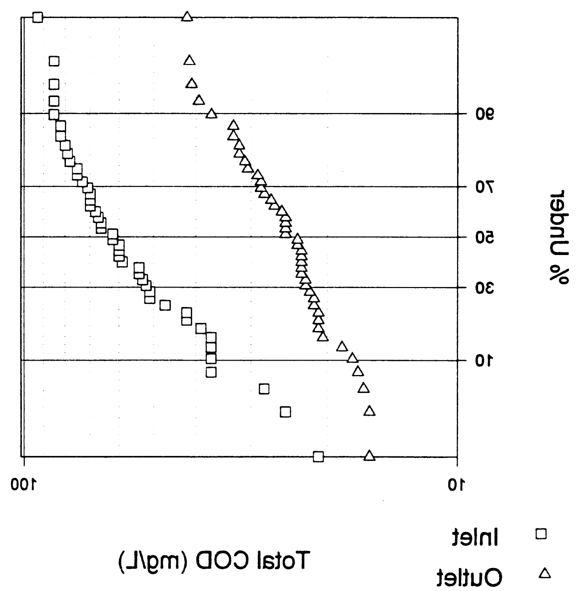
Observed Influent and Effluent Pollutant Concentrations. The following constituents had significant reductions between influent and effluent concentrations:

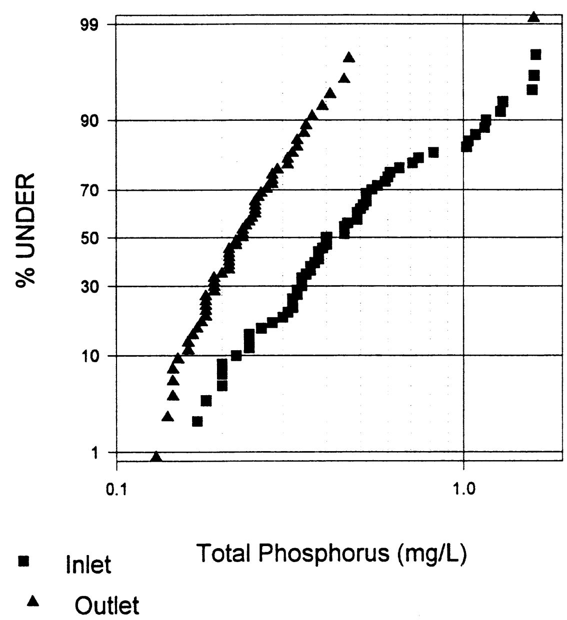
- Total solids
- Suspended solids
- Volatile solids
- Chlorides
- COD (all forms)
- Phosphorus (all forms)
- Phosphate
- TKN (total and particulate)
- Nitrate
- Copper (filtered)
- Zinc (all forms)

Probability plots of pond inlet and outlet pollutant concentrations are shown on Figure 99. Each constituent was found to be linearly distributed when plotted on log-normal plots. In most cases (except for TDS, chlorides, and filtered zinc), the inlet data values are plotted at higher values than the outlet data, indicating pollutant reductions in the pond.


99
90
70
50
30
10
1
% Under
1
10
100
1000
Particulant Residue (SS) (mg/L)
Inlet
Outlet


99
90
70
50
30
10
1
% Under
10
100
1000
10000
Filtered Residue (TDS) (mg/L)
Inlet
Outlet


90
70
50
30
10
% Under
100
10
Total COD (mg/L)
Inlet
Outlet


99
90
70
50
30
10
1
% UNDER
0.1
1.0
Total Phosphorus (mg/L)
Inlet
Outlet

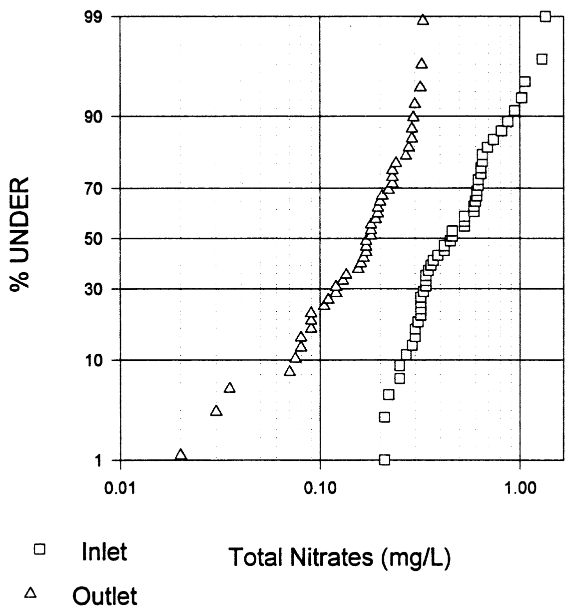


Figure 99. Probability plots of influent and effluent pollutant concentrations.

Particulate Pollutant Strength. Particulate pollutant strength (PPS) is the ratio of a particulate pollutant concentration to the suspended solid concentration, expressed in mg/kg. PPS was calculated for each pollutant with a particulate form and plotted on a probability versus strength chart (Figure 100). All pollutants had higher outlet than inlet PPS values due to preferential removals of large particles in the detention pond, leaving relatively more small particles in the discharge water. The small particles in stormwater have greater PPS values than the large particles. Wide differences indicates that the predominant components of the contaminant (such as for TKN and phosphorus) are associated with the fines that are not removed in the pond.

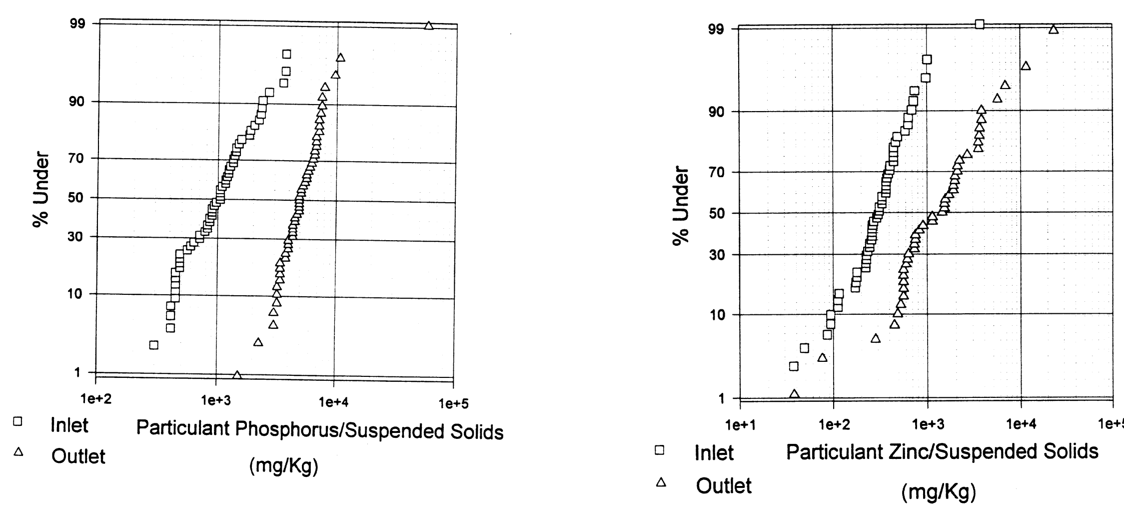


Figure 100. Particulate pollutant strengths.

Control of Pollutants. The reduction of pollutants was calculated from the difference in pollutant concentrations in the inlet and outlet water for each event, as shown on Table 33. As expected, control was higher for all particulate forms of the constituents than for filtered forms. Filtered constituents (<0.45 μm) are generally transported through the wet detention pond relatively unchanged, except for biochemical processes that may cause some precipitation or other reactions.

Table 33. Summary Table of Pollutant Control*

	10%	50% (median removal)	90%
Suspended solids	35	87	97
Total Residue	<0	52	86
Volatile Residue	<0	41	76
Filtered Residue	<0	<0	56
Particulate COD	15	80	95
Total COD	29	60	84
Filtered COD	<0	24	80
Particulate Phosphorus	-20	60	80
Total Phosphorus	<0	47	81
Filtered Phosphorus	<0	43	83
Particulate TKN	-40	40	80
Total TKN	<0	45	75
Filtered TKN	<0	12	68
Particulate Zinc	- 117	70	95
Total Zinc	<0	31	69
Filtered Zinc	<0	<0	59

*Copper and Lead observations were mostly below the detection limits and are therefore not shown.

Particle Size Distributions and Short-Circuiting

Seven events were studied to find the short-circuiting "n" factors using observed and predicted particle size distributions in effluent water. Particle size distributions were measured using the Sedigraph method at the USGS Denver laboratory. This technique measures settling rates of different size suspended solid particulates down to 2 μm. The value of n is calculated using the concentrations of large particles that are found in the effluent. In ideal settling, no particles greater than the theoretical critical size (about 5 μm for Monroe St.) should appear in the effluent. However, there is always a small number of these larger particles. It is generally assumed that short-circuiting is responsible for these large particles, although some monitoring in Canada has indicated that flocculation processes may also be responsible for the presence of large particles in ponds. The measured values for n were one, or less, indicating a high degree of short-circuiting in the pond. However, these observations were possibly affected by scour of bottom deposits near the subsurface effluent pipes. The maximum effect of short-circuiting on pond performance is shown in the following table, showing the average reduction in suspended solids removals for different n values, compared to the best performance (n value equal to 8):

n value	% SS removal (average)	reduction in % SS removal compared to n=8
8	85	
3	84	1
1	80.7	4.3
0.5	78.5	6.5
0.2	59	26

The calculated values of n (based on matching measured effluent particle size distributions with distributions calculated using different values of n) ranged from about 0.2 to 1, indicating "very poor performance", or worse.

The median value of n observed was about 0.35, indicating a degradation in annual average suspended solids capture efficiency of no more than about 10 percent. The effects of this short-circuiting, even with the extremely low values of n for Monroe St., only has a minimal effect on the suspended solids percentage removals. The Monroe St. pond provided an average suspended solids reduction of 87%, compared to the design goal of 90%. These values are quite close and the short-circuiting has a negligible effect on actual performance, as the pond surface is relatively large (0.6% of the drainage area) and the outlets were efficiently modified during the retrofitting activities.

Although the pond is producing very good suspended solids removals as designed, the particle size distributions of the effluent indicate some short circuiting (some large particles are escaping from the pond). The short circuiting has not significantly reduced the effectiveness of the pond (measured as the percentage of suspended solids captured). Therefore, care should be taken in locating and shaping ponds to minimize short circuiting problems, but not at the expense of other more important factors (especially size, or constructing the pond at all). Poor pond shapes probably cause greater problems by producing stagnant areas where severe aesthetic and nuisance problems originate.

Figure 101 shows the particle size distribution for the inflow events, including bedload. The median size is about 8 µm, but it ranges from about 2 to 30 µm. About 10% of the particles may be larger than 400 µm. The largest particle size observed was larger than 2 mm. The bedload added about 10% of the mass of these particulates and was associated with the largest sizes. The settling velocities of discrete particles can be predicted using Stoke's and Newton's settling equations. Probably more than 90% of all stormwater particulates (by volume and mass) are in the 1 to 100 µm range, corresponding to Laminar flow conditions. In most cases, stormwater particulates have specific gravities in the range of 1.5 to 2.5 (determined by conducting settling column, sieving, and microscopic evaluations of the samples, in addition to particle counting), corresponding to a relatively narrow range of settling rates for a specific particle size.

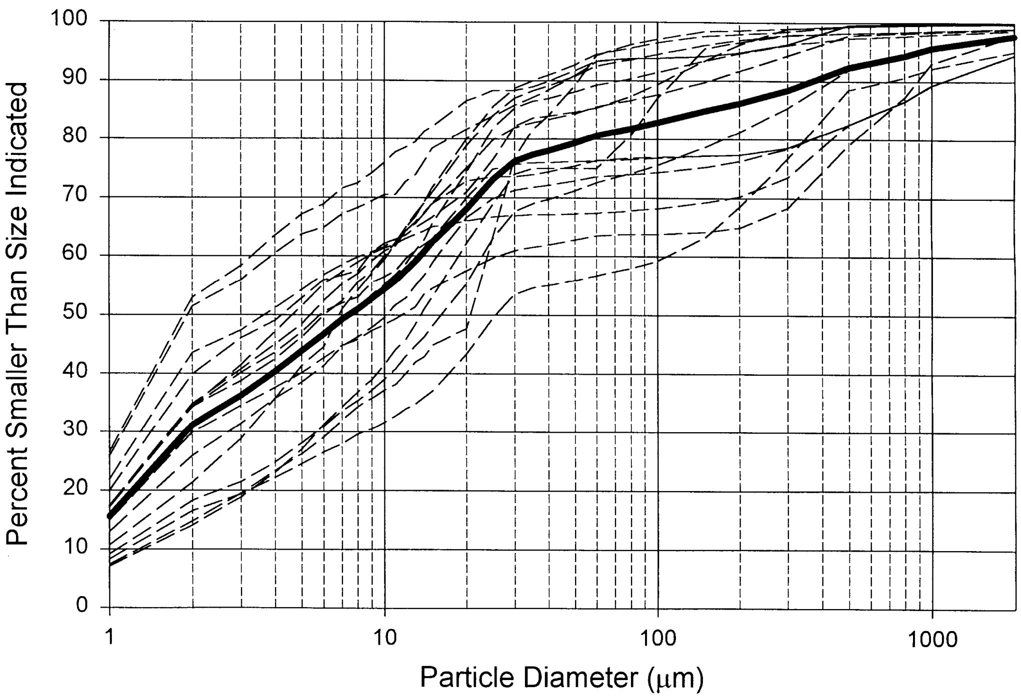


Figure 101. Inlet particle size distributions observed at the Monroe St. wet detention pond.

Monroe St. Pond Verification Conclusions

WinDETPOND successfully predicted the hydraulic, water quality, and particle size control at the Monroe St. detention pond in Madison, WI. In addition, WinDETPOND was successfully used to modify the outlet structure at the pond to enhance the pond's performance. The retrofitting of the Monroe St. wet detention pond was very successful. Changing the outlet structures from large rectangular weirs to v-notch weirs significantly reduced effluent flows and reduced downstream channel erosion. The modification also improved the water quality benefits of the pond.

All constituents had outflow concentrations lower than associated inlet concentrations, except for chlorides, TDS, and filtered zinc. Suspended solids had a median removal of 87%, the median particulate COD removal was 60%, the median removal for total forms of the nutrients (TKN and phosphorus) were 40 to 45% and the median removal for total zinc was 30%. (The median particulate zinc removal was 70%). A well-designed wet detention pond will remove 70 to 90% of suspended solids, 70% of COD, 60 to 70% of nutrients and 60 to 95% of the particulate forms of the heavy metals. The measured short-circuiting factor indicated a severe short-circuiting problem, but that could be a false indication due to minor scour near the effluent works in the pond. The Monroe Street pond is meeting all reasonable expectations in both downstream channel protection and in contaminant capture.

Verification Based on Measured Performance at a Landfill Pond in Birmingham, AL.

Another verification of the design criteria and the WinDETPOND model is available form the MSCE thesis prepared by Robert Creel (*Evaluating Detention Pond Performance with Computer Modeling Verification*, Dept. of Civil and Environmental Engineering, University of Alabama at Birmingham, AL. 1994, 137 pgs). Figure 102 shows the complete 41.3 ha drainage area and the pond. The drainage area has 20.3 ha of bare disturbed soil (the active landfill site), 4 ha of paved highways, and 13.3 ha of mature hardwood forests. The pond requires up to 2 ha for operation when completely full. Figure 103 is a schematic of the pond, showing the small isolated pre-settling pond (0.1 ha) at the upper end of the pond (about 1 ha), the locations of the major drainages entering the pond, and the polishing sand filter (140 m^2). The numbers on the schematic indicate the sampling locations used during this study. Figure 104 indicates the area and volumes of the pond for different pond surface elevations. Six storms were monitored between Nov 28, 1990 and January 10, 1991, having the following rain depths: 25, 16, 9, 20, 11, and 13 mm. Table 34 contains the particle size distributions of the influent to the pond for the six monitored events. Almost all of the monitored particles (using a Spectrex ILI 1000 laser particle counter and checked with a microscope) were in the range of 15 to 45 μm. Numerous turbidity measurements were made throughout the monitored events at the four sampling locations. Figure 105 is an example of the typical changes of turbidity during the first storm event. The turbidity of water leaving the small pond was very similar to the sheetflow water entering the small pond (several hundred to several thousand NTU), while the turbidity of the water leaving the large pond was greatly reduced (to between 20 and 50 NTU), which was further reduced by the sand filter (to about 1 to 10 NTU).

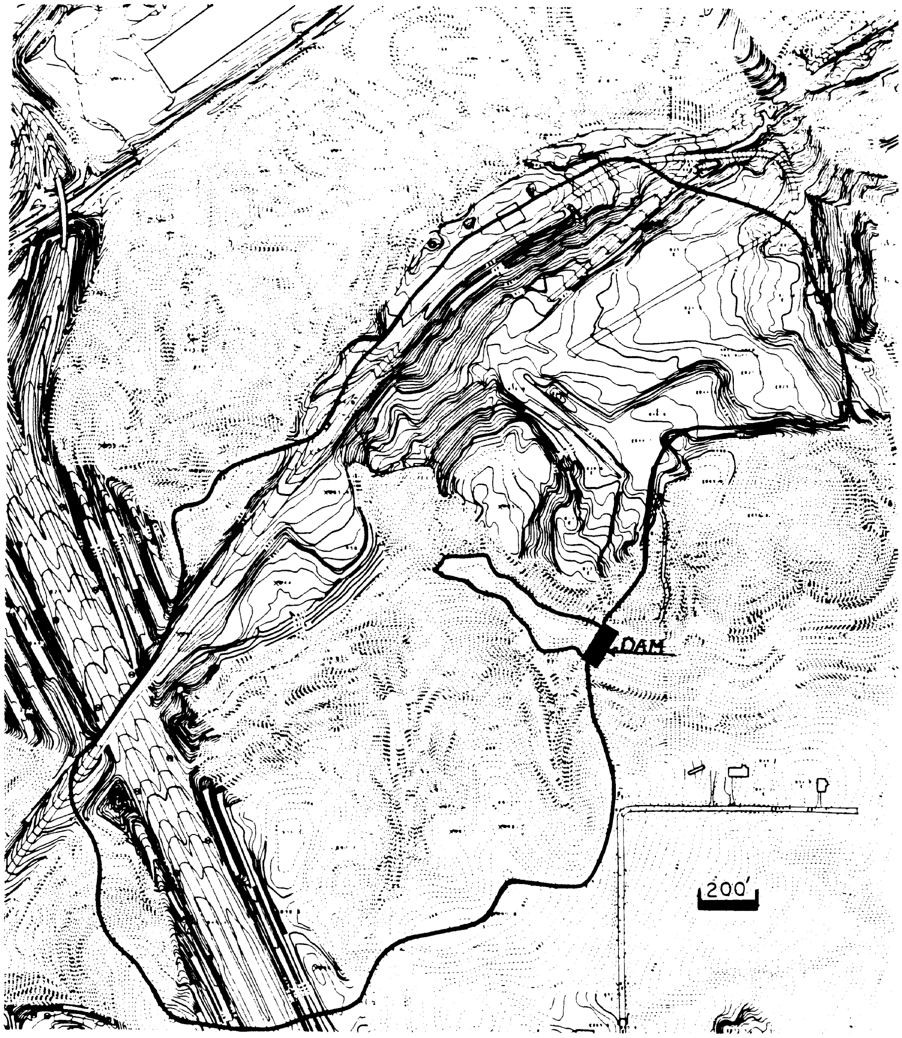


Figure 102. Birmingham landfill and pond watershed map.

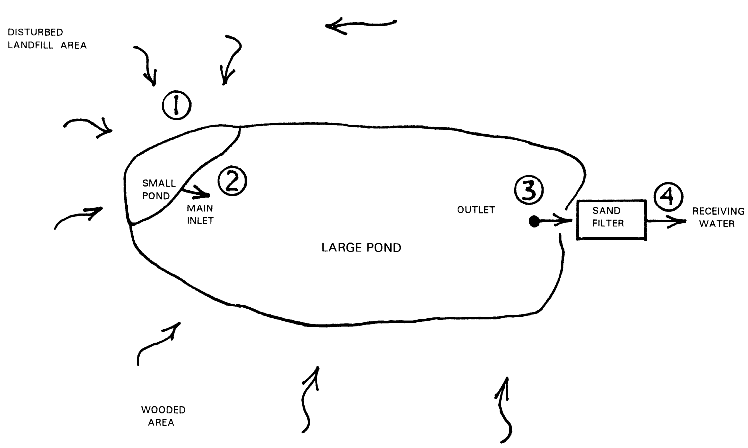


Figure 103. Birmingham landfill pond schematic.

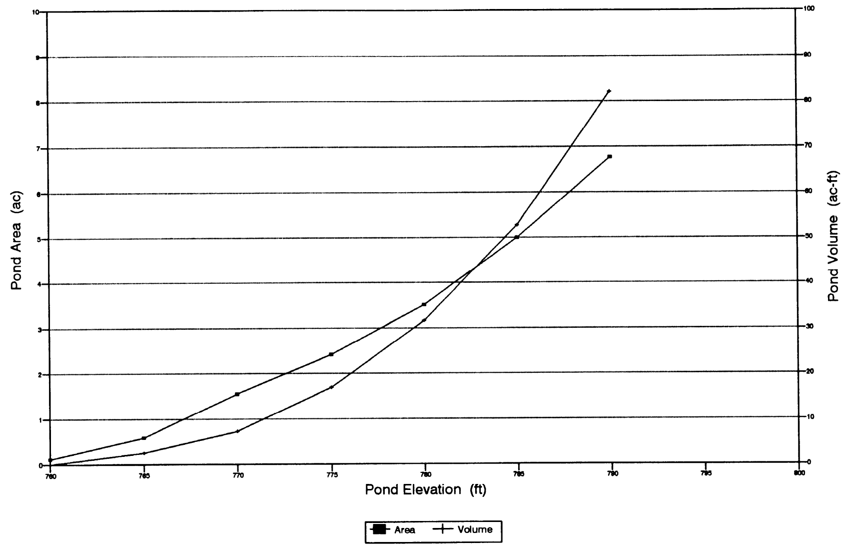


Figure 104. Birmingham landfill elevation-area and elevation-volumes curves.

Table 34. Observed Runoff Particle Sizes in Influent Water at Birmingham Landfill (percentage associated with each particle size range, by mass)

			Storm #			
Particle Size (μm)	**1**	**2**	**3**	**4**	**5**	**6**
<5	0	0	0	0	0	0
5-15	0.1	1.3	1.6	3.6	3.8	2.2
15-15	5.9	5.0	4.4	21.2	26.2	0.8
25-35	24.6	31.1	64.8	75.2	70.0	86.0
35-45	69.4	62.6	29.2	0	0	11.0
45-55	0	0	0	0	0	0
>55	0	0	0	0	0	0

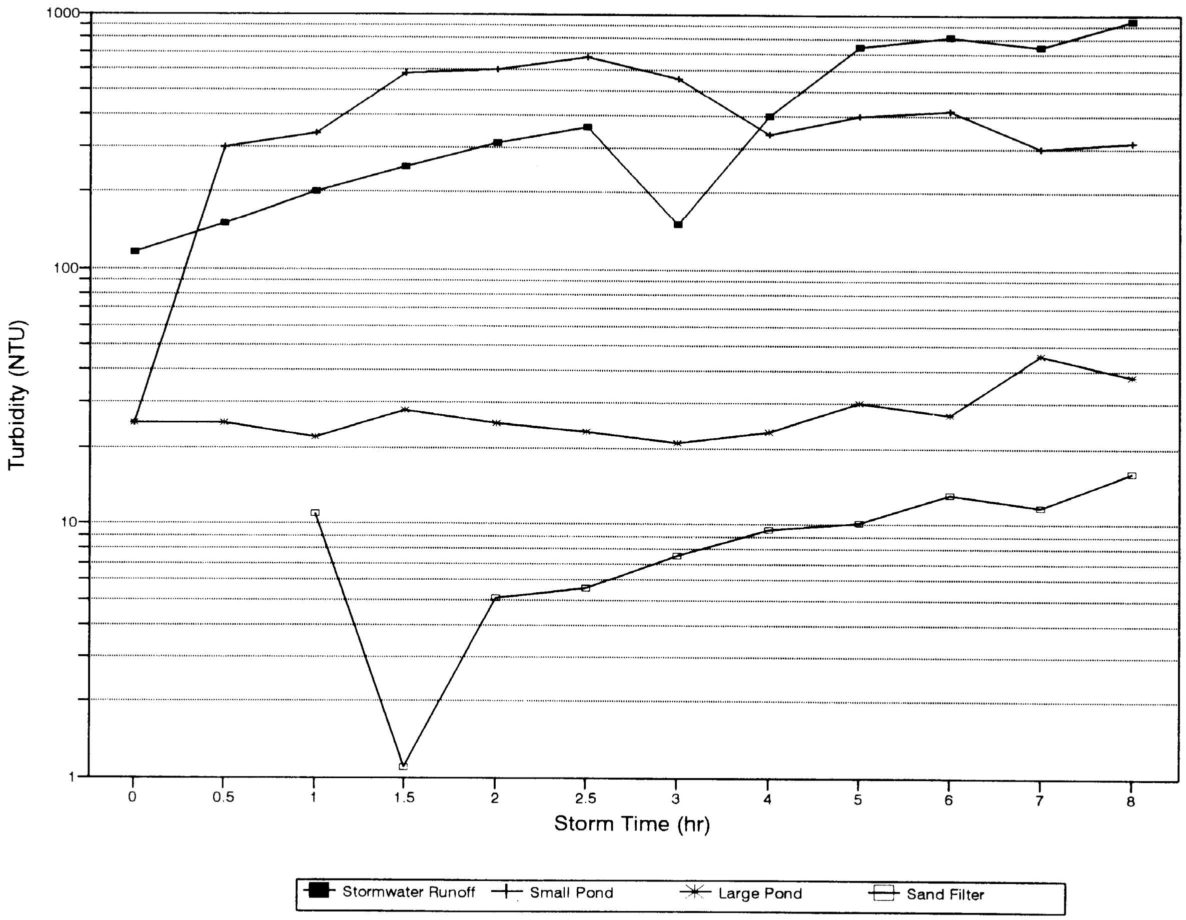


Figure 105. Turbidity changes with time for influent, initial small pond, pond, and sand filter effluent for storm #1.

This was a relatively large pond for the drainage area. The landfill was operating under a NPDES permit which restricted the turbidity of the effluent to 50 NTU. The pond was therefore designed and constructed larger than thought necessary in order to better meet this discharge limit. Since the sand filter clogged quickly and required manual cleaning, it was only used when necessary to ensure the effluent turbidity was less than the discharge limit. Figure 106 shows the successful predictions of the pond hydraulic performance using WinDETPOND, compared to the observed pond stages during the monitored storms over a wide range of conditions. Table 35 shows the predicted suspended solids removal by the pond, using WinDETPOND and the monitored particle size distributions and rain conditions, compared to the monitored suspended solids removal. Since the pond was over-sized for the site conditions, it was predicted (and shown to have) almost complete removal of the suspended solids.

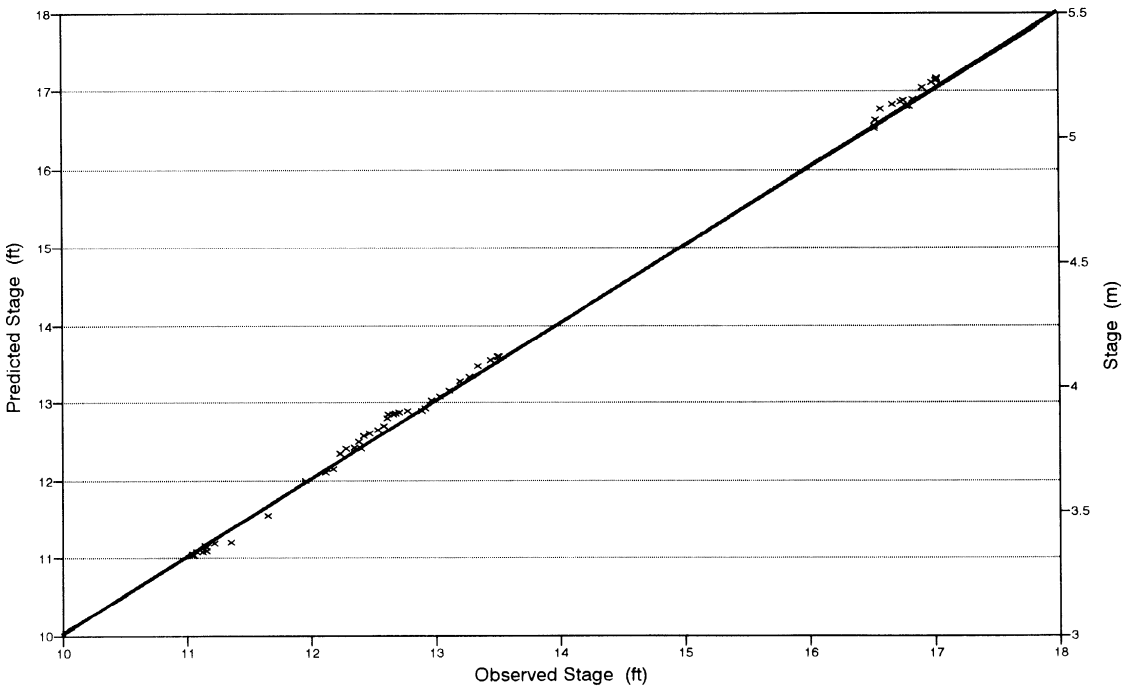


Figure 106. Observed and predicted pond stages for Birmingham landfill pond study.

Table 35. Predicted and Observed Suspended Solids Removal for Birmingham Landfill Pond

	Storm #					
	1	**2**	**3**	**4**	**5**	**6**
Predicted Removal, % (using WinDETPOND)	100	100	100	100	100	100
Observed Removal, %	100	100	99	100	99	100

Issues Associated with Using a Continuous Record of Rains vs. a Single Event Storm

Single-event designs for hydraulic devices have been used for many decades with reasonably good success. They were developed to evaluate single parameter conditions (especially peak flow rate or maximum stage in drainage design). They are used with the assumption that if the hydraulic structure is designed to withstand this critical event, all events less critical would be safely handled. The critical single event for drainage design is selected from a local intensity-duration-frequency (IDF) curve for the drainage area time of concentration. The level of service is selected based on the return frequency of the design event (such as a "10-year" storm) and the intensity for the design storm is selected based on this level of service and an event duration equal to the watershed time of concentration. This is an effective approach for the design of relatively simple hydraulic structures and was developed due to the impracticality of evaluating a large series of events during a time of manual calculations.

The current availability of inexpensive computer facilities and software has largely negated the need to use a single-event for design (James and Robinson 1982). A much more suitable approach is to use continuous models for an extended period of time. This is especially critical when non-linear processes interact in unpredictable ways for different conditions and when more than simple single-parameter evaluations are needed. Wet weather flow water quality evaluations are much more complex than drainage design evaluations and require continuous simulations for the best results. Specifically, continuous evaluations enable calculations of probabilities of certain levels of performance being exceeded, such as the percentage of flow treated to a certain level.

This is not to say that single-event design storms should not be used for preliminary designs. Sizing of a wet detention pond (or other control practice) for water quality improvement can usually be made using relatively simple guidelines, based on historical performance data, local land use information, and rainfall statistics. However, it is possible and sometimes necessary to evaluate this design with a model under continuous and long-term conditions. This evaluation will produce much more useful information and will enable the "preliminary" design to be modified to more effectively meet the project objectives. In most cases, this long-term simulation only requires several minutes of time to conduct.

Stream Habitat Benefits Associated with Peak Flow Reduction Criteria

Some of the most serious effects of urban runoff are on the aquatic habitat of the receiving waters. A significant indirect benefit of flow controls for stormwater management is the reduction in associated stream power. Increased flows are probably the best know example of impacts associated with urbanization. Most of the recognition has of course focused on increased flooding and associated damages. This has led to numerous attempts to control peak flows from new urban areas through the use of regulations that limit post development peak flows to pre-development levels for relatively large design storms. The typical response has been to use dry detention ponds. In addition to the serious issue of flooding, high flows also cause detrimental ecological problems in receiving waters. The following discussion presents several case studies where increased flows were found to have serious effects on stream habitat conditions, along with recommended approaches for their control.

The aquatic organism differences in urbanized and control streams found during the Bellevue Urban Runoff Program were probably mostly associated with the increased peak flows associated with urbanization. The increased flows in the urbanized Kelsey Creek resulted in increases in sediment carrying capacity and channel instability of the creek (Pederson 1981; Perkins 1982; Richey, *et al*. 1981; Richey 1982; Scott, *et al* . 1982). Kelsey Creek had much lower flows than the reference Bear Creek during periods between storms. About 30 percent less water was available in Kelsey Creek during the summers. These low flows may also have significantly affected the aquatic habitat and the ability of the urban creek to flush toxic spills or other dry weather pollutants from the creek system (Ebbert, *et al*. 1983; Prych and Ebbert undated). Kelsey Creek had extreme hydrologic responses to storms. Flooding substantially increased in Kelsey Creek during the period of urban development; the peak annual discharges almost doubled in the last 30 years, and the flooding frequency also increased due to urbanization (Ebbert, *et al.* 1983; Prych and Ebbert undated).

Snodgrass, *et al*. (1998) reported that in the Toronto, Ontario, area, flows causing bankfull conditions occur with a return frequency of about 1.5 years. Storms with this frequency are in general equilibrium with resisting forces that tend to stabilize the channel (such as vegetation and tree root mats), with increased flows overcoming these resisting forces causing channel enlargement. Infrequent flows can therefore be highly erosive. With urbanization, the flows that were bankfull flows during historical times now occur much more frequently (about every 0.4 years in Toronto). The channel cross-sectional areas therefore greatly increase to accommodate the increased stream discharges and power associated with the "new" 1.5 year flows that are trying to re-establish equilibrium.

Booth and Jackson (1997) found that the classical goal of detention ponds to maintain predevelopment flows was seriously inadequate because there is no control on the duration of the peak flows. They showed that a duration standard to maintain post development flow durations for all sediment-transporting discharges to predevelopment durations will avoid many receiving water habitat problems associated with stream instability. Without infiltration, the amount of runoff will obviously still increase with urbanization, but the increased water could be discharged from detention facilities at flow rates below the critical threshold causing sediment transport. The identification of the threshold discharge below which sediment transport does not occur, unfortunately, is difficult and very site specific. A presumed threshold discharge of about one-half of the pre-development 2-year flow was recommended for gravel bedded streams. Sand-bedded channels have sediment transport thresholds that are very small, with inevitable bed load transport likely to occur for most levels of urbanization.

MacRae (1997) presented a review of the development of the common zero runoff increase (ZRI) discharge criterion, referring to peak discharges before and after development. MacRae shows how this criterion has not effectively protected the receiving water habitat. He found that stream bed and bank erosion is controlled by the frequency and duration of the mid-depth flows (generally occurring more often than once a year), not the bank-full condition (approximated by the 2 yr event). During monitoring near Toronto, he found that the duration of the geomorphically significant pre-development mid-bankfull flows increased by a factor of 4.2 times, after 34% of the basin had been urbanized, compared to before development flow conditions. The channel had responded by increasing in cross-sectional area by as much as 3 times in some areas, and was still expanding. Table 36 shows the modeled durations of critical discharges for predevelopment conditions, compared to current and ultimate levels of development with "zero runoff increase" controls in place. At full development and even with full ZRI compliance in this watershed, the hours exceeding the critical mid-bankfull conditions will increase by a factor of 10, with resulting significant effects on channel stability and the physical habitat. MacRae (1997) concluded that an effective criterion to protect stream stability (a major component of habitat protection) must address mid-bankfull events, especially by requiring similar durations and frequencies of stream power (the product of shear stress and flow velocity, not just flow velocity alone) at these depths, compared to satisfactory reference conditions.

Table 36. Hours of Exceedence of Developed Conditions with Zero Runoff Increase Controls Compared to Predevelopment Conditions (MacRae (1997)

Recurrence Interval (yrs)	Existing Flowrate (m^3/s)	Exceedence for Predevelopment Conditions (hrs per 5 yrs)	Exceedence for Existing Development Conditions, with ZRI Controls (hrs per 5 yrs)	Exceedence for Ultimate Development Conditions, with ZRI Controls (hrs per 5 yrs)
1.01 (critical mid-bankfull conditions)	1.24	90	380	900
1.5 (bankfull conditions)	2.1	30	34	120

As seen, single-event criterion are not very effective for habitat protection unless relatively small events are used. Unfortunately, when only considering small events, serious drainage and flooding problems associated with large events may not be adequately mitigated. Therefore, flow criteria should consider at least several return frequency events (such as the recommended mid-bank flow condition, along with the less frequent drainage design storm). In addition, the duration of flows larger than critical sediment transport flows should also be controlled in order to provide protection of habitat. The use of continuous simulation including the more common events along with rarer storms causing flooding and drainage damage, should also be considered.

Untreated Flows Associated with Single-Event Criteria

Another important problem associated with single-event criteria is that many dry detention ponds built have low-flow channels to allow most of the annual flow to pass through the pond without any retention or opportunity for treatment. Only when the inflow exceeds the critical value does it back up in the pond. Therefore, most of the annual flow passes along a small concrete channel, with no treatment, with only a few events a year being treated at all. In these ponds, little scour of the settled particulates would likely occur because the long time period between flooding in the pond would allow incorporation of most of the settled material into the pond grass liner. If the pond was paved or lined with concrete, such as in some depressed tennis courts that are actually dry ponds, then scour may occur. In Bellevue, WA, where these "multi-use" ponds have been used, ramps lead down into the pond/tennis court to allow street cleaners to remove much of the settled sediment after a large rain, allowing little interruption of recreational use of the facility. In all cases, these pond designs, even though designed and operated to suppress large flows, actually treat very small amounts of the annual stormwater flows, with minimal water quality benefit.

Benefits of Using Continuous, Long-Term Simulations

Urban receiving water problems are related to many different conditions covering a wide range of rain characteristics. Reviews of numerous urban receiving water studies from throughout the U.S. have identified the following diverse list of receiving water problems that may be caused by stormwater (Pitt 1995a and 1995b):

- Sedimentation damage in stormwater conveyance systems and in receiving waters.
- Nuisance algae growths from nutrient discharges into quiescent waters.
- Inedible fish and undrinkable water caused by toxic pollutant discharges.
- Shifts to less sensitive aquatic organisms caused by contaminated sediments and habitat

destruction.

- Property damage from increased drainage system failures.
- Swimming beach closures from pathogenic microorganisms.
- Water quality violations, especially for bacteria and total recoverable heavy metals.

The first four problem areas are mostly associated with slug (mass) discharges (not instantaneous concentrations or rates), while the last three are mostly associated with instantaneous concentrations and high flow rates.

In order to predict receiving water problems caused by stormwater, accurate flow estimates and pollutant mass discharges must be known. Knowing where the potentially problem pollutants originate in the watershed is also valuable in order to select appropriate stormwater control candidates. Accurate knowledge of runoff volumes during different storms has been shown to be necessary when predicting pollutant discharges.

Most of the annual rain is associated with many small individual events, while most of the runoff volume and pollutant mass discharges are associated with a smaller set of intermediate events. The following discussion illustrates this, based on actual monitored rainfall and runoff distributions for Milwaukee, WI (data from the Milwaukee NURP project, Bannerman, *et al*. 1983), and analyses of long-term rainfall histories and predicted runoff for Minneapolis.

Figure 107 includes cumulative probability density functions (CDFs) of measured rain and runoff distributions for Milwaukee during the 1981 NURP monitored rain year (data from Bannerman, *et al*. 1983). CDFs are used for plotting because they clearly show the ranges of rain depths responsible for most of the runoff. Rains between 0.05 and 5 in. were monitored during this period, with two very large events (greater than 3 inches) occurred during this monitoring period which greatly distort these curves, compared to typical rain years. The following observations are evident:

- The median rain depth was about 0.3 in.
- 66% of all Milwaukee rains are less than 0.5 in. in depth.
- For medium density residential areas, 50% of runoff was associated with rains less than 0.75 in.
- A 100-yr., 24-hr rain of 5.6 in. for Milwaukee could produce about 15% of the typical annual runoff volume, but it only contributes about 0.15% of the average annual runoff volume, when amortized over 100 yrs.
- Similarly, a 25-yr., 24-hr rain of 4.4 in. for Milwaukee could produce about 12.5% of the typical annual runoff volume, but it only contributes about 0.5% of the average annual runoff volume, when amortized over 25 yrs.

Figure 108 shows CDFs of measured Milwaukee pollutant loads associated with different rain depths for a medium density residential area. Suspended solids, COD, lead, and phosphate loads are seen to closely follow the runoff volume CDF shown in Figure 107, as expected. Since load is the product of concentration and runoff volume, some of the high correlation shown between load and rain depth is obviously spurious. However, these overlays illustrate the range of rains associated with the greatest pollutant discharges.

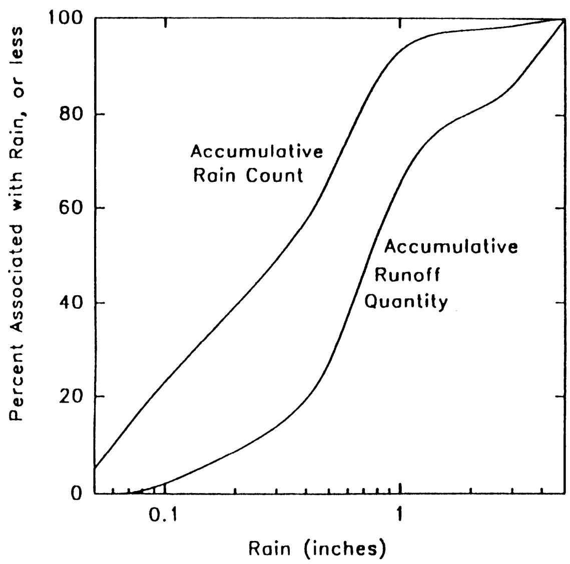


Figure 107. Milwaukee rain and runoff cumulative probability density functions (CDFs).

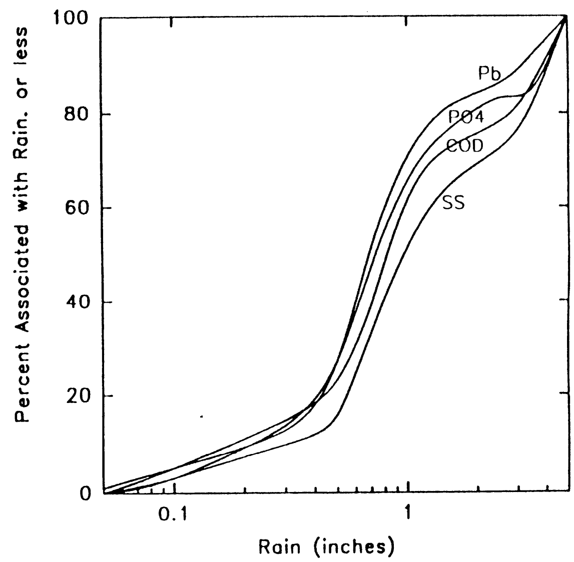


Figure 108. Milwaukee pollutant discharge cumulative probability density functions (CDFs).

The Milwaukee observations show that southeastern Wisconsin rainfall distributions can be divided into the following categories, with possible management approaches relevant for each category of rain:

- Common rains having relatively low pollutant discharges are associated with rains less than about 0.5 in. (12 mm) in depth. These rains account for most of the events, but little of the runoff volume, and are therefore easiest to control. They produce much less pollutant mass discharges and probably have less receiving water effects than other rains. However, the runoff pollutant concentrations likely exceed regulatory standards for several categories of critical pollutants, especially bacteria and some total recoverable metals. They also cause large numbers of overflow events in uncontrolled combined sewers. These rains are very common, occurring once or twice a week (accounting for about 60% of the total rainfall events and about 45% of the total runoff events that occurred), but they only account for about 20% of the annual runoff and pollutant discharges. Rains less than about 0.05 inches generally did not produce noticeable runoff during the field monitoring in Milwaukee, but the lower "cutoff" rainfall would be mostly dependent on the amount of pavement in the drainage. These are key rains when runoff-associated water quality violations, such as for bacteria and total recoverable heavy metals, are of concern. In most areas, runoff from these rains should be totally captured and either re-used for on-site beneficial uses or infiltrated in upland areas. For most areas, the runoff from these rains can be relatively easily removed from the surface drainage system.

- Rains between 0.5 and 1.5 in. (12 and 38 mm) are responsible for about 75% of the runoff pollutant discharges and are key rains when addressing mass pollutant discharges. These rains account for the majority of the runoff volume (about 50% of the annual volume for this Milwaukee example) and produce moderate to high flows. They account for about 35% of the annual rain events, and about 20% of the annual runoff events. These rains occur on the average about every two weeks during the spring to fall seasons and subject the receiving waters to frequent high pollutant loads and moderate to high flows. The small rains in this category can also be removed from the drainage system and the runoff re-used on site for beneficial uses or infiltrated to replenish the lost groundwater infiltration associated with urbanization. The runoff from the larger rains should be treated (such as in wet detention ponds) to prevent pollutant discharges from entering the receiving waters.

• Rains greater than 1.5 in. (38 mm) and less than 3 in (75 mm) are associated with drainage design and are only responsible for relatively small portions of the annual pollutant discharges. These rains produce the most damaging flows, from a habitat destruction standpoint, and occur every several months (at least once or twice a year) to every few years. These recurring high flows, which were historically associated with much less frequent rains, establish the energy gradient of the stream and cause unstable streambanks. Only about 2 percent of the rains are in this category and they are responsible for about 10 percent of the annual runoff and pollutant discharges. Typical storm drainage design events fall in the upper portion of this category. Extensive pollution control designed for these events would be very costly, especially considering the relatively small portion of the annual runoff associated with the events. However, discharge rate reductions are important to reduce habitat problems in the receiving waters. The infiltration and other treatment controls used to handle the smaller storms in the above categories would have some benefit in reducing pollutant discharges during these larger, rarer storms.

• In addition, extremely large rains >3 inches (>75 mm) also infrequently occur that can exceed the capacity of the drainage system and cause local flooding. This category is infrequently represented in field studies due to the rarity of these large events and the typically short duration of most field observations. The smallest rains in this category are included in design storms used for drainage systems in Milwaukee. These rains occur only rarely (once every several years to once every several decades, or less frequently) and produce extremely large flows. The 3-year monitoring period during the Milwaukee NURP program (1980 through 1983) was unusual in that two of these events occurred. Less than 2 percent of the rains were in this category (typically <<1% would be), and they produced about 15% of the annual runoff quantity and pollutant discharges. During a "normal" period, these rains would only produce a very small fraction of the annual average discharges. However, when they do occur, great property and receiving water damage results. The receiving water damage (mostly associated with habitat destruction, sediment scouring, and the flushing of organisms great distances downstream and out of the system) can conceivably naturally recover to before-storm conditions within a few years. These storms, while very destructive, are sufficiently rare that the resulting environmental problems do not justify the massive stormwater quality controls that would be necessary. The problem during these events is massive property damage and possible loss of life. These rains typically greatly exceed the capacities of the storm drainage systems, causing extensive flooding. It is critical that these excessive flows be conveyed in "secondary" drainage systems. These secondary systems would normally be graded large depressions between buildings that would direct the water away from the buildings and critical transportation routes and to possible infrequent/temporary detention areas (such as large playing fields or parking lots). Because these events are so rare, institutional memory often fails and development is allowed in areas that may not be indicated on conventional flood maps, but could suffer critical flood damage.

Example Use of WinDETPOND and Wet Detention Pond Analyses

Analysis of the Wet Stormwater Detention Pond for the Brook Highland Shopping Center

The following analysis was conducted by John Easton, a UAB graduate student, as part of a class assignment investigating current performance and possible retro-fit opportunities at existing wet detention ponds. The analyses included site surveys and peak flow evaluations using HydroCAD ® and water quality analyses using WinDETPOND. This wet detention pond is located between Highway 280 and the Wal-Mart at the Brook Highland Plaza Shopping Center, in Shelby County, AL. The contributing area was estimated at 18 acres.

General Quality Criteria

Pond Depth

A review of the plans and specifications, in addition to on-site field evaluations, indicates that the pond meets the depth criteria of 3 to 6 feet of permanent storage which is necessary to prevent scour, decrease light penetration (to minimize rooted aquatic plant growths), and to increase winter survival of fish. This review indicates that the pond will maintain approximately 4 feet of dead water storage, but does not provide for much sediment storage. The pond might benefit from a deepened sump near the pond inlets where sediment would preferentially be captured. This would likely lower the maintenance costs for the pond by allowing easy access for removal of these larger particles.

Safety Criteria
The pond side slopes are 1:2 near the water edge, steeper than preferred. A 15 foot wide shelf slightly below the water surface is provided.

Peak Reduction Factors (PRF)
The pond significantly reduces the peak outflow rates from the contributing area. Theoretically, the 100-year storm's runoff rate is reduced from 141 cfs to about 38 cfs. The peak reduction factor ($PRF = 1–Q_o/Q_I$), for this event is 0.74, corresponding to a 74% reduction of the inflow hydrograph in the pond.. For the 50-year and 25-year storms, the PRFs are 0.73. Even in the case of the 100-yr storm, the pond still has half of a foot of freeboard storage below the invert of the emergency spillway.

Upflow and Critical Settling Velocities
For the typical rain events, WinDETPOND simulations demonstrate that the pond satisfies the maximum upflow velocity (or critical settling velocity) maximum of 0.00013 ft/sec which is necessary for 5µm particle control.

Pond's Water Quality Storage
A pond's water quality storage should be equal to the runoff associated with 1-¼" rain based on the land use, and cover of the watershed served by the pond. HydroCAD, which uses SCS TR-20 methods for computing the composite curve number, calculated a CN of 95. This 95 CN is appropriate for a commercial area, and corresponds to approximately 0.85 inches of runoff for this rain size. Therefore, the minimum active pond storage (between the invert elevation of the lowest outlet and the secondary outlet discharge devices) required should be a least 1.3 acre-ft. The pond's water quality storage is approximately 1.6 acre-ft. There is an additional freeboard storage of 4.6 acre-ft for peak runoff rate reductions.

Pond's Surface Area Requirements
A pond's surface area should be sized as a percent of watershed's area based on land use and the particle size control desired. This site has commercial land use, with a recommended 1.7% of the watershed area needed for the pond surface area (or about 0.31 acres). The pond has a normal pool area of about 0.54 acres, exceeding this minimum recommendation.

Other Benefits
In dry weather, the pond will be available to provide water for emergency fire protection. This pond should be a pleasing amenity for the retail mall area. The use of appropriate grasses adjacent to the pond may provide a grass filter for additional pollutant reduction.

Background Information Related to Site Evaluation
Criteria Used to Estimate Peak Flowrates
The peak inflow hydrograph values were determined by HydroCAD's SCS TR-20 methodology. For the site, a SCS Type III rainfall IDF curve was selected. Rainfall depths for the 100-year, 50-year, and 25-year storms were approximately 8.6", 7.8", and 7.1" respectively. The time of concentration (Tc = 5.1 minutes) for the watershed was also calculated using HydroCAD's built-in TR-20 methods. Given that the site is commercial, with an estimated 85% impervious area, a curve number of 95 was assigned.

Watershed Areas, Slope, and Drainage Divides
Based on the information provided in the site's grading plans (given by Sain and Associates) and field observations, it was determined that the contributing watershed area that drains into the pond has an estimated area of approximately 18 acres. Slopes were determined to be very flat in the vicinity of the pond, approximately 1 foot per 100 feet, or 1%.

Analysis of Design Storms
HydroCAD ®
The HydroCAD Stormwater Modeling System (version 4.53) was used to analyze the pond for flow attenuation during drainage design storms. This computer program calculates inflowing hydrographs, based upon design storm

and watershed characteristics, and then routes these through a drainage system composed of subcatchments, reaches, and ponds.

The subcatchment component is used to model a given drainage area or watershed. In this case, there was only one subcatchment, with subcatchment 1 referring to the 18.0 acres of the Brook Highland commercial shopping center that drains into the pond next to Wal-Mart. The program uses built-in SCS TR-20 hydrology methods for determining the hydrograph characteristics. Next, the hydrograph is routed through a series of defined reaches and/or ponds. In this case, there is one hydrograph from the subcatchment, which is routed through a single pond.

The pond component of this model is described using a stage v. surface area curve. In addition, the model requires descriptions of the outlet structures. This data, as input to the model, is described in Figure 109 and Table 37.

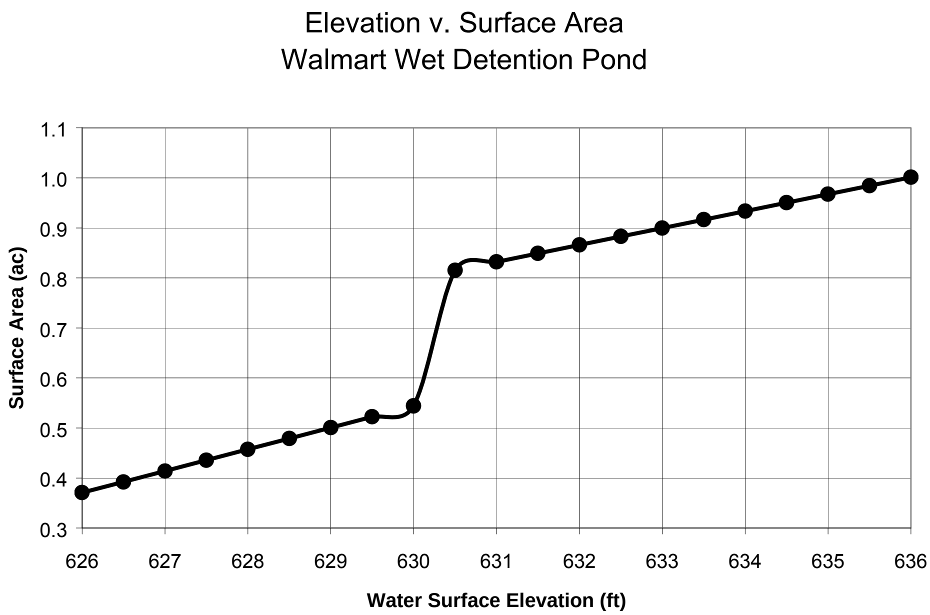


Figure 109. Stage v. Surface Area Curve

Table 37. Outlet Device Descriptions

#	Route	Invert	Outlet Devices
1	primary	630.0’	**36” culvert** n=0.013, length=38’, slope=0.13%, Ke=0.5, Cc=0.9
2	to #1	630.0’	**30” orifice**
2	to #2	630.0’	**22” orifice (two) (partially blocked by excessive cattail growths)**
3	to #2	632.5’	**sharp-crested rectangular weir** length=15.7’, height=3.5’ (square concrete box)
4	secondary	634.5’	**10’ broad-crested rectangular weir** emergency spillway

The HydroCAD simulations were run for three 24-hour, SCS type III design storm frequencies: 25-year (7.1”), 50-year (7.8”), and 100-year (8.6”). Table 38 summarizes these results. As previously mentioned, the peak reductions are about 73%, and the peak discharge lag is approximately 22 minutes. The peak elevation in the pond never reaches the maximum elevation (636 ft).

Table 38. Results of HydroCAD Simulations

Design Event	Rain Depth (in)*	Peak Elev. (ft)**	Peak Storage (AF)	Peak Qin (cfs)	Peak Qout (cfs)	Atten. (%)	Lag (min)
25-year	7.1	633.0	4.22	116.1	30.81	73	22.7
50-year	7.8	633.3	4.56	127.9	34.22	73	22.4
100-year	8.6	633.8	4.97	141.3	37.69	73	22.5

* Design storms are type III 24-hr for Shelby county (SCS methods).
** Flood elevation is at 636 feet.

<u>WinDETPOND</u>
WinDETPOND uses a simplified triangular hydrograph suitable for small rains. Therefore, the SCS hydrograph generated by HydroCAD was imported into WinDETPOND to simulate water quality benefits during these large "design" storms. A comparison of the hydraulic results from HydroCAD (Table 39) shows that the hydraulic results are similar. Even under these extreme rain conditions, the pond is expected to remove approximately 75% of the TSS.

Table 39. WinDETPOND Summary for Design Storms

Storm Year	Max. Stage (ft.)	Max. Inflow (cfs)	Max. Outflow (cfs)	Max particle size discharged (µm)	Avg. Min Particle Size Controlled (µm)	% TSS Removed
25	633.01	115.0	31.3	32.5	7.9	76.1
50	633.40	126.7	34.4	32.5	8.3	75.1
100	633.83	140.0	37.9	32.5	9.0	73.5

Analyses Using Long-Term Rainfall Records
<u>WinDETPOND</u>
The advantage to using WinDETPOND is that the program allows analyses of actual rainfall events over an extended period of time. Rain files contain start and end dates and times, plus the rain depth. The model determines the rain duration, rain intensities, and interevent periods. WinDETPOND then routes a simple triangular hydrograph through the pond to evaluate the expected particulate removal. For this evaluation, WinDETPOND simulations were conducted using rain files created from the 1976 Birmingham monitoring year (a "normal" rain year containing 112 events, based on long-term records), and also on the complete 1952 through 1989 rain record. There were 2 events (out of a total of 4,107 in the Bham5289 rain file), in which the pond stage rises to the level of the second outlet. In addition, it never reaches the emergency spillway.

Short-term Simulation Series. The results of the simulations using the Bham76 file are presented in Table 40. On average, the pond will collect particle sizes 1.17 µm and greater in size, which represents 97% TSS control. The average rain depth is 0.5 inches, and the average duration is 12 hours. For the smallest storms, this pond is achieving close to 100% control, and for the largest storm in 1976 the pond is still removing about 86% of the TSS. These high removals, in addition to the large peak flow rate reductions, indicates that the pond is likely over-sized, possibly in anticipation of additional area being directed to the pond as the shopping center is further developed.

Table 40. Water Quality Output Summary for 1976 Rain Year (112 events)

Statistic	Rain Depth (in)	Rain Duration (hrs)	Intrevt Duration (days)	Rain Intensity (in/hr)	Max Pond Stage (ft)	Flow-weighted Particle Size	Approx. Part. Res. Control* (%)	Peak Reduction Factor	Event Flushing Ratio
Mean	0.50	12.01	1.81	0.04	630.16	1.17	97	0.29	0.39
Std. Dev.	0.75	10.77	2.36	0.06	0.23	1.23	4	0.21	0.64
COV**	1.51	0.90	1.30	1.48	0.05	1.05	0.04	0.73	1.63
Min.	0.01	1.00	0.25	0.01	630.00	0.00	86	0.04	0.00
Max.	3.84	45	11.68	0.31	631.04	4.00	100	0.74	3.31

* Approximate Particle Residue Control (TSS).
** Coefficient of Variation – standard deviation divided by the mean.

Figure 110 shows the maximum pond stage versus the percent particle control. There is an expected trend as the control decreases with maximum stage, i.e., more water flowing into the pond.

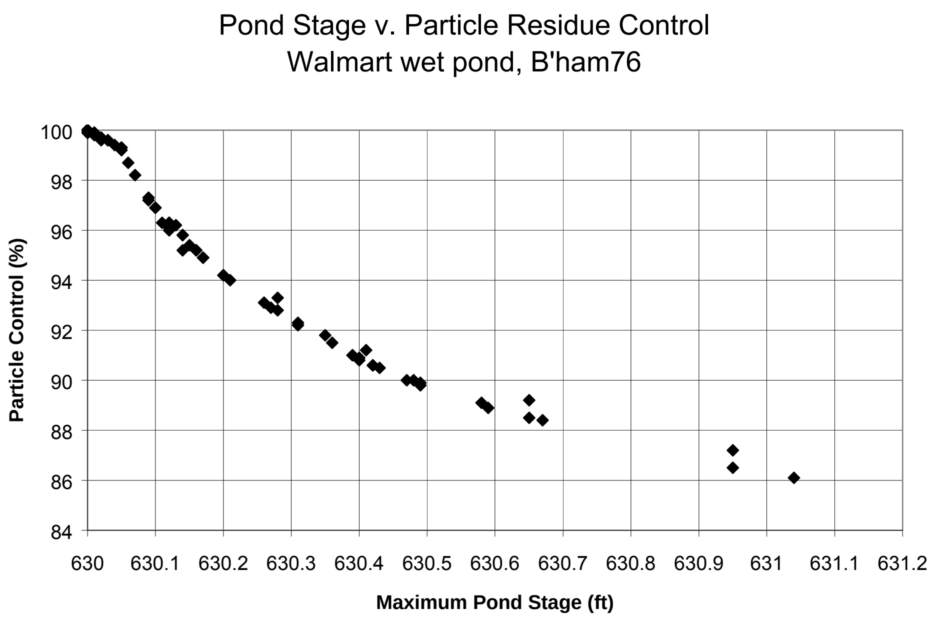


Figure 110. Pond Stage v. Particle Residue Control

Figure 111 shows the water quality performance of the pond (% particulate control) versus the rain depth in inches. Generally, the percentage TSS control decreases as the rain depth increases, as expected. The scatter is due the fact that rainfall/runoff characteristics are quite variable and depth is only one parameter. The results are similar to Figure 112 which shows the percentage TSS control versus rain intensity.

Long-term Simulation using Birmingham Rain, 1952-1989. Table 41 is a summary for the 4,107 rain events that occurred in Birmingham from 1952-1989. Notice that the minimum and maximum values are different than those from the 1976 simulations, but the mean values are quite similar, indicating that 1976 is likely a good indicator for a typical rain year. The mean particle control is about 95%, slightly less than the 97% value indicated for the 1976 rain year. This high removal rate over this extended period assumes that proper maintenance of the pond will occur.

Design Storm Runs Using WinDETPOND

The pond inflow hydrograph from the HydroCAD analyses were used as a "user defined hydrograph" for input into WinDETPOND to evaluate the water quality control during these low frequency design storms. The following is an example WinDETPOND output file for the 25-year design event.

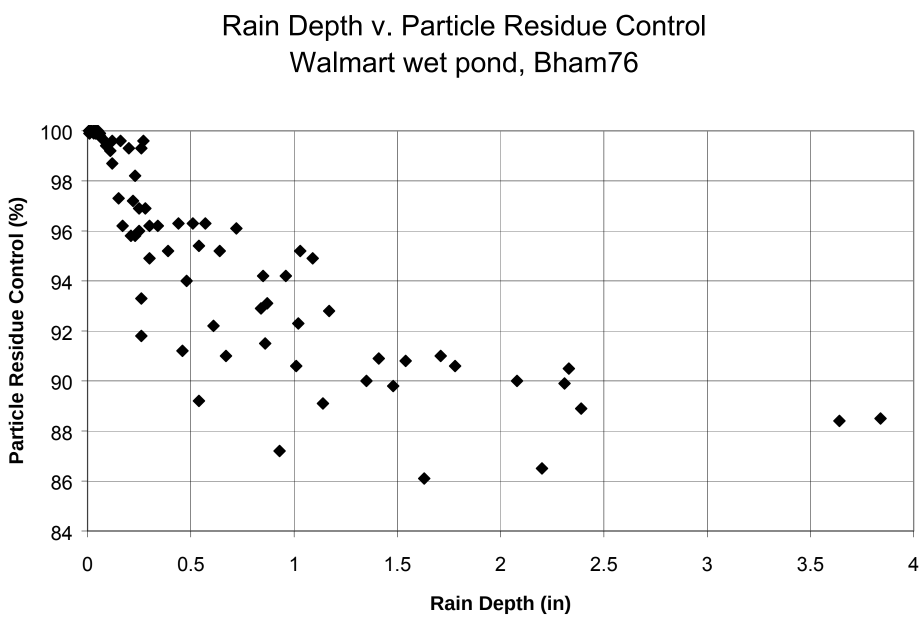


Figure 111. Rain Depth v. Particle Residue Control

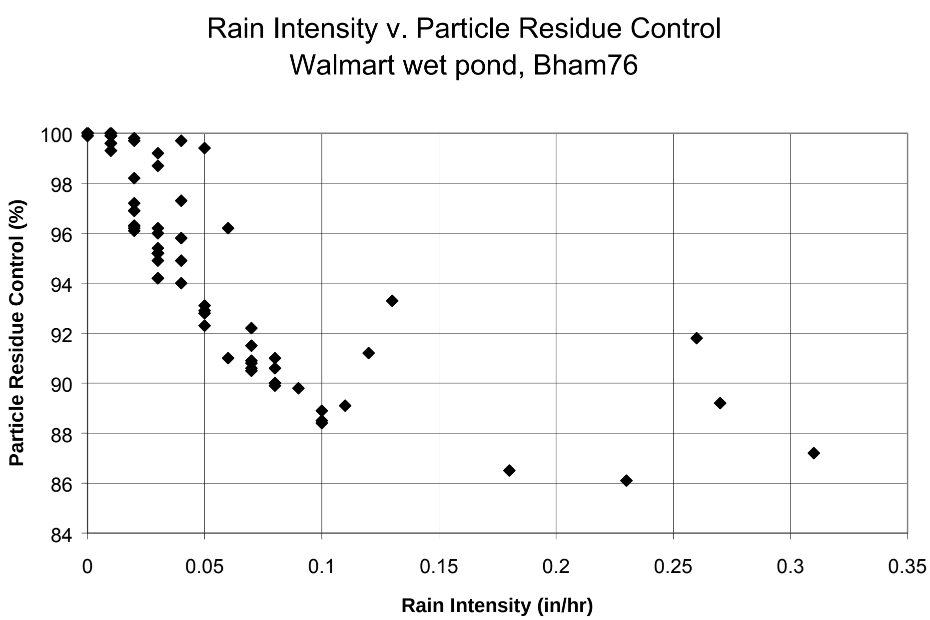


Figure 112. Rain Intensity v. Particle Residue Control

Table 41. Water Quality Output Summary for 1952-1989 Rain File

Statistic	Rain Depth (in)	Rain Duration (hrs)	Intrevt Duration (days)	Rain Intensity (in/hr)	Max Pond Stage (ft)	Flow-weighted Particle Size	Approx. Part. Res. Control* (%)	Peak Reduction Factor	Event Flushing Ratio
Mean	0.50	6.31	2.57	0.09	630.26	1.64	95	0.42	0.38
Std. Dv.	0.75	6.88	3.54	0.11	0.33	1.37	5	0.22	0.62
COV**	1.50	1.09	1.38	1.31	0.00	0.83	0.05	0.52	1.62
Min.	0.01	1.00	0.00	0.00	630.00	0.00	74	0.01	0.00
Max.	13.58	93	44.31	1.45	632.41	7.70	100	0.77	8.06

* Approximate Particle Residue Control (TSS).
** Coefficient of Variation – standard deviation divided by the mean.

25-year Design Event:

```
Time increment (min)=  6       Number of increments=  360
Rain depth (in) (N/A for user defined inlet hydrograph):        0.00
Rain duration (days):     0.42     Event duration (days):     0.50
Interevent duration (days):     0.00
Inflow rate to pond (cfs):  max=       115.0
Outflow rate from pond (cfs):  min=       0.0     max=      31.3
                               time weighted ave=         2.1
Net inflow volume (cu ft) -  event:    72615   cumulative:    72615
Total inflow volume to pond (cu ft):      348192
Outflow volumes (cu ft) - hydraulic:          275577
                        - seepage:                 0
                        - evaporation:             0
                        - total outflow:      275577
Pond storage above lowest invert (cu ft):  max =     106777
Pond storage below lowest invert (cu ft):      72615
Pond stage above datum for event (ft):  min=      0.30   max=        7.01
Pond surface area for event (sq ft):  min=     5379    max=    39223
Event flushing ratio (total inflow volume/pond storage below invert):        4.80
Upflow velocity for event (ft/hr):  min=   0.000     max=   2.871
Minimum particle size controlled (microns):   flow weighted average=       7.9
Particulate solids control (percent): min=    62.8    flow weighted average=    74.1
Peak Reduction Factor (PRF):        0.73
***  The largest ave particle size discharged during any time increment:   12.1
microns
```

Particle Size Distribution

Percent of Particles Larger than Size Indicated	Particle Size (microns)					
	Pond Inflow During Event	Pond Outflow During Event				
		Theoretical ***	n=8	n=3	n=1	User Defined n n = 5
0 >	2000.0	7.9	21.0	32.5	233.3	32.5
10 >	233.3	6.9	8.7	10.0	15.3	8.9
20 >	95.0	5.9	7.0	7.6	10.1	7.2
30 >	53.3	5.2	5.4	5.8	7.7	5.5
40 >	32.5	4.5	4.6	4.8	5.9	4.7
50 >	21.0	3.8	3.7	4.0	4.8	3.8
60 >	13.5	3.1	2.9	3.1	3.7	3.0
70 >	9.0	2.3	2.2	2.3	2.7	2.2
80 >	5.7	1.6	1.5	1.5	1.8	1.5
90 >	3.0	0.8	0.7	0.8	0.9	0.7
100 >	0.0	0.0	0.0	0.0	0.0	0.0
Row A:		12.1	21.0	32.5	233.3	32.5
Row B:		7.9				
Row C:		74.1	76.6	75.3	70.6	76.1

Row A: Largest ave particle size discharged (microns) during any time event
Row B: Flow weighted average minimum particle size controlled (microns)
Row C: Percent particulate solids removed


Figure 113. Facing west, inlets on right, obscured by cattails.


Figure 114. Facing North, showing parking area that drain to pond.

Figure 115. Outlet structure, showing cattails partially blocking the 22 in orifices through concrete wall.

The Use of WinDETPOND to Evaluate Wet Detention Pond for Minneapolis-St. Paul Airport

This discussion if summarized from a report originally prepared for Liesch Associates, Inc., in August 1999.

Long-Term Rain and Runoff Analyses for Minneapolis

The critical values defining the important rain categories affecting receiving water uses are highly dependent on local rain and development conditions. Computer modeling analyses for 7 years of rains for Minneapolis (1982 through 1989) were conducted to examine the runoff distributions for typical residential and commercial areas. The plots from this modeling activity (shown in Figure 116) indicate the rainfall and runoff probability distributions. The complexity of most receiving water quality problems prevents a simple analysis. The use of simple design storms, which was a major breakthrough in effective drainage design more than 100 years ago, is not adequate when receiving water quality issues must also be addressed.

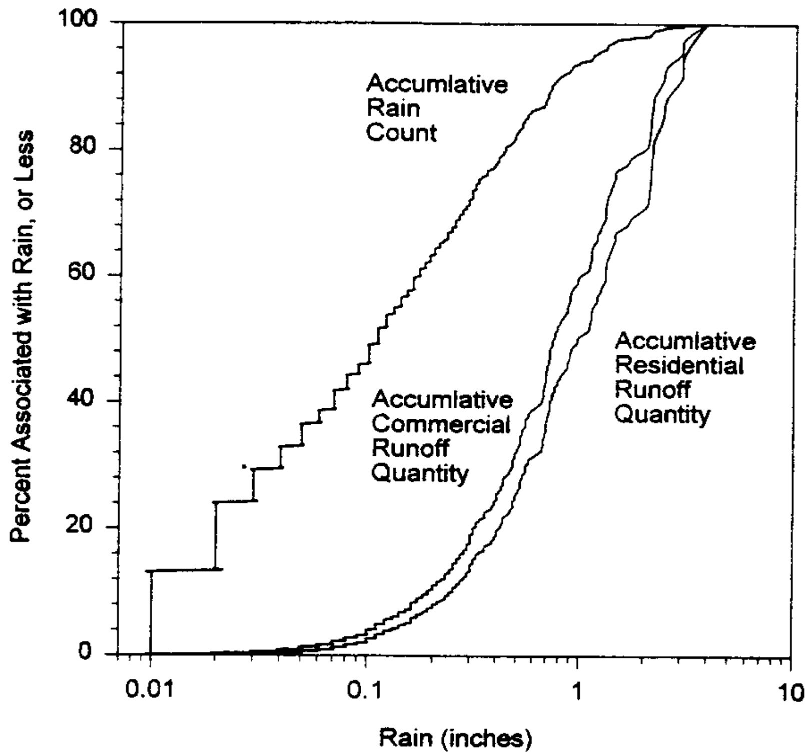


Figure 116. Recorded rain count and modeled runoff volume distributions for Minneapolis, MN (1983 through 1989).

These simulations were based on 7 years of rainfall records (1983 through 1989), from the NOAA station at the Minneapolis-St. Paul airport and were obtained from CD-ROMs distributed by EarthInfo of Boulder, CO. Hourly rainfall depths for the indicated periods were downloaded from the CD-ROMs into an Excel spreadsheet. The files were slightly modified (by eliminating the daily total rainfall column) and saved as a comma delineated file. This file was then read by an utility program included in the WinSLAMM package. This rainfall file utility combined adjacent hourly rainfall values into individual rains, based on user selections (at least 6 hrs of no rain was used to separate adjacent rain events and all rain depths were used, with the exception of the "trace" values). These rain files for each city were then used in SLAMM for typical medium density and strip commercial developments. The median rainfall was 0.11 inches, while rainfall depths of about 0.73 to 1.0 inch correspond to the median runoff depth, depending on the land use.

The CDF plot (Figure 116) shows two distinct "breakpoints" which separate the distributions into the following three general categories:

• less than lower breakpoint: small, but frequent rains. These generally account for 50 to 70 percent of all rain events (by number), but only produce about 10 to 20 percent of the runoff volume throughout the U.S. Figure 116 shows that the rain depth for this breakpoint was about 0.22 inches for Minneapolis during this 7 year period, and 68% of all rains were less than this value. Nine to 13% of the runoff volume was associated with these smaller rains, depending on the land use. These events are therefore most important because of their frequencies, not because of their mass discharges. These rains are therefore of great interest where water quality violations associated with urban stormwater occur. This would be most common for bacteria (especially fecal coliforms) and for total recoverable heavy metals which typically exceed receiving water numeric criteria during practically every rain event in heavily urbanized drainages having separate stormwater drainage systems.

• between the lower and upper breakpoint: moderate rains. These rains generally account for 30 to 50 percent of all rains events (by number), but produce 75 to 90 percent of all of the runoff volume throughout the U.S. Figure 116 shows that the rain depth of this upper breakpoint is about 2.8 inch for Minneapolis during this 7 year period, and about 84% of all runoff was between the two breakpoints, while only 32% of the rains were in this range. These intermediate rains also account for most of the pollutant mass discharges and much of the actual receiving water problems associated with stormwater discharges.

• above the upper breakpoint: large, but rare rains. These rains include the typical drainage design events and are therefore quite rare. During the period analyzed, less than 1 percent of the rains were greater than this breakpoint (only 11 events in 38 years, including a 10 inch rain that occurred on July 23, 1987, as shown in Table 42). These rare events accounted for about 5 percent of the runoff on an annual basis, as shown on Figure 116. Obviously, these events must be evaluated to ensure adequate drainage and for habitat protection.

Table 42. Very Large Rains Occurring from 1952 to 1990 at Minneapolis-St. Paul International Airport

Date	Rain Depth (inches)	Rain Duration (hours)
6/25/78	2.88	8
6/7/84	2.94	12
6/21/86	3	10
5/31/65	3.01	13
10/14/66	3.13	32
10/7/70	3.2	61
8/26/78	3.65	14
7/20/87	3.8	9
7/7/55	3.89	9
8/30/77	7.35	11
7/23/87	10	6

A continuous analysis of proposed water quality control practices is therefore needed in order to evaluate how the practices affect the stormwater discharges in each of these three major rainfall categories.

Estimated Performance of Minneapolis-St. Paul International Airport Pond Design

WinDETPOND was used to evaluate the proposed pond at the Minneapolis-St. Paul International Airport. Table 43 is a summary of the overall pond performance for the three major rain categories described above, while Figure 117 shows the expected performance plots for this pond, and Figure 118 shows the predicted rainfall-runoff relationship for the airport drainage. Obviously, the smaller rains and flows experience a much greater level of treatment than the larger rains. The following summarizes the overall expected pond performance:

• the flow-weighted particle size control is about 5.1 μm, corresponding to an estimated flow-weighted suspended solids control of about 89% using the “midwest” particle size distribution.

• if using the “low” particle size distribution (made up predominately of smaller particles), then the estimated flow-weighted suspended solids control would be about 65%.

• if using the “high” particle size distribution (made up predominately of larger particles), then the estimated flow-weighted suspended solids control would be about 97%.

Table 44 is a statistical summary of the modeled pond performance for this proposed pond for the 38 year analysis period. This period contained almost 4,000 events, ranging from 0.01 to 10 inches, with interevent periods ranging up to 34 days. Only about 1% of the total pond outflow occurred as evaporation and only about 10% of the pond water was displaced during the median rain event. The pond displacement volume (the water volume in the pond at the beginning of the event) was about equal to a 0.5 inch rainfall. The intermediate rainfall category (0.22 to 2.8 inches) had event flushing ratios ranging from 0.25 to 6.8, with most of the events in this critical category displacing several times the pond volume during the event period. In other words, most of the treatment is likely occurring during the relatively short runoff period (5 to 24 hours) as dynamic settling and not during interevent periods as quiescent settling.

Table 43. Predicted Wet Detention Pond Performance at Minneapolis-St. Paul International Airport

Rain category	Occurrence of rains in this category (% of all rains)	Rain range (inches)	Predicted critical particle size control (μm) (flow-weighted)	Predicted suspended solids control (%) (flow-weighted)	Percentage of annual runoff volume in category
Frequent, small rains	68%	<0.22 inch	1.0 μm	99%	8.8%
Common, intermediate rains	32%	0.22 to 2.8 inches	4.8 μm	89%	84.1%
Rare, large rains	<1% (11 in 38 years)	>2.8 inches	15 μm	71%	7.1%

The first category, the most frequent, but smallest rains, account for about 68% of all rains (by count), but only 8.8% of the airport runoff quantity. These rains are most significant from a water quality standard violation standpoint, as almost all rains are likely to exceed water quality standards for bacteria and some of the total recoverable heavy metals. Much of these flows would be infiltrated through the grass-lined drainages at the airport. The directly-connected impervious areas draining directly to the drainage systems and the proposed detention pond will contribute most of the expected flows during these small rains. The proposed detention pond will remove almost all of the suspended solids in the runoff, and much of the associated other pollutants (especially the heavy metals) during these small rains, greatly reducing the frequency of water quality violations.

The intermediate category of rains are responsible for most of the annual runoff volume (84.1%). Runoff from this category of rains would most likely be responsible for most of the receiving water problems. Much of the runoff from the smallest rains in this category would likely be infiltrated at the upland grass waterways, but the larger rains would produce some runoff from these "disconnected" areas in addition to most of the runoff from the directly connected paved areas. The proposed pond is estimated to remove most of the particulate pollutants greater than about 5 μm in size (and about 89% of the suspended solids) from the runoff from these rains.

The third category of rains (>2.8 inches) account for only 7.1% of the annual airport runoff, and originate from only 0.3% of the rain events. Fifteen events over the 38 years would have been expected to cause an overflow of the emergency spillway of the pond, possible causing catastrophic pond failure (especially the maximum 10 inch rain, while the other excessive rains would have produced much less of an overflow). The proposed pond design therefore has a bypass structure that will divert large flows around the pond and discharge them directly into the Minneapolis River untreated. A later discussion presents an analysis to recommend the bypass flow rate. The water treated in the pond in this category would provide capture of all particulates greater than about 15 μm, corresponding to a suspended solids level of control of about 71%.

The estimated long-term averaged suspended solids control is therefore about 88%, mostly associated with the intermediate-sized events.

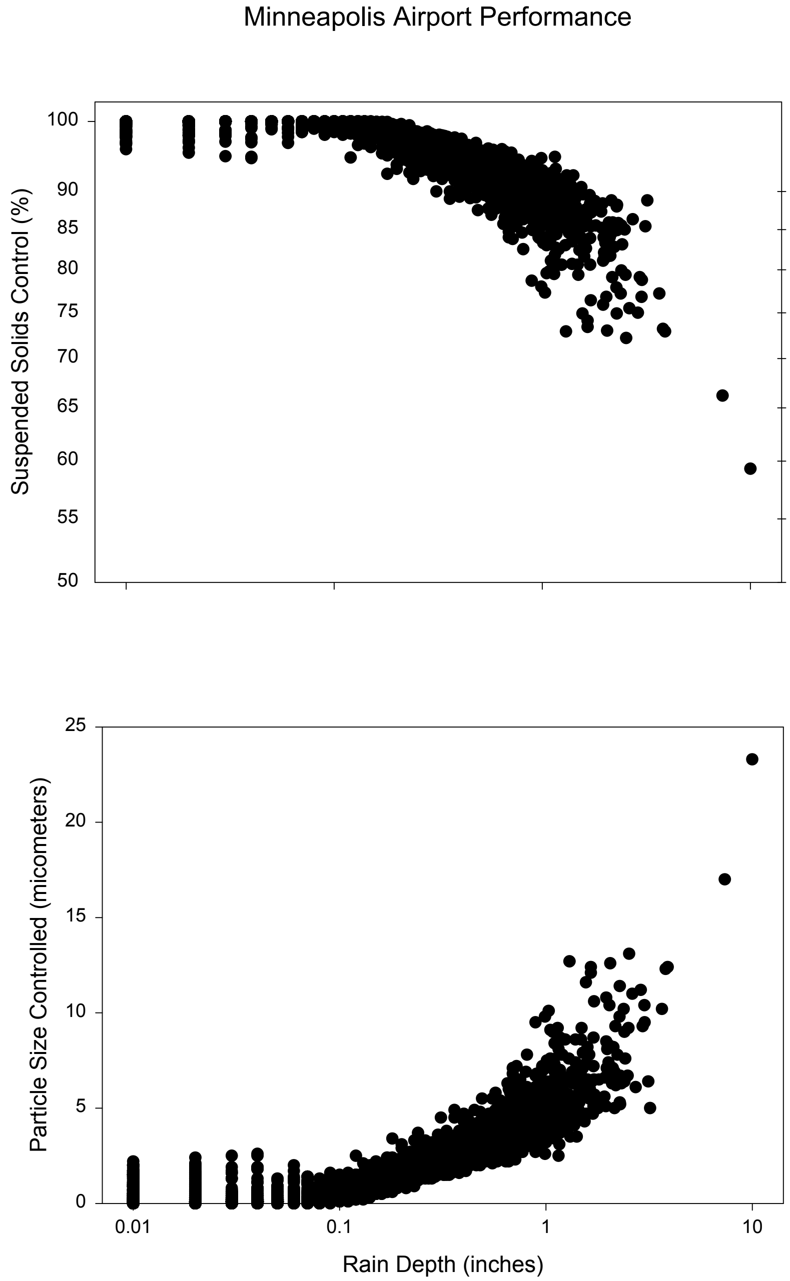


Figure 117. Suspended solids and particulate control as a function of rain depth.

Table 44. Pond Performance Summary for 38 Year Rain Series for Proposed Airport Pond Design

	Rain depth (in)	Rain duration (hrs)	Interevent duration (days)	Rain intensity (in/hr)	Maximum pond stage (ft)	Minimum pond stage (ft)	Event inflow volume (ac-ft)	Event hydraulic outflow (ac-ft)	Event evaporation outflow (ac-ft)	Event total outflow (ac-ft)	Flow-weighted particle size controlled (μm)	Approximate suspended solids control (%)	Peak reduction factor	Event flushing ratio
number	3997	3997	3997	3997	3997	3997	3997	3997	3997	3997	3997	3997	3680	3997
total	1033	24829	10647				40016	39535	444	39980				
% flow out								98.89	1.11					
num avg	0.26	6.21	2.66	0.05	6.25	5.24	10.01	9.89	0.11	10.00	1.45	97.31	0.68	0.61
fl wt avg											5.08	89.24		
median	0.10	4.00	1.46	0.02	5.81	5.20	1.67	2.24	0.05	2.36	0.60	99.90	0.76	0.10
min	0.01	1.00	0.00	0.00	4.89	4.77	0.01	0.00	0.00	0.01	0.00	59.30	0.02	0.00
max	10.00	79.00	34.31	1.67	25.34	6.48	807	807	1.25	807	23.30	100.00	1.00	49.04
st dev	0.43	6.8	3.4	0.08	1.24	0.17	23.7	23.4	0.14	23.4	1.94	4.40	0.26	1.44
COV	0.59	0.90	0.76	0.58	5.0	29	0.42	0.42	0.75	0.42	0.74	22	2.5	0.42
1%	0.01	1	0	0	5.05	4.96	0.014	0.008	0.002	0.02	0	81.4	0.07	0.001
5%	0.01	1	0	0.01	5.13	5.04	0.014	0.042	0.005	0.07	0	88.0	0.16	0.001
10%	0.01	1	0	0.01	5.18	5.08	0.014	0.095	0.009	0.13	0	91.3	0.25	0.001
20%	0.02	1	0.132	0.01	5.27	5.11	0.056	0.255	0.017	0.33	0.1	94.4	0.42	0.003
25%	0.03	2	0.24	0.01	5.32	5.13	0.123	0.387	0.022	0.47	0.1	95.6	0.49	0.007
30%	0.03	2	0.41	0.01	5.38	5.14	0.129	0.559	0.027	0.66	0.1	97.0	0.56	0.008
40%	0.06	3	0.88	0.02	5.54	5.17	0.506	1.14	0.039	1.2	0.2	98.9	0.67	0.031
50%	0.10	4	1.46	0.02	5.81	5.20	1.67	2.23	0.054	2.3	0.6	99.9	0.76	0.10
60%	0.16	5	2.16	0.03	6.15	5.23	4.24	4.17	0.077	4.2	1.2	100	0.83	0.25
70%	0.25	7	3.08	0.04	6.58	5.27	8.36	7.54	0.11	7.6	1.8	100	0.88	0.50
75%	0.31	8	3.72	0.05	6.84	5.30	10.9	10.2	0.14	10.	2.3	100	0.91	0.66
80%	0.40	10	4.46	0.06	7.14	5.33	15.1	13.9	0.17	14.	2.7	100	0.93	0.92
85%	0.50	12	5.48	0.08	7.50	5.38	20.0	19.0	0.22	19.	3.2	100	0.94	1.2
90%	0.69	15	6.93	0.11	7.98	5.44	29.1	27.8	0.28	28.	4.0	100	0.96	1.7
91%	0.74	15	7.32	0.12	8.11	5.46	30.8	30.4	0.30	30.	4.2	100	0.96	1.8
92%	0.78	16	7.77	0.13	8.27	5.48	33.6	33.1	0.32	33.	4.3	100	0.97	2.0
93%	0.88	17	8.27	0.14	8.40	5.51	37.9	37.7	0.35	37.	4.7	100	0.97	2.3
94%	0.96	18	8.86	0.15	8.61	5.54	41.4	40.9	0.37	41.	5.0	100	0.97	2.5
95%	1.05	20	9.67	0.17	8.81	5.58	46.1	44.9	0.39	45.	5.3	100	0.98	2.8
96%	1.16	22	10.63	0.19	9.02	5.63	51.8	50.6	0.43	50.	5.7	100	0.98	3.1
97%	1.30	24	11.93	0.22	9.26	5.68	57.8	57.5	0.48	57.	6.3	100	0.98	3.5
98%	1.51	27	13.07	0.27	9.53	5.77	69.0	69.8	0.58	69.	6.9	100	0.98	4.1
99%	1.99	32	15.85	0.37	10.12	5.90	93.8	92.7	0.74	92.	8.3	100	0.99	5.6
99.50%	2.32	38	19.28	0.46	10.69	5.99	112.	111.	0.90	111.	9.8	100	0.99	6.8
99.90%	3.65	48	24.69	0.89	12.15	6.25	192.	190.	1.12	190.	12.6	100	0.99	11.
100%	10	79	34.31	1.67	25.34	6.48	807.	807.	1.24	807.	23.3	100	1.00	49.

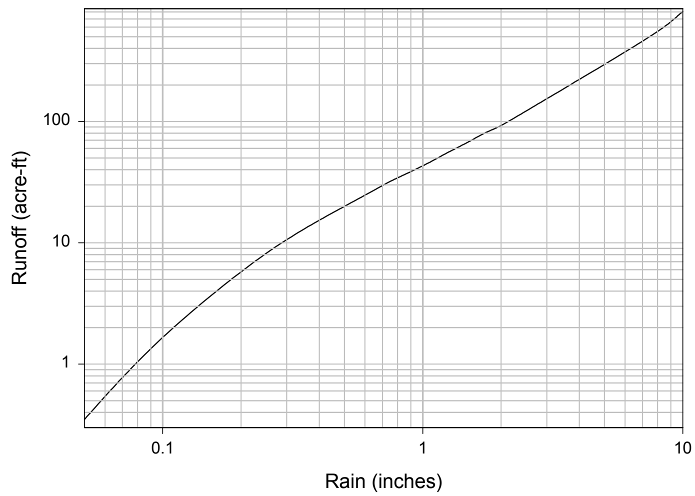


Figure 118. Rainfall-runoff relationship for airport drainage.

Bypass of Excessive Flows around Pond

All low flows will be directed to the pond. However, certain peak flows will be discharge directly to the Minnesota River without passing through the pond, although flows less than this cutoff flow will be discharged to the pond. The diversion will consist of an orifice at the bottom of the storm sewers which will direct the flows below the critical cutoff flow to the pond. A shallow dam will be located immediately downstream to create a head. The excessive flows overtopping this diversion dam will be directed to the river. This diversion structure will not have any type of machinery to ensure safe and unhampered operations.

Various bypass amounts were examined to prevent the pond from exceeding the 10 ft and 11 ft elevations, using a 3.89 inch type ll hydrograph for the site. All influent flows in the influent hydrograph greater than the bypass amount were replaced with the values shown as the bypass amount, allowing the truncated hydrograph to flow to the pond. It is assumed that the excessive flows would then be bypassed to the river directly. Table 45 summarizes the results of these analyses.

Table 45. Evaluation of Alternative By-pass Flows Around Proposed Airport Pond

bypass cutoff	peak stage	hrs >10 ft	hrs >11 ft	max size	%SS reduc
none	13.8 ft.	2.5 hrs	1.7 hrs	19.3 μm	75
500 cfs	11.7	2.2	1.2	15.5	79
400	11.2	2.0	0.7	14.2	81
350	10.9	1.8	0	13.5	82
300	10.6	1.5	0	12.6	83
200	10.0	0	0	6.8	85

Therefore, all flows greater than 200 cfs should be diverted around the pond to keep it from exceeding the 10 ft pond elevation (giving a one foot freeboard), while all flows greater than 350 cfs should be diverted around the pond to keep it from over-topping the 11 ft. roadway elevation.

A conservative estimate is that a “typical” 0.7 inch rain (having a duration of about 6 hrs) may produce a peak runoff of about 200 cfs, depending on the rain intensity variation during the rain. Similarly, a 1 inch rain (again with a duration of about 6 hrs) may produce a peak runoff rate of about 350 cfs. The 0.7 inch rain (or greater) occurs about 10 times a year (based on an analysis of 38 years of Minneapolis-St. Paul airport rainfall data), while the 1 inch rain (or greater) occurs about 5 times a year. As indicated above, the by-passes are for only portions of these events, not for the whole events. Very little of the runoff volume would be by-passed during rains close to these “cutoff” rains, but larger portions of larger events would be bypassed.

As shown on Table 46, about 95% of the annual runoff volume (would vary due to actual rain durations) would pass through the pond with a 200 cfs bypass. This assumes that the first 200 cfs of all events would pass through the pond, and the excess flows would be diverted around the pond untreated. This estimate assumes about 80% of the maximum volume calculated using a constant 200 cfs flow rate and the runoff duration would be treated in the pond. This decrease is due to delays in the rising limb and extended recession limbs of the inflow hydrographs (the geometry of the inflowing hydrograph would truncate the upper corners of the assumed rectangular hydrograph if using a constant 200 cfs flow for the total event duration). The water passing through the pond would receive good treatment, as noted elsewhere in this report (likely greater than 80% SS reduction, even using the "worst-case" type ll hydrograph for the largest events).

Table 46. Amount Treated by Pond, with By-Pass (First 200 CFS of Each Event Treated in Pond)

Rain Depth (in)	Approx. Runoff Duration (hrs)	Potential Volume Treated in Pond for this Duration (acre-ft)[1]	Runoff Depth for this Rain (ac-ft)	Volume Bypassed (ac-ft)	Volume Treated in Pond (ac-ft)	% of Runoff Treated in Pond	% of Annual Runoff Volume for this Rain Depth, and Less	% of Annual Runoff in Range[2]	Average % Treated in Range	Incremental Amount of Runoff Treated in Range
0.09	2	26	1	0	1	100	1	1.0	100	1.0
0.17	3	40	5	0	5	100	5	4.0	100	4.0
0.24	3	40	8	0	8	100	10	5.0	100	5.0
0.38	6	79	14	0	14	100	20	10.0	100	10.0
0.44	6	79	17	0	17	100	25	5.0	100	5.0
0.49	6	79	20	0	20	100	30	5.0	100	5.0
0.67	6	79	27	0	27	100	40	10.0	100	10.0
0.81	6	79	35	0	35	100	50	10.0	100	10.0
1.04	6	79	45	0	45	100	60	10.0	100	10.0
1.32	6	79	57	0	57	100	70	9.9	100	9.9
1.45	6	79	64	0	64	100	75	5.0	100	5.0
1.65	6	79	76	0	76	100	80	5.1	100	5.1
1.96	10	132	94	0	94	100	85	5.0	100	5.0
2.28	10	132	112	0	112	100	90	4.9	100	4.9
2.4	10	132	118	0	118	100	91	1.2	100	1.2
2.51	10	132	125	0	125	100	92	0.9	100	0.9
2.72	10	132	137	5	132	97	93	1.2	98	1.2
3.01	10	132	155	22	132	85	94	1.1	91	1.0
3.13	10	132	161	28	132	82	95	0.8	84	0.7
3.8	10	132	198	65	132	67	96	1.4	75	1.0
3.89	10	132	204	71	132	65	97	0.5	66	0.3
7.35	15	198	490	292	198	40	98	1.2	53	0.6
10	15	198	808	609	198	25	100	2.0	33	0.7
										sum: 97

[1] at 80% of maximum potential due to geometry of inflow hydrograph truncating upper corners of rectangular hydrograph
[2] between rain depth and next smaller rain depth

Short-Circuiting Factor Effects

The Hazen equation illustrates how WinDETPOND considers "short-circuiting". This method is based on a specific volume of water passing through the pond faster than the average residence time, providing less treatment. Short-circuiting is calculated assuming hypothetical ponds in series: little short-circuiting is possible if many ponds are connected in series, while more will occur when few ponds are connected. The main effect of short-circuiting is an increase in the number of large particles that may pass through a pond. Table 47 summarizes multiple evaluations of the proposed airport pond for different short-circuiting factors. The following list summarizes the observations from these analyses:

- short-circuiting increases the discharge of large particles, but with relatively small increases in suspended solids discharges.

- the effects of short-circuiting are worse for larger events (and for smaller ponds). The worst reduction in SS reductions was for the very large 10 inch rain, where the theoretical SS removal was 59%, while the SS removal for very poor short-circuiting conditions (n=1) was reduced to about 56%.

- the elongated pond shape and the isolated inlets and outlets are expected to result in a pond with little short-circuiting.

- the largest particle sizes discharged for very good short-circuiting conditions (n=8) is about 9 um for 0.25 inch rains, about 13.5 µm for 0.7 inch rains, 22 µm for 1 inch rains, and 34 µm for 2 inch rains. Again, few of these large particles would actually be discharged.

Table 47. Effects of Different Short-Circuiting Factors on Pond Performance

Event percentile	Exceedence frequency (#/yr)	Rain depth (in)	Rain duration (hrs)	Inflow volume (ft3)	Maximum pond stage (ft)	Flushing ratio	Peak reduction factor	Max. part. Size trapped (theoretical)	Flow-weighted part. Size (theoretical)	Percent suspended solids removal (theoretical)	Max. part. Size trapped (n=8)	Percent SS removed (n=8)	Max. part. Size trapped (n=3)	Percent SS removed (n=3)	Max. part. Size trapped (n=1)	Percent SS removed (n=1)
1	107	0.01	1	613	5.01	0	1	0	0	100	0	100	0	100	0	100
5	103	0.01	1	613	5.01	0	1	0	0	100	0	100	0	100	0	100
10	97	0.01	1	613	5.01	0	1	0	0	100	0	100	0	100	0	100
20	86	0.02	2	2,450	5.02	0	1	0	0	100	0	100	0	100	0	100
30	76	0.03	2	5,510	5.04	0.01	1	0	0	100	0	100	0	100	0	100
40	65	0.06	2	22,400	5.14	0.03	1	0	0	100	0	100	0	100	0	100
50	54	0.10	3	72,700	5.43	0.10	0.97	0.2	0.1	100	0	100	0	100	0	100
60	43	0.16	3	185,000	5.97	0.26	0.92	1.3	0.9	99.7	0	100	0	100	0	100
70	32	0.25	3	357,000	6.58	0.50	0.81	2.5	1.8	97.1	9	97.6	13.5	97.1	175	95.0
80	22	0.40	6	678,000	7.41	0.95	0.64	4.1	3.0	93.7	9	94.3	13.5	93.7	175	91.1
90	11	0.69	6	1,270,000	8.4	1.77	0.46	6.1	4.5	90.1	13.5	90.7	22.0	89.9	175	86.4
95	5	1.05	6	2,020,000	9.24	2.82	0.30	8.4	6.2	86.2	22.0	87.2	33.8	86.2	380	82.1
99	1	1.99	6	4,140,000	10.66	5.78	0.16	12.7	9.7	78.2	33.8	80.5	55.0	79.2	380	74.7
100	0	10	6	35,100,000	25.34	49.04	0.12	26.9	23.5	59.0	92.0	62.7	175	61.3	2000	56.4

Sizing and Performance of Airport Wet Detention Pond Based on Simple Design Criteria

As a comparison to the preliminary pond design, an airport wet detention pond was sized based on simple guidance, ignoring actual site constraints. The performance of this pond was also evaluated using 38 years of airport rainfall data.

The first criteria in sizing a detention pond for water quality is to provide a surface area equal to about 3% of the paved drainage area in order to control particles larger than about 5 μm. For the airport site, 353 acres of pavement will drain to the pond, along with 622 acres of sandy soil pervious areas and 210 acres of pavement that is drained through surface swales across the sandy soil. Because of the high rate of infiltration of the sandy soil, the pond can be sized only for the directly connected paved area. Therefore, the optimal pond design would include a permanent pond surface area of about 10.6 acres.

The second criteria in sizing a pond is to provide a "live" storage volume equal to the runoff volume associated with a rain of about 1.25 inches in depth. Figure 119 is a plot of the estimated runoff volumes (in acre-ft) associated with different rain depths. This plot was produced using WinDETPOND output data for almost 4,000 rains ranging from 0.05 to 10 inches and for the tributary areas shown above. This plot shows that a rainfall of 1.25 inches would produce about 55.6 acre-ft of runoff. Table 48 lists the resulting side slopes associated with different pond depths.

Table 48. Side Slope Calculations of Full-Size Airport Pond

Depth (above the normal water elevation)	Pond area at this depth	Resulting side slope of pond
2 ft.	46 acres	0.5%
3	27	1.3
4	17.0	3.9
5	11.7	25

In order to construct a pond having this volume, normal surface area, and a side slope of about 4%, the live storage pond depth above the normal water level would be about 4 feet. The surface area at 4 ft above the normal pond surface would therefore be about 17 acres.

The final criteria in sizing a wet detention pond is to select the outlet devices to provide at least 5 μm control at all pond stages. The critical settling velocity of a 5 μm particle is about 1.3×10^{-4} ft/sec. The maximum outlet discharge is equal to this velocity times the surface area (the surface overflow rate). Several choices are possible with this pond, including: a single 90° v-notch weir, two 60° v-notch weirs, a 5 ft. sharp-crested rectangular weir (a little too large), or two 36 inch vertical drop structures. Table 49 summarizes these outfall options.

Table 49. Alternative Discharge Devices for Full-Size Airport Pond

Stage above lowest invert	Pond area at this stage	Maximum allowable discharge at this stage for 5 μm control	Discharge for a single 90° v-notch weir	Total discharge for two 60° v-notch weirs	Discharge for a single 5 ft. sharp-crested rectangular weir	Total discharge for two 36" drop structures
0 ft	10.6 acres	60 cfs	0	0	0	0
1	12.2	69	2.5	2.8	16	14
2	13.8	78	14	16	43	56
3	15.4	87	39	56	80	84
4	17.0	96	80	92	110	84

The 60° v-notch weirs provide the best solution because they are the closest fit at the 4 ft stage, while providing substantially better performance at lower elevations than the rectangular weir or the drop structures.

In addition to these “water quality” discharges, another spillway needs to be provided for rarer events that may not be contained within these outlet devices. A rectangular weir 7.8 ft long and 2.5 ft high extending from the 4 ft stage (above the normal water surface) was included in the preliminary design and was therefore used for this design. In addition, a road crossing provides another emergency spillway for rare storms.

This pond design was evaluated using the rain history (3997 separate events) from the 38 year period from 1952 through 1989. Table 50 summarizes the performance of this hypothetical pond, for comparison to the proposed pond design. This larger pond provides a flow-weighted control for particles greater than 2.2 µm. For the “Midwest” particle size distribution, this corresponds to an approximate flow-weighted suspended solids control of about 96%. Using the “low” particle size distribution, this would correspond to an approximate flow-weighted suspended solids control of about 85%, and using the “high” particle size distribution, this would correspond to an approximate flow-weighted suspended solids control of about 99%. Particles larger than 5 µm (at least) would be theoretically trapped in the pond whenever the surface water elevation was below the rectangular weir. If the pond water elevation was near the invert of the v-notch weirs, then the particle size control would be much better. Similarly, whenever the pond water level is within the rectangular weir, particles larger than 5 µm would be discharged. Of course, it is likely that some particles larger than 5 µm would be discharged at lower pond surface elevations due to potential short-circuiting. As shown previously, with large short-circuiting (not expected with the elongated design of the pond) the discharge of some large particles would occur, but the pond suspended solids control is only reduced by a small amount. This larger pond therefore has a relatively large marginal improvement over the proposed pond design (96% vs. 88%), but at about three times the area. However, this larger pond is not suitable for the site because of limited available space at the airport.

Table 50. Pond Performance Summary for 38 Year Rain Series for Large Pond Design

	Rain depth (in)	Rain duration (hrs)	Interevent duration (days)	Rain intensity (in/hr)	Maximum pond stage (ft)	Minimum pond stage (ft)	Event inflow volume (ac-ft)	Event hydraulic outflow (ac-ft)	Event evaporation outflow (ac-ft)	Event total outflow (ac-ft)	Flow-weighted particle size controlled (μm)	Approximate suspended solids control (%)	Peak reduction factor	Event flushing ratio
number	3997	3997	3997	3997	3997	3997	3997	3997	3997	3997	3997	3997	3492	3997
total	1033	24829	10648				40016	38854	1113	39967				
% flow out								97.22	2.78					
num avg	0.26	6.21	2.66	0.05	5.99	5.34	10.01	9.72	0.28	10.00	0.60	99.27	0.76	0.35
fl wt avg											2.24	95.82		
median	0.10	4.00	1.46	0.02	5.70	5.29	1.67	2.83	0.13	3.14	0.20	100.00	0.83	0.06
min	0.01	1.00	0.00	0.00	4.95	4.79	0.01	0.00	0.00	0.01	0.00	72.10	0.00	0.00
max	10.00	79.00	34.31	1.67	20.46	6.75	807.71	804.65	3.10	805.15	12.90	100.00	1.00	28.51
st dev	0.43	6.9	3.5	0.081	0.89	0.22	24	23	0.37	23	0.87	1.8	0.21	0.84
COV	0.59	0.90	0.76	0.58	6.7	24	0.42	0.43	0.76	0.44	0.69	56	3.5	0.42
1%	0.01	1	0	0	5.10	5.01	0.014	0.021	0.005	0.045	0	92.2	0.08	0
5%	0.01	1	0	0.01	5.17	5.10	0.014	0.091	0.013	0.143	0	95.2	0.30	0
10%	0.01	1	0	0.01	5.22	5.13	0.014	0.182	0.021	0.260	0	97.4	0.44	0
20%	0.02	1	0.13	0.01	5.31	5.17	0.056	0.459	0.043	0.598	0	99.1	0.59	0.002
25%	0.03	2	0.24	0.01	5.36	5.19	0.123	0.681	0.054	0.847	0	99.7	0.65	0.004
30%	0.03	2	0.41	0.01	5.42	5.21	0.129	0.950	0.067	1.13	0.1	99.9	0.70	0.005
40%	0.06	3	0.88	0.02	5.54	5.25	0.506	1.69	0.096	1.96	0.1	100	0.77	0.018
50%	0.10	4	1.46	0.02	5.7	5.29	1.67	2.83	0.134	3.13	0.2	100	0.83	0.059
60%	0.16	5	2.16	0.03	5.9	5.33	4.24	4.78	0.195	5.08	0.4	100	0.87	0.15
70%	0.25	7	3.08	0.04	6.16	5.39	8.36	7.69	0.293	8.10	0.6	100	0.91	0.29
75%	0.31	8	3.72	0.05	6.35	5.43	10.9	10.1	0.356	10.4	0.9	100	0.92	0.38
80%	0.40	10	4.46	0.06	6.53	5.47	15.1	13.0	0.442	13.5	1.2	100	0.94	0.53
85%	0.50	12	5.48	0.08	6.78	5.53	20.0	17.4	0.556	17.8	1.4	100	0.95	0.70
90%	0.69	15	6.93	0.11	7.11	5.61	29.1	26.2	0.708	26.4	1.8	100	0.96	1.00
91%	0.74	15	7.32	0.12	7.23	5.63	30.8	28.3	0.766	28.6	1.9	100	0.96	1.08
92%	0.78	16	7.77	0.13	7.32	5.66	33.6	31.1	0.826	31.6	2	100	0.97	1.18
93%	0.88	17	8.27	0.14	7.45	5.69	37.9	35.1	0.889	35.5	2.1	100	0.97	1.34
94%	0.96	18	8.86	0.15	7.63	5.72	41.4	39.0	0.944	39.5	2.2	100	0.97	1.46
95%	1.05	20	9.67	0.17	7.75	5.77	46.1	42.8	1.00	43.1	2.4	100	0.97	1.62
96%	1.16	22	10.63	0.19	7.97	5.82	51.8	47.9	1.08	48.2	2.6	100	0.98	1.82
97%	1.30	24	11.93	0.22	8.17	5.89	57.8	56.2	1.19	56.8	2.8	100	0.98	2.04
98%	1.51	27	13.07	0.27	8.47	5.96	69.0	65.4	1.43	65.5	3.1	100	0.98	2.43
99%	1.99	32	15.85	0.37	8.92	6.11	93.8	87.3	1.85	87.6	3.6	100	0.99	3.31
99.50%	2.32	38	19.28	0.46	9.72	6.25	112.	109.	2.21	110.	4.4	100	0.99	3.96
99.90%	3.65	48	24.69	0.89	10.86	6.59	192.	181.	2.74	181.	5.6	100	0.99	6.81
100%	10.00	79	34.31	1.67	20.46	6.75	807.	804.	3.10	805.	12.9	100	1.00	28.5

Suggested Pond Modifications to Enhance Performance

The following discussion presents some suggestions to further enhance the performance of the proposed wet detention pond at the Minneapolis-St. Paul International Airport. The most important enhancements relate to special winter operations, where the pond water level should be drawn down during the winter to isolate the sediments by ice from snowmelt that may otherwise flow under the ice. This would also increase the effective storage volume for snowmelt and provide additional storage for winter runoff that may be contaminated by de-icing compounds. This would allow the winter runoff to be pumped to separate facilities for treatment of the de-icing compounds.

Another suggested enhancement would be to add a capability for surface aeration to the pond. This would increase mixing during interevent periods to reduce stratification, increase photo-degradation of toxicants, and provide an excess of dissolved oxygen (especially important considering the very high BOD_5 of common de-icing compounds that may enter the pond). Aeration could be used intermittently, depending on the pond conditions.

A subsurface outlet would enhance floatable control and would minimize icing problems. The outlet pipe should be located near the bottom of the pond, but on a sealed surface to minimize scour. The outlet pipe would then be connected to a large subsurface box where the outlet control weir is located. This box would also be outfitted with lower outlet controls for winter operation and for complete drainage of the pond for any required maintenance.

It is strongly suggested that a fore-bay be installed near the pond inlet to minimize the area where most of the sediment would accumulate. The area for the fore-bay should be between 10 and 20% of the total pond area and be separated from the main pond by a subsurface weir/dam (located below the low winter operational pond level). Special access provisions should be provided adjacent to this area to enable easy access to dredging equipment.

The inlet leading to the pond could also be provided with chemical feed facilities to allow chemical treatment under severe conditions. The use of alum has been shown to be problematic in northern areas where pH and buffering capacity of the water may be low, causing aluminum toxicity. However, alum is easy to apply and the floc can be discharged into the pond where it is relatively stable. Ferric chloride is generally a superior coagulant for stormwater, especially in northern areas, allowing the faster formation of a more stable floc that settles much more rapidly than an alum floc. Unfortunately, a ferric chloride floc becomes unstable under anaerobic conditions, which may occur near the sediment interface in a wet detention pond. Therefore, ferric chloride flocs are usually removed before discharge. It may be possible to capture most of the floc in the recommended fore-bay, and to ensure aerobic conditions there through the use of aeration in that area.

Special Issues Associated with Wet Detention Ponds at Airports

There are special recommendations for the use of wet detention ponds at airports that need to be addressed. These have to do with aircraft safety, especially by not providing an attraction to birds. Heavily vegetated perimeters of a pond generally decrease the pond's attractiveness to geese, but they also provide habitat to other wildlife and are not recommended by the FAA. The linear shape of the proposed pond meets the FAA's recommendations, but it is a wet pond, whereas they recommend dry ponds. Unfortunately, dry ponds do not provide adequate water quality treatment. They also recommend steep sides that are rip-rap lined, with minimal vegetation to discourage wildlife. The nearby location of the Valley National Wildlife Refuge and Meadow Lake may make this proposed wet detention pond much less attractive to wildlife than if it was the only body of water in the region.

The FAA published an Advisory Circular (No. 150/5200-33) on May 1, 1997 discussing hazardous wildlife attractants on or near airports. They list the wildlife that have been involved in damaging collisions with civilian aircraft in the U.S. in 1993 – 1995. Waterfowl were involved in 28% of the collisions and wading birds were involved in another 3%. Because of this, they are concerned about land use practices on and near airports that may attract waterfowl. The recommended distance between an aircraft's movement areas, loading ramps, or aircraft parking areas and any wildlife attractants is 10,000 ft for airports serving turbine-powered aircraft, and 5 miles if the wildlife attractant may cause hazardous wildlife movement across or into the approach or departure airspace.

They recommend that artificial marshes (wetland treatment systems for wastewater) not be located within these separation distances. They also recommend against the discharge of wastewater to unpaved airport areas, as the resultant soft or muddy conditions can severely restrict or prevent emergency vehicles from reaching accident sites in a timely manner. These incompatible land uses specifically deal with wastewater treatment facilities and not to stormwater. However, the issues may be similar. Obviously, many airports utilize grass swales to drain airport pavement areas. It is imperative that these swales are designed to minimize standing water and provide good infiltration conditions. Longitudinal infiltration trenches along the swale's lengths, or at least intermittent infiltration areas, could be provided to ensure adequate drainage in these areas. Wetland treatment of airport runoff may also be of concern.

The FAA also lists land uses that may be compatible with safe airport operations, specifically addressing stormwater dry and wet detention ponds. In general, the FAA does not consider these activities to be hazardous to aviation if there is no apparent attraction to hazardous wildlife, or wildlife hazard mitigation techniques are implemented to deal effectively with any wildlife hazard that may arise. They state that both dry and wet detention ponds control runoff (a necessary activity for safe aircraft operations), but also can attract hazardous wildlife. To best control hazardous wildlife, the FAA recommends using steep-sided, narrow, linearly-shaped, rip-rap lined dry detention ponds rather than wet detention ponds. Whenever possible, these ponds should be placed away from aircraft movement areas and that all vegetation in or around dry or wet detention ponds that provide food or cover for hazardous wildlife be eliminated. They also state that if soil conditions permit, the use of underground stormwater infiltration systems, such as French drains or buried rock field be used because they are less attractive to wildlife. Ponds at airports can be very effective. As an example, Yang, *et al*. (2002) reported on the use of aerated wet ponds at CVG airport (Cincinnati/Northern Kentucky) to treat glycol-contaminated streams in cold temperatures. The operation of the pond resulted in a nearly 50% BOD_5 removal and intermittent discharges that appear to have resulted in the reduction of nuisance growth in the once-polluted streams.

Design Suggestions for In-Receiving Water Detention

A preliminary investigation to estimate the level of stormwater control that may be possible by using the flow balancing method (EquiFlow®) at Waller Creek in Austin, TX, was conducted by Pitt (1995b). The FBM technology has been in use in Sweden for several decades for the control of stormwater (Pitt and Dunkers 1992 and 1993; and Pitt 1995). It has recently also been demonstrated in the U.S. for CSO control in New York City (Forndran, *et al*. 1991; Field and Pitt 1994a and 1994b; Field, *et al.* 1994; and Field, *et al*. 1995).

The FBM is constructed using a series of pontoons forming multiple cells in a waterbody. Weighted PVC curtains hang from the pontoons containing the stormwater that enters the FBM from the stormwater discharge location. The curtains divide the FBM into multiple cells that are interconnected by openings. In freshwater applications, the polluted stormwater moves through the FBM in plug flow, passing through successive compartments until its discharge into the receiving water.

Some FBM facilities are connected to a treatment facility on-shore for high levels of phosphate removal using ferric chloride precipitation (Pitt 1995). However, the FBM alone is capable of acting like a stormwater wet detention pond, with similar removals for particulate pollutants. New concepts for the FBM use wetland cells for increased passive removal of nutrients (Fresh Creek Technologies, West Caldwell, NJ, personal communication). The FBM can therefore be evaluated using conventional wet detention pond procedures (Pitt 1993a and 1993b).

In this example design, the maximum surface area of the FBM is limited by (1) a maximum width of 1/4 of the width of Town Lake and (2) the length is restricted by the closest upstream and downstream major stormwater outfalls from other watersheds. In addition, the FBM must be compatible with the rowing club operations near the creek outfall. The preliminary plan is for a six cell FBM extending from a location just downstream from an existing 30 inch storm drain from the downtown area to a location just upstream from an existing 72 inch storm drain from a highway. The upper cell would capture flow from the proposed Waller Creek bypass and would then join the remainder of the Waller Creek discharge in the second cell. The creek stormwater would then flow through cells C through F before final discharge into Town Lake. The final four cells (C, D, E, and F) can be wetland cells providing additional treatment, compared to simple sedimentation. The maximum overall length of the FBM system that could be used (without accepting additional flows) is therefore about 4,000 feet, with five dividing pontoons sections, each about 300 feet long. The maximum total pontoon and curtain length would therefore be about 5,500 feet.

WinDETPOND Input File for Proposed Minneapolis Airport Wet Detention Pond

Pond file name: K:\WDP71\AIRPORT.PND
Pond file description: basic Minn/St Paul airport file
Particle Size file name: K:\WDP71\MIDWEST.CPZ
Output Format Option: Water Quality Summary: One Line per Event
Output device: Print Output to File (extension .DPO)
Date: 06-29-1999

Drainage Basin Runoff Procedure:
Combined Surface Characteristics
1. All directly connected impervious areas (acres): 353
2. All pervious areas (acres): 622
3. All impervious areas draining to pervious areas (acres): 210

Outlet Characteristics:
Outlet number 1
Outlet type: V - Notch Weir
1. Weir angle (degrees): 90
2. Weir height from invert: 6
3. Invert elevation above datum (ft): 5

Outlet Characteristics:
Outlet number 2
Outlet type: Rectangular Weir
1. Weir length (ft): 7.8
2. Weir height from invert: 2.5
3. Invert elevation above datum (ft): 8.5

Outlet Characteristics:
Outlet number 3
Outlet type: Evaporation

Month Number	Evaporation (in/day)
1	.01
2	.01
3	.03
4	.06
5	.1
6	.13
7	.18
8	.18
9	.14
10	.1
11	.04
12	.01

Initial stage elevation (ft): 5
User defined pond efficiency factor (n): 5

Pond Stage, Surface Area, and Stage-related Outfall Devices (if applicable)

Entry Number	Stage (ft)	Pond Area (acres)	Natural Seepage (in/hr)	Other Outflow (cfs)
0	0.00	0.0000	0.00	0.00
1	1.00	3.0500	0.00	0.00
2	2.00	3.3500	0.00	0.00
3	3.00	3.6500	0.00	0.00
4	4.00	4.0000	0.00	0.00
5	5.00	4.4000	0.00	0.00
6	6.00	4.8000	0.00	0.00
7	7.00	5.2000	0.00	0.00
8	8.00	5.6500	0.00	0.00
9	9.00	6.1500	0.00	0.00
10	10.00	6.6500	0.00	0.00
11	11.00	7.2000	0.00	0.00

Rain Information
Rain file name: K:\wdp71\MINN5289.RAN
Rain starting date : 01/09/52
Rain ending date : 12/31/89

This FBM has a maximum surface area of about 23 acres. The depth in the FBM (at normal Town Lake levels) is as deep as about 10 feet, with an average depth assumed to be 4.25 feet. The maximum FBM volume is therefore about 100 acre-feet, or 30 million gallons. A very rough estimated turnkey cost for this FBM would be about $1.5 to 2.0 million, including wetland planting in about 15 acres.

Cell A is adjacent to the rowing club location. This cell (and possibly cell B) would be usable as small craft areas. A short section of the outer curtain at these locations could be supported by barely submerged floats allowing small boats to pass into Town Lake. The pontoons would also provide protected small areas, about 600 feet long and about 150 feet wide. In addition, the pontoons could act as floating walkways as part of the shoreline recreational area.

The flows in Town Lake are summarized in Table 51. These worst-case flow rates were estimated from Town Lake cross sections in the Waller Creek area and from Corps of Engineers water depth and discharge data. The Town Lake cross-sectional areas have the estimated FBM cross-sectional areas subtracted. It is expected that the FBM can withstand all of these flows, with the possible exception of the 500-year event (at 13 ft/sec). In cases of excessive flows in the Colorado River through Town Lake, the FBM would rise with the lake water, with the curtains lifting off the lake bottom and ballooning towards shore, as the river currents require. The FBM would therefore have minimal affects on flood flows as the pontoons and curtains rise in the water with increasing water depths and will balance water on both sides of the curtains (*e.g.*: Flow Balancing Method).

Table 51. Expected Town Lake Water Velocities

Return Period	Discharge (CFS)*	Town Lake Cross-Sectional Area (ft^2)	FBM Cross-Sectional Area (ft^2)	Net Cross-Sectional Area (ft^2)***	Velocity (ft/sec)
1-year	16,000**	6,600	640	6,000	3
10-year	38,700	9,000	1,300	8,000	5
50-year	78,650	14,000	2,700	11,000	7
100-year	102,100	16,000	3,300	13,000	8
500-year	240,000	23,000	5,200	18,000	13

* U.S. Army Corps of Engineers, Travis Co. FIS, March 1979.
** The average of the observed instantaneous peak discharges for each year from 1981 through 1994.
*** Estimated cross-sectional area of Town lake near Waller Creek after subtracting FBM cross-sectional area.

Waller Creek flows are also of interest because they affect the force applied to the FBM curtain opposite the discharge location and the amount of water discharged determines the level of stormwater treatment obtainable. Table 52 summarizes these expected flows at the mouth of Waller Creek. The 1-year discharge may be very roughly estimated to be about 40% of the 10-year value, based on the ratio of the reported discharges in Town Lake. The high flow rates for the smaller events will likely be dissipated near the mouth of the creek, but the design of the FBM will need to consider these high flow rates.

Table 52. Waller Creek Flows at Confluence with Town Lake

Return Period	Discharge (CFS)*	Cross-Section Area (ft^2)	Velocity (ft/sec)
10-year	5,444	500	11
25-year	7,035	1,000	7
100-year	9,424	1,625	6

* 3-hour duration design storms for the City of Austin

The most significant factor affecting wet detention pond performance is the surface of the pond compared to the drainage area. This ratio is a surrogate for the runoff volume expected and the volume of the pond. Table 53 shows the land uses in the Waller Creek watershed and the expected annual runoff volume. Waller Creek is unusual in that the educational land use (the main campus of the University of Texas) comprises the largest flow contributor (about 32%). The single family residential area is next, at about 25%. It is expected that the annual 31.5 inches of rainfall at Austin falls during about 400 hours.

Table 53. Land Uses and Annual Runoff for Waller Creek

Land Use	Area (acres)	Estimated Annual Rv	Annual Runoff Volume (acre-ft)*	Percent of Annual Runoff
Vacant/Undeveloped	77	0.1	20	0.5
Park	127	0.1	33	0.8
Single family residential	1358	0.3	1100	25
Multiple family resid.	234	0.4	250	6
Office	247	0.5	330	8
Commercial	416	0.5	550	13
Industrial	146	0.6	230	5
Major roadways	180	0.85	400	9
Utilities	9	0.5	12	0.3
Civil/educational	866	0.6	1400	32
Water	2	1.0	6	0.1
Total	3662		4300	100

* assuming an annual rainfall of 31.5 inches.

Table 54 shows the recommended "wet detention pond" surface area for Waller Creek for two levels of control. The 5 μm level (practically all particles having greater settling rates than 5 μm particles would be trapped) corresponds to a suspended solids control of about 90% for stormwater, while the 20 μm level corresponds to a suspended solids control level of about 65%. Table 55 shows estimated control levels for other pollutants for these two surface areas. These values do not include any additional control associated with the establishment of a wetland system within the FBM. The use of wetland attributes can be expected to increase the removals of most of the pollutants during the active growing season. The maximum available surface area for the FBM (about 23 acres) indicates that the 20 μm level of control may be a reasonable expectation for this proposed installation (65% control for suspended solids, 40% for COD, BOD5 and phosphorus, and 60% for lead and copper, with increased control levels during the active growing season associated with the wetland cells in the FBM). A small suction dredge would have to be periodically used to remove the captured sediments from the FBM.

Table 54. Recommended Wet Pond Surface Areas

Land Use		5 μm control		20 μm control	
	Area (acres)	% of area	Area (acres)	% of area	Area (acres)
Vacant/Undeveloped	77	0.6	0.46	0.2	0.15
Park	127	0.6	0.76	0.2	0.25
Single family residential	1358	0.8	11	0.3	4.1
Multiple family resid.	234	0.8	1.9	0.3	0.7
Office	247	1.7	4.2	0.6	1.5
Commercial	416	1.7	7.1	0.6	2.5
Industrial	146	2.0	2.9	0.8	1.2
Major roadways	180	2.8	5.0	1.0	1.8
Utilities	9	1.7	0.15	0.6	0.05
Civil/educational	866	1.7	15	0.6	5.2
Water	2	0	0	0	0
Total	3662	1.3	48	0.5	17

Table 55. Estimated Pollutant Control for Two Surface Areas

Pollutant	5 μm control (48 acres)	20 μm control (17 acres)
Suspended solids	90 %	65 %
COD	50	40
BOD_5	50	40
Phosphorus	50	40
Nitrate	50	40
TKN	40	25
Lead	80	60
Copper	80	60
Zinc	50	40

Another method of predicting the FBM performance is by examining settling profiles in the cells. The annual peak instantaneous flow rate through the FBM from Waller Creek is expected to be about 1.3 ft/sec (associated with a discharge of about 2200 CFS, or about 40% of the 10-year discharge rate of 5444 CFS, with an FBM cross-sectional area of about 1625 ft^2). The annual average wet-weather flow rate is only expected to be about 0.1 ft/sec through the FBM cells. Table 56 shows the expected worst-case particle sizes controlled by plug flow conditions, while Table 57 shows the annual average flow particle settling conditions. The 100 ft. flow length corresponds to an area near the outfall within the first cell, while the 500 ft. flow length is approximately after the first cell. The first cell near the discharge location (either the bypass or the natural creek confluence) would therefore result in about a 70 percent suspended solids reduction for average flow conditions, degrading to about 40 percent for peak annual flows. These tables show that the use of all six cells would result in levels of control similar to the levels predicted previously using the surface area ratio values. An FBM system half as long as the six-celled unit shown here would cost much less, interfere less with Town Lake activities, and provide about 85% of the pollutant control as the full length version. It was recommended that this smaller unit be initially constructed and monitored as a demonstration facility. If the performance is as expected and additional control is desired, then the facility can be expanded.

Table 56. Annual Peak Flow (Worst Case) Particle Settling in FBM*

Flow Length (feet)	Travel Time (minutes)	Critical Particle Settling Rate (cm/sec)	Critical Particle Size (μm)**	Approx. Suspended Solids Control (%)
100	1.3	1.7	150	20
500	6.4	0.34	60	40
1,000	13	0.17	55	50
2,000	25	0.087	35	55
3,000	38	0.057	25	60
4,000	50	0.043	20	65

* assuming an average FBM depth of 4.25 feet and a velocity of 1.3 ft/sec.
** assuming particles have a specific gravity of 2.65 and are spherical.

Table 57. Annual Average Flow Particle Settling in FBM*

Flow Length (feet)	Travel Time (minutes)	Critical Particle Settling Rate (cm/sec)	Critical Particle Size (μm)**	Approx. Suspended Solids Control (%)
100	21	0.10	40	45
500	100	0.021	15	70
1,000	210	0.011	12	75
2,000	415	0.0054	7	80
3,000	620	0.0035	6	85
4,000	830	0.0026	5	90

* assuming an average FBM depth of 4.25 feet and a velocity of 0.08 ft/sec.
** assuming particles have a specific gravity of 2.65 and are spherical.

The large size of the Waller Creek watershed (and corresponding large stormwater flows) requires a large "end-of-pipe" treatment device for significant pollutant reductions. The maximum FBM that could be used at the Town Lake site would have about 23 acres of surface, equivalent to about 0.6 percent of the drainage area. This maximum sized FBM is expected to control suspended solids at the 65% level for the peak one-year flow conditions and at the 90% level for the average annual flow conditions. The first cell of the proposed six cell FBM would control at least 20% of the suspended solids associated with the annual peak flow conditions. The last four cells could have wetland attributes for much improved pollutant removals during the growing season.

Calculation of Groundwater Rise underneath a Seepage Basin using WinDETPOND

Potential Groundwater Contamination Beneath Seepage Basins

Pitt, *et al.* (1994; 1996) examined the potential effects of stormwater on groundwater quality based on the likely presence of problem constituents in the stormwater, their mobility through soils, the type of treatment received before infiltration, and the infiltration method used. The constituents of most concern include chloride, certain pesticides (lindane and chlordane), organic toxicants (1,3-dichlorobenzene, pyrene and fluoranthene), pathogens, and some heavy metals (nickel and zinc). Reported instances of groundwater contamination associated with stormwater was rare in residential areas where infiltration occurred through surface soils (except for chloride), but was more common (especially for toxicants) in commercial and industrial areas where subsurface infiltration was used.

There are many types of artificial stormwater infiltration mechanisms that have been used in urbanizing areas in order to decrease discharges of stormwater to surface waters and to help preserve groundwater recharge. These are described in many stormwater design manuals. The following infiltration techniques are most commonly used:

- surface infiltration devices (grass filters and grass-lined drainage swales; infiltration is usually dominant stormwater treatment mechanism; infiltration occurs through turf and surface soils, providing the most opportunities for pollutant trapping before the water reaches groundwater);

• french drains or soak-aways (small source area subsurface infiltration pits, most typically used for infiltrating drainage from roofs; usually simple gravel-filled dug holes, but can be an empty perforated container);

• porous pavements or grid pavers (replace impervious pavements, overlain on a relatively thick storage layer of coarse material; may include drainage pipes to collect excess water that cannot be infiltrated into underlying soil);

• drainage trenches (collect and infiltrate runoff from adjacent paved areas; generally long, moderately wide, and shallow in dimensions; filled with coarse gravel to provide storage);

• infiltration wells, or dry wells (deep, relatively small diameter holes allowing stormwater to be discharged to deep soil horizons, sometimes directly into saturated zones, commonly located at storm drainage inlet locations serving up to a few hectares of drainage area, with overflows discharged to storm or combined drainage system);

• perforated drainage pipes (conventional separate storm drainage, but with perforations through pipe or gaps between pipe segments; usually wrapped in geotextile fabric with coarse gravel used as trench backfill material);

• dry (percolating) basins (usually large storage areas typically located at end of drainage system before discharge into receiving water; commonly used as recreation facilities during dry weather; also provides infiltration through turf and surface soils).

All infiltration devices redirect runoff waters from the surface to the sub-surface environments. Therefore, they must be carefully designed using sufficient site specific information to protect the groundwater resources and to achieve the desired water quality management goals.


Pond close to groundwater table (Berlin, Germany)


Pond close to groundwater table (Madison, WI)

Figure 119. In most cases, ponds need to be located well above the local groundwater table to minimize groundwater contamination.

Table 58 is a summary of the pollutants found in stormwater that may cause groundwater contamination problems for various reasons. This table does not consider the risk associated with using groundwater contaminated with these pollutants. However, the Groundwater Recharge Committee of the National Academy of Science (Andelman, *et al.* 1994) examined risks associated with consuming contaminated groundwater.

General causes of concern indicating probable groundwater contamination potential are:

- high mobility (low sorption potential) in the vadose zone,
- high abundance (high concentrations and high detection frequencies) in stormwater, and
- high soluble fractions (small fraction associated with particulates which would have little removal potential using conventional stormwater sedimentation controls) in the stormwater.

The contamination potential is defined as the most critical rating of the influencing factors. As an example, if no pretreatment was to be used before percolation through surface soils, the mobility and abundance criteria are most important. The filterable fraction is not as important as no treatment is being used, based on the assumption that physical removal of particulates is the most important removal process for stormwater. If a compound was mobile, but was in low abundance (such as for VOCs), then the groundwater contamination potential would be low because the concentrations are low to begin with. However, if the compound was mobile and was also in high abundance (such as for sodium chloride, in certain conditions), then the groundwater contamination potential would be high.

If sedimentation pretreatment is to be used before surface infiltration, then some of the pollutants will likely be removed before infiltration. In this case, all three influencing factors (mobility, abundance in stormwater, and soluble fraction) would be considered important.

If subsurface injection (with minimal pretreatment) is to be used, then only the abundance factor is significant. If the pollutant is present in adverse concentrations, it will likely have an adverse effect on the groundwater. Attenuation through the vadose zone (as reflected in the mobility factor) may be insignificant as the water would bypass the vadose zone for a deep injection well. Similarly, the filterable fraction of the pollutant would be less important as no treatment is conducted before disposal. However, pollutants that are mostly in filterable forms would likely have a greater effect on the groundwater quality than those mostly associated with particulates.

As an example, chlordane would have a low contamination potential with sedimentation pretreatment, while it would have a moderate contamination potential if no pretreatment was used. However, if subsurface infiltration/injection was used instead of surface percolation, both the mobility and the abundance factors would be important, with some regard given to the filterable fraction information for operational considerations.

This table is only appropriate for initial estimates of contamination potential because of the simplifying assumptions made, such as the likely worst case mobility measures for sandy soils having low organic content. If the soil was clayey and had a high organic content, then most of the organic compounds would be less mobile than shown on this table. The abundance and filterable fraction information is generally applicable for warm weather stormwater runoff at residential and commercial area outfalls. The concentrations and detection frequencies would likely be greater for critical source areas (especially vehicle service areas) and critical land uses (especially manufacturing industrial areas). Other, more detailed methods are possible to access the potential problems caused by stormwater infiltration, such as proposed by Martinelli and Alfakih (1998).

The stormwater pollutants of most concern (those that may have the greatest potential adverse impacts on groundwaters) include:

• nutrients: nitrate has a low to moderate groundwater contamination potential for both surface percolation and subsurface infiltration/injection practices because of its relatively low concentrations found in most stormwaters. If the stormwater nitrate concentration was high, then the groundwater contamination potential would likely also be high.

• pesticides: lindane and chlordane have moderate groundwater contamination potentials for surface percolation practices (with no pretreatment) and for subsurface injection (with minimal pretreatment). The groundwater contamination potentials for both of these compounds would likely be substantially reduced with adequate sedimentation pretreatment.

• other organics: 1,3-dichlorobenzene may have a high groundwater contamination potential for subsurface infiltration/injection (with minimal pretreatment). However, it would likely have a lower groundwater contamination potential for most surface percolation practices because of its relatively strong sorption to vadose zone soils. Both pyrene and fluoranthene would also likely have high groundwater contamination potentials for subsurface infiltration/injection practices, but lower contamination potentials for surface percolation practices because of their more limited mobility through the unsaturated zone (vadose zone). Others (including benzo(a)anthracene, bis (2-ethylhexyl) phthalate, pentachlorophenol, and phenanthrene) may also have moderate groundwater contamination potentials, if surface percolation with no pretreatment, or subsurface injection/infiltration is used. These compounds would have low groundwater contamination potentials if surface infiltration was used with sedimentation pretreatment. Volatile organic compounds (VOCs) may also have high groundwater contamination potentials if present in the stormwater (likely for some industrial and commercial facilities and vehicle service establishments, but unlikely for most other areas).

• pathogens: enteroviruses likely have a high groundwater contamination potential for all percolation practices and subsurface infiltration/injection practices, depending on their presence in stormwater (likely, especially if contaminated with sanitary sewage). Other pathogens, including *Shigella*, *Pseudomonas aeruginosa*, and various protozoa, would also have high groundwater contamination potentials if subsurface infiltration/injection practices are used without disinfection. If disinfection (especially by chlorine or ozone) is used, then disinfection byproducts (such as trihalomethanes or ozonated bromides) would have high groundwater contamination potentials.

• heavy metals: nickel and zinc would likely have high groundwater contamination potentials if subsurface infiltration/injection was used. Chromium and lead would have moderate groundwater contamination potentials for subsurface infiltration/injection practices. All metals would likely have low groundwater contamination potentials if surface infiltration was used with sedimentation pretreatment.

• salts: chloride would likely have a high groundwater contamination potential in northern areas where road salts are used for traffic safety, irrespective of the pretreatment, infiltration or percolation practice used.

The control of these compounds will require a varied approach, including source area controls, end-of-pipe controls, and pollution prevention. All dry-weather flows should be diverted away from infiltration devices because of their potentially high concentrations of soluble heavy metals, pesticides, and pathogens. Similarly, all runoff from manufacturing industrial areas should also be diverted from infiltration devices because of their relatively high concentrations of soluble toxicants. Combined sewer overflows should also be diverted because of sanitary sewage contamination. In areas of extensive snow and ice, winter snowmelt and early spring runoff should also be diverted from infiltration devices.

All other runoff should include pretreatment using sedimentation processes before infiltration, to both minimize groundwater contamination and to prolong the life of the infiltration device (if needed). This pretreatment can take the form of grass filters, sediment sumps, wet detention ponds, etc., depending on the runoff volume to be treated and other site specific factors. Pollution prevention can also play an important role in minimizing groundwater contamination problems, including reducing the use of galvanized metals, pesticides, and fertilizers in critical areas. The use of specialized treatment devices can also play an important role in treating runoff from critical source areas before these more contaminated flows commingle with cleaner runoff from other areas. Sophisticated treatment schemes, especially the use of chemical processes or disinfection, may not be warranted, except in special cases, especially considering the potential of forming harmful treatment by-products (such as THMs and soluble aluminum).

The use of surface percolation devices (such as grass swales and percolation ponds) that have a substantial depth of underlying soils above the groundwater, is preferable to using subsurface infiltration devices (such as dry wells, trenches or French drains, and especially injection wells), unless the runoff water is known to be relatively free of pollutants. Surface devices are able to take greater advantage of natural soil pollutant removal processes. However, unless all percolation devices are carefully designed and maintained, they may not function properly and may lead to premature hydraulic failure or contamination of the groundwater.

It has been suggested that, with a reasonable degree of site-specific design considerations to compensate for soil characteristics, infiltration can be very effective in controlling both urban runoff quality and quantity problems (EPA 1983). This strategy encourages infiltration of urban runoff to replace the natural infiltration capacity lost through urbanization and to use the natural filtering and sorption capacity of soils to remove pollutants. However, potential groundwater contamination through infiltration of some types of urban runoff requires some restrictions. Infiltration of urban runoff having potentially high concentrations of pollutants that may pollute groundwater requires adequate pretreatment, or the diversion of these waters away from infiltration devices.

Table 58. Groundwater Contamination Potential for Stormwater Pollutants

	Compounds	Mobility (worst case: sandy/low organic soils)	Abundance in storm-water	Fraction filterable	Contamination potential for surface infilt. and no pretreatment	Contamination potential for surface infilt. with sediment-ation	Contamination potential for sub-surface injection with minimal pretreatment
Nutrients	nitrates	mobile	low/moderate	high	low/moderate	low/moderate	low/moderate
Pesticides	2,4-D	mobile	low	likely low	low	low	low
	γ-BHC (lindane)	intermediate	moderate	likely low	moderate	low	moderate
	malathion	mobile	low	likely low	low	low	low
	atrazine	mobile	low	likely low	low	low	low
	chlordane	intermediate	moderate	very low	moderate	low	moderate
	diazinon	mobile	low	likely low	low	low	low
Other organics	VOCs	mobile	low	very high	low	low	low
	1,3-dichloro-benzene	low	high	high	low	low	high
	anthracene	intermediate	low	moderate	low	low	low
	benzo(a) anthracene	intermediate	moderate	very low	moderate	low	moderate
	bis (2-ethylhexyl) phthalate	intermediate	moderate	likely low	moderate	low?	moderate
	butyl benzyl phthalate	low	low/moderate	moderate	low	low	low/moderate
	fluoranthene	intermediate	high	high	moderate	moderate	high
	fluorene	intermediate	low	likely low	low	low	low
	naphthalene	low/inter.	low	moderate	low	low	low
	penta-chlorophenol	intermediate	moderate	likely low	moderate	low?	moderate
	phenanthrene	intermediate	moderate	very low	moderate	low	moderate
	pyrene	intermediate	high	high	moderate	moderate	high
Pathogens	enteroviruses	mobile	likely present	high	high	high	high
	Shigella	low/inter.	likely present	moderate	low/moderate	low/moderate	high
	Pseudomonas aeruginosa	low/inter.	very high	moderate	low/moderate	low/moderate	high
	protozoa	low/inter.	likely present	moderate	low/moderate	low/moderate	high
Heavy metals	nickel	low	high	low	low	low	high
	cadmium	low	low	moderate	low	low	low
	chromium	inter./very low	moderate	very low	low/moderate	low	moderate
	lead	very low	moderate	very low	low	low	moderate
	zinc	low/very low	high	high	low	low	high
Salts	chloride	mobile	seasonally high	high	high	high	high

Source: Pitt, *et al.* 1994; 1996

Hantush Method to Calculate Groundwater Rise

WinDETPOND will predict the height of the groundwater mound that forms beneath a seepage field using the analytic groundwater mounding model developed by Hantush (Hantush 1968). This model predicts the rise and fall of a groundwater mound beneath a rectangular seepage field over a specified time period. There are three conditions that affect the mounding process. These conditions are when infiltration begins, when it forms a mound under steady flow, and when the mound intersects the field.

The purpose of including this model in WinDETPOND is to provide a simple way of entering the infiltration rate of a seepage field, and from that, the changing stage and outflow rates of a pond connected to the field under various infiltration conditions. Note that this is a simplification of the problem of modeling infiltration because of the changing conditions that occur as infiltration progresses.

Seepage field infiltration obeys a form of Darcy's Law when infiltration begins and the mound forms:

qi = Ki(dh/dL), where: qi = infiltration rate [L/T]
Ki = infiltration hydraulic conductivity [L/T], and
dh/dL = hydraulic gradient [L/L]

However, because the seepage field initially is operating in unsaturated soil, the infiltration rate is primarily a function of the negative pressure head and the volumetric water content of the soil. This infiltration rate is much less than the saturated hydraulic conductivity, K. Initially it may be higher than Ki (assuming the gradient is unity) due to the negative head in the soil and the increased gradient. Once the percolation rate becomes steady, the gradient is unity and the final infiltration rate approaches a "resaturated" Ki that is about one-half of the saturated K (Bouwer 1978).

When the seepage field water reaches the water table, it forms a mound that develops both lateral and vertical hydraulic gradients to transport this recharge water through the saturated zone. However, if the rising mound intersects the seepage field, the infiltration rate decreases even though hydraulic conductivity is now at its highest, saturated rate. This is because the gradient, which was formerly equal to one, is now a function of the difference between the pond surface elevation and the mound height above the seepage field, and the sloping water table. Because this dynamic interaction between the mound and the seepage field is difficult to model, WinDETPOND only indicates whether the mound intersects the field (this is defined as a failure in the model output); it does not recalculate the flow rate of water entering the seepage field from the pond.

Hantush, in developing his mounding model, makes a number of assumptions. They are:

1) The aquifer is homogenous and isotropic.
2) The aquifer rests on an impermeable, horizontal base.
3) The percolation rate is small, relative to the saturated hydraulic conductivity.
4) The aquifer is infinite in areal extent.

Hantush develops his mounding equation by combining the continuity equation for unconfined flow with accretion and storage. These two equations are combined to form the Boussinesq equation for unsteady flow in an unconfined aquifer. Hantush uses various Laplace and Fourier transformations to solve the equation for the mound height.

This program calculates the maximum mound height beneath the center of the seepage field. It uses the principle of superposition in time, allowable because the mound height equation is linear, to calculate the mound height at time t. The calculated timeline illustrates the solution process. When the pond is above a net pond stage value established for flow into the seepage field, the field is operating and the mound is rising, and vice versa. The total mound height, h, at time t3 , therefore is:

h = h1- h2 + h3 , where

h1 is obtained from the mound height equation for t = t3 - to

h2 is obtained from the mound height equation for t = t3 - t1

h3 is obtained from the mound height equation for t = t3 - t2

The table provided by Hantush to find the Laplace transforms necessary to calculate the mound height has been entered as a data table in the program. The program makes a double linear interpolation for the given values to calculate the transform. Finally, because the average height of the water table changes as the mound height changes, the program iteratively solves for the mound height by recalculation, using the previous iteration's mound height value.

The model output is the mound height at time t. The distance between the seepage field bottom is compared to the mound height to determine if the mound has intersected the field.

Model Limitations

A number of theoretical and practical limitations arise when the mounding equations are applied to a seepage field. These limitations are due to field factors that the model is unable to consider, such as soil heterogeneity, changing initial water table elevations, aquifer thickness, and the water table slope.

Hantush assumes the soil system is homogeneous. If there is a wide variation in hydraulic conductivity beneath the seepage field, this will affect the mound rise rate and shape.

If the water table beneath the field varies, then the seepage system will not fit with Hantush's static water level assumption, and so could cause disagreements between predicted and actual mound heights.

The aquifer thickness is another source of error. Bouwer (1978) suggested that Hantush's equations should be used with care if the depth to the initial water table elevation exceeds the seepage field width. If this occurs, the flow is not evenly distributed over the entire aquifer depth, but rather concentrates in the upper portion of the aquifer. The uneven flow violates the Dupuit-Forscheimer assumption needed to develop the equation. To account for this problem, Bouwer suggests the use of the "effective transmissivity," KW, instead of the actual transmissivity KH. Therefore, it would be appropriate for the user to set the initial depth to the water table to no more than the seepage field width.

One of Hantush's unstated assumptions is that the initial ambient flow beneath a seepage field is zero. However, if the slope is small, this problem should not affect the most critical model results directly beneath the center of the seepage field.

Groundwater Mounding in WinDETPOND

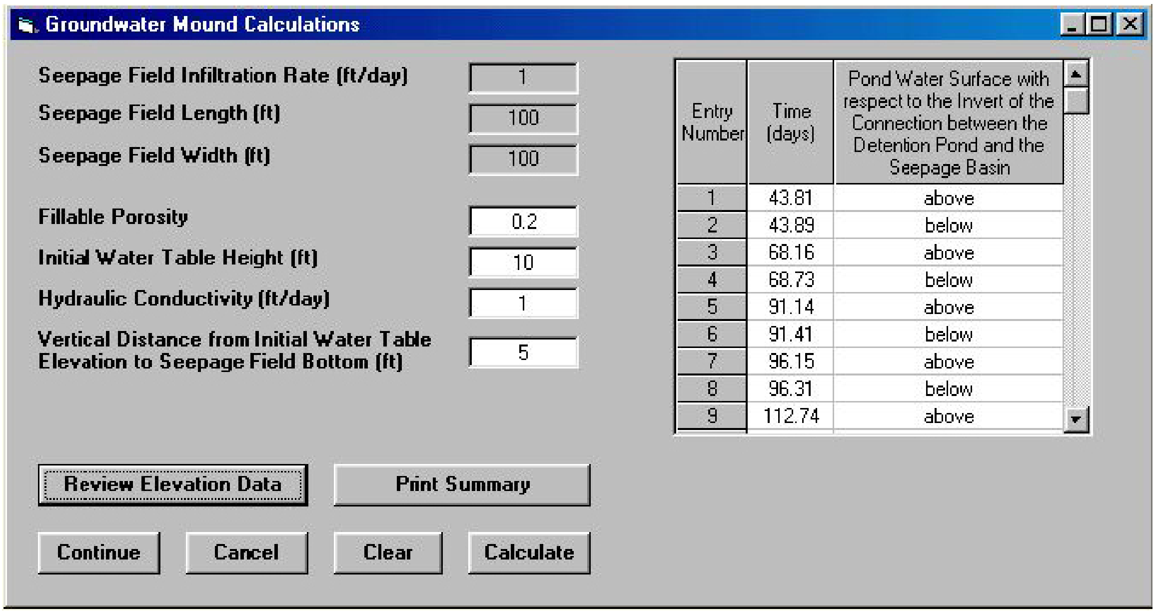
Determining the groundwater mound height beneath a seepage basin in WinDETPOND is a four-step process, as described below:

1. Select Output Option 9 from the "File/Output Options" dropdown menu item.

2. Enter the data required for the seepage basin, including the invert elevation of the seepage field invert above the datum (the elevation above the pond bottom where water begins to flow into the seepage basin). It is this value, along with the vertical distance from the initial water table elevation to the seepage field bottom, that determines whether the groundwater mound intersects the seepage field bottom, and therefore fails.

3. Press the "Calculate" button to run the model after all other pond data is entered.

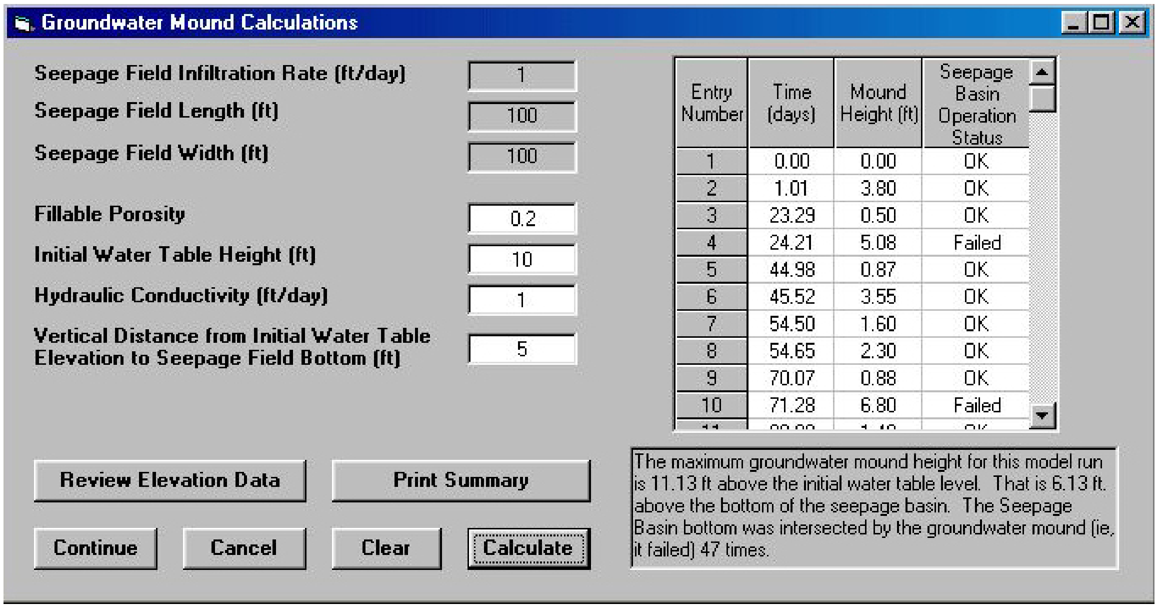
4. Select "Groundwater Mound Height Beneath Seepage Basin" from the "Tools" dropdown menu. The form below appears, without the data grid on the right.



This form already includes the data from the seepage field - the infiltration rate, length, and width. You need to enter the:

1. Fillable porosity
2. Initial water table height
3. Hydraulic conductivity
4. Vertical distance from the initial water table elevation to the seepage field bottom

The model calculates the location of the pond water surface relative to the invert elevation of the pipe, or other connection to the seepage field. The model determines the time the pond water surface is either above or below the seepage field invert. To examine this information, select the button “Review Elevation Data.” The grid in the form below, on the right, appears. This grid lists the times, in days from the start of the model run, that the pond surface is above or below the seepage field invert.

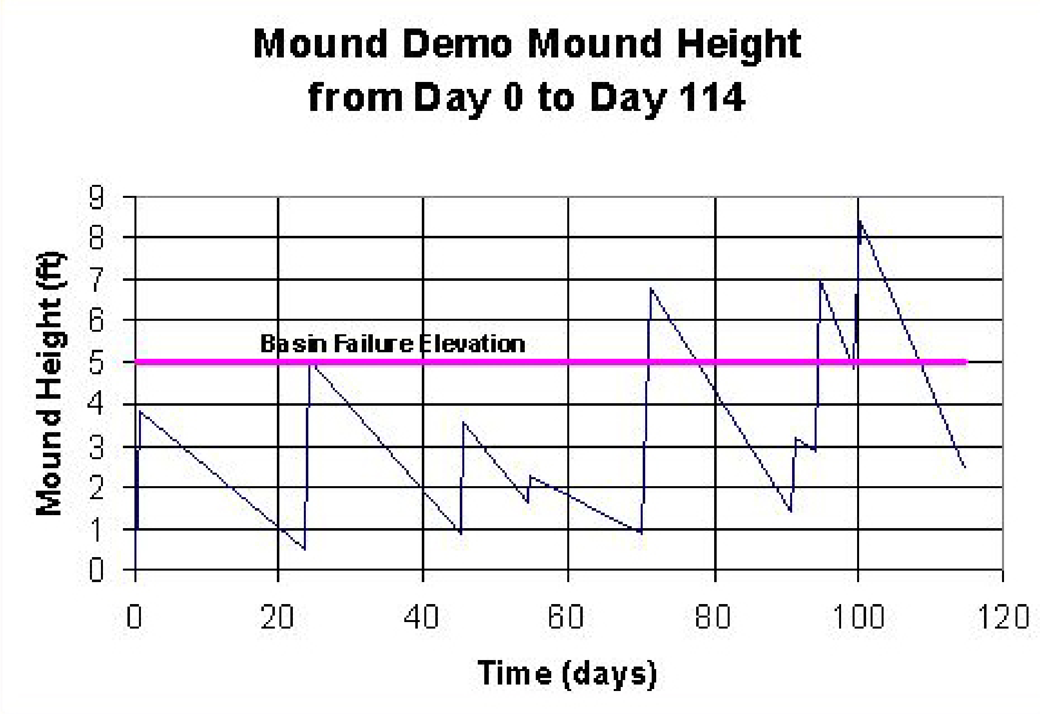


To calculate the mound height, select the button "Calculate" after entering the groundwater parameters described above. The grid and summary describe the mound height and seepage basin operation status, with either "OK" or "Failed" listed in the grid. The "Failed" status indicates that the top of the mound has intersected the seepage basin bottom. Note that although the seepage basin is submerged whenever the "Failed" status is reached, the model continues to calculate the mound height as if the intersection did not occur. This is because the Hantush algorithm used to model the mound height does not account for complications such as source interception. This allows a continuous approximation of the severity of the rising groundwater table beneath the seepage basin.

The model output also includes a summary that lists the maximum mound height, the distance the maximum height is either above or below the seepage basin bottom, and how many times the seepage basin bottom was intersected by the groundwater mound.

View the file "PondFileName.MHT" that is located in the pond data directory on the computer for more detailed information that includes a list of the time and elevation data shown in the grid. This file is a comma-separated text file that lists the row number (starting with row 2, row 1 always has values of 0,0), the time since the start of the model run, and the mound height at that time. Select the "Print Summary" button to get a printout of the input and output data summary.

The following graph (prepared in Excel using the imported comma separated *.MHT file) shows the mound height calculated from the MoundDemo.PND file. The mound height line is jagged rather than smooth because the model only calculates the mound height at selected time intervals: the times when the pond water level, going either up or down, has reached the elevation of the invert of the seepage basin inlet. The graph shows that during the time period from day 0 to day 114, the groundwater mound intersected the seepage basin bottom, located 5 feet from the initial water table height, four times.



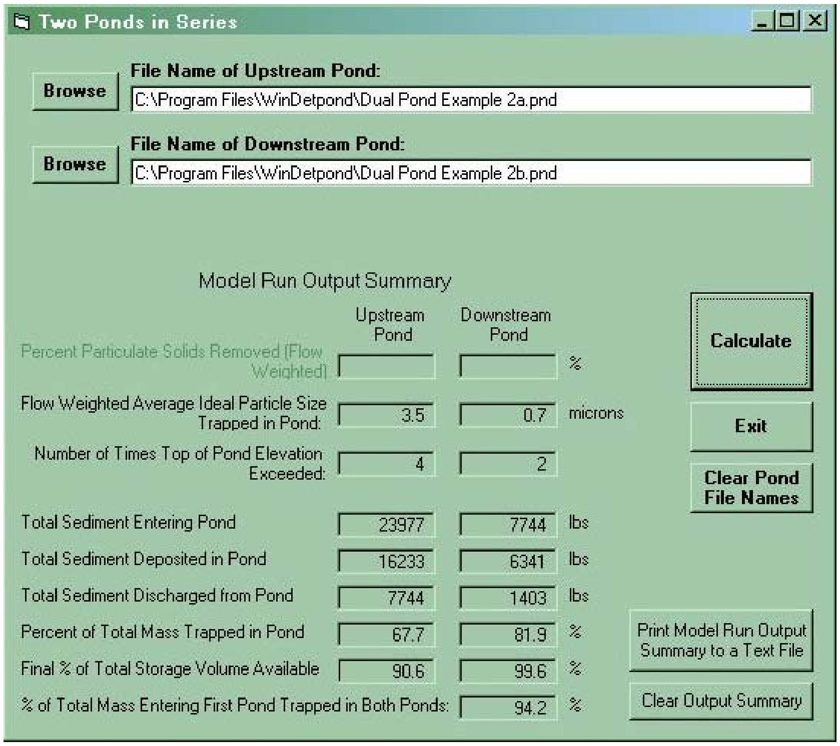
Evaluation of Multiple Ponds in Series using WinDETPOND

The use of ponds in series can be effective, if carefully done. The increasing use of pond forebays, or smaller pretreatment ponds before the major pond, can significantly reduce maintenance costs. Most of the sediment will be captured in the smaller forebay, which can be more easily dredged than the larger pond which is still needed for capturing finer particulates. However, if two ponds in series are the same size, or if the larger pond receives the water first, the second pond will provide little benefit, beyond a minor improvement associated with short-circuiting prevention: most of the sediment that can be captured will accumulate in the first pond.

This scenario would be like using two filters or sieves on top of each other. If they have the same size openings, almost all of the material that is larger than the sieve openings will be captured on the top sieve, with the bottom sieve only capturing material that leaks through small holes in the sieve fabric of the upper sieve (representing short circuiting). If the top sieve has smaller openings than the bottom sieve, very little, if any, material can make it through the top sieve and still be captured on the bottom sieve which has larger openings. The desirable approach is to have the sieve with larger openings on top of the sieve having smaller openings. This will distribute the sediment onto the two sieves. It is possible to design a pond with a forebay to optimize sediment capture while decreasing maintenance costs. WinDETPOND can be useful in examining the effects of two ponds.

Using WinDETPOND to Evaluate Two Ponds in Series

This option allows the user to predict the combined effects of two detention ponds in series. This option is particularly useful if the user wants to determine the size of a sediment forebay in a pond by calculating how quickly the forebay will fill with sediment. The following screen is used, from the "Tools" dropdown menu:



To use this option, first create two separate WinDETPOND files, one file for the upstream pond, and one file for the downstream pond. The model evaluates the two ponds in series by saving the hydrograph and the outflow particle size distribution for each event from the first pond and then using these values as inputs for the second pond. The current version of the model treats the two ponds as hydraulically disconnected by assuming that they are separated by enough distance so that no backwater effects occur.

To calculate sediment loads, the upstream pond must have a *.ORP (Outfall Runoff and Particulate loading) file created in WinSLAMM entered as the “Drainage Basin Runoff Procedure” for the upstream pond. The downstream pond does not require anything listed for the drainage basin characteristics, as it will only use the outfall hydrograph (and sediment particle sizes) from the upstream pond.

Once the names of the upstream and downstream pond files are entered, press the calculate button to evaluate the two ponds in series. A progress bar will appear on the form below the name of the downstream pond file twice; once for each pond of the series. The model run output summary data will appear on the form after the second pond has successfully run. To print this information to a text file, select the “Print Model Run Output Summary to a Text File” button.

There are three criteria that must be followed when setting up the two pond files:

1. Both of the pond files must have the same rain files, with the same path (location on the hard drive), and the same beginning and ending rainfall dates.

2. Do not enter any areas in the downstream file “Basin Runoff Procedure” because the second file will use the hydrograph created by the first file. Disregard the “Incomplete” status label after the “Drainage Basin Runoff Procedure” edit button for the downstream pond file, if it appears.

3.If the sediment load output is calculated for the upstream pond, the *.ORP (Outfall Runoff and Particulate loading) file created in WinSLAMM must be entered. The rainfall dates and file paths and names must be identical for all three files.

The WinSLAMM outfall output file option: Enter the WinSLAMM Outfall Data File Name (to predict pond sediment loading). This option provides runoff volumes and particulate solids loadings for each rainfall event that are generated from a WinSLAMM model run. This option provides the user with the most accurate runoff volumes because it uses WinSLAMM small storm hydrology algorithms, including all control devices available in WinSLAMM. In addition, because it imports WinSLAMM outfall particulate loadings, it can predict pond sediment loadings by event, and so determines when the pond will need to be dredged. When using this feature, the rainfall file and rainfall start and ending dates must be the same in both the WinSLAMM and WinDETPOND model runs. To create the output file in WinSLAMM, select "File/Output Options" from the main menu, and select the checkbox "Save Outfall Runoff and Particulate Loading for WinDETPOND Analysis." The file extension for this option is .ORP (Outfall Runoff and Particulate loading).

The *.ORP file contains the following information:

The first line includes:

1. The number of rainfall events evaluated.
2. The rain file name and path.
3. The starting date of the model run.
4. The ending date of the model run.

Each line of the balance of the file includes:

1. The event number.
2. The runoff volume of the event at the outfall of the modeled area.
3. The particulate loading of the event at the outfall of the modeled area.

Retro-fit Examples for Providing Water Quality Benefits in Existing Dry Detention Ponds

Evaluation and Recommended Modifications to a Small Dry Stormwater Detention Pond

This example analysis, similar to the previous example for the Brook Highland Plaza pond, was also conducted by John Easton, while he was a UAB graduate student. This example differs in that it was for a small dry detention pond at an apartment complex. The pond was evaluated in its present state, and then modifications were recommended and the hypothetically modified pond was re-evaluated. The pond is located at Stonecrest at Double Oak Mountain Apartments, in Shelby, Co., AL.

This detention pond does not meet the general requirements for a well-designed stormwater quality control practice as summarized previously. Obviously, this detention pond was designed to be a dry pond and it is purely for peak flow rate reductions. It was not intended to provide any water quality benefits. The information used in this analysis was gathered from on-site field evaluations that were limited in scope, as no engineering details were available. The contributing area was estimated at 41.4 acres (apartment complex, 26.4 ac, and uphill woodland area, 15 ac).
This analysis makes suggestions for converting this pond to a wet pond, with enhanced water quality benefits. Because of the cost associated with moving the pond or its outlet structures, the recommended changes only consider additional excavation below the outlet.

The redesign presented here will include the preferred depth of six feet, requiring excavation from 720 to 714 feet, and a minimal three foot ledge at 719 feet. The side slope will be 1:1 from 720 feet to the ledge at 719, and also 1:1 from the interior of the ledge to 714 feet. Assuming a prismatic cross-section, the additional wet storage to be constructed below the 720 feet of elevation is about 0.204 ac-ft.

<u>Depth and Criteria</u>

This pond is designed such that the invert of the lowest output device is level with the pond bottom. This pond is potentially hazardous as the side slopes are about 1:2. Also, this steep slope is quite long (approximately 75 feet). This apartment complex is a new development; the majority of the construction occurred in 1997. Therefore, some of the landscaping is not complete as yet. There are some shrubs and small trees planted around the perimeter, however these do not completely surround the pond or form a suitable barrier.

Peak Reduction Factors (PRF)

The pond only slightly reduces the peak outflow rates. The expected 100-year storm's runoff rate is reduced from 153 cfs to about 145 cfs, with a peak reduction factor of only 0.05 (corresponding to a 5% reduction of the inflow hydrograph in the pond). For the 50-year and 25-year storms, the PRFs are 0.06 and 0.07, respectively. Even in the case of the 25-yr storm, the pond exceeds the maximum stage of 633 feet and may cause frequent flooding of the frontage road. In addition, there is 13 feet of head in the pond when it is full, producing very high outflow rates, including about 60 cfs flowing in the emergency spillway. This is especially problematic because the water coming through the spillway flows directly onto Bowling Drive, and Bowling drive is curbed so the water would flow down the hill and out onto Highway 280. This would be an extremely dangerous situation because the highway has high volume, high speed traffic.

Upflow and Critical Settling Velocities

The water quality goal for the re-designed pond is approximately 90% total suspended solids (TSS) removal (maximum upflow velocity, or critical settling velocity) maximum of 0.00013 ft/sec). Even though the re-designed pond only provides a worst-case upflow velocity of 0.0016 ft/sec, the annual average TSS control for the 1976 typical rain year approaches 86%. The lowest TSS removal is only about 56% during this rain year.

Pond's Water Quality Storage

A pond's water quality storage should be equal to the runoff associated with 1-¼" rain based on the land use of the watershed served by the pond. The composite curve number for the apartment complex was calculated to be about 87, while the woodland area had a curve number of 55. This yields a total site composite CN of 75. This 75 CN corresponds to approximately 0.40 inches of runoff for the 1-1/4 inch rain. Therefore the minimum active pond storage (between the invert elevation of the lowest outlet and the secondary outlet discharge devices) required should be a least 1.4 acre-ft. However, due to limited space, the redesigned pond's water quality storage is only 0.67 acre-ft, less than the minimum recommended area. Even though the annual average TSS removal is reasonable, the individual event TSS removals vary considerably.

Since this pond is designed to reduce extreme peak inflow discharge rates in addition to providing water quality improvement, there is an additional freeboard storage (the volume between the lowest outlet and the top of the dam) of 2.34 acre-ft in the pond.

Pond's Surface Area Requirements

A pond's surface area should be sized as a percent of watershed's area based on land use and the level of control desired. The Stonecrest apartment site has residential and woodland land uses. The pond surface area recommendation is about 0.33 acres, which is close to the minimum surface area of the redesigned pond (0.31 acres).

Other Benefits

In dry weather, the redesigned pond will be available to provide water for emergency fire protection. This pond should be a pleasing amenity for the apartment residents. The use of appropriate grasses adjacent to the pond may provide a grass filter for additional pollutant reduction.

Background Information Related to Site Evaluation

Criteria Used to Estimate Peak Flowrates

The peak inflow hydrograph values were determined by HydroCAD's SCS TR-20 methodology. For the site, a SCS Type III rainfall IDF curve was selected. Rainfall depths for the 100-year, 50-year, and 25-year storms were approximately 8.6", 7.8", and 7.1" respectively. The time of concentration for the watershed was also calculated using HydroCAD's built-in TR-20 methods; Tc = 24.3 minutes for the apartment complex area, and Tc = 33.8 minutes for the woodland area.

Land Use, Development, Cover, Soils Type, and CNs

SCS soil maps for the Shelby County were examined, and it was determined that the Stonecrest site consisted of Nauvoo-Sunlight complex, with 15 to 25 percent slopes, and Townley silt loam, with 12 to 18 percent slopes. The SCS Hydrologic Soil Groups for these soils are type B and type C respectively. Research conducted at UAB has shown that development, due to construction disturbances, compaction, and soil mixing, can significantly reduce the actual infiltration rates from those assumed. Therefore, the curve number assigned to the developed area was for the worst case, type D soil. However, the undeveloped woodland area, mostly Nauvoo soil, was assigned a curve number based upon the type B type. Therefore for the developed area of 26.4 acres, a composite CN of 87 was assumed (based on 16 acres of residential land use with 1/8 acre lots, SCS soil type D, and 65% impervious cover, plus the remaining 10.4 acres of open lawns with good grass cover, and type D soil). A curve number of 55 was assumed for the woodland area of 15 acres, corresponding to woods with good hydrologic condition and type B soils.

Analysis of Design Storms

HydroCAD ™

The HydroCAD Stormwater Modeling System (version 4.53) was used to analyze the pond for flow behavior during large design storms. The program does not consider the dead storage below the first outlet, assuming that this is always full of water, therefore the hydraulic behaviors of both the existing pond and the redesigned pond are identical.

The subcatchment component of HydroCAD was used to model the two subcatchments: subcatchment 1 refers to the 26.4 acres of the apartment complex, and subcatchment 2 consists of the 15 acres of woodland area that drains to the complex. This data, as input to the model, is described in Figure 120 and Table 59.

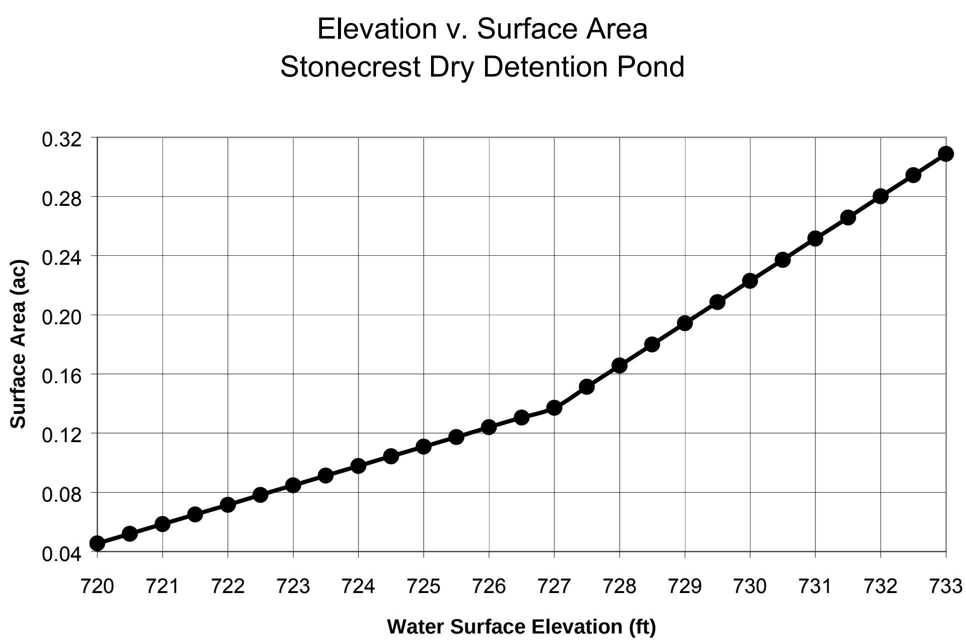


Figure 120. Stage v. Surface Area Curve.

Table 59. Outlet Device Descriptions

#	Route	Invert	Outlet Devices
1	primary	720’	**24” culvert** n=0.013, length=185’, slope=0.02 ‘/’, Ke=0.5, Cc=0.9
2	to #1	720’	**12” orifice**
3	to #1	727’	**sharp-crested rectangular weir** length=12’, height=8” (square concrete box with cap)
4	secondary	732’	**10’ broad-crested rectangular weir** emergency spillway

The HydroCAD simulations were made for three SCS type III design storm frequencies: 25-year (7.1”), 50-year (7.8”), and 100-year (8.6”). Table 60 summarizes the model’s output for these three storms, showing the hydrographs peaks and volumes. The most significant contribution to the hydrograph flowing into the pond comes from the apartment complex area, as expected.

Table 60. Subcatchment Summaries for Design Storms

Subcat #	description	Design Storm Frequency	Rainfall (in)	Peak (cfs)	Volume (ac-ft)
1	apartment complex	25-yr	7.1	102.6	10.79
2	woodland	25-yr	7.1	19.13	2.46
1	apartment complex	50-yr	7.8	114.4	12.04
2	woodland	50-yr	7.8	23.59	2.99
1	apartment complex	100-yr	8.6	127.9	13.47
2	woodland	100-yr	8.6	28.94	3.64

Table 61 summarizes the pond routing calculations. As previously noted, the peak reductions are quite low (5-7%), and the peak discharge lag is only 5 to 10 minutes. The peak elevation in the pond is higher than the maximum elevation in the pond, 733 ft. This is a dangerous situation because it means that the water is flowing uncontrollably over the dam. This could damage the emergency spillway and cause erosion of the dam itself. Notice that these events occur even at the lowest storm frequency modeled, 25-yr. It would appear that the pond is inadequate for the amount of runoff generated by these storms.

Replacing the secondary outlet with one that is less restrictive could mitigate the danger of overflowing the pond’s dam. This would also probably require enlarging the 24” culvert that flows under Bowling Drive, and would worsen already poor peak flow reduction characteristics of the pond. However, this would decrease the frequency in which water reaches the emergency spillway, flows out onto Bowling Drive and continues down the hill onto Highway 280. It is interesting to note that a similar detention pond recently constructed several miles away at a new shopping center failed, releasing large quantities of water onto Highway 280.

Table 61. Pond Results of HydroCAD simulations

Design Event	Rain Depth (in)*	Peak Elev. (ft)**	Peak Storage (ac-ft)	Peak Qin (cfs)	Peak Qout (cfs)	Peak Qoutlet† (cfs)	Peak Qemer‡ (cfs)	Atten. (%)	Lag (min)
25-year	7.1	733.4	2.11	118.30	110.20	50.51	59.73	7	8.1
50-year	7.8	733.8	2.21	134.20	126.10	51.00	75.15	6	7.2
100-year	8.6	734.1	2.31	152.60	144.90	51.59	93.36	5	5.8

* Design storms are type III 24-hr for Shelby County (SCS methods).
** Flood elevation is at 633 feet.
† Peak flow through the first and second outlets to 24” culvert.
‡ Peak flow in the emergency spillway (flowing onto Bowling Drive).

WinDETPOND

As in the previous example, the 25-year, 50-year, and 100-year hydrographs generated using HydroCAD's TR-20 methods were imported into WinDETPOND to estimate the TSS removals during these large rains. A comparison of the hydraulic results from HydroCAD with the WinDETPOND results in Table 62 indicates similar values. Even under these severe conditions, the pond is removing approximately 50% of the TSS.

Table 62. WinDETPOND Summary for Design Storms

Storm Year	Max. Stage (ft.)	Max. Inflow (cfs)	Max. Outflow (cfs)	Max particle size discharged (µm)	Avg. Min Particle Size Controlled (µm)	% TSS Removed
25	733.46	118.0	100.1	95.0	26.5	50.8
50	733.96	134.2	118.0	95.0	28.1	48.6
100	734.48	152.6	136.6	95.0	29.7	46.7

Analysis Using Actual Long-Term Rainfall Records

WinDETPOND

WinDETPOND simulations were conducted using rain files created from the 1976 Birmingham monitoring data, and also the 1952 through 1989 rain record. There are 23 events, out of a total of 4,107 in the 1952 to 1989 rain record for Birmingham, AL, in which the pond stage rises to the level of the second outlet. Water quality evaluations in the existing dry pond were not conducted as they are assumed to be negligible.

Short-term simulations using Bham76. The results of the simulations using the 1976 rain year data for Birmingham, AL, file are presented in Table 63. On average, in a typical year, the pond will collect particle sizes 4 µm and greater in size, which represents approximately 86% TSS control. This re-designed pond only contains runoff from a 0.5 inch rain, far short of the preferred 1-1/4 inch rain recommendation. Even though the average control is a desirable 86%, the worst-case removals are much less.

Table 63. Water Quality Output Summary for 1976 Rain File

Statistic	Rain Depth (in)	Rain Duration (hrs)	Intrevt Duration (days)	Rain Intensity (in/hr)	Max Pond Stage (ft)	Flow-weighted Particle Size	Approx. Part. Res. Control* (%)	Peak Reduction Factor	Event Flushing Ratio
Mean	0.50	12.01	1.81	0.04	6.30	4.26	86	0.07	1.75
Std. Dv.	0.75	10.77	2.36	0.06	0.51	4.23	13	0.07	2.41
COV	1.51	0.90	1.30	1.48	0.08	0.99	0.16	1.00	1.37
Min.	0.01	1.00	0.00	0.00	6.00	0.00	57	0.01	0.00
Max.	3.84	45	11.68	0.31	8.84	15.70	100	0.31	9.34

* Approximate Particle Residue Control (TSS).

Figure 121 shows the maximum pond stage, axis labels denote the elevation above the pond bottom (6' corresponds to 720' msl elevation, the invert of the first outlet device) versus the percent particle control. There is an expected trend, the TSS control decreases with maximum stage, i.e., more water flowing into the pond.

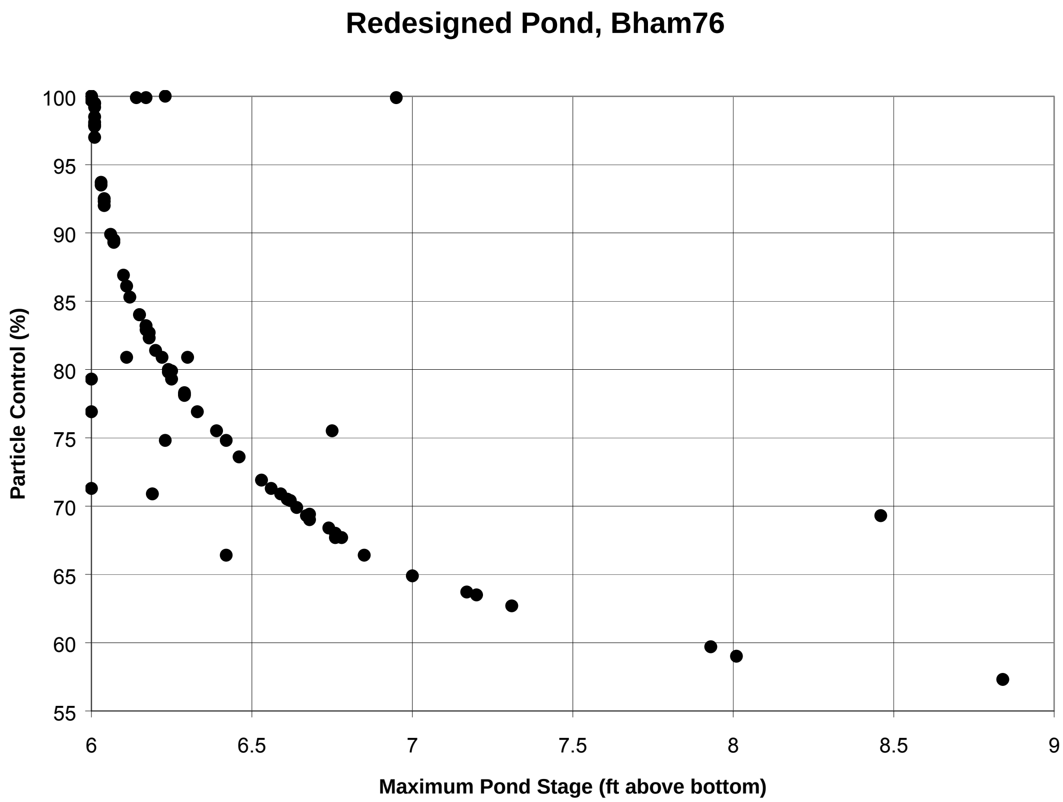


Figure 121. Pond Stage v. Particle Residue Control

Figure 122 shows the water quality performance of the redesigned pond (% particulate control) versus the rain depth in inches and Figure 123 shows water quality performance versus rain intensity. Generally, percent TSS control decreases as the rain depth, or the rain intensity, increase, as expected.

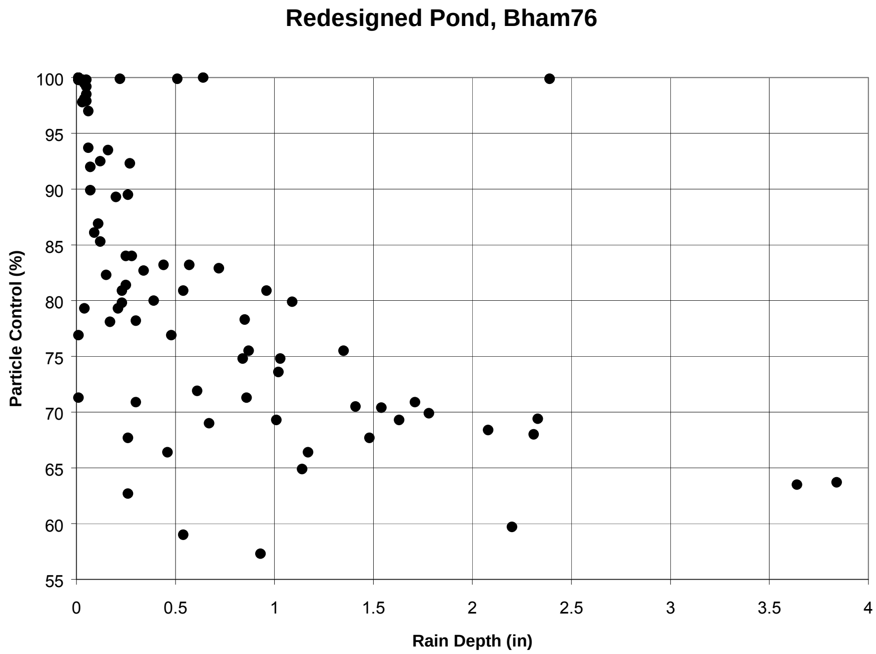


Figure 122. Rain Depth v. Particle Residue Control

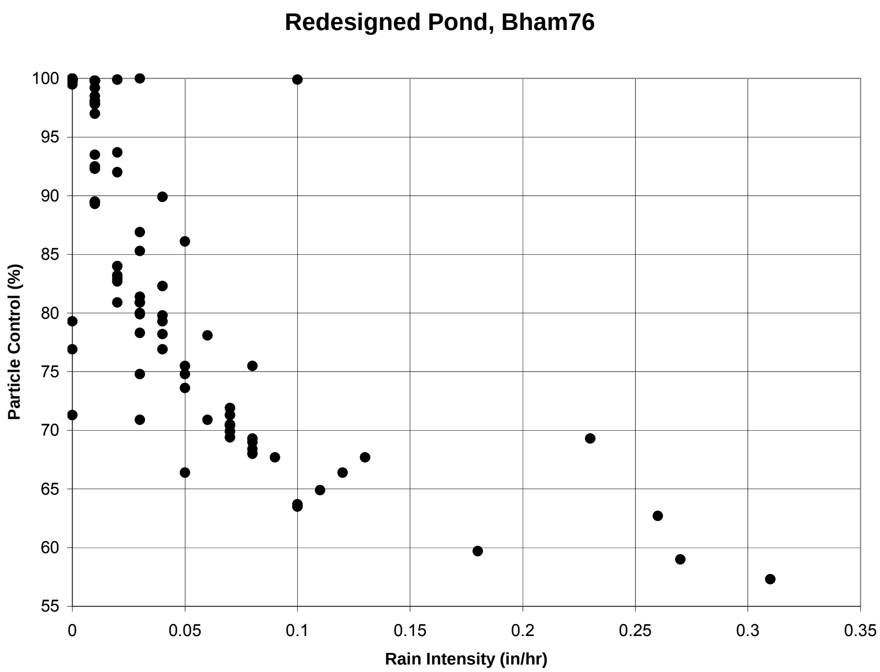


Figure 123. Rain Intensity v. Particle Residue Control

Long-term Simulation using Birmingham Rain, 1952-1989. Table 64 contains WinDETPOND analysis summaries for the 4,107 rain events that occurred in Birmingham from 1952-1989. With close to forty years of rains, the redesigned pond still averages 80% TSS removal.

Table 64. Water Quality Output Summary for 1952-1989 Rain File

Statistic	Rain Depth (in)	Rain Duration (hrs)	Intrevt Duration (days)	Rain Intensity (in/hr)	Max Pond Stage (ft)	Flow-weighted Particle Size	Approx. Part. Res. Control* (%)	Peak Reduction Factor	Event Flushing Ratio
Mean	0.50	6.31	2.57	0.09	6.65	6.43	80	0.13	1.91
Std. Dv.	0.75	6.88	3.54	0.11	1.06	4.91	14	0.10	2.44
COV	1.50	1.09	1.38	1.31	0.16	0.76	0.18	0.76	1.28
Min.	0.01	1.00	0.00	0.00	6.00	0.20	48	0.00	0.00
Max.	13.58	93	44.31	1.45	13.73	23.2	100	0.57	9.96

* Approximate Particle Residue Control (TSS).

Design Storm Runs Using WinDETPOND

The pond inflow hydrograph from the HydroCAD runs was used as a "user defined hydrograph" for input into WinDETPOND to evaluate the water quality control during these low frequency design storms. The following is the output from WinDETPOND for the 25-year design storm:

25-year Design Event

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*  This pond stage elevation is higher than the highest control outlet structure (el: 19 ft).
The pond bank is overtopped - increase the size of the emergency spillway.
This stage value assumes outflow is constant and equal to the highest value on the rating curve.
Time increment (min)=  6       Number of increments=  363
Rain depth (in) (N/A for user defined inlet hydrograph):        0.00
Rain duration (days):     0.42     Event duration (days):     0.51
Interevent duration (days):     0.00  Inflow rate to pond (cfs):  max=       118.3
Outflow rate from pond (cfs):  min=       0.0     max=     100.1  time weighted ave=         4.4
Net inflow volume (cu ft) -  event:     8168   cumulative:     8168
Total inflow volume to pond (cu ft):      576684
Outflow volumes (cu ft) - hydraulic:          568516
                        - seepage:                 0
                        - evaporation:             0
                        - total outflow:      568516
Pond storage above lowest invert (cu ft):  max =      93000
Pond storage below lowest invert (cu ft):       8168
Pond stage above datum for event (ft):  min=      1.21   max=       19.46
Pond surface area for event (sq ft):  min=     1307    max=    14102
Event flushing ratio (total inflow volume/pond storage below invert):       70.61
Upflow velocity for event (ft/hr):  min=   0.000     max=  25.547
Minimum particle size controlled (microns):   flow weighted average=      26.5
Particulate solids control (percent): min=    35.8    flow weighted average=    45.8
Peak Reduction Factor (PRF):       0.15
***  The largest ave particle size discharged during any time increment:   38.7 microns
                       Particle Size Distribution
 Percent of |<===============  Particle    Size    (microns)   ==============>|
  Particles    Pond    |<=======   Pond   Outflow   During    Event    =======>|
   Larger     Inflow   |<=======                                         User==>|
 than Size    During    Theoretical    n=8         n=3      n=1      Defined n
 Indicated     Event        ***                                      n = 5
     0  >     2000.0       26.5       95.0       95.0      233.3       95.0
    10  >      233.3       20.1       22.2       25.0       32.1       23.0
    20  >       95.0       16.0       15.8       16.9       20.3       16.1
    30  >       53.3       12.6       11.9       12.4       14.4       12.0
    40  >       32.5       10.1        9.2        9.7       11.0        9.3
    50  >       21.0        8.0        7.4        7.6        8.4        7.4
    60  >       13.5        6.2        5.6        5.8        6.4        5.7
    70  >        9.0        4.7        4.3        4.4        4.7        4.3
    80  >        5.7        3.2        2.9        3.0        3.2        2.9
    90  >        3.0        1.6        1.5        1.5        1.6        1.5
   100  >        0.0        0.0        0.0        0.0        0.0        0.0
      Row A:               38.7       95.0       95.0      233.3       95.0
      Row B:               26.5
      Row C:               45.8       51.2       49.9       45.7       50.8
Row A:  Largest ave particle size discharged (microns) during any time event
Row B:  Flow weighted average minimum particle size controlled (microns)
Row C:  Percent particulate solids removed
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Figure 124. Emergency spillway flowing directly onto Bowling Drive.


Figure 125. Facing north, from left to right: emergency spillway, secondary outlet, primary outlet, large inlet.

Figure 126. Facing North, dam.

Retrofit of Dry Detention Pond in Sunnyvale, CA

South San Francisco Bay, CA, has serious heavy metal problems, especially for copper, and numerous methods are being investigated to reduce the discharges of metals. Woodward Clyde Consultants (1994) conducted a retrofit project for the Santa Clara Valley Nonpoint Source Pollution Control Program to demonstrate the benefits of modifying an existing dry detention pond for enhanced water quality benefits. The basic information in the following discussion is mostly extracted from that report.

According to an inventory conducted by Woodward-Clyde (1990), there are 17 municipally-owned and operated pump stations in Santa Clara Valley. These pump stations generally consist of pumps, storage units such as a sump or a detention basin, and inlet and outlet works. Sumps and detention basins are designed to reduce the capacity of the pumps that would otherwise be needed to pass the peak flood flows. The purpose of the pump stations is to provide flood protection to low lying areas which have historically subsided and are now protected by levees. These pump stations have generally been operated as single-purpose flood control facilities. The pump operating schedules are designed such that the pumps go on as soon as water begins to fill the basin with the goal of emptying the basin as soon as possible after the event. One retrofitting option to achieve water quality benefits would be to change the pump operating schedule in order to increase detention time and to provide for a seasonal wet pond. A preliminary evaluation of retrofitting detention basins was encouraging and a pilot study to actually retrofit a facility and conduct testing to measure water quality benefits and costs was conducted. This study was conducted from August 1990 through July 1993.

The following tasks were conducted as part of this study:

• Retrofit the pump station and modify pump schedules to improve stormwater pollutant removal,
• Conduct water quality sampling to estimate the pollutant removal effectiveness of the retrofitted detention basin, and
• Measure sediment concentrations in the basin in order to evaluate if sediments are classified as hazardous waste.

The detention basin has a channel between the inlet and outlet that, prior to the modifications, encouraged short-circuiting. A gabion weir was installed at the outlet to reduce short circuiting and to provide better distribution of flow into the outlet. Rock was dumped into the channel leading from the inlet, and a drainage pipe that ran below the channel was blocked off. Operational changes consisted of modifying the pump schedule to create a two-foot permanent pool at the outlet and to provide temporary storage and slow release of water over the depth range of 2 to 2.4 feet.

Site Description
The northern portion of Santa Clara Valley has a history of subsidence caused by groundwater pumping. In order to protect these areas from flooding, a system of levees and pump stations has been built. The pump stations are designed to collect and pump stormwater runoff from these low lying areas through the levees into the flood control channels. In order to accommodate large flows and to reduce the number and pumping capacity of the pumps, some pump stations include relatively large sumps or detention basins. An inventory of the pump stations indicates that there are nine such facilities in the Valley with relatively large detention basins (WCC 1990). The design and operating philosophy of these systems is to: 1) attenuate the peak flow to reduce pump size, and 2) drain the basins as soon as possible following the storm so that flood capacity is available for subsequent storms.

An example of one of these pump stations is Sunnyvale Pump Station No. 2, located at the junction of the Milpitas/Alviso Road (Route 237) and Calabazas Creek. The Pump Station consists of four primary pumps, each rated at 39 cfs capacity, and one auxiliary electric pump (capacity 9 cfs). The detention basin area is approximately 4.4 acres and has a capacity of approximately 30 acre-feet (Figure 127). It receives runoff from a 463-acre catchment consisting of the following land uses: industrial park (30 percent), commercial (10 percent), and residential (60 percent). There is a seven-foot diameter concrete reinforced pipe that drains into the basin. A second 36-inch diameter line drains a 250 acre catchment (primarily open space) and bypasses the basin to the north and directly enters the pump house.

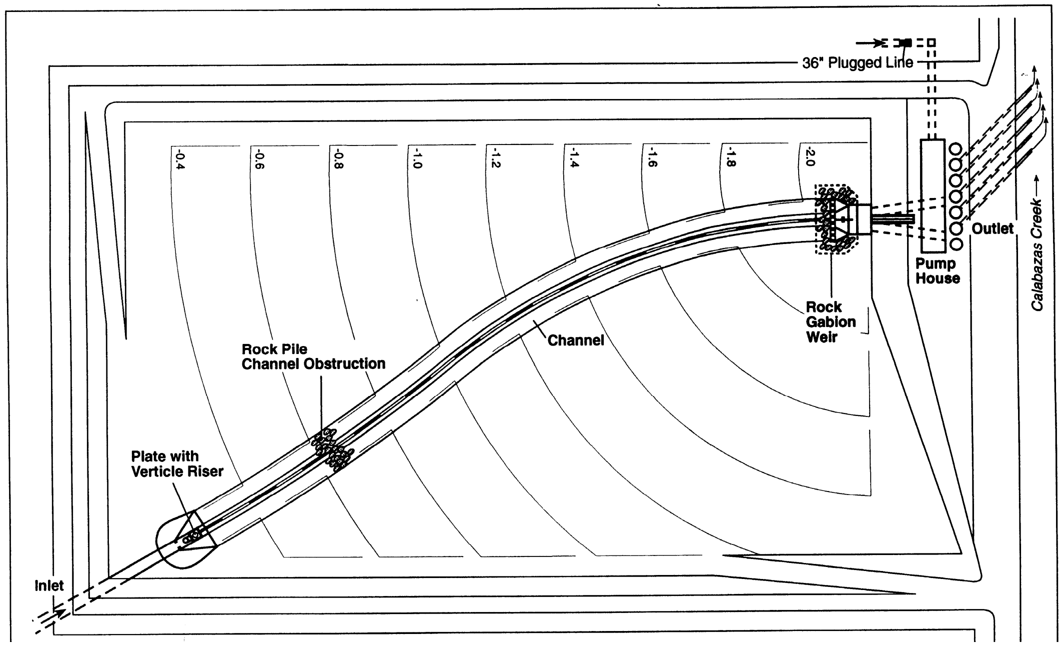


Figure 127. Sunnyvale Pump Station No. 2 dry detention pond.

Treatment Concepts and Retrofitting Objectives

The major premise for this project was that pump stations may provide an opportunity to reduce nonpoint source loads entering the South Bay if they can be cost-effectively retrofitted and maintained. The primary means of treatment is settling of particulates. Settling can be an effective treatment for some pollutants that are mostly in the particulate fraction in stormwater. Typical ranges of the particulate fraction for locally collected stormwater are 36 to 94 percent (mean of 69 percent) for copper and 24 to 97 percent (mean of 66 percent) for lead. Because of these high particulate fractions, sedimentation could be an effective control practice.

The retrofitting scheme is to increase the detention time to allow more particulates to settle out into the basin while not significantly increasing the flood risk. A goal of this retrofit was to prevent high flows from resuspending previously settled sediments in the detention pond. Scour protection was provided by having at least a two foot permanent wet pool during the wet weather season.

They concluded that a 24 to 40 hour hydraulic detention time for a pool several feet deep was necessary to effectively settle out most of the suspended sediment in the local stormwater.

In all cases, the basin must maintain a relatively large flood control capacity and associated outlet works and pumps in order to meet the necessary flood control objectives.

Description of Sunnyvale Retrofit Activities

Change Pump Operational Rules to Create a Permanent Pool and Temporary Storage. In order to create the permanent pool in the pond, the pumps were set to turn off when water levels in the basin (as measured at the outlet) dropped below two feet. In order to create temporary storage, pump settings were adjusted to phase in (and out) very slowly for depths between 2 and 2.4 feet. These operational conditions created a temporary storage depth above the permanent pool of 0.4 feet with a capacity of 1.75 acre feet. Because this is an existing flood control facility, the temporary storage depth was determined primarily based on flood control and secondarily on water quality considerations. The temporary storage depth was the maximum depth that would still allow the basin to pass the 100 year flood.

Prevent Short Circuiting. The pond has a trapezoidal open pilot channel (8-foot bottom width, 17-foot top width, and 4.5-foot depth) between the inlet and outlet (Figure 127). In addition to this open channel, a 30-inch reinforced concrete pipe (RCP) was below the channel to convey low flows between the inlet and outlet. These conveyances effectively "short circuited" flows between the inlet and outlet, a condition that is highly unsuitable for water quality control.

In order to limit this short-circuiting, three modifications were made. At the outlet weir near the pump house, a gabion wall was constructed around the original outlet weir to prevent short-circuiting of flows along the channel and to also promote a better distribution of flow from the basin into the outlet weir. A second modification involved placing rock into the channel near the inlet. The third modification involved covering the entrance of the 30-inch RCP with a steel plate and vertical riser that reduced the rate at which flow would enter the drain below the trapezoidal channel.

Plug Storm Drain that Directly Entered Pump House. A 36-inch RCP drained a 250-acre undeveloped area west of the detention basin directly to the pump house sump. This pipe was plugged with sand bags in one of the manholes upstream of the sump to prevent the runoff from this drainage area to mix with outflow from the detention basin in the sample collection area.

Problems Encountered

No problems were encountered during the structural retrofitting of the detention basin. However, the pump control system needed major repairs in order to operate the basin within the water level tolerances required for the study. Specific problems were encountered with the liquid level sensors and transmitter (inaccurate flow monitoring because of the very low flow rates), voltage instabilities caused when certain pumps came on line, and fluctuations in the power supply. Therefore, an important aspect in evaluating the feasibility of retrofitting pump stations is the design and condition of the pump control system and the possible need for repairs and upgrading.

Monitoring Program

The goal of this study was to measure the total runoff and collect flow-weighted composite water samples at both the inlet and outlet of the detention basin during and after storms in order to estimate pollutant removal performance. Sediment samples were also taken to characterize basin sediments.

Station Design and Equipment. Automated flow and water quality monitoring stations were located at the inlet and the outlet to the basin. The inlet pipe was a 7 -foot diameter reinforced concrete pipe which was quite low and tended to be full of water during most of the wet weather season. The inlet sampling station was located 35 feet upstream of the end of the pipe and consisted of a Druck pressure transducer, velocity meter, ISCO Model 3700 automatic water quality sampler and Campbell Scientific CR-10 data logger/controller. At the inlet, the initial plan was to collect flow-weighted composite samples based on flow volumes estimated using the measured velocity times the area of the pipe. Initially, a Montadero-Whitney electromagnetic velocity meter was used. However, the velocities in the pipe were too low to measure with this instrument and it was replaced in March, 1992 by a Detectonics I.S. Surveylogger, which relies on the Doppler effect and suspended sediment passing the instrument. When compared with estimates of anticipated runoff volumes, neither instrument appears to have measured flow velocities in a consistent and accurate manner. The primary cause appears to be the relatively low velocities in the large pipe.

The outlet sampling was conducted in the pump house where a Druck pressure transducer, an ISCO Model 2700 automatic sampler, and CR-10 datalogger/controller were installed. The initial plan was to start sampling based on estimated flow through the pumps. These estimates were based on the pump run times and pump characteristic curves (which show the relationship between flow and head for the design rpm of 700). To achieve this, the datalogger was connected to the pump house control panel to determine pump run times and calculate discharge from the sump. Field visits during the 1991-92 season revealed that the pumps did not operate at the design speed, especially during the warm-up period, resulting in inaccurate flow estimates much of the time.

Sampling Methods. At the inlet, a pressure sensor was used to estimate the water level in the detention basin. During each sampling event, flow was calculated as a product of velocity and area by the CR-10 microprocessor.

Based on the flow estimate (which was generally poor), the CR-10 initiated and continued water quality sampling at pre-specified flow intervals. During a sampling event, instantaneous velocity and pressure were recorded each time a water quality sample was taken. Based on anticipated rainfall, the sampling algorithm in the CR-10 was designed to instruct the water quality sampler to collect twenty 500-mL subsamples in a 10L borosilicate bottle over the duration of the storm event. Following the sampling event, the pressure sensor was also used to measure water level drops in the pond.

At the outlet, the average hourly flowrate was estimated based on the pump run times and the pump characteristic curves (also inaccurate), and was recorded over the duration of the wet weather season. To begin an event, field crews manually initiated the automated samplers based on anticipated flow volumes for that storm. As with the inlet, the automated samplers recorded instantaneous flow measurements when each sample was collected.

Stations were visited prior to, during, and after monitored events to ice samples, exchange sample bottles, and ensure proper equipment operation. Measurements of pH, conductivity, and temperature were made during the site visits.

Data Collected. A total of eight storm events were sampled. For six of these events, flow-weighted composite water quality samples and hydrologic measurements were taken at both inlet and outlet stations. In most cases, only partial flow measurements were made because of either equipment malfunction, below threshold velocities, and/or problems with the pump control system.

Due to the uncertainty in flow volume measurements, pollutant loads were not used to estimate treatment effectiveness. Instead, effectiveness was estimated based on the flow composite water quality concentration data, using the reasonable assumption that the inlet and outlet volumes for an event are equal.

Sediment samples were taken at three locations: in the center of the basin, near the inlet, and near the outlet. Three sets of sediment samples were collected during dry periods when the basin was empty, or nearly empty, of water (June 15, 1990, May 14, 1992, and on July 12, 1993). The first samples were obtained using a 4-inch stainless steel hand auger, while the other samples were collected by scraping the top half-inch of sediment with a Teflon™-coated scraper.

Flooding Analysis, Storm Hydrology, Water Quality and Sediment Monitoring Results

Flooding Analysis. Woodward Clyde used a reservoir routing model to estimate water levels in the basin for the 100-year inflow event and for two pump operating scenarios. The first pumping scenario corresponded to the original pumping schedule used for flood control. The second scenario corresponded to the revised pumping schedule appropriate for a multipurpose flood control and water quality control facility. Based on the results of the model, the maximum water level in the basin for the 100-year flood did not change by modifying the pump operation schedule.

Precipitation. Rainfall was measured with a tipping bucket rain gage, which registered the time when the bucket collected 0.1 inches of rainfall. The range of storm volumes during the sampling period were from 0.4 to 2.2 inches and the storm durations ranged from 6 to 60 hours. Historical rainfall data collected by the National Weather Service at the San Jose Airport (Gage No. 7821) was used to examine the long period characteristics of the local rainfall by using the Synoptic Rainfall Analysis Program (SYNOP). The median event rainfall volume for the San Jose Airport gage for the period from 1948 to 1989 was 0.5 inches.

Runoff. Flow measurements collected at the inlet and outlet for various events were compared with rainfall to calculate the volumetric runoff coefficients. The flow measurements at both the inlet and outlet stations were not considered very reliable, as the measured runoff coefficients ranged from 0.1 to 1.89. Woodward Clyde estimated that the actual values would be about 0.5-0.8 for these rains and watershed characteristics.

Comparison of Inlet Water Quality to Other Santa Clara Stormwater Monitoring Station Data. Laboratory chemical analyses were conducted on the water samples collected at the basin inlet and outlet stations during the six storm events. The median flow-weighted composite concentrations of total metals (cadmium, chromium, copper, lead, nickel, and zinc) from the inlet station are summarized in Table 65. The table shows median concentrations obtained from other Santa Clara Valley stormwater monitoring stations representing residential-commercial, industrial, and open land uses. The inlet concentrations of copper, lead, nickel, and zinc are higher than concentrations from open land use, but lower than concentrations at residential-commercial and industrial land use stations. The cadmium concentration appears to be very similar to the residential-commercial land use, while the chromium concentration is closer to the open land use.

Table 65. Comparison of Median Metal Concentrations at Inlet to Retrofitted Basin to other Santa Clara Valley Stormwater Monitoring Station Data (μg/L)

	Inlet to Retrofit Basin (n=6)	Residential/ Commercial Land Use Station (n=21)	Industrial Land Use Station (n=25)	Open Space Land Use Station (n=4)
Cadmium	1.1	1.0	3.9	0.3
Chromium	12	16	24	11
Copper	24	33	51	11
Lead	38	45	91	2.0
Nickel	21	30	46	5.0
Zinc	180	240	1150	5.0

Pollutant Removal Effectiveness. Table 66 summarizes inlet and outlet concentrations for total and dissolved metals (cadmium, chromium, copper, lead, nickel, and zinc), TSS, hardness and total oil and grease. Based on these data, pollutant reductions were estimated as the outlet minus inlet concentration divided by the inlet concentration. The average pollutant removal effectiveness for the metals ranged from about 30 to 50 percent. The metals removal data indicated that the removal of total chromium, copper, lead, nickel and zinc were well correlated with TSS removal.

Table 66. Inlet and Outlet Observed Concentrations and Pollutant Removals

	Cadmium (μg/L)		Chromium (μg/L)		Copper (μg/L)		Lead (μg/L)		Nickel (μg/L)		Zinc (μg/L)		TSS (mg/L)	TH (mg/L)	O&G (mg/L)
	nf	f	nf	f	nf	f	nf	f	nf	f	nf	f			
SE 17															
Inlet	0.4	<0.2	3.6	1.8	8.7	5.4	6.4	2.2	1.7	<2	46	28	12	97	1.5
Outlet	0.2	<0.2	2.7	1.1	6.8	4.7	3.4	1	1.7	<2	26	19	73	120	1.4
Reduction	--	--	25%	--	22%	--	47%	--	0%	--	43%	--	39%	--	7%
SE 20															
Inlet	6.6	1.3	12	1	24	3	45	1	16	1	180	19	90	110	0.2
Outlet	4.8	2.5	6	1	9	3	10	1	4	1	73	22	24	63	<0.2
Reduction	--	--	50%	--	63%	--	78%	--	75%	--	59%	--	73%	--	--
SE 21b															
Inlet	1.1	0.2	18	1	24	2	53	<1	25	<1	180	5	140	--	--
Outlet	1.5	<0.2	14	1	16	2	35	<1	19	<1	120	7	93	--	--
Reduction	--	--	22%	--	33%	--	34%	--	24%	--	33%	--	34%	--	--
SE 23															
Inlet	1	0.2	11	<1	27	5	30	1	13	3.9	190	41	74	100	0.7
Outlet	0.6	<0.2	8.3	1.4	12	4.7	12	<1	5.8	2.2	82	45	31	90	0.5
Reduction	--	--	25%	--	56%	--	60%	--	55%	--	57%	--	58%	--	--
SE 24															
Inlet	1.6	<0.2	21	1.1	40	2.1	76	<1	42	9.6	270	22	180	140	0.6
Outlet	1.3	0.2	15	8.6	24	5	40	1.4	29	15	160	31	96	140	3.5
Reduction	--	--	29%	--	40%	--	47%	--	31%	--	41%	--	47%	--	--
SE 27															
Inlet	1	0.5	6.3	1.4	14	5.4	13	<1	83	63	70	35	30	110	1.6
Outlet	0.6	0.4	4.9	1.7	8.9	4.5	6.6	<1	25	20	47	26	15	220	1.3
Reduction	--	--	22%	--	36%	--	49%	--	70%	--	33%	--	50%	--	--
Average Reduction	--	--	29%	--	42%	--	53%	--	51%	--	44%	--	50%	--	--

nf: non-filtered (total)
f: filtered ("dissolved")
TSS: total suspended solids
TH: total hardness, as $CaCO_3$
O&G: oil and grease
removals are only given if most observations were >PQL

Comparison to Water Quality Objectives (WQOs). Of these metals, total and dissolved chromium, lead and nickel did not exceed the acute WQOs. Total and dissolved cadmium exceeded the WQO in only one storm out of six monitored storms. Total copper at the inlet station exceeded WQOs in four out of six storm events. However, concentrations at the outlet station never exceeded WQOs (though the outlet concentration was essentially equal to the WQO for one event). None of the dissolved copper concentrations exceeded the acute WQOs. Total zinc concentrations at the inlet and outlet stations exceeded the acute WQOs for all six storms. Dissolved concentrations of zinc at the outlet station exceeded the WQOs in three of the six events.

Sediment Quality. The objectives of the sediment sampling was to characterize sediment quality in the detention basin and to compare the sediment concentrations to hazardous waste criteria. Results of these sediment sample analyses are summarized in Table 67.

Table 67. Sediment Observations (mg/kg)

	% TOC	Cadmium	Chromium	Copper	Iron	Lead	Manganese	Nickel	Zinc
6/15/90 core	--	2.2	--	92	--	36	--	61	320
5/14/92 surface									
Inlet	3.8	23	200	150	49,000	280	610	94	750
Middle	5.5	17	220	140	38,600	350	640	87	570
Outlet	1.9	35	140	47	47,700	18	680	76	260
7/12/93 surface									
Inlet	2.4	1.0	170	110	34,000	260	560	96	220
Middle	0.65	0.2	120	37	36,000	12	700	75	85
Outlet	0.93	0.3	110	43	30,000	24	570	73	63
TTLC		100	2,500	2,500	--	1,000	--	2,000	5,000

TTLC: Total Threshold Limit Concentration

In the second and third rounds of sampling, the highest concentrations for copper, nickel and zinc were found at the inlet station. Cadmium, chromium and lead were also highest at the inlet station for the July 12, 1993 sampling round. The high concentration of the majority of the metals near the inlet is consistent with other studies.

Average sediment concentrations observed in Pump Station No. 2 are compared in Table 68 with sediment data collected from other detention basins in the Valley and elsewhere. Results from the various basins differ substantially and indicate that sediment quality is highly site specific and varies depending on soils, catchment land use, and other factors, especially time when the samples were analyzed (for lead).

To evaluate whether the sediments were hazardous, concentrations were, compared to standards established in the California Administrative Code, Title 22. Under Title 22, there are two criteria for designating solids as hazardous waste. The first criterion is that the sediment concentrations not exceed the Total Threshold Limit Concentrations (TTLC). The second criterion is that the extract obtained from the WET extraction method not exceed the Soluble Threshold Limit Concentrations (STLC). For this pilot-scale screening level of analysis, it was considered adequate to compare with the TTLC only. In situations where disposal is being considered, the WET extraction test should also be conducted.

None of the sediment sample concentrations collected in the Sunnyvale Pump Station basin exceeded the TTLC. The highest concentrations of cadmium, lead, and zinc were 4, 3, and 7 times lower than the TTLC, respectively. The highest concentrations reported for chromium, copper, and nickel were 11, 17 and 21 times lower than the TTLC, respectively. Based on these sediment concentrations, these sediments are not considered hazardous.

Table 68. Comparison of Average Sediment Concentrations from Detention Basins and Swales (mg/kg)

Detention Basin	Cadmium	Copper	Nickel	Lead	Zinc
This Retrofit Basin	**11.2**	**88**	**80**	**140**	**324**
Other Santa Clara County					
Eastside Basin A	0.37	32	36	17	68
Eastside Basin B	0.37	36	40	6	73
Eastside Basin C	1	71	100	11	330
River Oaks	nd	24	72	14	84
Fresno NURP					
Recharge F	--	37	32	713	--
Recharge G	--	25	37	487	--
Recharge M	--	55	53	1333	--
Recharge EE	--	25	22	297	--
Recharge MM	--	9.5	11	93	--
Wigington (1983)					
Bulk Mail Basin	2.8	19	--	112	224
Kmart Basin	0.8	13	--	368	114
Nightingale (1975)					
Detention Basin	--	20	--	224	107
Special Pit	--	23	--	801	236
Wigington (1986)					
Fairidge Swales	0.26	4.2	--	42	102
Stratton Woods Swales	0.18	10	--	18	70
Rte. 234 Rd. Swales	0.82	23	--	936	106

Cost Effectiveness Evaluation

The mean annual runoff volume (351 acre-feet) was estimated based on mean annual rainfall (13 inches) in the vicinity of the basin, an assumed runoff coefficient (0.7), and the area of the catchment (463 acres). Mean concentrations and removal efficiencies are averages of observed data. For the metals, annual load reductions ranged from 0.6 lbs for cadmium to 65 lbs for zinc. For copper, the annual load reduction is estimated at 9 lbs, which represents approximately 40 percent of the total copper that enters the basin. Table 69 summarizes the estimated cost-effectiveness for the removal of heavy metals from the pond.

Table 69. Estimated Mean Annual Load reduction and Cost-Effectiveness*

	Mean Concentration at Inlet (mg/L)	Average Annual Removal Efficiency	Load Reduction (lbs/yr)	Cost Effectiveness (lbs/$1,000)
Cadmium	0.002	0.35	0.6	0.07
Chromium	0.012	0.29	3.3	0.40
Copper	0.023	0.42	9.1	1.1
Lead	0.037	0.53	18	2.2
Nickel	0.038	0.51	18	2.2
Zinc	0.156	0.44	65	7.9
TSS	87	0.50	41,000	5,000

*Assuming an annual runoff volume of 350 acre-ft

Solids Accumulation and Removal. About 41,000 lbs. of suspended solids would be collected annually in the retrofitted detention basin, which represents about one-half of the annual input of solids. Assuming a specific gravity of about 1.5, this would correspond to about 16 cubic yards of material annually. If uniformly distributed over the 4.4 acre basin, the mean annual accumulation rate would be 0.03 inches per year. Sediments are expected to accumulate near the inlet and, in this specific case, in the pilot channel. In ten years, this accumulation rate would equal about 0.1 acre-feet compared with the capacity of the basin that is 30 acre-feet. Therefore, this accumulation of sediments does not pose a risk to reducing the flood control capacity of the basin. Accumulation of at least 6 inches of sediment is required before removal is practical. This amount of sediment may take as long as 10 or 20 years to accumulate.

Capital, Operation and Maintenance Costs. Capital and O&M costs were estimated for the retrofitted pump station and are shown in Table 70. Costs were classified as capital expenditures, operation and maintenance, and disposal. Capital costs for the structural retrofitting were based on actual costs; whereas the costs for repair of the pump electronic control systems were estimated. Operations and maintenance assumes 100 hours per year labor in addition to that already being conducted to operate and maintain the facility for flood control. Disposal costs assume disposal is conducted every 10 years and include estimated future costs for landfill fees, trucking, and excavation. The total annualized cost is therefore estimated to be $8,200 for the 463 acre watershed, or about $18 per acre of watershed per year. The removal costs for copper were estimated to be about 1.1 lbs removed per $1,000, which compares very favorably with other stormwater control alternatives. As an example, it was estimated that street cleaning would remove about 1.5 lbs of copper from the streets per $1,000 of expenditures. However, the actual cost of removing copper from runoff by street cleaning would be about ten times this amount (Pitt 1979, 1985, and 1987) (or about 0.15 lbs per $1,000).

Table 70. Estimated Annualized Costs for Capital Expenditures and Operation

1. Capital Expenditures Structural retrofitting = $15,000 Amortized over 20 years at 8%	$1,500
2. Operations and Maintenance Inspection and repair (100 hours @ $50/hr)	$5,000
3. Disposal (every 10 years) Landfill (160 yd3 @ $50/yd3 = $8,000) Trucking (16 trips @$75/hr x 2 hrs/trip = $2,400) Excavation (160 yd3 @ $10/yd3 = $1,600) Disposal Subtotal = $12,000 Amortized over 10 years at 8%	$1,700
4. Total Cost per Year	$8,200

Conclusions

Implications for Other Facilities. According to an inventory conducted by Woodward-Clyde in 1990, nine of the existing 17 municipal pump stations in Santa Clara Valley are designed with detention basins (rather than sumps) and are, therefore, suitable for comparison with the pilot project. The detention basins range from 1.5 to 14 acres, with capacities of 4.5 to 148 acre-feet. The watershed area for each pump station ranges from 25 to 1,000 acres, and the total watershed area served by all nine stations is 4,260 acres, or 6.6 square miles. This is about 2 percent of the 350 square mile area of the Santa Clara Valley below the upland reservoirs. If we assume that other similar facilities could be retrofitted to achieve a performance comparable to that measured at Pump Station No. 2, the net reduction in copper load to the Bay would be about 100 lbs. This is only about 1 percent of the estimated mean annual load of 14,000 lbs of copper entering San Francisco Bay.

• A 100-year flood analysis indicated that modification of the pump schedule to achieve water quality benefits did not increase the maximum 100-year elevation in the pond.

• Based on measured inlet and outlet flow composite concentrations from 6 storm events, the average pollutant removal efficiencies were: total chromium, 29 percent; total copper, 42 percent; total lead, 53 percent; total nickel, 51 percent; total zinc, 44 percent; and total suspended solids, 50 percent.

• The removal efficiencies for chromium, copper, lead, nickel and zinc correlated well with TSS removal, indicating that suspended solids may be used as a surrogate parameter to monitor effectiveness of metals removal in detention basins.

• Metal concentrations of basin sediments were generally highest at the inlet location.

• None of the sediment concentrations exceeded the Total Threshold Limit Concentration (TTLC) standard, indicating that the sediments are not hazardous.

• The estimated mean annual load reduction of metals ranged between 0.6-65 lbs., depending on the metal. The mean annual load reduction for copper was 9 lbs.

• The amortized annual capital and O&M cost for retrofitting the Sunnyvale Pump Station No. 2 is estimated at $8,200. The cost effectiveness removal rate for copper is 1.1 lb/$1,000.

• Solids accumulation rates are very low and are estimated to be approximately 0.1 acre-feet over 10 years. Given that the basin has a capacity of 30 acre-feet, increased deposition caused by retrofitting does not increase flood risk.

Implications for Management.
• The total watershed area in Santa Clara Valley served by the nine pump stations with retention basins is approximately 6.6 square miles (only 2 percent of the total area of the Valley downstream of the reservoirs). Thus, even if an improved treatment performance could be obtained from these basins, the total load reduction to the Bay would be minimal. For example, the load reduction of copper would only be 100 lbs., which is less than 1 percent of the estimated mean copper load to the Bay.

• Since pump stations are relatively easy to retrofit, water quality benefits could be achieved by simply changing the pumping schedule.

• If a retrofitting program is to be pursued, it would be important to ensure that the pump control equipment is operational and well maintained, and that staff are well trained in its use.

Retrofit Case Examples from the Center for Watershed Protection (Claytor 1998)
The following two short case studies were provided by Richard Claytor of the Center for Watershed Protection, Ellicott City, Maryland (1998).

Example of Retrofitting an Existing Stormwater Detention Facility, Wheaton Branch, Montgomery County, Maryland
The Wheaton Branch facility, located near Wheaton, Maryland, is a well-know example of a former dry detention facility retrofitted to provide water quality and channel protection controls. The facility, constructed in 1990, drains an 800 acre watershed that is over 50% impervious. A unique design feature was the three cell wet pond (constructed around an existing sanitary sewer trunk main) to provide water quality controls. Extended detention controls for the 1½ inch design rainfall were incorporated for channel protection. The three cell pond has a complex flow path for both baseflows and small stormflows to facilitate maximum settling of solids. Controls for larger storms (i.e, 2 to 100 year events) were balanced against upstream backwater constraints and dam safety considerations. Figure 128 illustrates the key operational and design elements of the project.

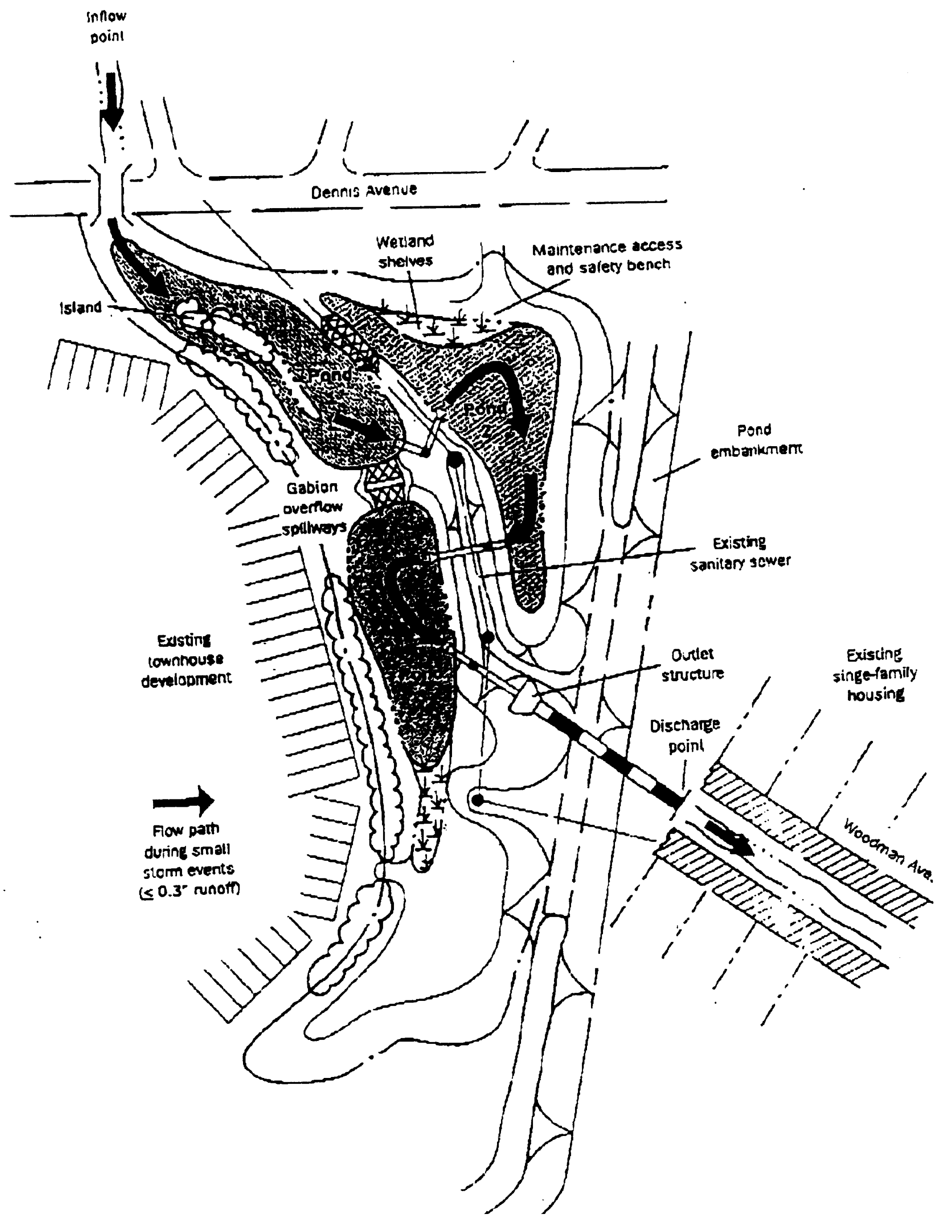


Figure 128. Wheaton Branch, Maryland, detention facility retrofit project (Claytor 1998).

The first cell of the facility, or forebay, provided almost a tenth of an inch per impervious acre (this is a good target minimum volume for most retrofits). A 25 ft wide access ramp with a level 30 ft by 30 ft pad was provided for future dredging. During the design phase, it was estimated that dredging of the forebay would be necessary every 5 years of so. The first cleanout of the forebay occurred in July 1997, a little over 7 years after completion of the project.

The Wheaton Branch retrofit facility was also part of the larger Sligo Creek watershed restoration project. Downstream habitat improvement and native fish restocking projects accompanied the retrofit and have proved very successful over their seven year trial period. John Galli (MWCOG), and his colleague Jim Commins (ICPRB) have published several reports and articles on the success of the stream restoration efforts in Wheaton Branch.

Some important design lessons are also illustrated by the Wheaton project. The existing hydraulic characteristics of the facility were first analyzed to assess the types of control originally provided. The original facility provided partial control of the 2, 10, and 100-year storm and safely passed the probable maximum flood (PMF) through a massive emergency spillway. The retrofit required a balancing act to maximize water quality control, while maintaining enough control for larger storms to avoid impacting downstream houses or the 100-year floodplain. Routing storms through the 3-cell pond was extremely difficult due to the very low head conditions and the unusual backwater created by downstream ponds. The original pond bottom was excavated for much of the permanent pool storage (for pond and wetland components), the emergency spillway was modified to maintain passage of the PMF and the outlet control structure was completely overhauled.

All of these measures added up to quite an expensive project. The total cost for the facility, including engineering, construction, and construction inspection was approximately $800,000. Although this was certainly a large total sum, it was approximately $640,000 per square mile of drainage area, somewhat less than the typically quoted figure of approximately one million dollars per square mile of drainage for average effective retrofitting projects in urban areas (Karouna 1989).

Example of a Retrofit in a Highway Right-of-Way, Bear Gutter Creek, Westchester County, New York

The Bear Gutter Creek Retrofit is one of many stormwater control programs recently designed to protect the Kensico Reservoir (one of the principle components of New York City's drinking water system) from impacts of stormwater runoff. The Bear Gutter watershed is approximately a square mile in area and drains an area having mixed land uses of approximately 30% impervious area directly into the Kensico Reservoir. Note that this is an unfiltered drinking water system that serves millions of New Yorkers. The retrofit is located immediately below a state road culvert and within the NY Route 22 Right-of-Way.

Interesting design features include a flow diversion weir at the downstream end of an existing large diameter road culvert which diverts baseflow and stormflow for up to the 1½ inch rainfall into a primary settling area. Storms larger than the 1½ inch rainfall are diverted to a stabilized downstream channel below the facility. The primary settling chamber is sized for about a third of an inch per impervious acre and has both a wet component and storm storage above the wet pool. An existing 1½ acre emergent wetland, adjacent to the facility, receives runoff as a polishing treatment below the primary settling chamber. Figure 129 is an illustration of the facility and representative design features.

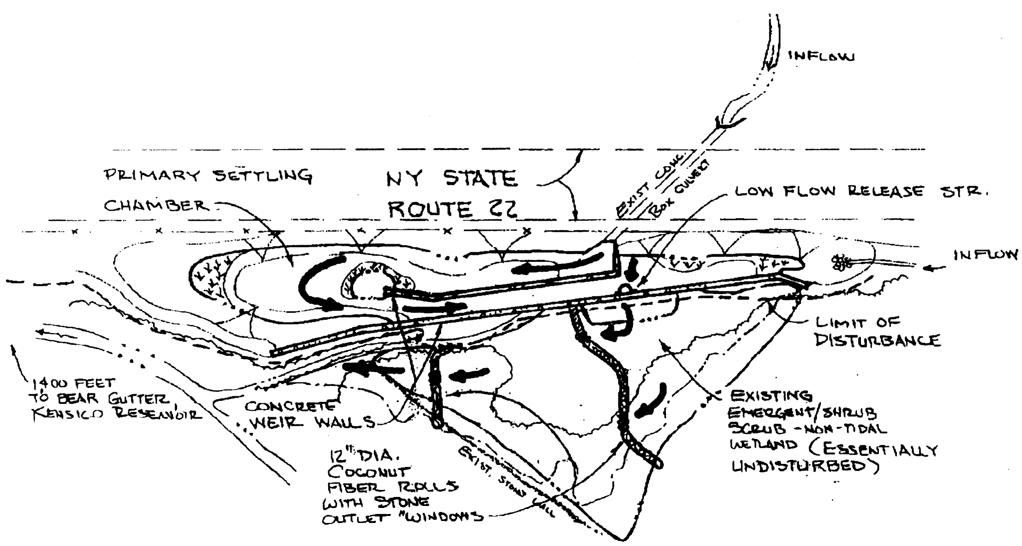


Figure 129. Bear Gutter Creek highway right-of-way urban stormwater retrofit (Claytor 1998).

The design criteria for the Bear Gutter Creek project (as well as all of the Kensico stormwater control practices) was to provide a facility with a minimum storage volume necessary to maximize particulate settling, and provide long detention times to allow for fecal coliform dieoff. An original design concept called for siting the facility within the middle of the 1½ acre wetland. Unfortunately, very little space was available within the road right-of-way or anywhere else outside of the existing wetland. The solution was to use a flow diversion structure coupled with a concrete weir and baffle to maximize a flow path within the primary settling chamber and then utilize the wetland as a "polishing" treatment. Coconut rolls were specified within the wetland to encourage additional detention for control of larger storms.

Other Examples using Sedimentation for the Control of Stormwater Pollutants

Stormwater Treatment at Critical Source Areas using the Multi-Chambered Treatment Train (MCTT)

The information presented in the following discussion is based on the results from a series of related projects sponsored by the U.S. EPA (Pitt, *et al.* 1996, Clark and Pitt 1999, Pitt, *et al*. 1999, and Clark 2000). The Multi-Chambered Treatment Train (MCTT) was developed to control toxicants in stormwater from critical source areas. The MCTT is most suitable for use at relatively small areas, about 0.1 to 1 ha in size, such as vehicle service facilities, convenience store parking areas, equipment storage and maintenance areas, and salvage yards. The MCTT is an underground device and is typically sized between 0.5 to 1.5 percent of the paved drainage area. It is comprised of three main sections, an inlet having a conventional catchbasin with litter traps, a main settling chamber having lamella plate separators and oil sorbent pillows, and a final chamber having a mixed sorbent media (usually peat moss and sand). During monitoring, the pilot-scale MCTT provided median reductions of >90% for toxicity, lead, zinc, and most organic toxicants. Suspended solids were reduced by 83% and COD was reduced by 60%. The full-scale tests substantiated these excellent reductions.

Phase 1 of this research included analyzing stormwater samples collected from many source areas in Birmingham, AL. Only a few of the analyzed runoff samples had detected organic toxicants (as is typical for stormwater evaluations), but the majority of samples analyzed had detected heavy metals (Pitt, *et al*. 1995 and Pitt, *et al*. 1999). The study also confirmed that many toxicants are associated with particulate matter in the runoff. Industrial/commercial areas are likely to be the most significant pollutant source areas, with the highest toxicant concentrations and most frequent occurrences found at vehicle service and parking/storage areas. The duration of the antecedent dry period before a storm and the intensity of the storm event were found to be significant factors influencing the concentrations of most of the toxicants detected. These critical areas were further evaluated during later treatability tests. The associated treatability study (as summarized in Appendix B) found that settling, screening, and aeration and/or photo-degradation treatments showed the greatest potential for toxicant reductions, as measured by the reduction in toxicity of the samples, using the Microtox™ toxicity screening test. The third project phase examined the toxicant reduction benefits of large-scale applications of the most suitable treatment unit processes investigated.

Aeration at Jones Falls, Baltimore, MD

Pond fountain aeration at Dayton, OH, shopping center

Pond fountain aeration at residential area, Lake Oswego, OR
Figure 130. Pond aeration to enhance circulation, aesthetics, and oxygenation of wet detention ponds.

The third phase of this research examined the use of a multi-chambered treatment tank (MCTT) to collect and treat runoff from critical stormwater source areas, including gas stations, oil change facilities, transmission repair shops, and other auto repair facilities. In an MCTT, the collected runoff is first treated in a catchbasin chamber where larger particles are removed by settling. The water then flows into a main settling chamber containing oil sorbent material where it undergoes a much longer treatment period (24 to 72 h) to remove finer particles and associated pollutants. The final chamber contains mixed media (typically comprising equal amounts of sand and peat). This final chamber acts as a polishing “filter” to remove some of the filterable toxicants from the runoff by other processes, such as ion exchange and sorption.

The pilot- and full-scale tests showed that the MCTT provides substantial reductions in stormwater toxicants (both in particulate and filtered phases) and suspended solids. Increases in color and a slight decrease in pH also occurred during the final treatment step when using peat. The main settling chamber provided substantial reductions in total and dissolved toxicity, lead, zinc, certain organic toxicants, SS, COD, turbidity, and color. The sand-peat chamber also provided additional filterable toxicant reductions. However, the catchbasin/grit chamber did not provide any significant improvements in water quality, although it is an important element in reducing maintenance problems by trapping bulk material.

Zinc and toxicity are examples where the use of the final chamber was needed to provide high levels of control. Otherwise, it may be tempting to simplify the MCTT by removing the last chamber, and rely mostly on physical settling. Another option would be to remove the main settling chamber and only use the pre-treating capabilities of the catchbasin as a grit chamber before the peat “filtration” chamber (similar to many stormwater filter designs). This option is not recommended because of the short life that the filter would have before it would clog (Clark and Pitt 1999; Clark 2000). In addition, the bench-scale tests showed that a treatment train was needed to provide some redundancy because of frequent variability in sample treatability storm to storm, even for a single sampling site.

The MCTT is capable of reducing a broad range of stormwater pollutants that cause substantial receiving-water problems (Pitt 1995a and 1995b; Burton and Pitt 2001). The MCTT has a high potential for cost-effective use as an integrated component in watershed management programs designed to protect and enhance receiving waters.

Description of the MCTT
Figure 131 shows a cross section of the MCTT. The catchbasin functions primarily as a protector for the other two units by removing large, grit-sized material. The setting chamber is the primary treatment chamber for removing settleable solids and associated constituents. The sand-peat filter is for final polishing of the effluent, using a combination of sorption and ion exchange for the removal of soluble pollutants, for example.

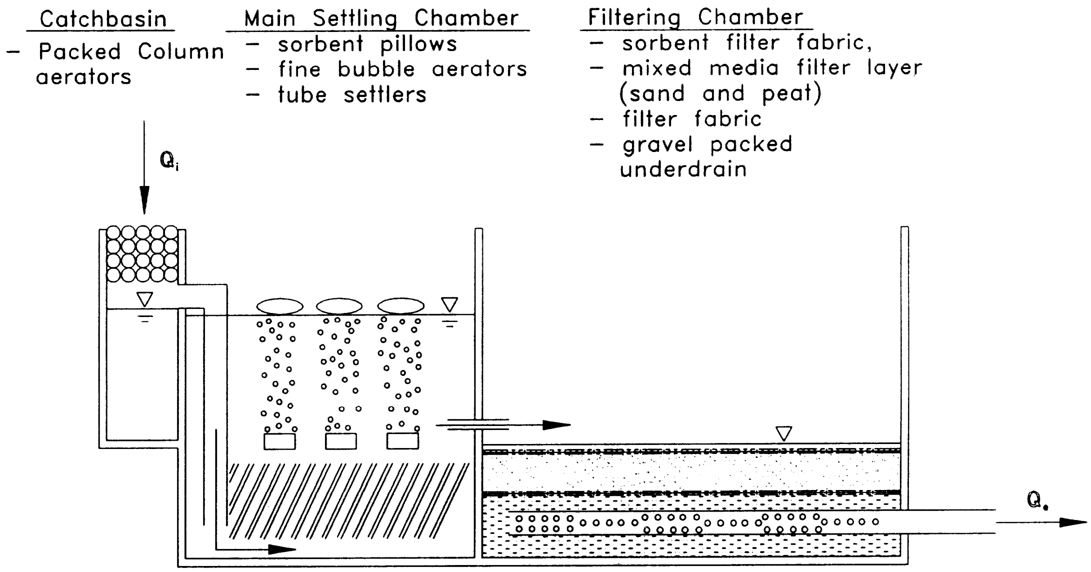


Figure 131. MCTT cross-section.

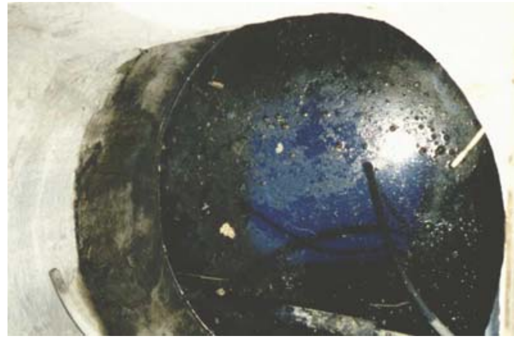
The treatability and source area information described in the main research report (Pitt, *et al.* 1999) can be used to develop other source area or outfall stormwater controls. As an example, it would be relatively easy to enhance the performance of typical wet detention ponds by adding some of the unit processes investigated. The most important control process would be to enhance the capture of small particles. In addition, water circulation and aeration may also enhance toxicant control by better utilizing photo-degradation and aeration processes. Care obviously needs to be taken to minimize scour of the deposited sediments. Conventional aeration design usually results in a circulation and aeration system than would have about 1/10 of the energy requirements needed for bottom scour. Subsurface discharges would also be an important addition in a wet detention pond to maximize capture of floatable debris and oils. Obviously, many other small units like the MCTT can be conceived and used for stormwater control at critical areas also. Typical goals would be to use a treatment unit having redundant processes, is easy to maintain, is robust for the changing conditions expected, and has the least cost possible for the needed level of stormwater control.

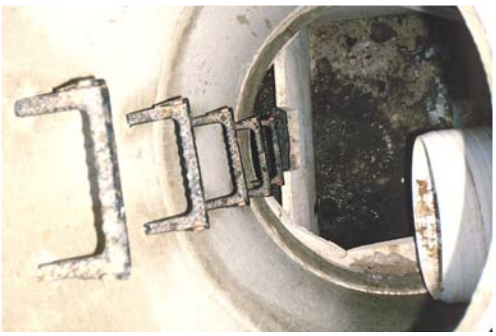
Catchbasin/grit chamber. Catchbasins have been found to be effective in removing coarser runoff solids. Moderate reductions in total and suspended solids (SS) (up to 45%, depending on the inflowing water rate) have been indicated by prior studies (Lager, *et al.* 1977, Aronson, *et al.* 1983, Pitt 1979, and Pitt 1985). While relatively few pollutants are associated with these coarser solids, their removal decreases maintenance problems of the other MCTT chambers.

Pitt and Field (1998) also evaluated three storm drain inlet designs in Stafford Township, NJ, as part of this EPA research: a conventional catchbasin with a sump, and two representative designs that used filter fabric material. The inlet devices were located in a residential area. Twelve storms were evaluated for each of the three inlet units by taking grab composite samples using a dipper sampler throughout the events. Influent and effluent samples were analyzed for a broad range of conventional pollutants, metals, and organic toxicants, both in total and filtered forms. The catchbasin with the sump was the only device that showed significant removals for suspended solids (0 to 55%, average 32%).

The MCTT catchbasin/grit chamber design is based upon a recommended design from previous studies of catchbasins (Lager, *et al.* 1977 and Aronson, *et al.* 1983). This design suggests using a circular catchbasin with the diameter 4 times the diameter of the circular outlet. The outlet is then placed 1.5 times its diameter from the top and 4 times its diameter from the bottom of the catchbasin, thus providing a total depth of 6.5 times the outlet diameter. The size of the MCTT catchbasin is controlled by three factors: the runoff flow rate, the SS concentration in the runoff, and the desired frequency at which the catchbasin will be cleaned so as not to sacrifice efficiency.


Lakewood Park and Ride, Caltrans, Los Angeles, CA, MCTT inlet chamber


Ruby Garage, Milwaukee, WI, MCTT inlet chamber


Minocqua, WI, MCTT inlet chamber


Ocean County, NJ, inlet tests

Figure 132. MCTT inlet chambers.

Main settling chamber. The main settling chamber mimics the completely mixed settling column bench-scale tests previously conducted and uses a hydraulic loading rate (depth to time ratio) for removal estimates. This loading rate is equivalent to the conventional surface overflow rate (SOR), or upflow velocity, for continuous-flow systems, or the ratio of water depth to detention time for static systems. The MCTT can be operated in both modes. If it uses an orifice, to control the settling chamber outflow, then it operates in a similar mode to a conventional wet detention pond and the rate is the upflow velocity (the instantaneous outflow divided by the surface area of the tank). If the outflow is controlled with a float switch and a pump, then it operates as a static system and the hydraulic loading rate is simply the tank depth divided by the settling time before the pump switches on to remove the settled water.

In addition to housing plate or tube settlers, the main settling chamber also contains floating sorbent “pillows” to trap floating oils and a fine bubble aerator that operates during the filling time of the MCTT. Plate settlers (or inclined tubes) increase solids removal by reducing the distance particles travel to the chamber floor and by reducing scour potential. Plate settler theory is described by Davis, *et al.* (1989). The main settling chamber operates much like a settling tank, but with the plate settlers increasing the effective surface area of the tank. The increase in performance is based on the number of plate diagonals crossing the vertical. If the plates are relatively flat and close together, the increase in performance is greater than if the plates are steeper and wider apart. The effective increase is usually about 3 to 5 fold.

The fine bubble aerator serves two functions: to support aerobic conditions in the settling chamber and to provide dissolved air flotation of particles. Aeration was used during the pilot-scale MCTT tests, but was not used during the full-scale Wisconsin or Caltrans MCTT tests. Dissolved air flotation has been utilized in industrial applications and combined sewer overflows (Gupta, *et al*. 1977). The settling time in the main settling chamber typically ranges from 1 to 3 d, and the settling depth typically ranges from 0.6 to 2.7 m (2 to 9 ft). These depth to time ratios provide for excellent particulate (and associate pollutant) removals in the main settling chamber.

Bench-scale tests found that depth/time ratios of at least 3 X 10^{-5} m/s (1 X 10^{-4} ft/s) are needed to obtain a median toxicity reduction of at least 70 percent in the main settling chamber. If the main settling chamber tank were one meter (3.3 ft) deep, then the required detention time would have to be at least 0.4 days to obtain this level of treatment. If the tank were twice as deep, the required detention time would be 0.8 days. The tank surface area is therefore based on the volume of runoff to be detained and the settling depth desired/available. Shallow tanks require shorter detention times than deeper tanks, but the surface areas are correspondingly larger, and scour may be more of a problem. Since the MCTT is placed underground, a tank having a large surface area (and a shallower depth) may be much more expensive than a deeper tank requiring a longer detention time.

The design of a stormwater treatment device, including the MCTT, is greatly dependent on the rainfall pattern for a specific area. In water quality evaluations, a single "design storm" is not evident because of the many factors comprising runoff quality (runoff volume, runoff flow rate, water temperature, concentrations of many different pollutants, etc.). It is not very clear under which storm condition the combination of these factors is critical for the local beneficial uses. In addition, targeting a specific size storm is no guarantee that all storms of lesser magnitude will also be adequately controlled. Continuous simulation is therefore needed to effectively design and evaluate most stormwater quality controls.

If the rains are infrequent, long detention periods are easily obtained without having "left-over" water in the tank at the beginning of the next event. However, if the rains are frequent, the available holding times are shortened, requiring shallower main settling chamber tanks for the same level of treatment. A spreadsheet model was used to develop design curves for many locations of the U.S. based on long-term rain records, desired levels of control, and tank geometry. These design curves are included in the EPA report (Pitt, *et al.* 1999).

This model was used to investigate various storage capacities, holding periods, and settling tank depths for 21 cities throughout the U.S. having annual rains from about 180 – 1500 mm (7 – 60 in.). The model used the rain depths and durations, the time interval between the consecutive storm events, the dimensions of the subsurface tank, and the tank pumpout or drainage time. A random set of 100 rain events from the past 5 to 10 years (from EarthInfo CD-ROMs, Boulder, CO,) was used for each city in these simulations. The annual toxicity reductions were calculated by knowing the individual storm median toxicity reductions and the annual percentage of runoff treated. As an example, if the holding period was 24 h for a 2.1 m (7 ft) deep settling chamber, the individual median storm toxicity reduction would be about 75%. If the MCTT were large enough to contain the runoff from a 38 mm (1.5 in) rain, then about 98% of the annual runoff would be treated, for an annual expected toxicity reduction of 73% (0.75 X 0.98 = 0.73).

Figure 133 is a plot for Birmingham, AL, for different annual control levels associated with holding periods from 6 – 72 h and storage volumes from 2.5 – 51 mm (0.1 – 2.0 in.) of runoff for a 2.1 m (7 ft) deep MCTT. This figure can be used to determine the size of the main settling chamber and the minimum required detention time to obtain a desired level of control (toxicity reduction). If the MCTT were full from a previous rain (because of the required holding period), the next storm would bypass the MCTT with no treatment. Birmingham, AL, rains typically occur about every 3 to 5 d, so it would be desirable to have the holding period less than this value. Similarly, if the storage volume was small, only a small fraction of a large rain would be captured and treated, requiring a partial bypass for most rains.

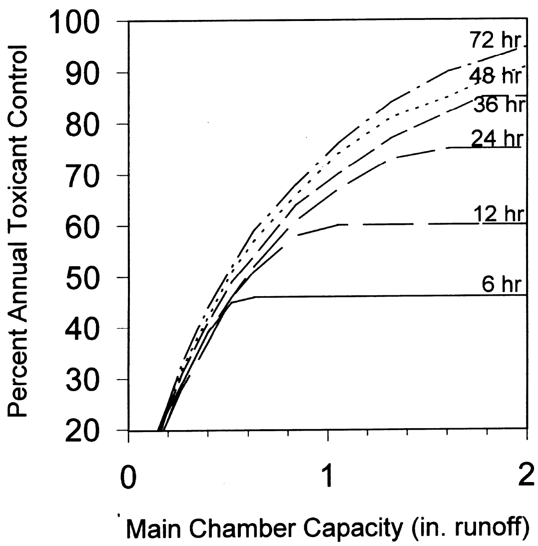
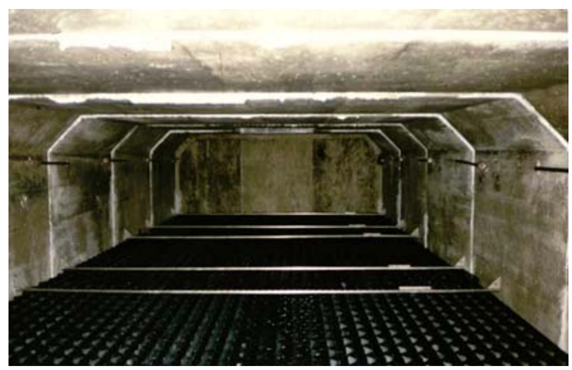


Figure 133. Effects of storage volume and treatment time on annual toxicity reduction, 2.1 m settling depth) (Example storage-treatment plot for Birmingham, AL).

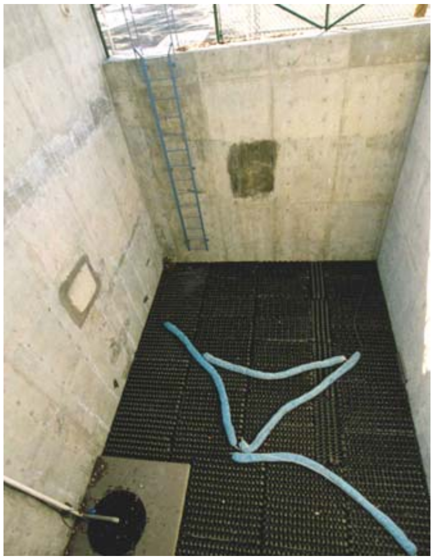
This plot shows that the most effective holding time and storage volume for a 70% toxicity reduction goal is 72 hours and 22 mm (0.86 inch) of runoff storage. A shorter holding period would require a larger holding tank for the same level of control. Shorter holding periods may only be more cost-effective for small removal goals (<50%). If a 6-hour holding time were used, the maximum toxicant removal would only be about 46% for this tank depth.

Lakewood Park and Ride, Caltrans, Los Angeles, CA, MCTT main settling chamber



Minocqua, WI, MCTT main settling chamber

Ruby Garage, Milwaukee, WI, MCTT main settling chamber



Via Verde Park and Ride, Caltrans, Los Angeles, CA, MCTT main settling chamber

Lamella plates and aerator distributors at pilot-scale MCTT, Birmingham, AL

Inclined-tube settlers at Minocqua, WI, MCTT

Figure 134. MCTT main settling chanbers, with inclined plates or tubes, and sorption pillows.

Filter/ion exchange chamber. The final MCTT chamber is a mixed media filter (sorption/ion exchange) device. It receives water previously treated by the grit and the main settling chambers. The initial designs used a 50/50 mix of sand and peat moss, while the Ruby Garage full-scale MCTT in Milwaukee used a 33/33/33 mixture of sand, peat moss, and granulated activated carbon. The MCTT can be easily modified to contain any mixture of media in the last chamber. However, care must be taken to ensure an adequate hydraulic capacity. As an example, peat moss alone was not effective because it compressed quickly, preventing water from flowing through the media. However, when mixed with sand, the hydraulic capacity was much greater and didn't change rapidly with time.

Initial bench-scale tests showed that sand by itself (especially if recently installed) did not permanently retain the stormwater toxicants (which are mostly associated with very fine particles and which were mostly washed from the sand during later events). This lack of ability to permanently retain stormwater toxicants prompted the investigation of other filtration media. Further research as part of this U.S. EPA supported cooperative research agreement (Clark and Pitt 1999 and Clark 2000) examined the pollutant removal benefits and design criteria for several candidate media.

Combinations of filtration media, including organic materials (peat moss, activated carbon, composted leaves, and a cotton processing waste material), Zeolite, and sand, were investigated for their ability to more permanently retain stormwater pollutants. Sand was mixed with most of these materials in order to maintain adequate hydraulic capacities, especially for peat. Some clogging tests have shown that channeling still occurred in the Zeolite-sand combination media, significantly decreasing the performance by decreasing the contact time provided by simple gravity flow. The use of a restrictive filter fabric placed on top of the peat-sand filter in the MCTT allows the water to spread over the filter and help prevent preferential channel flow.

The sand-peat filter possesses ion exchange, adsorption, and filtration reduction mechanisms. As the media ages, the performance of these processes will change. Ion exchange capacity and adsorption sites, primarily associated with the peat moss, will be depleted. Filtration, primarily associated with the sand, however, is expected to increase, especially for the trapping of smaller particles. Improved performance of sand filters with age has been documented by Darby, *et al*. (1991). Eventually though, the sand-peat filter will become clogged by solids and the exchange capacity of the peat will be exceeded, requiring replacement of the media. Replacement of the media in the MCTT is expected to be necessary about every 3 to 5 years.

Initial Pilot-Scale Tests

Pilot-scale tests on the campus of the University of Alabama at Birmingham at a long-term parking lot and vehicle service area verified the design procedures and indicated very high pollutant removal capabilities. The pilot-scale MCTT was evaluated for 13 storm events. Based solely upon the design of the settling chamber, percent toxicity reductions were predicted to be near the 90% reduction level. Actual performance of the overall MCTT was found to have a median value of 96%. The median toxicity reduction of the filtered samples was found to be 87%.

Exact 1-sided probabilities were calculated by the Wilcoxon Signed Rank Test for paired observations using StatXact-Turbo™ software by Cytel Software Corporation. The exact probability calculated is based upon sign and magnitude of concentration differences occurring across each chamber and across the entire MCTT, while omitting zero differences. The software calculated an exact p value as opposed to a p value obtained asymptotically which would inherently decrease accuracy for the relatively small sample size. The software also expedited data analysis by performing the statistical tests in a batch mode.

Table 71 shows performance summaries for the settling chamber, sand-peat chamber, and for the overall MCTT for the major constituents of interest. The catchbasin was not found to provide significant toxicity reductions, as expected, and is therefore not included on this summary table. The catchbasin was used to provide grit and other coarse solids control to reduce maintenance in the other chambers.

By design, the settling chamber was assumed to provide most of the pollutant reductions. The other two chambers and secondary features were added for extra benefit, especially to reduce variations in performance for the highly variable runoff conditions. As an example, good toxicant reductions occurred in both the settling chamber and the sand-peat filter.

Lakewood Park and Ride, Caltrans, Los Angeles, CA, MCTT filtration/sorption chamber

Ruby Garage, Milwaukee, WI, MCTT filtration/sorption chamber

Minocqua, WI, MCTT filtration/sorption chamber

Via Verde Park and Ride, Caltrans, Los Angeles, CA, MCTT filtration/sorption chamber

Figure 135. MCTT filtration/sorption chambers.

Wisconsin Full-Scale MCTT Tests

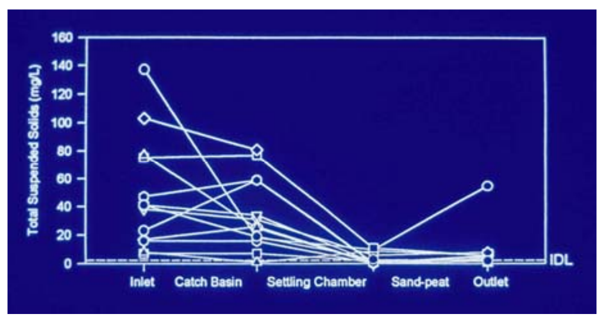
Full-scale units were installed in Milwaukee and Minocqua, WI, and monitored for a one-year period. Results from the full-scale tests of the MCTT in Wisconsin (Corsi, *et al.* 1999) were encouraging and collaborated the high levels of treatment observed during the pilot-scale tests. Table 72 shows the treatment levels that have been observed during seven tests in Minocqua (during one year of operation) and 15 tests in Milwaukee (also during one year of operation). These data indicate high reductions for SS (83 to 98%), COD (60 to 86%), turbidity (40 to 94%), phosphorus (80 to 88%), lead (93 to 96%), zinc (90 to 91%), and for many organic toxicants (generally 65 to 100%). The reductions of dissolved heavy metals (filtered through 0.45 μm filters) were also all greater than 65% during these full-scale tests. None of the organic toxicants were ever observed in effluent water from either full-scale unit, even considering the excellent detection limits available at the Wisconsin State Department of Hygiene Laboratories that conducted the analyses. The influent organic toxicant concentrations were all less than 5 μg/L and were only found in the unfiltered sample fractions. The Wisconsin MCTT effluent concentrations were also very low for all of the other constituents monitored: <10 mg/L for SS, <0.1 mg/L for phosphorus, <5 μg/L for cadmium and lead, and <20 μg/L for copper and zinc. The pH changes in the Milwaukee MCTT were much less than observed during the Birmingham pilot-scale tests, possibly because of the added activated carbon in the final chamber in Milwaukee. Color was also much better controlled in the full-scale Milwaukee MCTT.

Table 71. Median Percent Reductions by Chamber

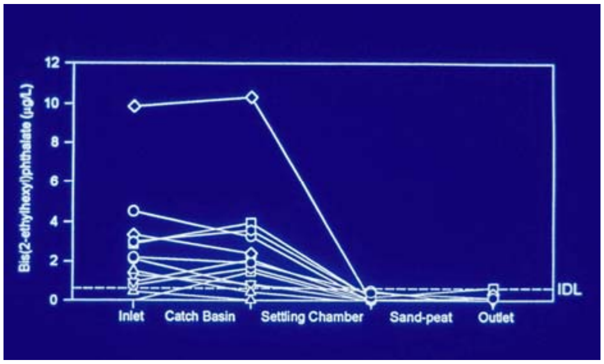
Constituent	Main Settling Chamber (percent)	Sand-Peat Chamber (percent)	Overall Device (percent)
Common Constituents			
total solids	***31***[a]	2.6	***32***
suspended solids	***91***	-400	***83***
turbidity	***50***	***-150***	***40***
conductivity	-15	***21***	***11***
apparent color	***16***	***-75***	***-55***
pH	-0.3	***6.7***	***7.9***
COD	***53***	-55	***54***
Nutrients			
nitrate	***27***	-5	***24***
ammonia	***-62***	-7	***-400***
Toxicants			
Microtox™ toxicity (unfiltered)	***18***	***70***	***96***
Microtox™ toxicity (filtered)	***69***	67	***87***
lead	***88***	***18***	***93***
zinc	***39***	62	***91***
n-Nitro-di-n-propylamine	81	64	92
hexachlorobutadiene	29	97	100
pyrene	100	25	100
bis (2-ethylhexyl) phthalate	***99***	N/A	***99***

[a] Note: Bold italics indicate Wilcoxon 1-sided p values of ≤0.05

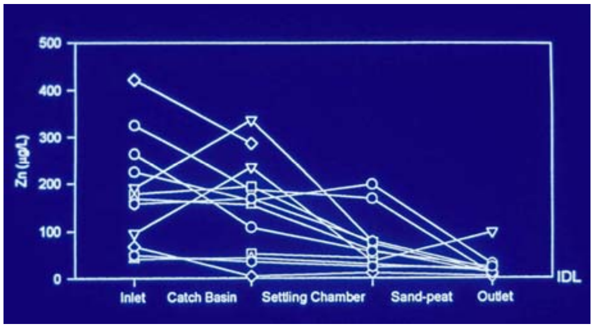

Pilot-scale setup in Birmingham, AL.



Suspended solids removals for different unit processes in MCTT



Phthalate ester removal for different unit processes in MCTT



Zinc removal for different unit processes in MCTT

Figure 136. Pilot-scale MCTT and performance plots.

Table 72. Performance Data[(1)] for WI Full-Scale MCTT Tests (median percent reductions and median effluent quality)

	Milwaukee MCTT (15 events)	Minocqua MCTT (7 events)
suspended solids	98 (<5 mg/L)	85 (10 mg/L)
volatile suspended solids	94 (<5 mg/L)	na[a]
COD	86 (13 mg/L)	na
turbidity	94 (3 NTU)	na
pH	-7 (7.9 pH)	na
ammonia	47 (0.06 mg/L)	na
nitrates	33 (0.3 mg/L)	na
Phosphorus (total)	88 (0.02 mg/L)	>80 (<0.1 mg/L)
Phosphorus (filtered)	78 (0.002 mg/L)	na
Microtox® toxicity (total)	Na	na
Microtox® toxicity (filtered)	Na	na
Cadmium (total)	91 (0.1 µg/L)	na
Cadmium (filtered)	66 (0.05 µg/L)	na
Copper (total)	90 (3 µg/L)	65 (15 µg/L)
Copper (filtered)	73 (1.4 µg/L)	na
Lead (total)	96 (1.8 µg/L)	nd (<3 µg/L)
Lead (filtered)	78 (<0.4 µg/L)	na
Zinc (total)	91 (<20 µg/L)	90 (15 µg/L)
Zinc (filtered)	68 (<8 µg/L)	na
benzo(a)anthracene	>45 (<0.05 µg/L)	>65 (<0.2 µg/L)
benzo(b)fluoranthene	>95 (<0.1 µg/L)	>75 (<0.1 µg/L)
dibenzo(a,h)anthracene	89 (<0.02 µg/L)	>90 (<0.1 µg/L)
fluoranthene	98 (<0.1 µg/L)	>90 (<0.1 µg/L)
indeno(1,2,3-cd)pyrene	>90 (<0.1 µg/L)	>95 (<0.1 µg/L)
phenanthrene	99 (<0.05 µg/L)	>65 (<0.2 µg/L)
pentachlorophenol	na	na
phenol	na	na
pyrene	98 (<0.05 µg/L)	>75 (<0.2 µg/L)

[1] Samples analyzed in accordance with approved EPA or Standard Methods and in accordance with the pre-approved Quality Assurance Project Plan.
na[a] : not analyzed   nd[b]: not detected in most of the samples

The Milwaukee installation was at a public works yard and served about 0.1 ha (0.25 acre) of pavement. This MCTT was designed to withstand very heavy vehicles driving over the unit. The estimated cost was $54,000 (including a $16,000 engineering cost), but the actual total capital cost was $72,000. The high cost was due to uncertainties associated with construction of an unknown device by the contractors and because it was a retrofitted installation.

The Minocqua site was a 1 ha (2.5 acre) newly paved parking lot for a state park and commercial area. It was located in a grassed area and was also a retrofitted installation, designed to fit within an existing storm drainage system. The installed capital cost of this MCTT was about $95,000 and included the installation of the MCTT plus the parking area paving. The MCTT was built using 3.0 m X 4.6 m (10 ft X 15 ft) box culverts for the main settling chamber (13 m, or 42 ft long) and for the filtering chamber (7.3 m, or 24 ft long). These costs are about equal to the costs of installation of porous pavement (about $40,000 per acre of pavement).

Ruby Garage, Milwaukee, WI, drainage area

Ruby Garage, Milwaukee, WI, drainage area

Construction of Ruby Garage MCTT

Construction of Minocqua, WI, MCTT

Figure 137. Full-sized MCTTs and tributatory drainage area in Wisconsin.

Caltrans Full-Scale MCTT Tests

Three MCTT units were planned for the Caltrans stormwater monitoring project in Los Angeles County, CA. Two of the facilities have been completed and monitored for two years. Both sites are Park & Ride lots and range from about 0.4 to 0.8 ha (1 to 2 acre). Both drainage areas are 100% impervious. At these installations, pumps were used to ensure that the stormwater remained in the sedimentation chamber for at least 24 h. The filter chambers have a 450 mm (18 in.) layer of mixed media (50/50 mixture of sand and peat moss). The filter areas were sized using a loading rate of 5,000 g SS/m^2/yr (1 lb/ft^2/yr).

Major maintenance items for MCTTs include removal of sediment from the sedimentation basin when the accumulation exceeds 150 mm (6 in.) and removing and replacing the filter media about every 3 years. Neither of these activities was required during the first two years of the Caltrans study. After two wet seasons, the total accumulated sediment depth was less than 25 mm (1 in.), indicating that sediment removal may not be needed for about 10 years. The sorbent pillows were scheduled to be replaced annually, or sooner if darkened by oily stains. Weekly general inspections were conducted during the wet season for such things as trash removal from the inlet and outlet structures. Monthly inspections were also conducted to identify damage to inlet and outlet structures, and evidence of graffiti or vandalism. Because the MCTT test units used by Caltrans were above ground and not initially covered, the permanent pools were available for mosquito breeding. The Via Verde site was finally completely enclosed to prevent mosquito access.

Table 73 is a summary of the average influent and effluent concentrations averaged for the two year monitoring period, and resulting reductions, for these Caltrans tests (Michael Barrett, University of Texas, personal communication). Statistical tests showed no significant differences between the two MCTT sites, so their data were combined for this table. These data indicated comparable performance to the Austin sand filter design that was also tested, even with the additional peat moss and the pre-treatment provided in the MCTT. This was likely due to the low influent concentrations observed at these two parking lot sites and the absence of more contaminated runoff for which the MCTT was designed. Caltrans ranked the performance of the stormwater controls in the following general order (based on SS performance): MCTT and Austin media filter; wet basin; infiltration devices; Delaware media filter; biofilter strip; dry detention basin; biofilter swale; StormFilter®; and drain inlet inserts. Further information concerning the Caltrans stormwater program is available at: www.dot.ca.gov/hq/env/stormwater/.

Table 73. Initial Caltrans Test Results for MCTTs

Constituent	Average Influent Concentration (mg/L)	Average Effluent Concentration (mg/L)	Concentration Reduction (%)
TSS	29.6	6	80
Nitrate	0.42	0.68	-62
TKN	1.27	0.82	35
N Total	1.69	1.50	11
P Total	0.18	0.11	39
Cu Total	0.008	0.005	38
Pb Total	0.006	0.003	50
Zn Total	0.086	0.013	85
Cu Dissolved	0.004	0.003	25
Pb Dissolved	0.001*	0.001*	NA
Zn Dissolved	0.050	0.013	74
TPH-Oil	0.34	0.20*	>41
TPH-Diesel	1.43	0.21	85
Fecal Coliform	973 MPN/100mL	171 MPN/100mL	82

*equals value of reporting limit

Note– TPH and Coliform collected by grab method and may not accurately reflect removal. The concentrations are the mean of the event mean concentrations (EMCs) for the entire monitoring period.

Summary of MCTT Performance

The pilot- and full-scale test results show that the MCTT provides substantial reductions in stormwater toxicants (both in particulate and filtered phases) and suspended solids. Increases in color and a slight decrease in pH also occurred during the final treatment step when using peat as part of the filtering/ion-exchange media. The main settling chamber provided substantial reductions in total and dissolved toxicity, lead, zinc, certain organic toxicants, SS, COD, turbidity, and color. The sand-peat chamber also provided additional filterable toxicant reductions. However, the catchbasin/grit chamber did not provide any significant improvements in water quality, although it is an important element in reducing maintenance problems by trapping bulk material.

Zinc and toxicity are examples where the use of the final chamber was needed to provide high levels of control. Otherwise, it may be tempting to simplify the MCTT by removing the last chamber. Another option would be to remove the main settling chamber and only use the pre-treating capabilities of the catchbasin as a grit chamber before the peat "filtration" chamber (similar to many stormwater filter designs). This option is not recommended because of the short life that the filter would have before it would clog from the silt and fine sand in stormwater. In addition, the bench-scale treatability tests conducted during the development of the MCTT (Pitt, *et al.* 1999) showed that a treatment train was needed to provide some redundancy because of frequent variability in sample treatability storm to storm, even for a single sampling site.

The MCTT operated as intended: it provided very effective reductions for both filtered and particulate stormwater toxicants and SS. Because of its high cost, it may only be suitable for critical source areas where high levels of toxicant reductions are needed. Much of the added expense is associated with the underground installation of the MCTT to enable it to be located in areas having little room for alternative stormwater control options. In addition, the pilot-scale and full-scale installations described in this paper were all designed for very high levels of control. This research also examined treatability of stormwater toxicants in general, and this information can be used to develop or improve other stormwater treatment devices.

Example Calculation of Sediment Capture Behind Filter Fence

Filter fabric fences commonly used at construction sites actually remove sediment through sedimentation in the water ponded behind the fence and not through filtering. The following photographs clearly show accumulated sediment in the pool area, with relatively little trapped sediment captured on the fabric material:

Figure 138. Filter fence installations showing areas of sedimentation where water pools behind the fences

It is possible to calculate the expected level of control for a filter fence at a specific construction site using the upflow velocity concept presented earlier:

$$v = \frac{Q_{out}}{A}$$

Where *v* is the settling rate for the critical particle, with faster settling particles being trapped, Q_{out} is the discharge from the pond at a specific water depth, and A is the pond surface area at that depth. The performance of a filter fence can therefore be calculated by knowing the ratio of the discharge through the fence divided by the surface area of the ponded area. Both of these values are directly related to the depth of water detained behind the filter fence. This value can be easily calculated assuming an even slope uphill from the fence and using the manufacture's value for unit area flow capacity. The ponded surface area increases directly with the water depth, depending on the slope. The total outfall rate also increases directly with the water depth. Therefore, the critical particles being trapped in the pond behind the filter fence is only dependent on the slope and fabric flow capacity. Figure 139 is a plot of the particle size controlled, in µm, for different ground slopes (%) and filter fabric flow rates (ft/sec), using Stokes' law for calculating the critical particle sizes associated with the upflow velocity:

$$v = \frac{1}{18}\left[\frac{g}{\kappa}(spgr - 1)\right]d^2$$

where:
v= settling rate of particle, cm/sec
g = 981 cm/sec^2
k = kinematic viscosity = 0.01 cm^2/sec
spgr = specific gravity of particulate = 2.65
d = particle diameter, cm

The particle size distribution can be used to estimate the approximate suspended solids control corresponding to the critical particle size. For example, if the calculated critical particle size was 10 µm (such as for a 2% slope and a 0.02 ft/sec filter fabric flow rate), the expected SS control would be about 25 to 45% for the size distributions likely appropriate for construction site runoff. A 5% slope and 0.25 ft/sec flow rate would result in about a 60 µm critical particle size, and the SS control would only be about 5 to 15%.

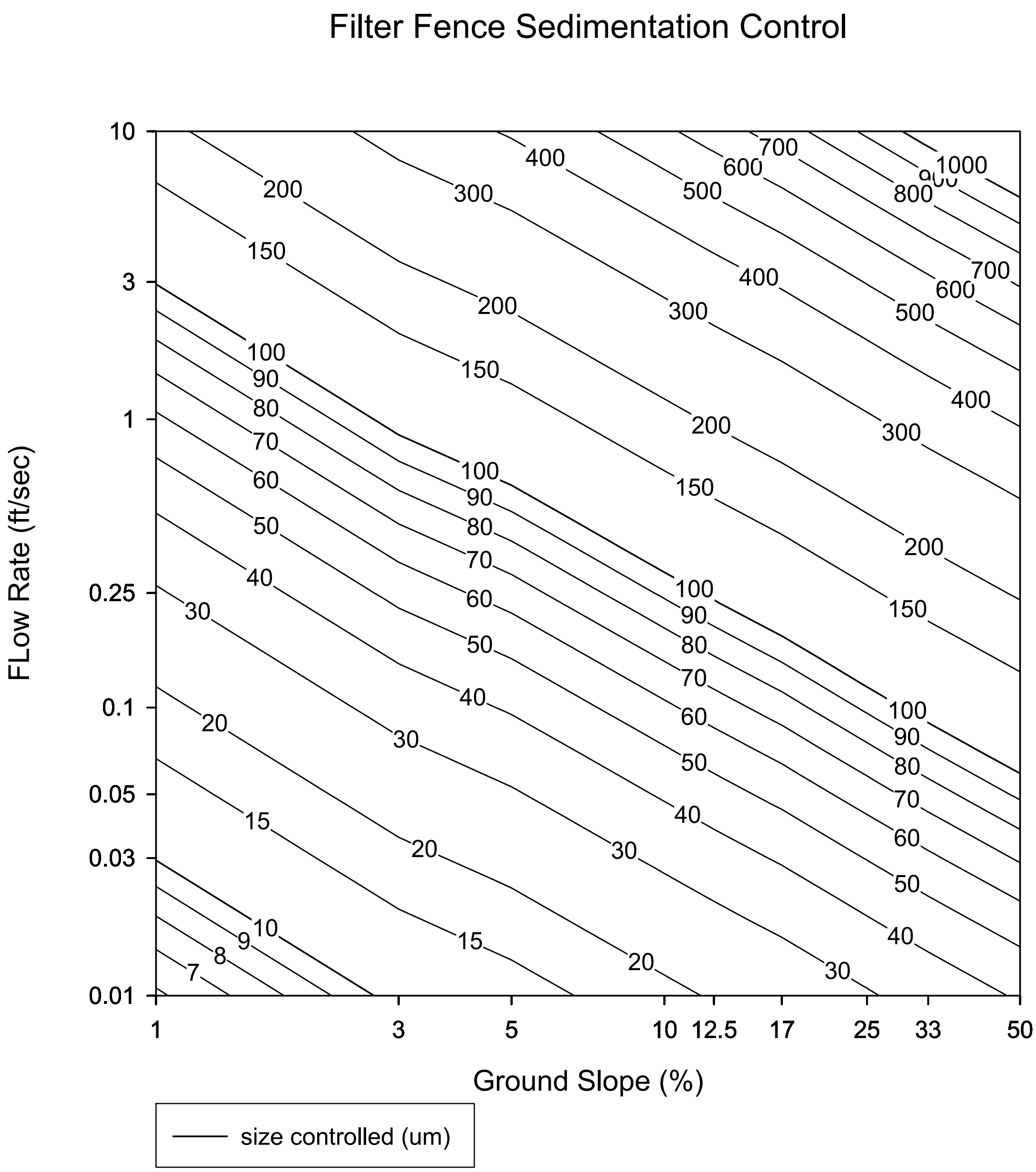


Figure 139. Filter fabric conditions and critical particle size controlled.

Conclusions

This chapter has shown that the use of relatively simple design criteria can be used to provide excellent water quality benefits over a wide range of storm conditions. WinDETPOND can be used to evaluate a wide variety of pond designs and can be used to develop appropriate design guidelines for different climatic conditions. Wet detention ponds for water quality control can also be used to provide drainage and flood control benefits by providing additional free board storage. However, a detailed hydrologic investigation of the complete watershed is necessary to make sure that these detention ponds do not actually increase drainage and flooding problems downstream.

Detention ponds are probably the most commonly used stormwater quality devices and have substantial literature documenting their performance and problems. Wet detention ponds have been shown to be very effective, if their surface area is large enough in comparison to the drainage area and expected runoff volume. Small wet ponds and all dry ponds have been shown to be much less effective. Detention ponds can be easily integrated into a comprehensive stormwater management program, but only if land is available and if installed at the time of development. They are very difficult and expensive to retrofit into existing areas. Care must also be taken to minimize safety and environmental hazards associated with ponds in urban areas. In addition to safety concerns, contaminated sediment management and poor water quality are major issues.

Monitoring of stormwater detention ponds is needed to confirm the adequacy of any stormwater control design criteria, including the simple criteria as presented in this paper. If the performance is different than desired, then the criteria should be appropriately adjusted. Because of the relatively large volume of water contained in detention ponds, long-term continuous monitoring of influent and effluent quality is needed. Haphazard storm event monitoring can result in inaccurate evaluations of detention ponds. The effluent of the pond for relatively small storms may not be related to the current storm's influent, but can actually be mostly made of displaced water that had resided in the pond since previous events. Also, in order to effectively design wet detention ponds, along with many other sediment practices (including grass filters, catch basins, and other types of sumps) particle size and/or settling rate analyses are necessary. This information can be obtained using conventional settling column tests directly resulting in settling velocity information. Small sieves, ranging from 20 µm to up to several hundred µm, can also be used along with total solids gravimetric analyses to obtain particle size data. These tests would result in particle diameter measurements and specific densities would have to be assumed or measured using other procedures in order to calculate settling velocities. The use of laser or other types of particle counters may also be worthwhile in order to rapidly obtain the needed particle size data.

Wet detention ponds have been shown to be an extremely robust stormwater control practice. Even though their cost may be high, their level of pollutant reduction is also high, resulting in very cost-effective pollutant removals. Physical sedimentation is the main removal process occurring in wet ponds, resulting in much better removals of particulate bound pollutants than "filterable" forms of pollutants. Fortunately, for many of the stormwater pollutants of concern, particulate forms are much more abundant then filterable forms. Wet detention ponds can also be optimized to encourage biochemical processes that can further reduce many filterable pollutants. Even though wet detention ponds have been demonstrated to provide high levels of control, they may not be the best control for all conditions. Combinations of controls, determined using a comprehensive watershed evaluation tool, are likely to result in the best stormwater control program.

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Appendix A: User Guide for WinDETPOND

The following example shows the initial steps in designing a wet detention pond and the development of a WinDETPOND file for that pond in order to enable water quality evaluations. The pond sizing criteria can be examined in relation to site constraints and the pond design modified, if needed, based on these evaluations.

Example Design Calculations and Evaluation Using WinDETPOND

The following discussion presents a calculation example using the design criteria presented earlier:

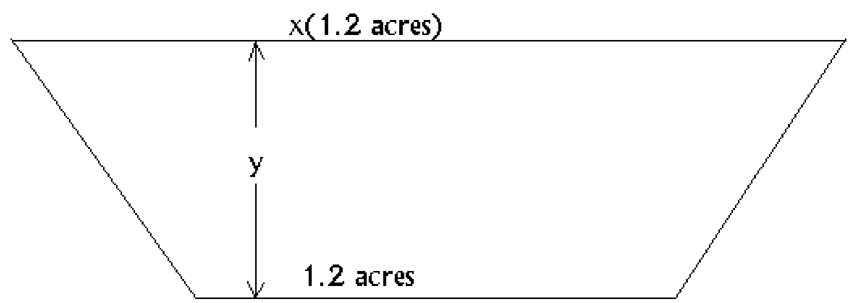
- Assume a medium density residential area of 150 acres with a goal of approximately 90% suspended solids control (corresponding to 5μm critical particle size).

- The wet pond surface would therefore be: 0.008(150 acres) = 1.2 acres

- The runoff volume for 1.25" rain => 0.5" runoff (based on typical development conditions and small storm hydrology; CN= 90 and Rv= 0.4).

- Therefore, wet storage volume: 0.5"(150 acres) => 6.3 acre-feet

- The depth associated with the wet storage volume can be estimated assuming a prismatic cross-section (simplified, compared to a conical section):



Approximately: $[1.2 + x(1.2)]y/2 = 6.3$ acre-ft.

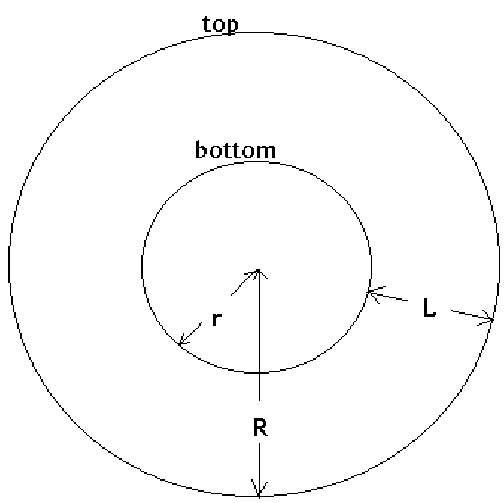
re-arranging gives: $x =[(10.5)/y] - 1$

The following table can be used to give simultaneous depths for different x multipliers and top of pond areas for the "live-storage" area of the pond (the section affected by the primary water quality outlet device and located on top of the permanent pool depth, and below the invert of the emergency spillway and additional storage needed for flood control):

y (depth, ft)	x (multiplier)	top area
2	4.3	4.3 (1.2 acres) = 5.2 acres
3	2.5	3.0 acres
4	1.6	1.9 acres
5	1.1	1.3 acres

Depths less than 2 feet are too shallow and could require very large pond top surface areas for this example. "Live depths" greater than 5 feet may be too deep for most locations and obviously result in very steep side slopes for this example.

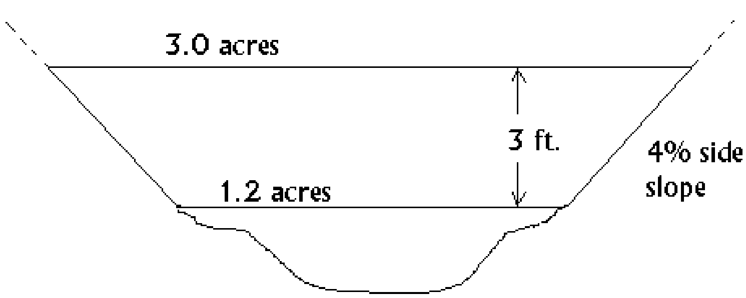
The following table summarizes the calculations for the side slopes of the pond (assuming a simple circular shaped pond, as shown below):



$r = (A/\pi)^{1/2} = [1.2\text{acres}(43{,}560\text{ ft}^2\text{ per acre})/\pi)]^{1/2} = 130\text{ ft}$

Depth (ft)	Top Area (acres)	Top Radius (ft)	Slope Length (ft)	Side Slope
2	5.2	270	270 - 130 = 140	2/140 = 1.4%
3	3.0	200	200 - 130 = 70	3/70 = 4.3%
4	1.9	160	160 - 130 = 30	4/30 = 13%
5	1.3	135	135 - 130 = 5	5/5 = 100%

• The preliminary pond cross-section is therefore:



• The outfall device is selected by comparing the maximum allowable discharge rate for the surface area of the pond at several pond depth increments. These maximum allowable discharges are compared with weir ratings (as tabulated in the text, for example) to select the permissible weirs that can be used:

$Q_{out} = vA$
$v = 1.3 \times 10^{-4}$ ft/sec for 5 µm particle

Stage (above normal water surface, ft)	Pond Area (acres)	Maximum Allowable Discharge (cfs)
0	1.2	6.8
0.5	1.5	8.5
1	1.8	10
1.5	2.1	12
2	2.4	14
3	3.0	17 (usually most critical)

Therefore, use a single 45° V-notch weir, or two 22-1/2° V-notch weirs.

• Select emergency spillway (mandatory) and additional flood control storage volume (if necessary) using NRCS TR-55 (SCS 1986) procedures.

• Figure A-1 is an example program check sheet for a WinDETPOND model evaluation, while the next section shows how this information is entered into a data file for analysis.

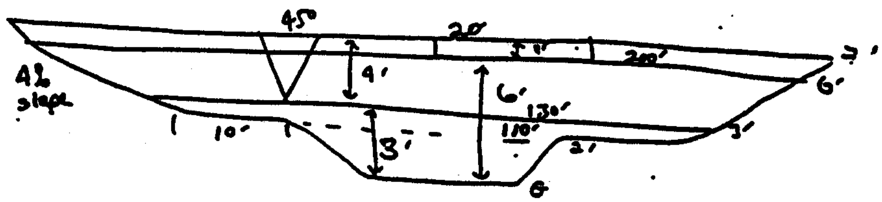
Figure A-1a. WinDETPOND model check sheet for example calculation.

150 acre MDR "90%" control

n=5 | directly connected impervious 45 acres (30%)
medium.cpz | pervious 75 acres (50%)
Brown76 + Brown Fld | impervious areas draining to pervious 30 acres (20%)
150 acres

Wet Detention Ponds W

Diagram Outlet Structures in space below



1. Initial stage elevation (ft) above datum: 3
2. Number of stage elevation increments required: (constant 0→7' by ½')
3. Total number of outlets (10 max): 2
4. Stage, pond area, seepage, and other outlet information:

Entry Number	Stage (ft)	Pond Area (Acres)	Natural Seepage (cfs)	Other Outflow (cfs)
0	0	* 0		
1	0.5	0.1		
2	1	0.13		
3	1.5	0.17		
4	2	0.2		
5	2.5	0.9		
6	3	1.2		
7	3.5	1.5		
8	4	1.8		
9	4.5	2.1		
10	5	2.4		
11	5.5	2.7		
12	6	3.0		
13	6.5	3.3		
14	7	3.6		
15				
16				
17				
18				
19				
20				
21				
22				

Figure A-1b. WinDETPOND model check sheet for example calculation.

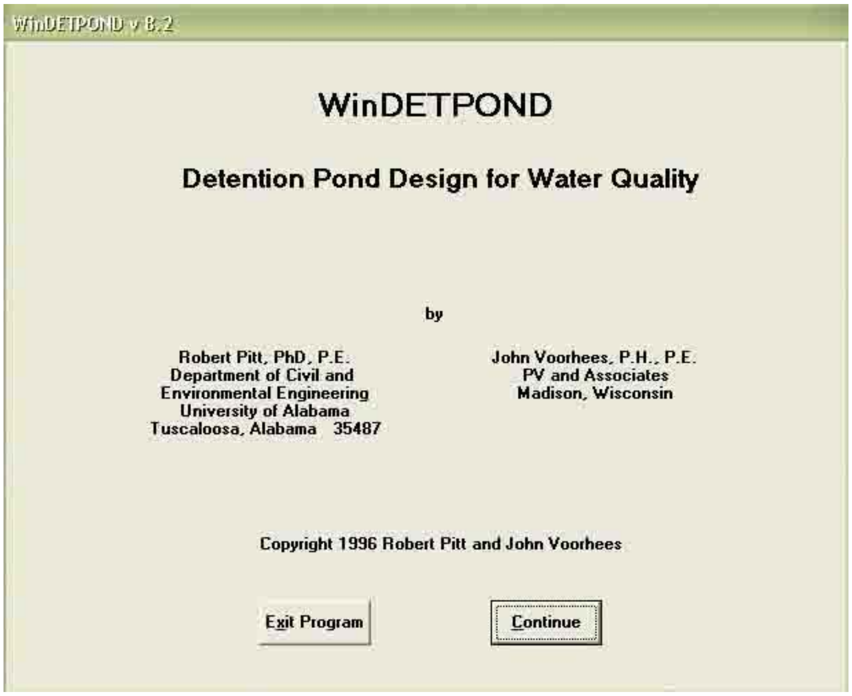
Wet Detention Ponds (continued)

5. Other outlet characteristics
   1. Rectangular Weir
      1. Weir length (ft): 20'
      2. Height from bottom of weir opening (invert) to top of weir: 1'
      3. Height from datum to bottom of weir opening (invert) (ft): 6'
   2. V-Notch Weir Characteristics:
      A) Weir angle:
         1. 22.5 degrees
         2. 30 degrees
         3. 45 degrees
         4. 60 degrees
         5. 90 degrees
         6. 120 degrees
      B) Height from bottom of weir opening (invert) to top of weir: 4'
      C) Height from datum to bottom of weir opening (invert) (ft): 3'
   3. Orifice characteristics:
      1. Orifice diameter (ft): ______
      2. Invert elevation above datum (ft): ______
   4. Seepage Basin characteristics:
      1. Infiltration rate (inches/hr): ______
      2. Width of device (ft): ______
      3. Length of device (ft): ______
      4. Invert elevation of seepage basin inlet above datum (ft): ______
   5. Monthly Evaporation Rate

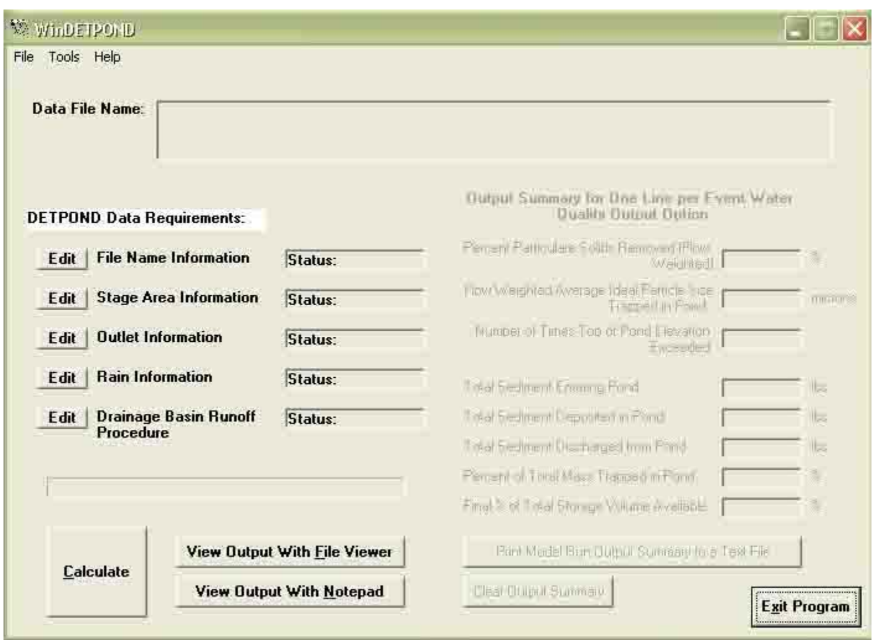
Month Number	Month	Evaporation (in/day)
1	January	
2	February	
3	March	
4	April	
5	May	
6	June	
7	July	
8	August	
9	September	
10	October	
11	November	
12	December	

Steps in Entering Data for Evaluation in WinDETPOND

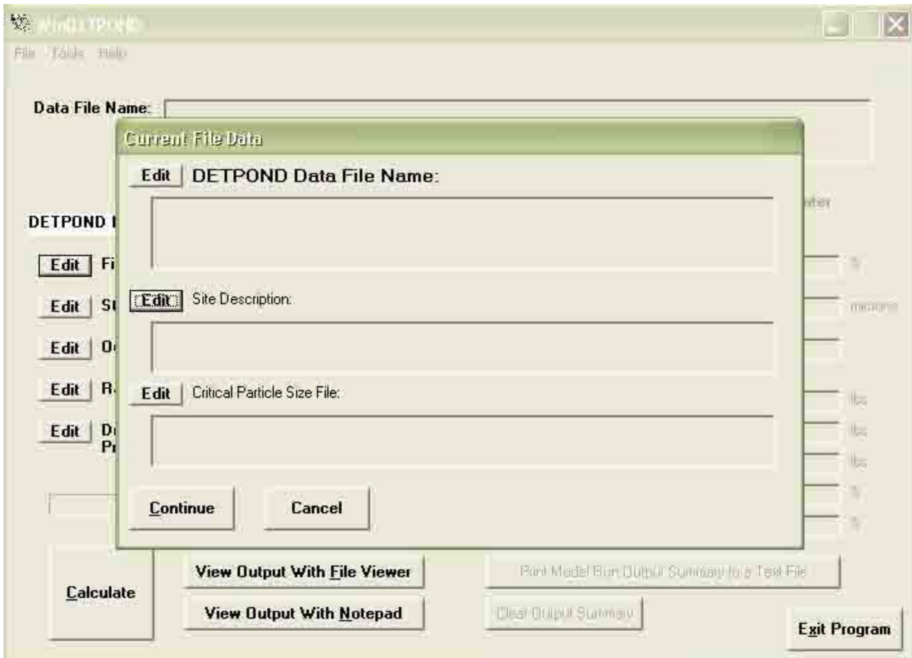
Enter the main WinDETPOND program by double-clicking on the WinDetpond.exe file located in the directory where the program was installed, or select the file from the "start, programs, WinDETPOND" list. The following window will open:

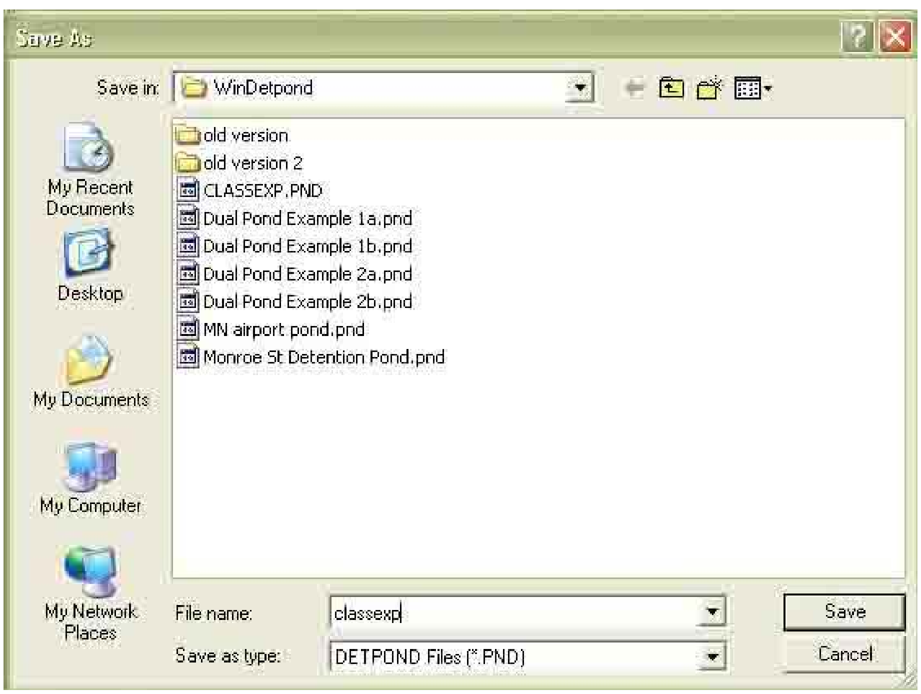


Select the "continue" button to open the following window:

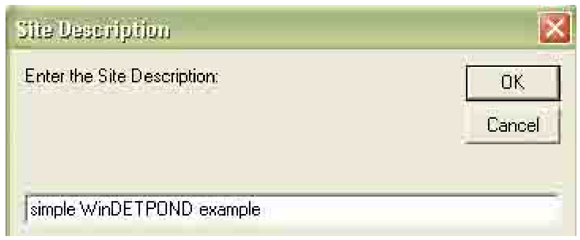


Notice that the status for each of the five main categories are listed as “incomplete.” The next steps in creating the file include entering this data. The first step for this window is to select the file name “edit” box and entering a file name, site description, and selecting the critical particle size file, as shown below:

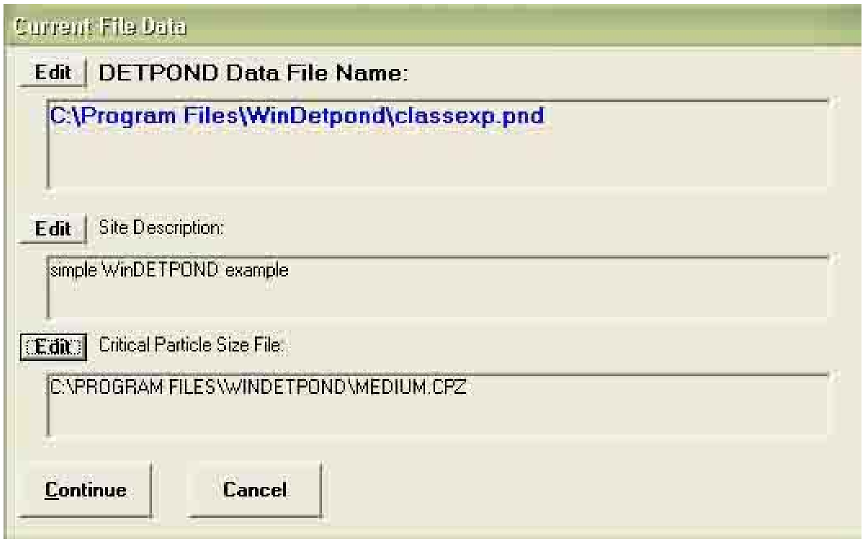




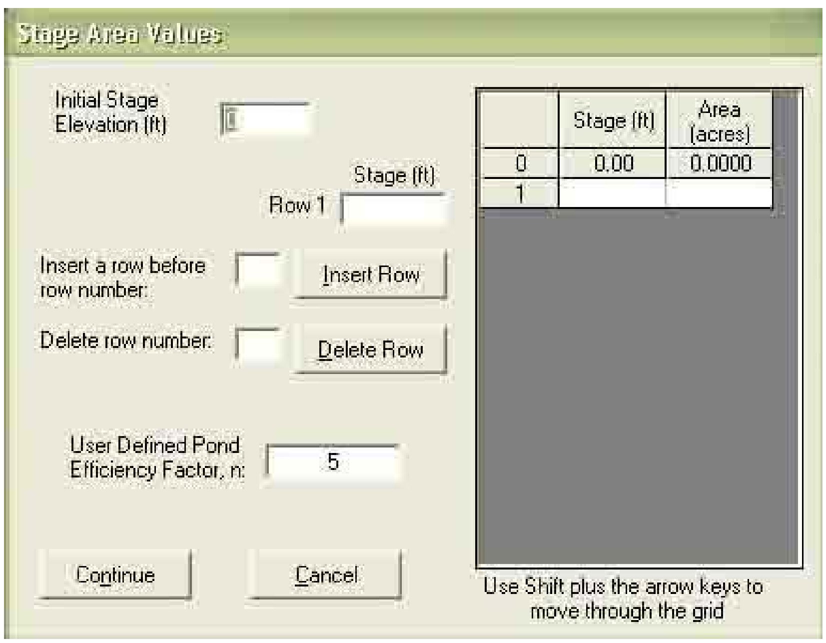
After the file name is typed in, click on the save button, after ensuring that the correct directory is listed. The next step under "file name information" is to enter a site description. Any short statement can be entered that will enable tracking the files or the site test conditions. The last part of this element is selecting the particle size file, as shown below:



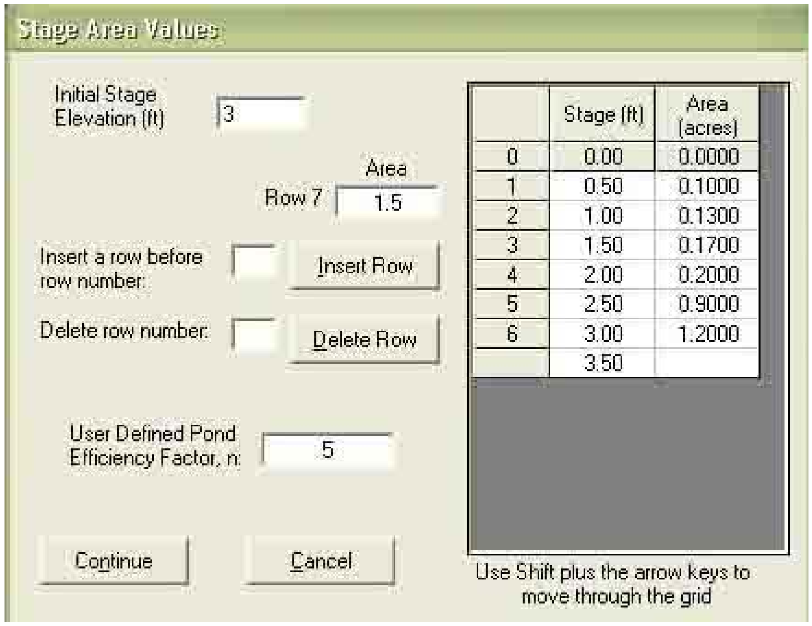
All available particle size files are listed. If the desired file is not listed, check the directory to ensure that the correct directory is shown. When the desired file is selected, click "OK."



The next major category of information is the stage-area values. When that "edit" box is selected, the following window is displayed:



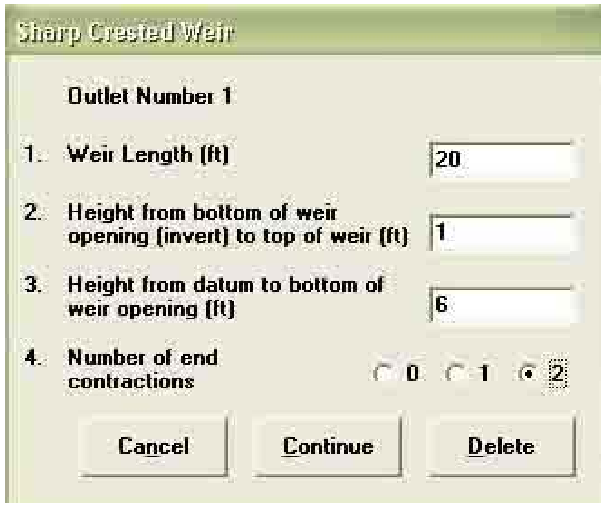
The first information to be entered is the initial stage elevation. This is the water depth in the pond at the beginning of the study period. It is generally the normal water elevation (above the pond bottom datum). However, it can be different reflecting actual conditions (such as being lower than the lowest invert because of evaporation that may have occurred during an extended dry period, or higher because the pond has not completely drained since the preceding rain). When that number is entered, the program automatically starts requesting stage and surface area data. The bottom-most stage (at depth zero) is already entered (required to have a surface area of zero acres). When all of the stage-area data is entered, select continue, or change the user defined pond efficiency factor first. The sequence is displayed in the following window:



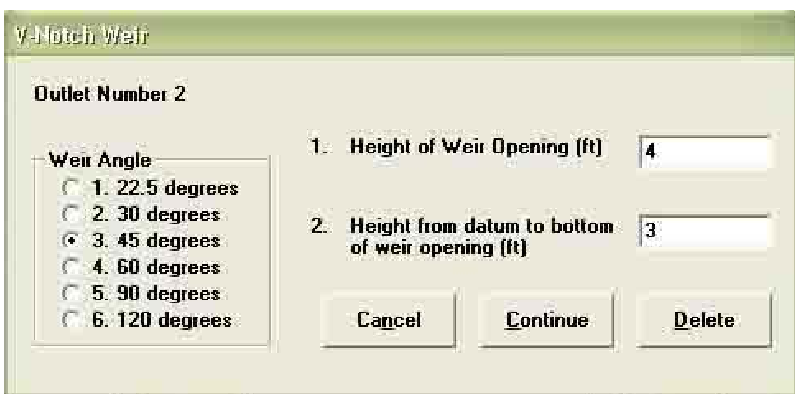
The “User Defined Pond Efficiency Factor, n” is given as 5, but can be changed by over-typing. This is the n factor used in the Hazen equation and is equivalent to the number of pond cells. Large numbers imply very little short-circuiting, while small numbers imply that substantial numbers of large particles may be leaving the pond.

The next major data requirement group is the outlet information. Select “edit” to bring up the following window:

When the sharp-crested weir is selected, the following window is brought up to enable the user to describe the weir dimensions and location:

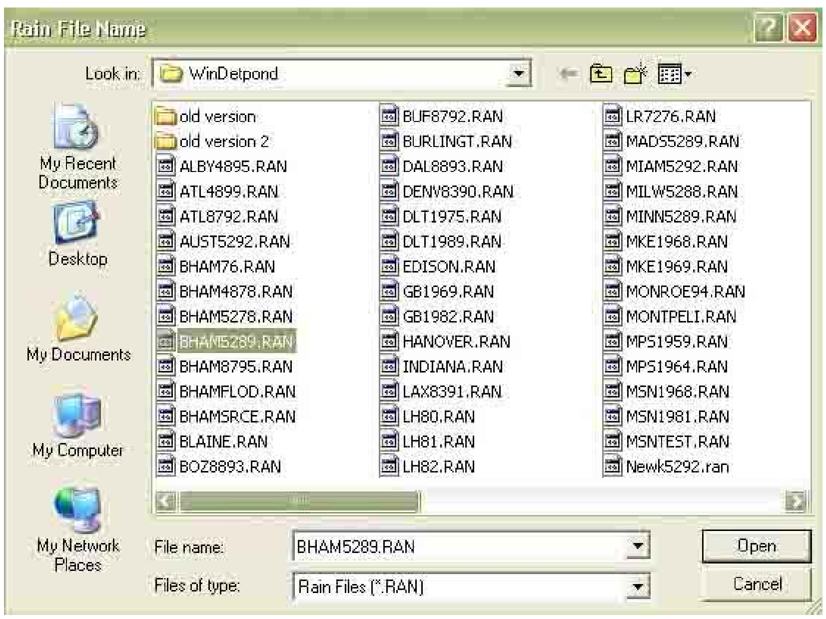


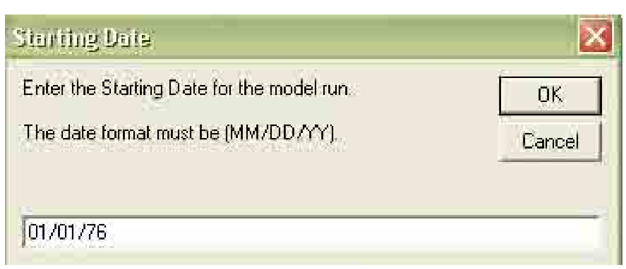
The user needs to refer to the diagram (on Figure A-1) to ensure that the weir heights are correct. The program also checks to make sure that the sum of the "height of bottom of weir opening to top of weir" plus the "height from datum to bottom of weir opening" adds up to equal the total depth of the pond entered previously. After entering the data and clicking on "continue", the user selects the V-notch weir for this example, bringing up the following window:



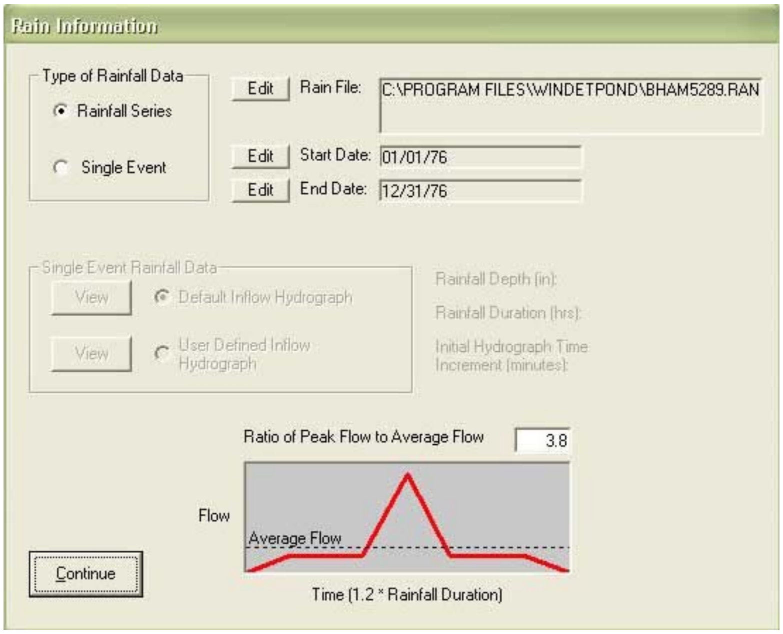
The user selects the v-notch weir angle and the height data, and then clicks "continue."

The next data requirement set relates to the rain file. A rainfall series is selected from the available list, and the starting and ending dates contained in the file are automatically listed. If these dates are not correct, they can be edited by selecting the "edit" button near each date, as shown in the following window, and typing in the desired dates:

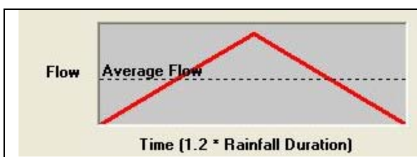
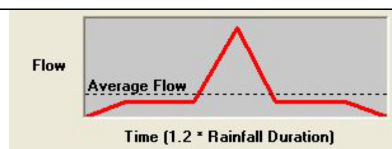
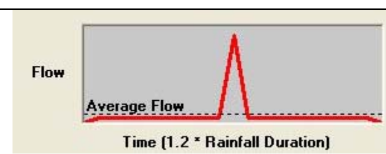




The ratio of peak flow to average flow for the inflow hydrograph is also changed, if desired. A plot of the hydrograph is then shown:

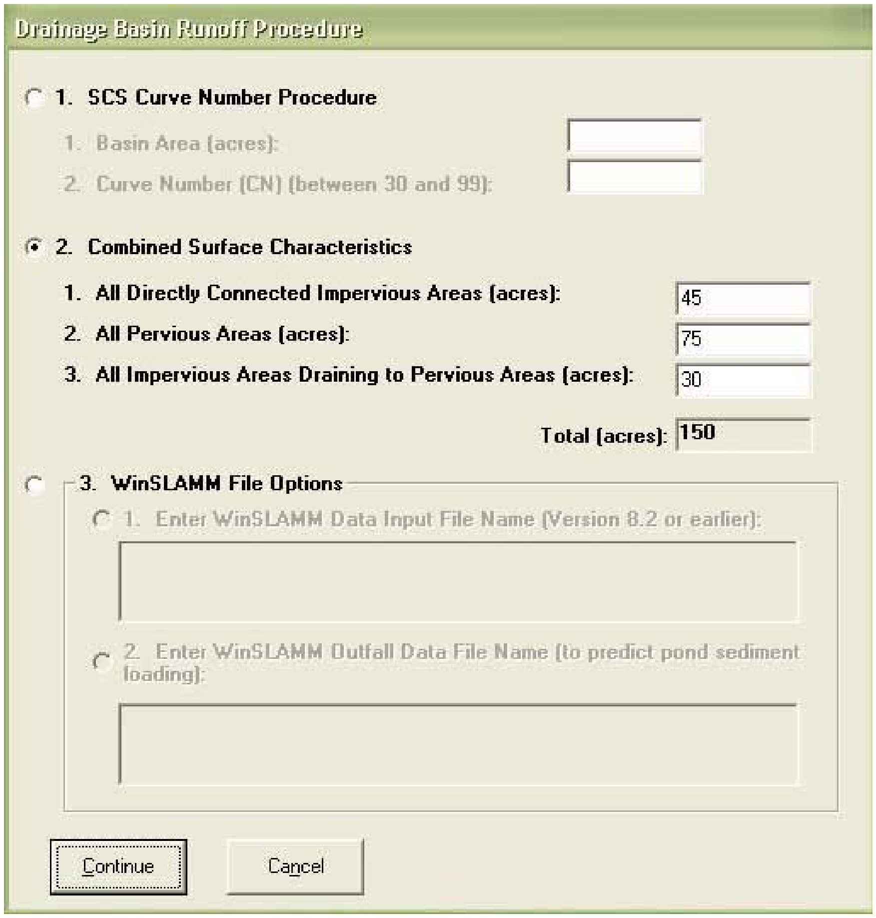


The ratio of the peak to average flows for the hydrographs for each event is suggested to be 3.8, a typical value based on monitoring. A simple triangular hydrograph corresponds to a ratio of 2.0 and may be representative of large areas during relatively small rains. For small source areas and for moderate to larger rains, higher values than this ratio are appropriate. The following are several plots representing different ratios of peak to average flows. In all cases, the same runoff volume calculated for the contributing area is used, but the flow rates are distributed according to the hydrograph shape.

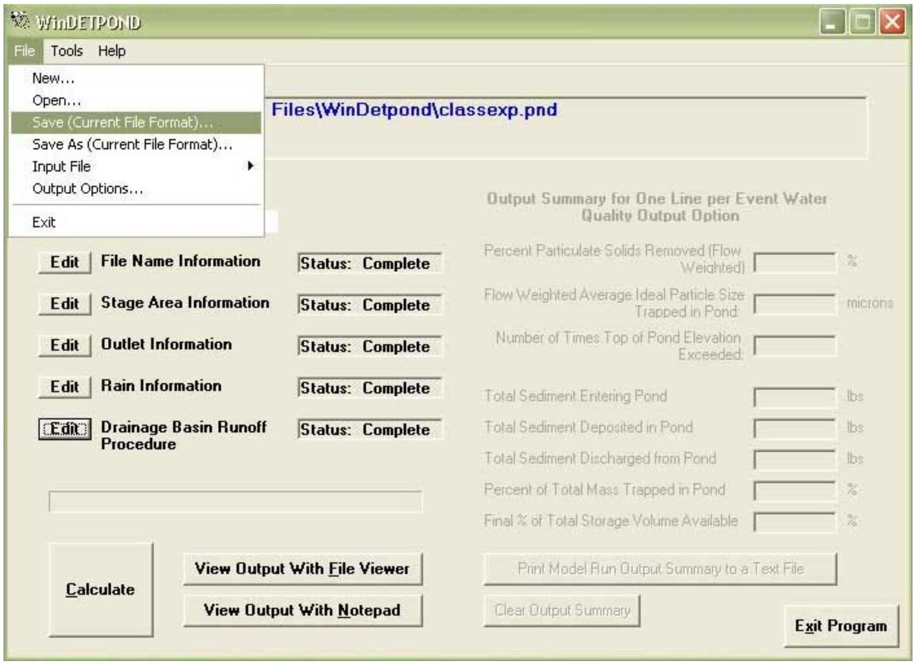
 Ratio = 2.0	 Ratio = 3.8	 Ratio = 10

If a user-defined hydrograph is to be evaluated (such as for entering a single design storm calculated using TR-55, for example, or to enter actual observed inflow rates), then the “single event” type of rainfall data is selected and the program prompts for that information.

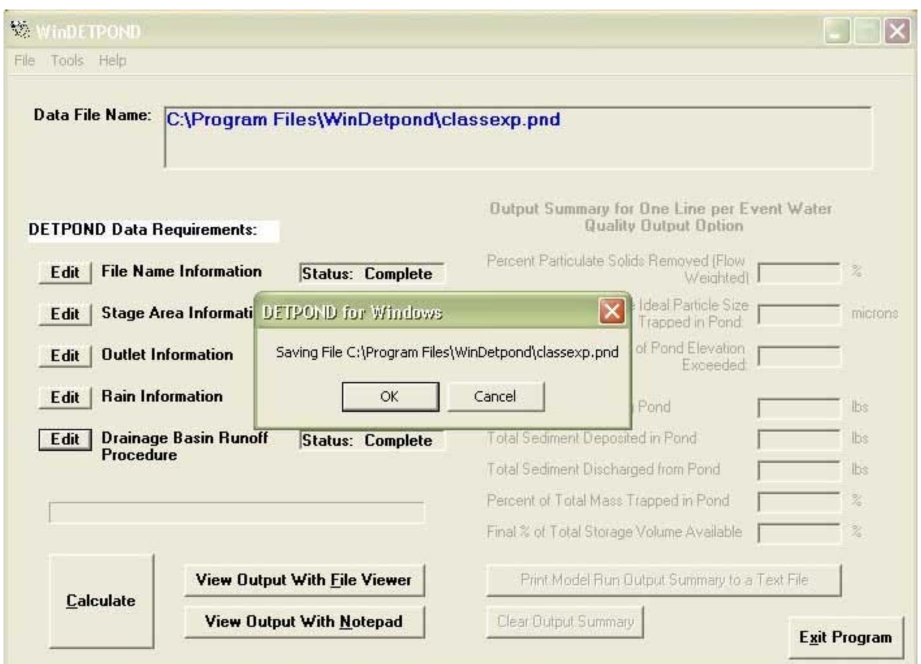
The last series of data requirements is the drainage basin information, as shown in the following window:



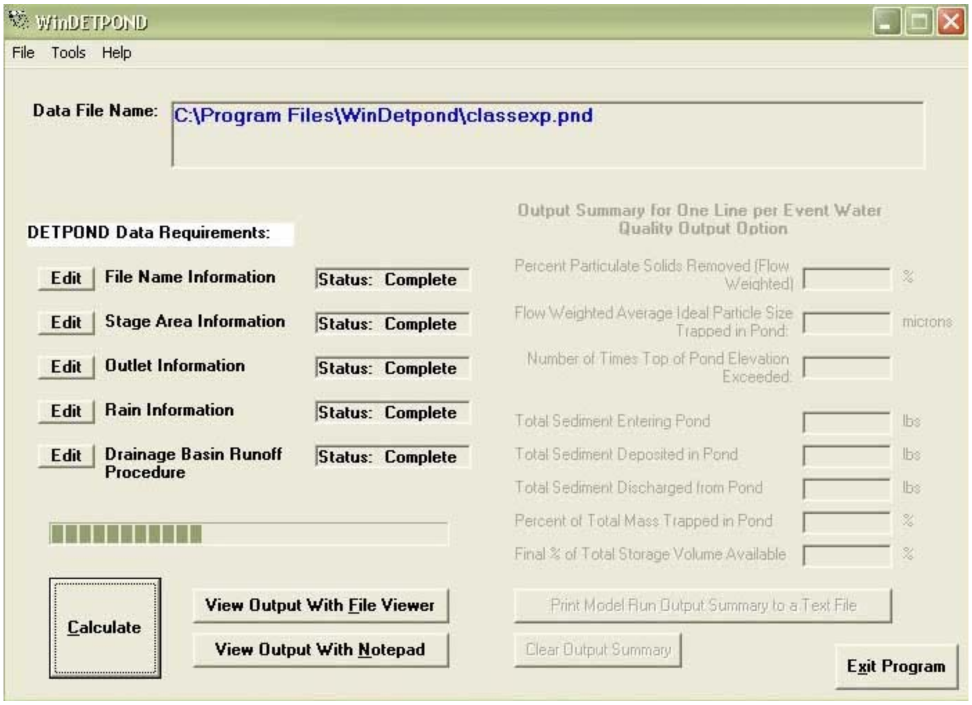
In our example, the "combined surface characteristics" is selected, which uses the correct runoff characteristics associated with small and intermediate-sized events. The area associated with each surface category is entered, and then the "continue" button is clicked. The "SCS Curve Number Procedure" simply uses a constant curve number for each event, but still uses the basic triangular hydrograph (and not the TR-55 tabular hydrograph, which is not accurate for these smaller rains). The SLAMM data file option allows more resolution in describing the surface areas, and is especially helpful if the same file is being used for a SLAMM analysis, but the greater detail in WinDETPOND is desired for an outfall wet detention pond. When these data are entered, the main screen shows that the status of each data requirement category is "complete." The file needs to be saved again, as shown in the following window:



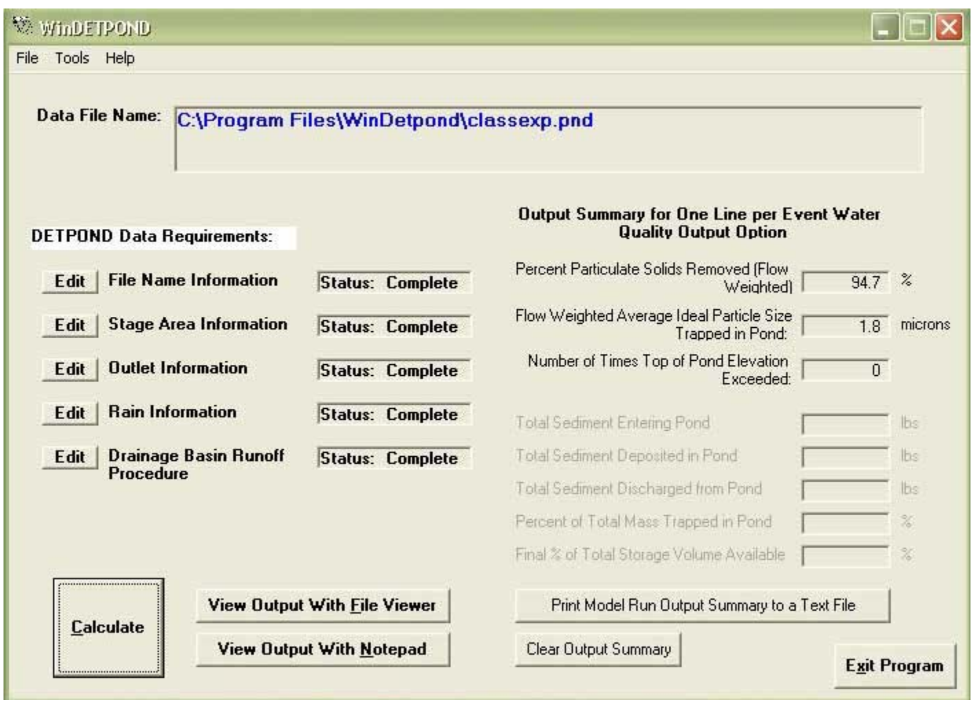
The file name is verified by clicking on "OK" in the following dialog box:



Finally, the large "calculate" button is clicked and after a few seconds, the program is completed.

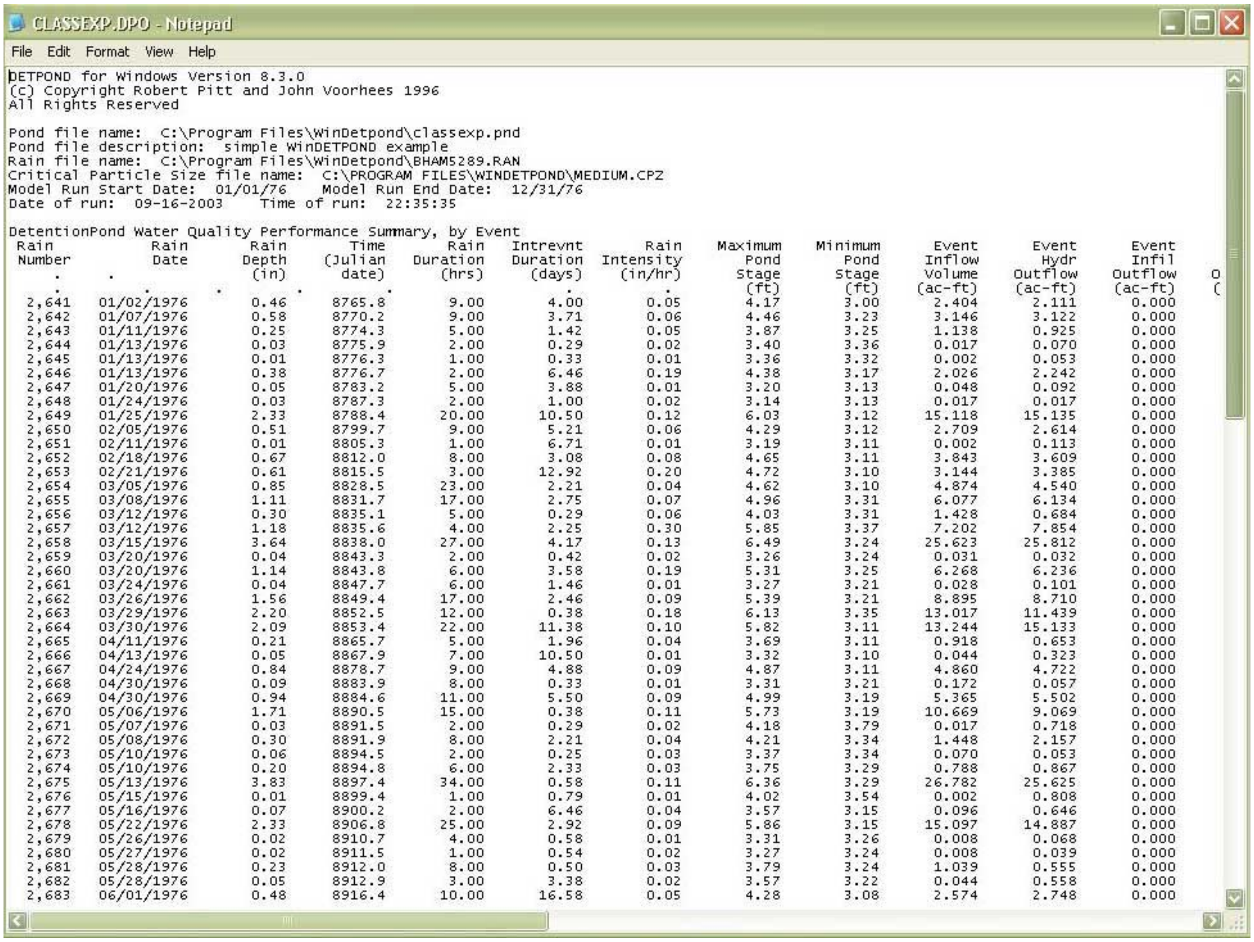


The following screen displays the basic pond performance information:



The file viewer is then clicked and the output file is selected.

The following window then appears:



CLASSEXP.DPO - Notepad

File Edit Format View Help

```
DETPOND for Windows Version 8.3.0
(c) Copyright Robert Pitt and John Voorhees 1996
All Rights Reserved

Pond file name:  C:\Program Files\WinDetpond\classexp.pnd
Pond file description:  simple WinDETPOND example
Rain file name:  C:\Program Files\WinDetpond\BHAM5289.RAN
Critical Particle Size file name:  C:\PROGRAM FILES\WINDETPOND\MEDIUM.CPZ
Model Run Start Date:  01/01/76    Model Run End Date:  12/31/76
Date of run:  09-16-2003    Time of run:  22:35:35

DetentionPond Water Quality Performance Summary, by Event
```

Rain Number	Rain Date	Rain Depth (in)	Time (Julian date)	Rain Duration (hrs)	Intrevnt Duration (days)	Rain Intensity (in/hr)	Maximum Pond Stage (ft)	Minimum Pond Stage (ft)	Event Inflow Volume (ac-ft)	Event Hydr Outflow (ac-ft)	Event Infil Outflow (ac-ft)	O (
2,641	01/02/1976	0.46	8765.8	9.00	4.00	0.05	4.17	3.00	2.404	2.111	0.000	
2,642	01/07/1976	0.58	8770.2	9.00	3.71	0.06	4.46	3.23	3.146	3.122	0.000	
2,643	01/11/1976	0.25	8774.3	5.00	1.42	0.05	3.87	3.25	1.138	0.925	0.000	
2,644	01/13/1976	0.03	8775.9	2.00	0.29	0.02	3.40	3.36	0.017	0.070	0.000	
2,645	01/13/1976	0.01	8776.3	1.00	0.33	0.01	3.36	3.32	0.002	0.053	0.000	
2,646	01/13/1976	0.38	8776.7	2.00	6.46	0.19	4.38	3.17	2.026	2.242	0.000	
2,647	01/20/1976	0.05	8783.2	5.00	3.88	0.01	3.20	3.13	0.048	0.092	0.000	
2,648	01/24/1976	0.03	8787.3	2.00	1.00	0.02	3.14	3.13	0.017	0.017	0.000	
2,649	01/25/1976	2.33	8788.4	20.00	10.50	0.12	6.03	3.12	15.118	15.135	0.000	
2,650	02/05/1976	0.51	8799.7	9.00	5.21	0.06	4.29	3.12	2.709	2.614	0.000	
2,651	02/11/1976	0.01	8805.3	1.00	6.71	0.01	3.19	3.11	0.002	0.113	0.000	
2,652	02/18/1976	0.67	8812.0	8.00	3.08	0.08	4.65	3.11	3.843	3.609	0.000	
2,653	02/21/1976	0.61	8815.5	3.00	12.92	0.20	4.72	3.10	3.144	3.385	0.000	
2,654	03/05/1976	0.85	8828.5	23.00	2.21	0.04	4.62	3.10	4.874	4.540	0.000	
2,655	03/08/1976	1.11	8831.7	17.00	2.75	0.07	4.96	3.31	6.077	6.134	0.000	
2,656	03/12/1976	0.30	8835.1	5.00	0.29	0.06	4.03	3.31	1.428	0.684	0.000	
2,657	03/12/1976	1.18	8835.6	4.00	2.25	0.30	5.85	3.37	7.202	7.854	0.000	
2,658	03/15/1976	3.64	8838.0	27.00	4.17	0.13	6.49	3.24	25.623	25.812	0.000	
2,659	03/20/1976	0.04	8843.3	2.00	0.42	0.02	3.26	3.24	0.031	0.032	0.000	
2,660	03/20/1976	1.14	8843.8	6.00	3.58	0.19	5.31	3.25	6.268	6.236	0.000	
2,661	03/24/1976	0.04	8847.7	6.00	1.46	0.01	3.27	3.21	0.028	0.101	0.000	
2,662	03/26/1976	1.56	8849.4	17.00	2.46	0.09	5.39	3.21	8.895	8.710	0.000	
2,663	03/29/1976	2.20	8852.5	12.00	0.38	0.18	6.13	3.35	13.017	11.439	0.000	
2,664	03/30/1976	2.09	8853.4	22.00	11.38	0.10	5.82	3.11	13.244	15.133	0.000	
2,665	04/11/1976	0.21	8865.7	5.00	1.96	0.04	3.69	3.11	0.918	0.653	0.000	
2,666	04/13/1976	0.05	8867.9	7.00	10.50	0.01	3.32	3.10	0.044	0.323	0.000	
2,667	04/24/1976	0.84	8878.7	9.00	4.88	0.09	4.87	3.11	4.860	4.722	0.000	
2,668	04/30/1976	0.09	8883.9	8.00	0.33	0.01	3.31	3.21	0.172	0.057	0.000	
2,669	04/30/1976	0.94	8884.6	11.00	5.50	0.09	4.99	3.19	5.365	5.502	0.000	
2,670	05/06/1976	1.71	8890.5	15.00	0.38	0.11	5.73	3.19	10.669	9.069	0.000	
2,671	05/07/1976	0.03	8891.5	2.00	0.29	0.02	4.18	3.79	0.017	0.718	0.000	
2,672	05/08/1976	0.30	8891.9	8.00	2.21	0.04	4.21	3.34	1.448	2.157	0.000	
2,673	05/10/1976	0.06	8894.5	2.00	0.25	0.03	3.37	3.34	0.070	0.053	0.000	
2,674	05/10/1976	0.20	8894.8	6.00	2.33	0.03	3.75	3.29	0.788	0.867	0.000	
2,675	05/13/1976	3.83	8897.4	34.00	0.58	0.11	6.36	3.29	26.782	25.625	0.000	
2,676	05/15/1976	0.01	8899.4	1.00	0.79	0.01	4.02	3.54	0.002	0.808	0.000	
2,677	05/16/1976	0.07	8900.2	2.00	6.46	0.04	3.57	3.15	0.096	0.646	0.000	
2,678	05/22/1976	2.33	8906.8	25.00	2.92	0.09	5.86	3.15	15.097	14.887	0.000	
2,679	05/26/1976	0.02	8910.7	4.00	0.58	0.01	3.31	3.26	0.008	0.068	0.000	
2,680	05/27/1976	0.02	8911.5	1.00	0.54	0.02	3.27	3.24	0.008	0.039	0.000	
2,681	05/28/1976	0.23	8912.0	8.00	0.50	0.03	3.79	3.24	1.039	0.555	0.000	
2,682	05/28/1976	0.05	8912.9	3.00	3.38	0.02	3.57	3.22	0.044	0.558	0.000	
2,683	06/01/1976	0.48	8916.4	10.00	16.58	0.05	4.28	3.08	2.574	2.748	0.000	

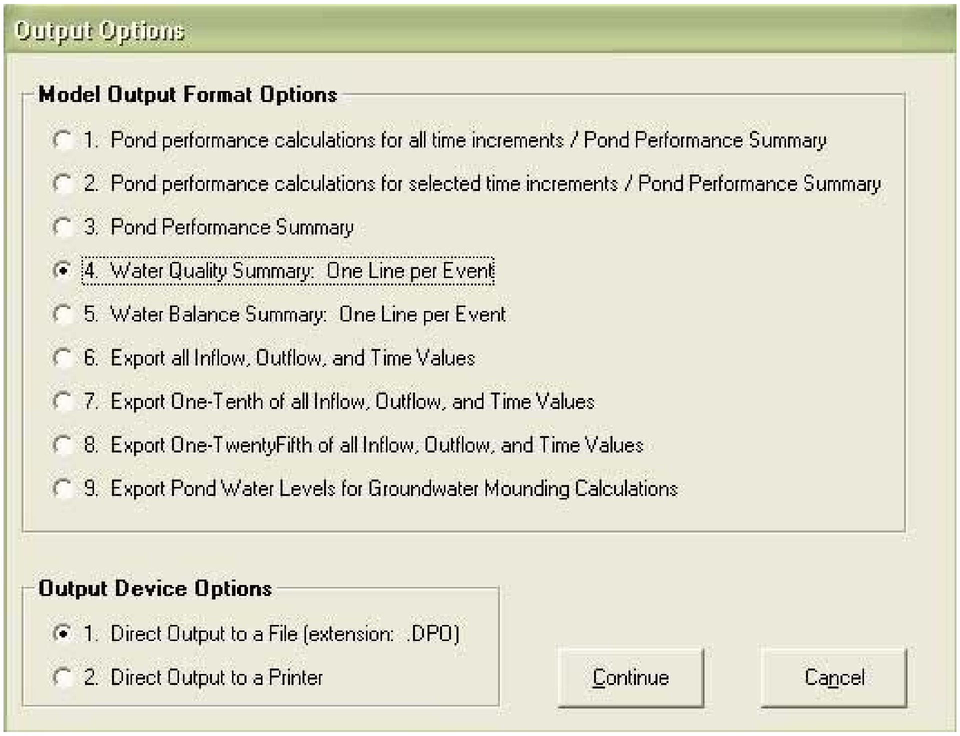
The screen can be scrolled to see other parts of the output calculations. At the bottom of the screen, the following performance summary is displayed:

CLASSEXP.DPO - Notepad

File Edit Format View Help

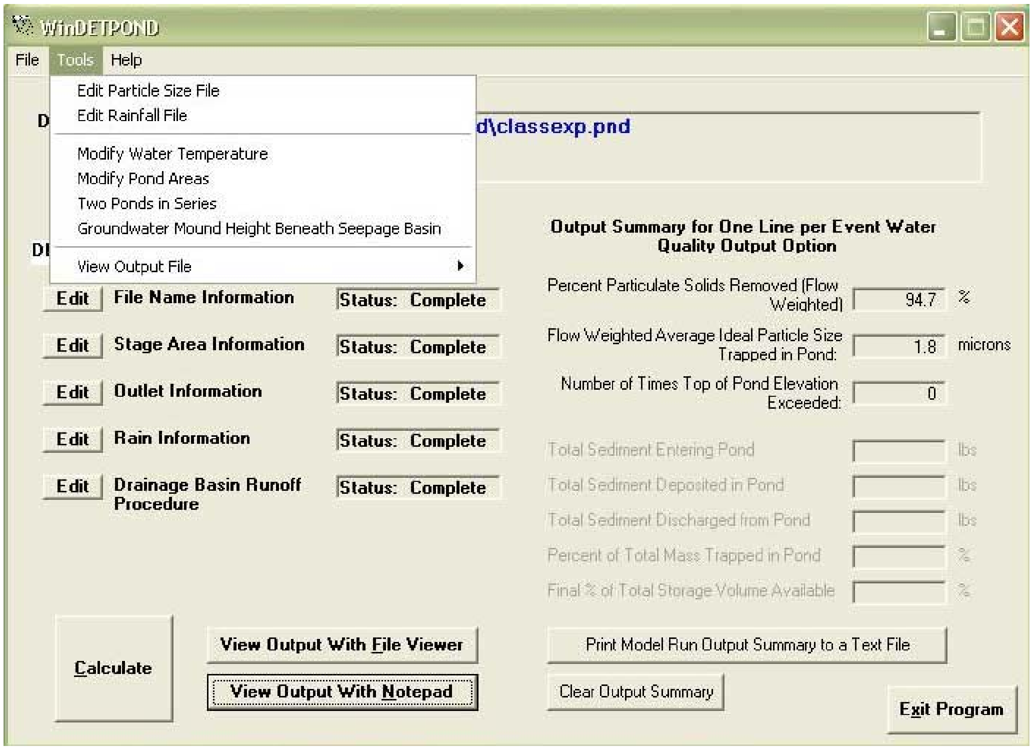
```
  2,709   09/04/1976       0.05     9011.2      7.00       0.58       0.01       3.64       3.43      0.044      0.365      0.000
  2,710   09/05/1976       0.44     9012.0     14.00       1.00       0.03       4.21       3.43      2.284      2.131      0.000
  2,711   09/06/1976       0.04     9013.6      1.00       0.75       0.04       3.54       3.39      0.032      0.240      0.000
  2,712   09/07/1976       0.11     9014.4      2.00       2.42       0.06       3.56       3.26      0.290      0.478      0.000
  2,713   09/10/1976       0.01     9016.9      1.00      11.00       0.01       3.26       3.09      0.002      0.220      0.000
  2,714   09/21/1976       0.06     9028.0      2.00       5.38       0.03       3.15       3.09      0.070      0.062      0.000
  2,715   09/26/1976       0.12     9033.4      2.00       0.67       0.06       3.36       3.10      0.360      0.092      0.000
  2,716   09/27/1976       0.03     9034.2      1.00       1.54       0.03       3.31       3.23      0.018      0.121      0.000
  2,717   09/28/1976       2.39     9035.8     16.00       6.67       0.15       6.20       3.17     15.133     15.212      0.000
  2,718   10/06/1976       0.04     9043.1      2.00       0.38       0.02       3.19       3.17      0.031      0.014      0.000
  2,719   10/06/1976       0.01     9043.5      1.00       1.46       0.01       3.18       3.15      0.002      0.037      0.000
  2,720   10/08/1976       0.01     9045.0      1.00       0.50       0.01       3.16       3.15      0.002      0.010      0.000
  2,721   10/08/1976       0.15     9045.6      5.00       7.88       0.03       3.50       3.13      0.529      0.551      0.000
  2,722   10/16/1976       0.05     9053.7      6.00       3.13       0.01       3.16       3.12      0.044      0.054      0.000
  2,723   10/20/1976       0.15     9057.0      2.00       4.88       0.08       3.48       3.12      0.514      0.450      0.000
  2,724   10/25/1976       0.64     9062.0     14.00       4.38       0.05       4.45       3.17      3.486      3.423      0.000
  2,725   10/30/1976       0.54     9067.0     11.00      11.96       0.05       4.33       3.11      2.825      2.972      0.000
  2,726   11/11/1976       0.23     9079.4     13.00       2.21       0.02       3.68       3.11      1.046      0.798      0.000
  2,727   11/14/1976       0.91     9082.1     19.00       5.33       0.05       4.77       3.19      5.200      5.337      0.000
  2,728   11/20/1976       0.22     9088.3      7.00       5.71       0.03       3.70       3.17      0.888      0.920      0.000
  2,729   11/26/1976       0.12     9094.3      9.00       0.79       0.01       3.39       3.17      0.356      0.156      0.000
  2,730   11/27/1976       0.02     9095.4      2.00       0.46       0.01       3.32       3.28      0.008      0.055      0.000
  2,731   11/28/1976       0.73     9096.0     22.00       7.50       0.03       4.50       3.15      4.057      4.235      0.000
  2,732   12/06/1976       0.59     9104.4     19.00       3.92       0.03       4.30       3.15      3.167      3.056      0.000
  2,733   12/11/1976       1.09     9109.1     38.00       2.17       0.03       4.66       3.23      6.309      6.144      0.000
  2,734   12/14/1976       0.25     9112.8      5.00       4.88       0.05       3.93       3.19      1.138      1.356      0.000
  2,735   12/20/1976       0.87     9117.9      9.00       4.92       0.10       4.94       3.20      5.048      5.033      0.000
  2,736   12/25/1976       1.35     9123.2     13.00       4.71       0.10       5.46       3.21      8.352      8.341      0.000
  2,737   12/30/1976       0.01     9128.5      1.00       0.29       0.01       3.21       3.21      0.002      0.014      0.000
  2,738   12/30/1976       0.19     9128.8      7.00       2.75       0.03       3.63       3.21      0.725      0.660      0.000

Detention Pond Water Quality Performance Summary, by Event
    .          .           Rain       Time       Rain   Intrevnt       Rain    Maximum    Minimum      Event      Event      Event
      .        .          Depth   (Julian   Duration   Duration  Intensity       Pond       Pond     Inflow       Hydr      Infil
     .    .                (in)      date)      (hrs)     (days)    (in/hr)      Stage      Stage     Volume    Outflow    Outflow    O
     .     .          .     .     .                          .          .       (ft)       (ft)    (ac-ft)    (ac-ft)    (ac-ft)    (
 Number_of_Events  .         98          .         98         98         98         98         98         98         98         98
 Total             .      55.15        n/a      742.0      335.1        n/a        n/a        n/a      318.2      317.9          0
 Equiv_Annual_Ttl  .      55.46        n/a      746.2      337.0        n/a        n/a        n/a      320.0      319.7          0
 Minimum           .   0.010000          0      1.000     0.2500   0.005000      3.099      3.003   0.001924   0.004586          0
 Maximum           .      3.830          0      38.00      16.58     0.5400      6.487      3.834      26.78      25.81          0
 Average          .      0.5628          0      7.571      3.419    0.07694      4.190      3.232      3.247      3.244          0
 Flow-weighted_Avg .        n/a        n/a        n/a        n/a        n/a        n/a        n/a        n/a        n/a        n/a
 Median            .     0.2400          0      5.000      2.375    0.04795      3.964      3.193      1.043     0.9006          0
 Stnd_Deviation    .     0.7599          0      7.713      3.402    0.09113     0.9176     0.1536      5.062      4.980          0
 COV               .      1.350          0      1.019     0.9950      1.184     0.2190    0.04754      1.559      1.535          0

   First Rain Date:  01/02/1976
   Last Rain Date:  12/30/1976
   Total Time Period (yrs):   0.9944064

 Number of Times Water Surface Elevation Exceeded Sharp Crested Weir Elevation:       6
 Number of Times Water Surface Elevation Exceeded V-Notch Weir Elevation:             98
 Number of Times Water Surface Elevation Exceeded Broad Crested Weir Elevation:
 Number of Times Water Surface Elevation Exceeded Vertical Stand Pipe Weir Elevation:
 Number of Times Water Surface Elevation Exceeded Top of Pond Elevation:
 Ideal column will reflect reduced performance due to scour from shallow ponds.  See Scour help topic.
```

This example shows the default file output format, or one line per event. The “file, output” drop down menu offers several other options:

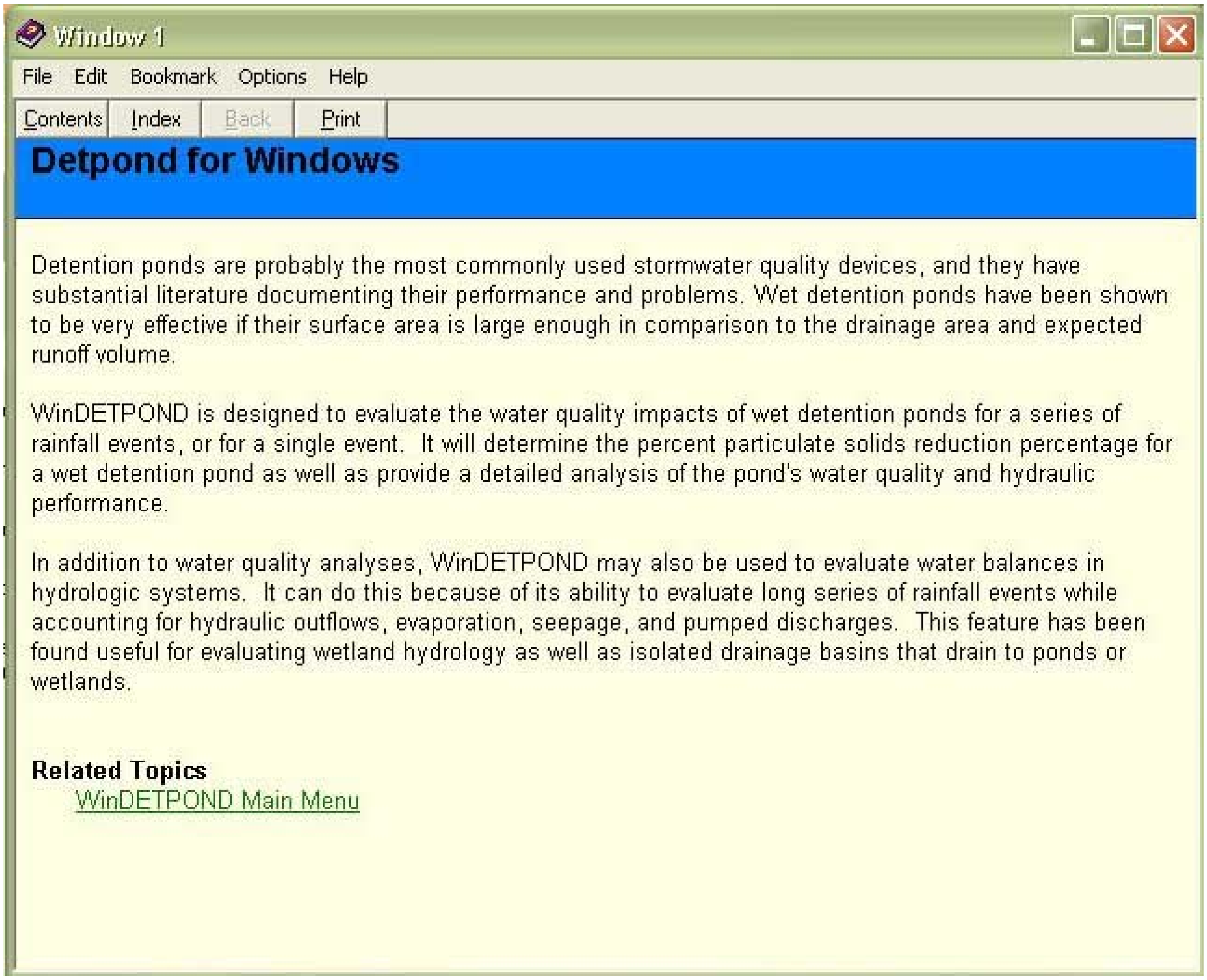


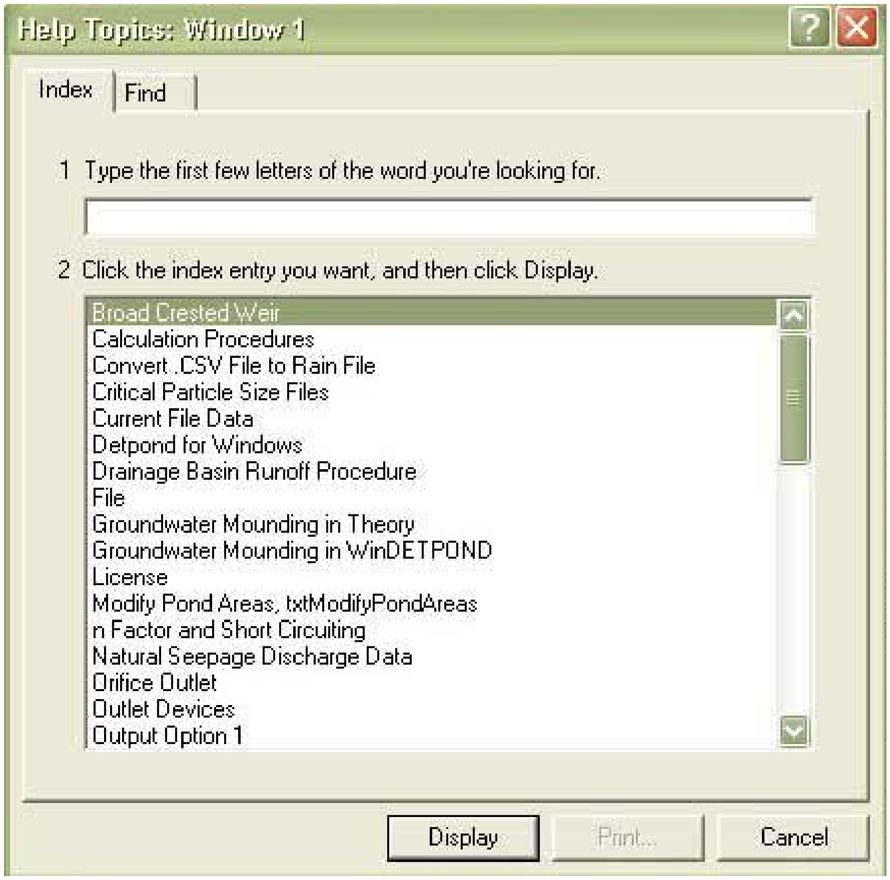
The file is automatically saved as a comma separated value (CSV) file that can be directly opened with a spreadsheet program. In addition, the input file can also be saved to a file that can be opened in a spreadsheet for examination. The input file for this example is shown as Table A-1, while the output file (after adding some column statistics in Excel) is shown in Table A-2. It is also possible to plot these data from within the spreadsheet, or in any graphing program.

The “Tools” drop down menu also displays several program options:

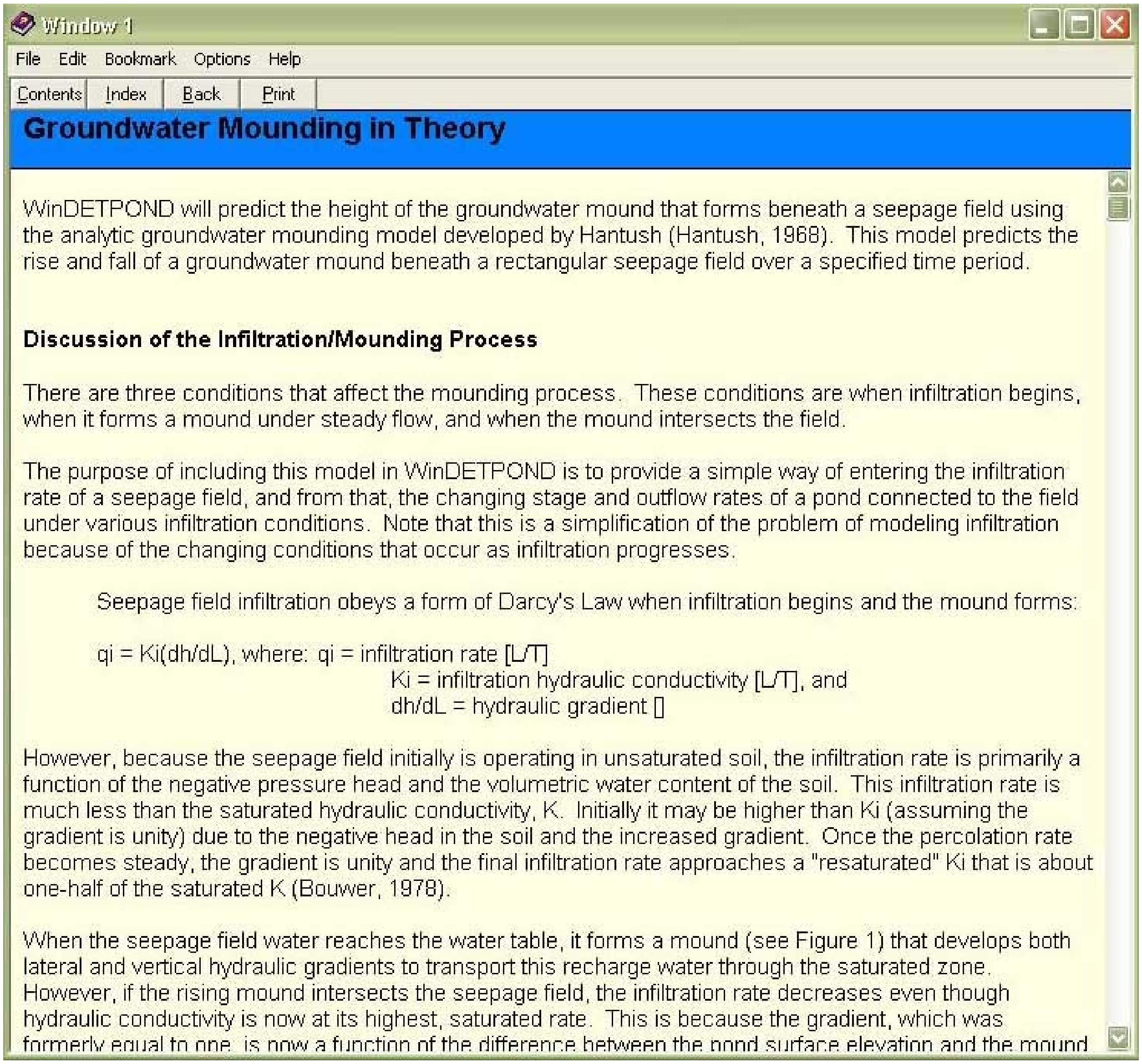


It is possible to modify the water temperature for each month to simulate varying conditions. These changes can cause large changes in pond performance under severe snowmelt conditions, for example. Other pond tools include being able to automatically modify the pond area with a multiplier. As an example, the pond in the above example is likely larger than necessary, as the predicted suspended solids removal rate is almost 95%. This tool makes it easy to change the size of the pond with a multiplier for further analyses. It is also possible to analyze two ponds in series. This tool collects the correct file data for the two ponds (which must have the same start and end dates and rain files). In addition, another DETPOND option allows groundwater mounding to be calculated beneath a percolation pond. This tool menu also allows easy review and editing of the necessary rain and particle size files. The Help drop down many contains detailed instructions for these, and all other model options. It is also possible to press the F1 function key anywhere in the model to receive additional help. The Help menu displays the following:


Window 1
File Edit Bookmark Options Help
Contents Index Back Print
Detpond for Windows
Detention ponds are probably the most commonly used stormwater quality devices, and they have substantial literature documenting their performance and problems. Wet detention ponds have been shown to be very effective if their surface area is large enough in comparison to the drainage area and expected runoff volume.
WinDETPOND is designed to evaluate the water quality impacts of wet detention ponds for a series of rainfall events, or for a single event. It will determine the percent particulate solids reduction percentage for a wet detention pond as well as provide a detailed analysis of the pond's water quality and hydraulic performance.
In addition to water quality analyses, WinDETPOND may also be used to evaluate water balances in hydrologic systems. It can do this because of its ability to evaluate long series of rainfall events while accounting for hydraulic outflows, evaporation, seepage, and pumped discharges. This feature has been found useful for evaluating wetland hydrology as well as isolated drainage basins that drain to ponds or wetlands.
Related Topics
WinDETPOND Main Menu



The following are example help screens for the groundwater mounding options, for example:



Window 1

File Edit Bookmark Options Help

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Groundwater Mounding in Theory

WinDETPOND will predict the height of the groundwater mound that forms beneath a seepage field using the analytic groundwater mounding model developed by Hantush (Hantush, 1968). This model predicts the rise and fall of a groundwater mound beneath a rectangular seepage field over a specified time period.

Discussion of the Infiltration/Mounding Process

There are three conditions that affect the mounding process. These conditions are when infiltration begins, when it forms a mound under steady flow, and when the mound intersects the field.

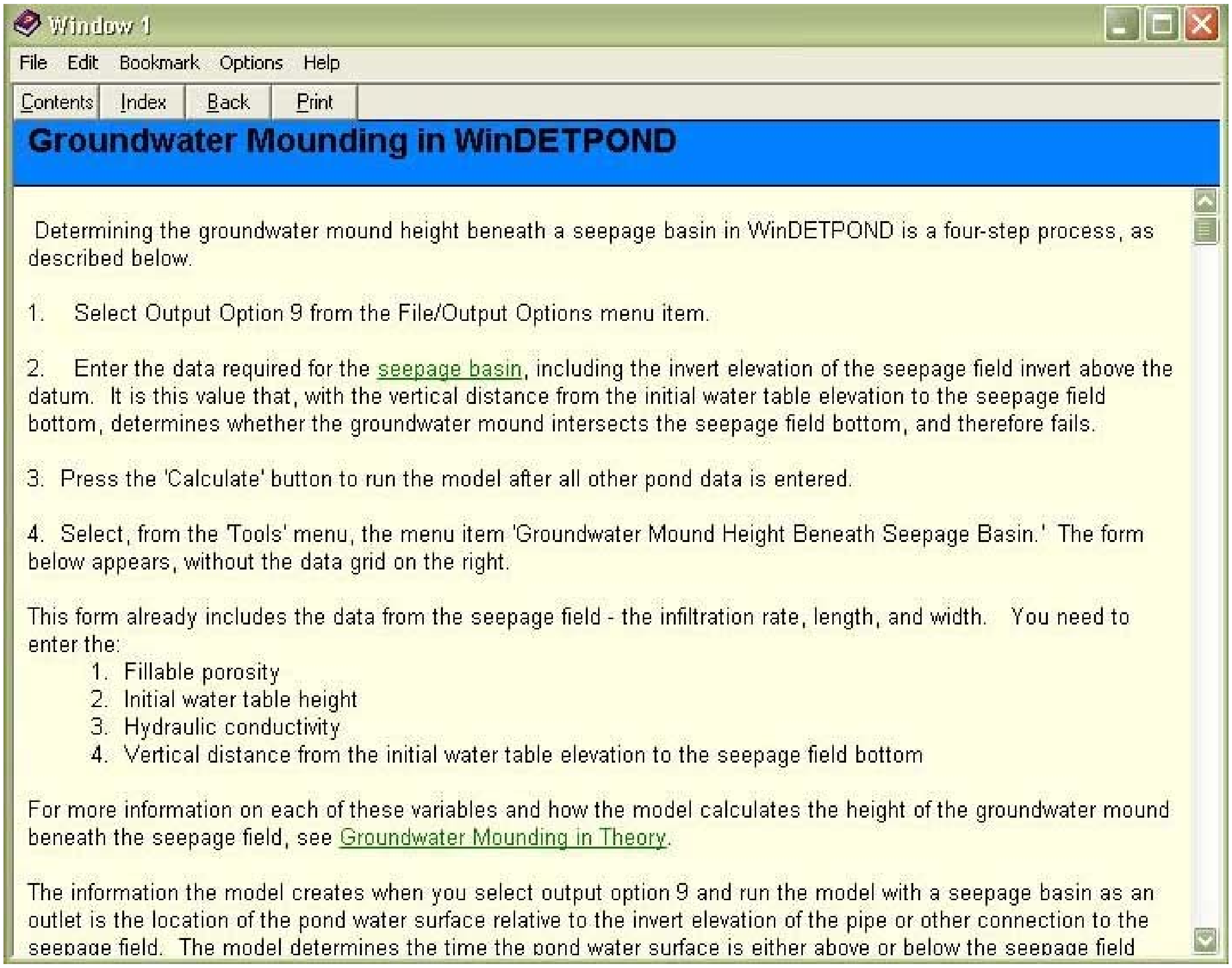
The purpose of including this model in WinDETPOND is to provide a simple way of entering the infiltration rate of a seepage field, and from that, the changing stage and outflow rates of a pond connected to the field under various infiltration conditions. Note that this is a simplification of the problem of modeling infiltration because of the changing conditions that occur as infiltration progresses.

Seepage field infiltration obeys a form of Darcy's Law when infiltration begins and the mound forms:

qi = Ki(dh/dL), where: qi = infiltration rate [L/T]
Ki = infiltration hydraulic conductivity [L/T], and
dh/dL = hydraulic gradient []

However, because the seepage field initially is operating in unsaturated soil, the infiltration rate is primarily a function of the negative pressure head and the volumetric water content of the soil. This infiltration rate is much less than the saturated hydraulic conductivity, K. Initially it may be higher than Ki (assuming the gradient is unity) due to the negative head in the soil and the increased gradient. Once the percolation rate becomes steady, the gradient is unity and the final infiltration rate approaches a "resaturated" Ki that is about one-half of the saturated K (Bouwer, 1978).

When the seepage field water reaches the water table, it forms a mound (see Figure 1) that develops both lateral and vertical hydraulic gradients to transport this recharge water through the saturated zone. However, if the rising mound intersects the seepage field, the infiltration rate decreases even though hydraulic conductivity is now at its highest, saturated rate. This is because the gradient, which was formerly equal to one, is now a function of the difference between the pond surface elevation and the mound



Window 1

File Edit Bookmark Options Help

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Groundwater Mounding in WinDETPOND

Determining the groundwater mound height beneath a seepage basin in WinDETPOND is a four-step process, as described below.

1. Select Output Option 9 from the File/Output Options menu item.

2. Enter the data required for the seepage basin, including the invert elevation of the seepage field invert above the datum. It is this value that, with the vertical distance from the initial water table elevation to the seepage field bottom, determines whether the groundwater mound intersects the seepage field bottom, and therefore fails.

3. Press the 'Calculate' button to run the model after all other pond data is entered.

4. Select, from the 'Tools' menu, the menu item 'Groundwater Mound Height Beneath Seepage Basin.' The form below appears, without the data grid on the right.

This form already includes the data from the seepage field - the infiltration rate, length, and width. You need to enter the:

1. Fillable porosity
2. Initial water table height
3. Hydraulic conductivity
4. Vertical distance from the initial water table elevation to the seepage field bottom

For more information on each of these variables and how the model calculates the height of the groundwater mound beneath the seepage field, see Groundwater Mounding in Theory.

The information the model creates when you select output option 9 and run the model with a seepage basin as an outlet is the location of the pond water surface relative to the invert elevation of the pipe or other connection to the seepage field. The model determines the time the pond water surface is either above or below the seepage field

Table A-1. Input File Associated with Example Problem

```
Pond file name:  C:\Program Files\WinDetpond\classexp.pnd
Pond file description:  simple WinDETPOND example
Particle Size file name:  C:\PROGRAM FILES\WINDETPOND\MEDIUM.CPZ
Output Format Option:  Water Quality Summary:  One Line per Event
Output device:  Print Output to File (extension .DPO)
Date:  09-16-2003


Drainage Basin Runoff Procedure:
         Combined Surface Characteristics
         1.  All directly connected impervious areas (acres):   45
         2.  All pervious areas (acres):   75
         3.  All impervious areas draining to pervious areas (acres):   30
Outlet Characteristics:
    Outlet number 1
      Outlet type:  Sharp Crested Weir
           1.  Weir length (ft):   20
           2.  Weir height from invert:   1
           3.  Invert elevation above datum (ft):   6
           4.  Number of end contractions:   2
Outlet Characteristics:
    Outlet number 2
      Outlet type:  V - Notch Weir
           1.  Weir angle (degrees):  45
           2.  Weir height from invert:   4
           3.  Invert elevation above datum (ft):   3

Initial stage elevation (ft):   3
User defined pond efficiency factor (n):   5

Pond Stage, Surface Area, and Stage-related Outfall Devices (if applicable)
Entry    Stage     Pond Area     Natural Seepage    Other Outflow
Number    (ft)      (acres)          (in/hr)            (cfs)
   0      0.00       0.0000             0.00              0.00
   1      0.50       0.1000             0.00              0.00
   2      1.00       0.1300             0.00              0.00
   3      1.50       0.1700             0.00              0.00
   4      2.00       0.2000             0.00              0.00
   5      2.50       0.9000             0.00              0.00
   6      3.00       1.2000             0.00              0.00
   7      3.50       1.5000             0.00              0.00
   8      4.00       1.8000             0.00              0.00
   9      4.50       2.1000             0.00              0.00
  10      5.00       2.4000             0.00              0.00
  11      5.50       2.7000             0.00              0.00
  12      6.00       3.0000             0.00              0.00
  13      6.50       3.3000             0.00              0.00
  14      7.00       3.6000             0.00              0.00

Rain Information
Rain file name:  C:\Program Files\WinDetpond\BHAM5289.RAN
         Rain starting date : 01/01/76
```

Rain ending date : 12/31/76

Temperature and Specific Gravity Information:
Average Annual Water Temperature (degrees F): 50.
Particle Specific Gravity: 2.5

Groundwater Mounding Data
1. Seepage field infiltration rate (ft/day): 0
2. Seepage field length (ft): 0
3. Seepage field width (ft): 0
4. Fillable porosity (no units): 0
5. Initial water table height (ft): 0
6. Hydraulic conductivity (ft/day): 0
7. Vertical distance from initial water table elevation to seepage field bottom (ft): 0

Table A-2. Output Data for Example Analysis (one-line per event, partial output example showing only selected columns and some rows)

DETPOND for Windows Version 8.3.0

Pond file name: C:\Program Files\WinDetpond\classexp.pnd

Pond file description: simple WinDETPOND example

Rain file name: C:\Program Files\WinDetpond\BHAM5289.RAN

Critical Particle Size file name: C:\PROGRAM FILES\WINDETPOND\MEDIUM.CPZ

Model Run Start Date: 1/1/1976 Model Run End Date: 12/31/1976

Date of run: 9/16/2003 Time of run: 22:35:35

Water Rain Number	Quality Rain Date	Rain Depth (in)	Rain Duration (hrs)	Intrevnt Duration (days)	Rain Intensity (in/hr)	Maximum Pond Stage (ft)	Minimum Pond Stage (ft)	Event Inflow Volume (ac-ft)	Event Hydr Outflow (ac-ft)	Peak Reduction Factor (Fraction)	Event Flushing Ratio	Flow-weighted Particle Size (Ideal)	%_Part Solids Removed (Ideal)
2,642	1/7/1976	0.58	9	3.71	0.06	4.46	3.23	3.146	3.122	0.8	2.913	2.7	91.4
2,643	1/11/1976	0.25	5	1.42	0.05	3.87	3.25	1.138	0.925	0.91	1.053	1.6	95.8
2,644	1/13/1976	0.03	2	0.29	0.02	3.4	3.36	0.017	0.07	0.67	0.016	0.9	99.1
2,645	1/13/1976	0.01	1	0.33	0.01	3.36	3.32	0.002	0.053	n/a	0.002	0.8	99.2
2,646	1/13/1976	0.38	2	6.46	0.19	4.38	3.17	2.026	2.242	0.94	1.876	2.2	93.5
2,647	1/20/1976	0.05	5	3.88	0.01	3.2	3.13	0.048	0.092	0.95	0.045	0.3	99.7
2,648	1/24/1976	0.03	2	1	0.02	3.14	3.13	0.017	0.017	0.97	0.016	0.3	99.7
2,649	1/25/1976	2.33	20	10.5	0.12	6.03	3.12	15.118	15.135	0.42	13.998	5.7	80.6
2,650	2/5/1976	0.51	9	5.21	0.06	4.29	3.12	2.709	2.614	0.83	2.508	2.3	92.9
2,651	2/11/1976	0.01	1	6.71	0.01	3.19	3.11	0.002	0.113	0.76	0.002	0.3	99.7
2,652	2/18/1976	0.67	8	3.08	0.08	4.65	3.11	3.843	3.609	0.79	3.559	3.1	89.9
2,653	2/21/1976	0.61	3	12.92	0.2	4.72	3.1	3.144	3.385	0.9	2.911	2.8	91
2,654	3/5/1976	0.85	23	2.21	0.04	4.62	3.1	4.874	4.54	0.57	4.513	3.1	89.7
2,655	3/8/1976	1.11	17	2.75	0.07	4.96	3.31	6.077	6.134	0.6	5.627	3.7	87.3
2,656	3/12/1976	0.3	5	0.29	0.06	4.03	3.31	1.428	0.684	0.89	1.322	2.3	92.7

Table A-2. Output Data for Example Analysis (one-line per event, partial output example showing only selected columns and some rows) (continued)

2,657	3/12/1976	1.18	4	2.25	0.3	5.85	3.37	7.202	7.854	0.78	6.669	5.2	82.1
2,658	3/15/1976	3.64	27	4.17	0.13	6.49	3.24	25.623	25.812	0.23	23.725	6.8	76.9
2,659	3/20/1976	0.04	2	0.42	0.02	3.26	3.24	0.031	0.032	0.94	0.028	0.5	99.5
2,660	3/20/1976	1.14	6	3.58	0.19	5.31	3.25	6.268	6.236	0.79	5.804	4.3	85.2
2,661	3/24/1976	0.04	6	1.46	0.01	3.27	3.21	0.028	0.101	0.78	0.026	0.5	99.5
2,662	3/26/1976	1.56	17	2.46	0.09	5.39	3.21	8.895	8.71	0.55	8.236	4.5	84.2
2,663	3/29/1976	2.2	12	0.38	0.18	6.13	3.35	13.017	11.439	0.56	12.053	6.5	77.8
2,664	3/30/1976	2.09	22	11.38	0.1	5.82	3.11	13.244	15.133	0.4	12.263	5.2	82.1
2,665	4/11/1976	0.21	5	1.96	0.04	3.69	3.11	0.918	0.653	0.94	0.85	1.2	97.7
2,666	4/13/1976	0.05	7	10.5	0.01	3.32	3.1	0.044	0.323	0.75	0.04	0.5	99.5
2,667	4/24/1976	0.84	9	4.88	0.09	4.87	3.11	4.86	4.722	0.76	4.5	3.4	88.3
2,668	4/30/1976	0.09	8	0.33	0.01	3.31	3.21	0.172	0.057	0.93	0.16	0.6	99.4
2,669	4/30/1976	0.94	11	5.5	0.09	4.99	3.19	5.365	5.502	0.69	4.968	3.6	87.5

	Rain Depth (in)	Rain Duration (hrs)	Intrevnt Duration (days)	Rain Intensity (in/hr)	Maximum Pond Stage (ft)	Minimum Pond Stage (ft)	Event Inflow Volume (ac-ft)	Event Hydr Outflow (ac-ft)	Peak Reduction Factor (Fraction)	Event Flushing Ratio	Flow-weighted Particle Size (Ideal)	%_Part Solids Removed (Ideal)
Number of Events	98	98	98	98	98	98	98	98	98	98	98	98
Total	55.15	742	335.1	n/a	n/a	n/a	318.2	317.9	n/a	n/a	n/a	n/a
Equiv Annual Ttl	55.46	746.2	337	n/a	n/a	n/a	320	319.7	n/a	n/a	n/a	n/a
Minimum	0.01	1	0.25	0.005	3.099	3.003	0.0019	0.0046	0	0.0018	0.1671	76.92
Maximum	3.83	38	16.58	0.54	6.487	3.834	26.78	25.81	0.9951	24.8	6.8	99.83
Average	0.5628	7.571	3.419	0.07694	4.19	3.232	3.247	3.244	0.7434	3.006	n/a	n/a
Flow-weighted Avg	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	0.7654	2.164	1.827	94.74
Median	0.24	5	2.375	0.04795	3.964	3.193	1.043	0.9006	0.7982	0.9656	1.962	94.3
Stnd Deviation	0.7599	7.713	3.402	0.09113	0.9176	0.1536	5.062	4.98	0.2421	4.687	1.771	6.615
COV	1.35	1.019	0.995	1.184	0.219	0.04754	1.559	1.535	0.3256	1.559	0.7928	0.07097

Total Time Period (yrs): 0.994406

Number of Times Water Surface Elevation Exceeded Sharp Crested Weir Elevation: 6

Number of Times Water Surface Elevation Exceeded V-Notch Weir Elevation: 98

Number of Times Water Surface Elevation Exceeded Top of Pond Elevation:

Ideal column will reflect reduced performance due to scour from shallow ponds. See Scour help topic.

Appendix B: Treatability Tests for the Development of New Controls for Critical Source Areas

Toxicant Reduction Tests for Various Treatment Unit Processes

The EPA-funded project for the development of critical source area controls conducted by Pitt, *et al.* (1999) examined several methods to reduce stormwater toxicity by using a variety of conventional bench-scale treatment processes, including settling, sieving, aeration, photodegradation, floatation, some chemical addition tests, and combinations. The data from an earlier project phase identified the critical source areas that generally had the highest toxicant concentrations for study during these treatability tests: storage/parking areas, vehicle service areas, and automobile salvage yards.

The objective of these treatability tests was to quantify improvements in stormwater toxicity using different stages of several bench-scale treatment methods. These data were used to indicate the relative effectiveness of different treatment efforts and processes. To meet this objective and the resource restraints of cost and time, the Azur Environmental Microtox™ screening toxicity test was chosen to indicate the relative changes in toxicity.

The treatability tests included intensive analyses of samples from twelve sampling locations in the Birmingham, AL, area. Table B-1 lists the source area categories, and relative toxicity category prior to treatment. Independent replicates (obtained during separate analysis runs) were used to determine the measurement errors associated with the Microtox™ procedure. Samples B and D were initially extremely toxic, while the remainder of the samples were moderately toxic. All samples were reduced to "non-toxic" levels after various degrees of treatment.

Table B-1. Treatability Sample Descriptions

Sample Source	Initial Toxicity[a] (%)	Number of Analyses	Standard Deviation[b]	Coefficient of Variation[b] (%)
	Automobile Service Area Samples			
B	78	28	7.6	9.8
C	34	42	2.9	8.5
E	43	74	1.3	3.0
H	50	88	1.5	3.0
	Industrial Loading & Parking Area Samples			
D	67	74	2.1	3.1
F	31	88	1.5	4.9
G	53	88	3.0	5.7
I	55	89	1.9	3.4
J	49	89	1.1	2.3
K	28	89	2.2	8.1
	Automobile Salvage Yard Samples			
L	26	89	1.4	5.5
M	54	89	1.8	3.4

[a] Toxicity measured as percent light reduction after 35 minute exposure.
[b] Applies to replicate samples only. From 5 to 10 replicates were conducted for initial toxicity for each sample.

Treatability Tests

The selected source area runoff samples all had elevated toxicant concentrations, compared to the other urban source areas initially examined, allowing a wide range of laboratory partitioning and treatability analyses to be conducted. The treatability tests conducted were:

- Settling column (37 mm x 0.8 m Teflon™ column).
- Floatation (series of eight glass narrow neck 100 mL volumetric flasks).
- Screening and filtering (series of eleven stainless steel sieves, from 20 to 106 µm, and a 0.45 µm membrane filter).
- Photo-degradation (2 liter glass beaker with a 60 watt broad-band incandescent light placed 25 cm above the water, stirred with a magnetic stirrer with water temperature and evaporation rate also monitored).
- Aeration (the same beaker arrangement as above, without the light, but with filtered compressed air keeping the test solution supersaturated and well mixed).
- Photo-degradation and aeration combined (the same beaker arrangement as above, with compressed air, light, and stirrer).
- Undisturbed control sample (a sealed and covered glass jar at room temperature).

Because of the difficulty of obtaining large sample volumes from many of the source areas that were to be examined, these bench-scale tests were all designed to use small sample volumes (about one liter per test). Each test (except for filtration, which was an "instantaneous" test) was conducted over a duration of 3 days. Subsamples (40 mL each) were obtained for toxicity analyses at 0, 1, 2, 3, 6, 12, 24, 48, and 72 hours. In addition, settling column samples were also obtained several times within the first hour, at: 1, 3, 5, 10, 15, 25, and 40 minutes.

The Microtox™ procedure allowed toxicity screening tests to be conducted on each sample partition during the treatment tests. This procedure enabled more than 900 toxicity tests to be made. Turbidity tests were also conducted on all samples. Figures B-1 to B-24 are graphical data plots of the toxicity reductions observed during each treatment procedure examined, including the control measurements. These figures are grouped in threes for each treatment type. One group contains the treatment responses for the automobile service facility areas (samples B, C, E, and H), another group is for the industrial loading and parking areas (samples D, F, G, I, J, and K), and the last group is for the automobile salvage yards (samples L and M). These plots indicate the reduction in toxicity as the level of treatment increased. As an example, Figures B-1 through B-3 show three separate plots for the undisturbed samples undergoing very little change, except for samples F (which increased in toxicity with time) and C (which decreased in toxicity with time). In contrast, Figures B-4 through B-6 show the dramatic improvements available with plain physical settling. All samples, except for B, showed dramatic reductions in toxicity with increasing settling times. Even though the data are separated into these three groups, very few consistent differences are noted in the way the different sample types responded to various treatments. As expected, there are greater apparent differences between the treatment methods than between the sample groupings.

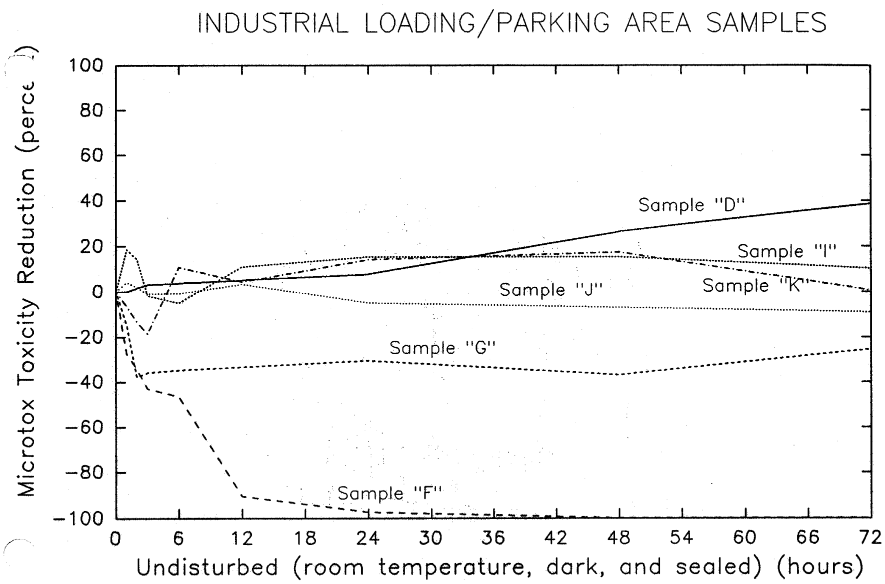


Figure B-1. Toxicity reduction on control samples - industrial loading and parking areas.

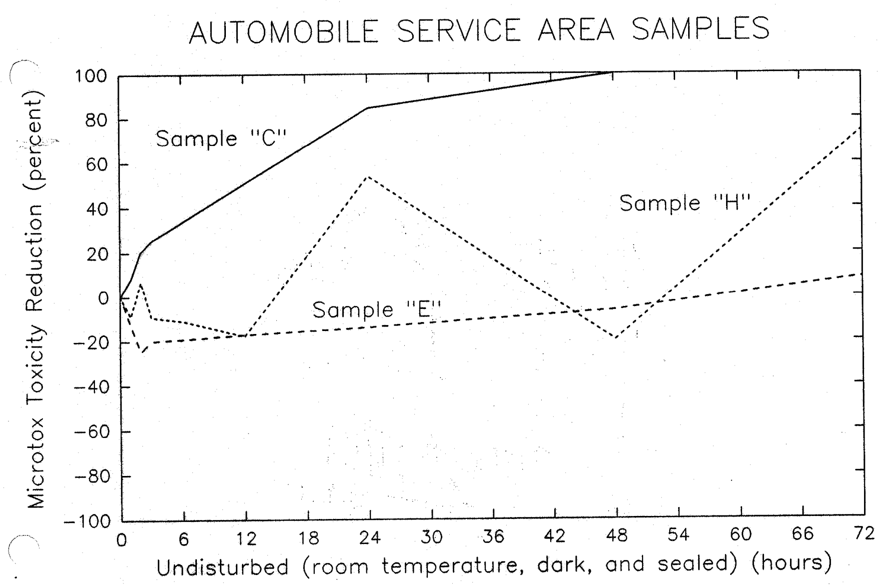


Figure B-2. Toxicity reduction on control samples - automobile service facilities.

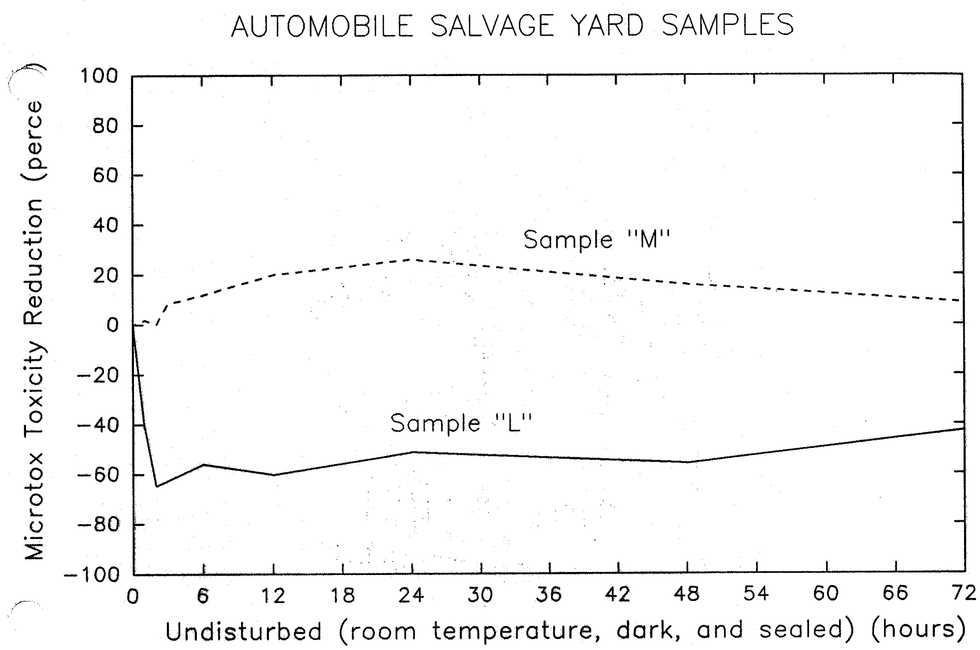


Figure B-3. Toxicity reduction on control samples - automobile salvage yards.

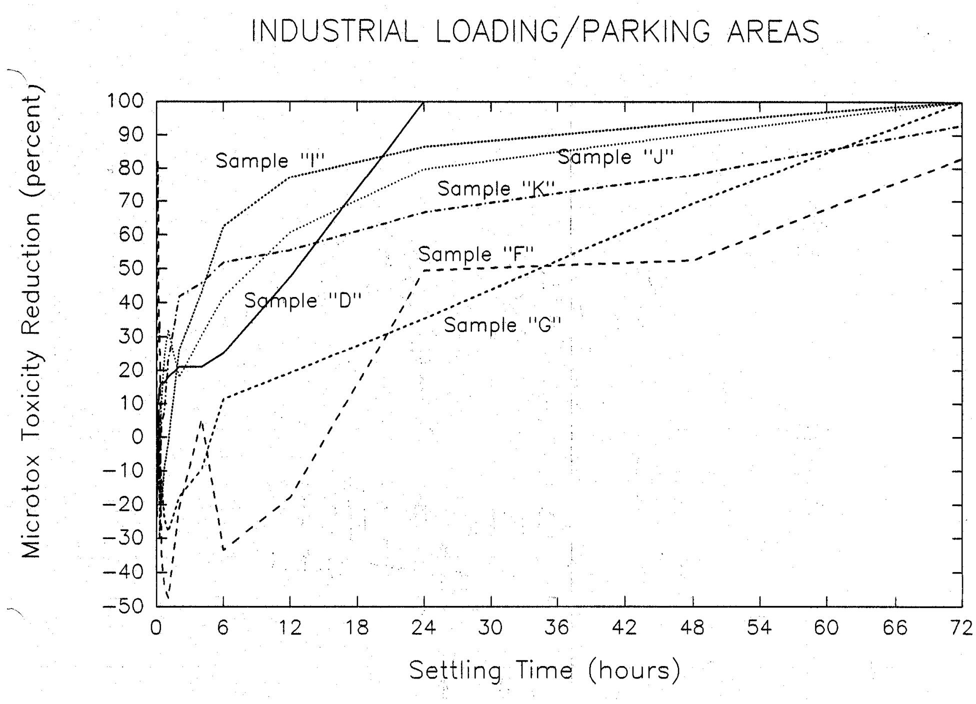


Figure B-4. Toxicity reduction from settling treatment - industrial loading and parking areas.

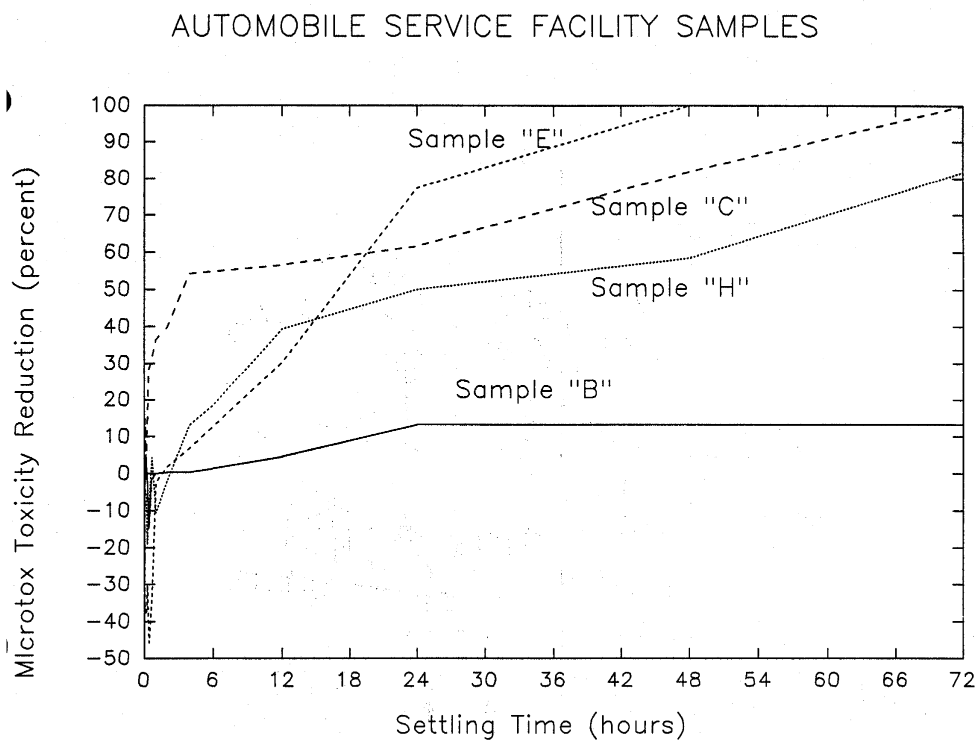


Figure B-5. Toxicity reduction from settling treatment - automobile service facilities.

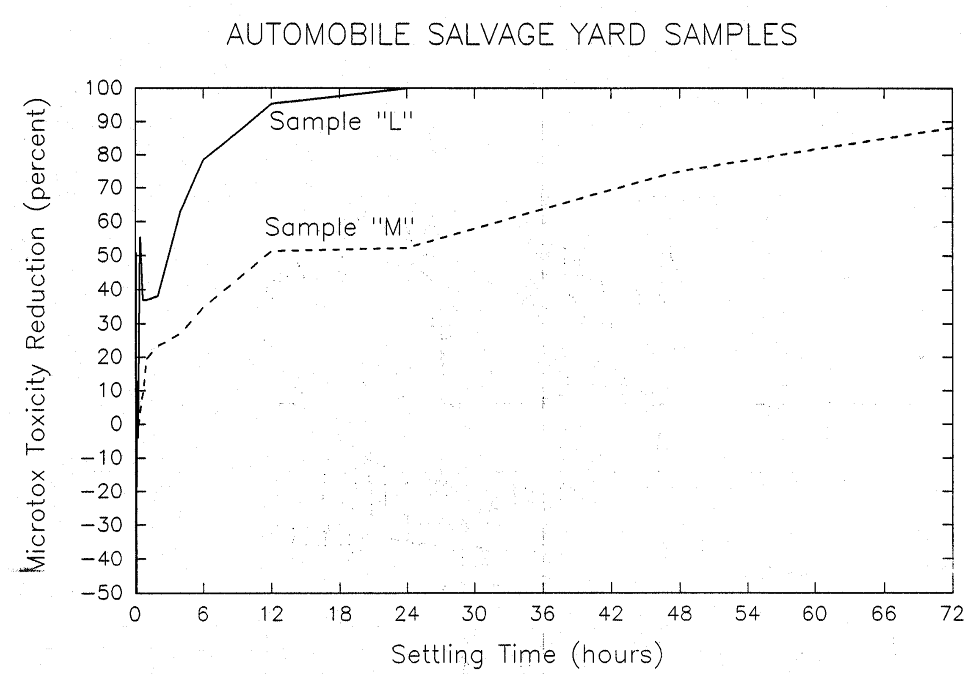


Figure B-6. Toxicity reduction from settling treatment - automobile salvage yards.

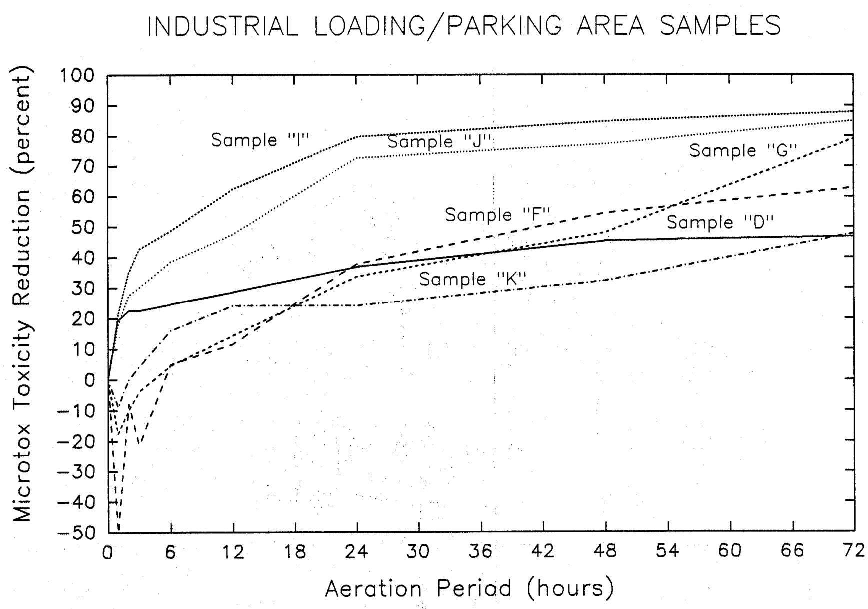


Figure B-7. Toxicity reduction from aeration treatment - industrial loading and parking areas.

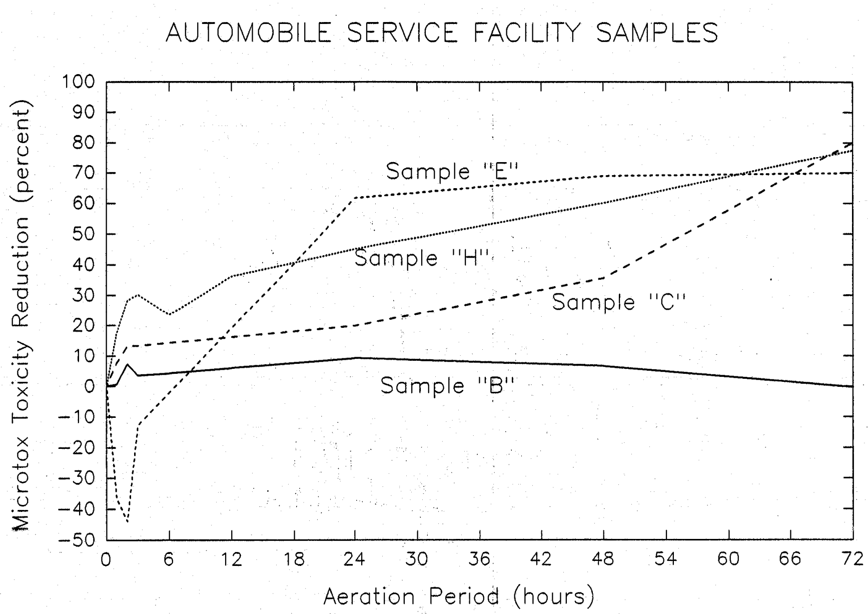


Figure B-8. Toxicity reduction from aeration treatment - automobile service facilities.

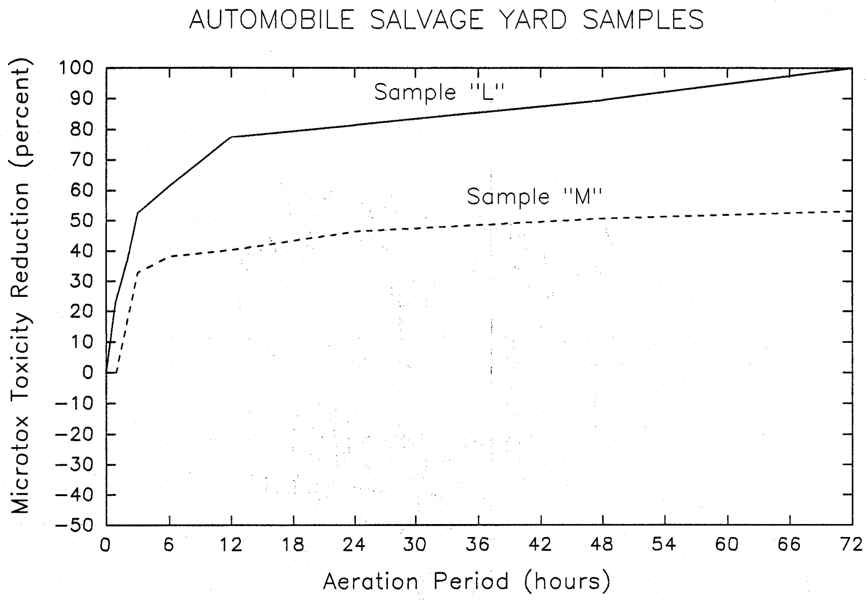


Figure B-9. Toxicity reduction from aeration treatment - automobile salvage yards.

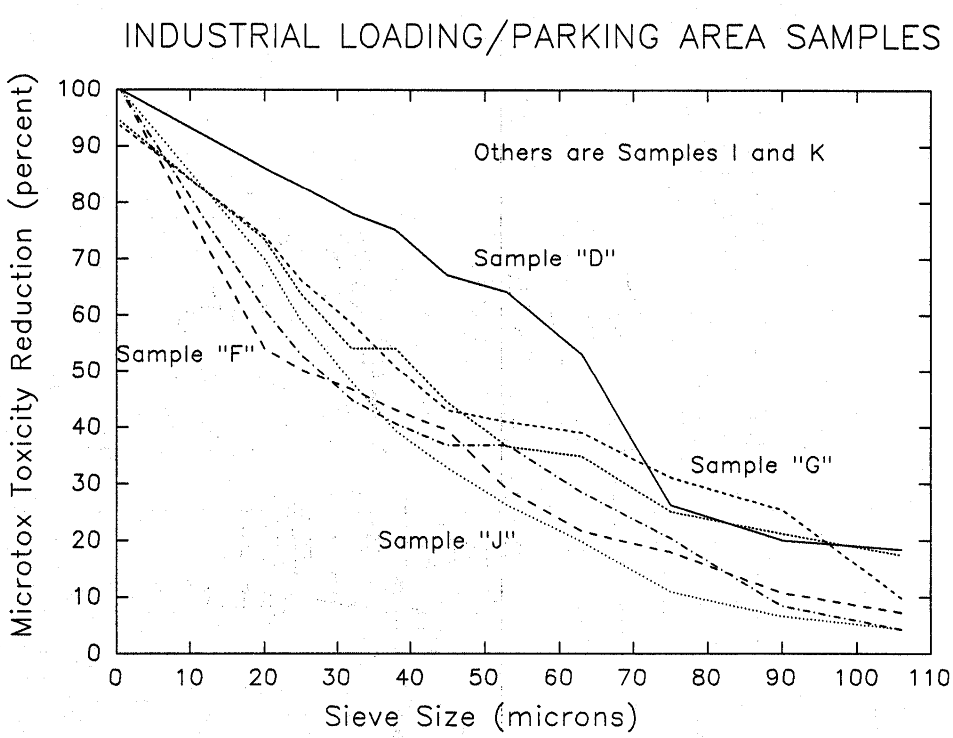


Figure B-10. Toxicity reduction from sieve treatment - industrial loading and parking areas.

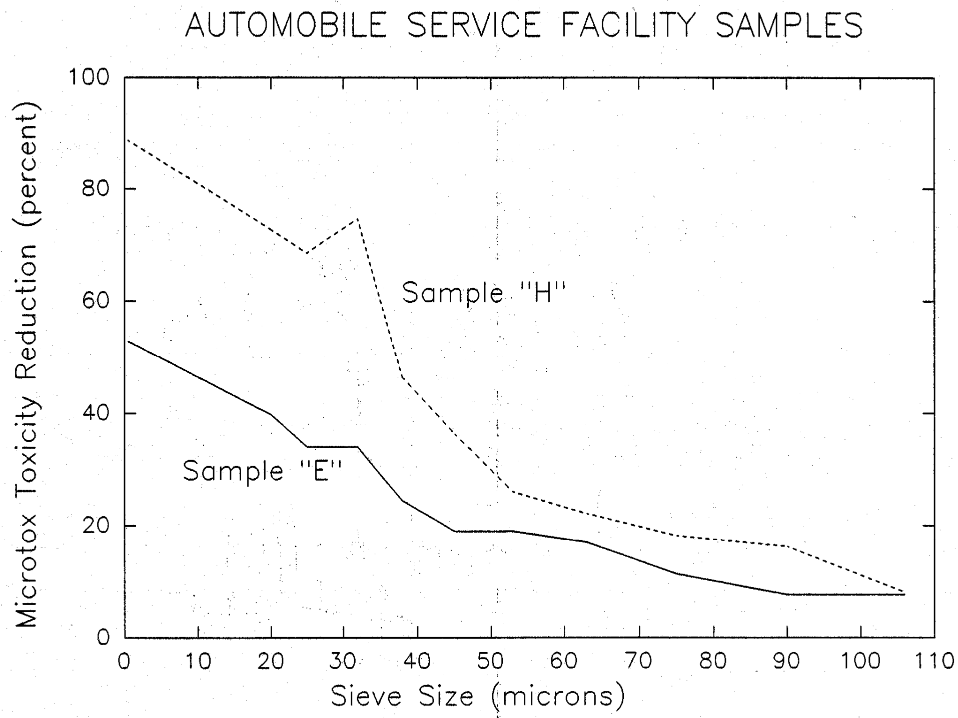


Figure B-11. Toxicity reduction from sieve treatment - automobile service facilities.

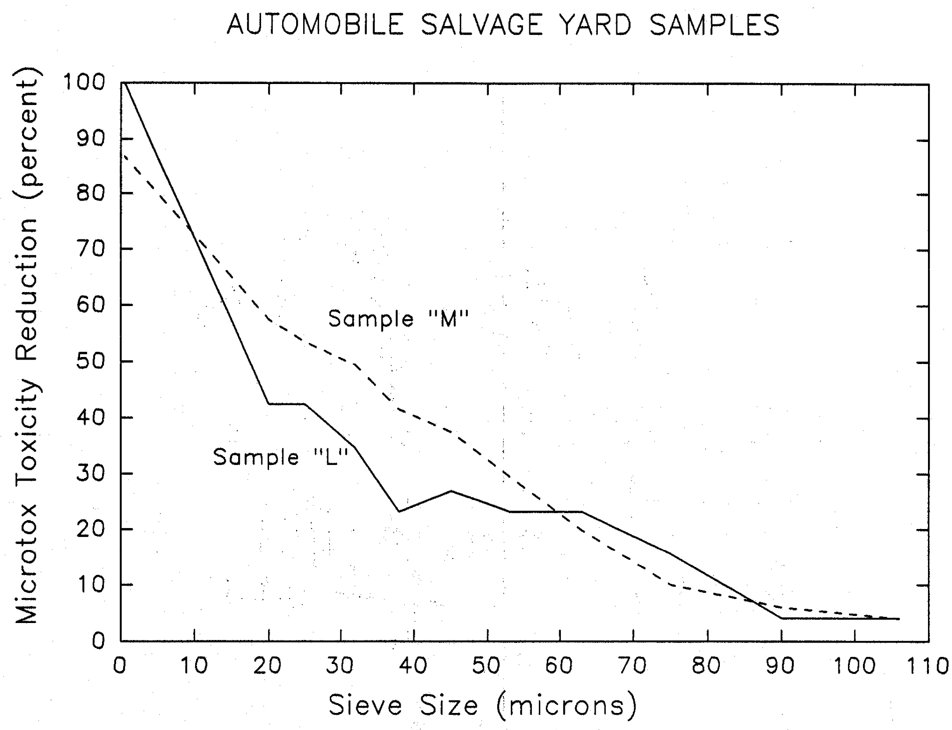


Figure B-12. Toxicity reduction from sieve treatment - automobile salvage yards.

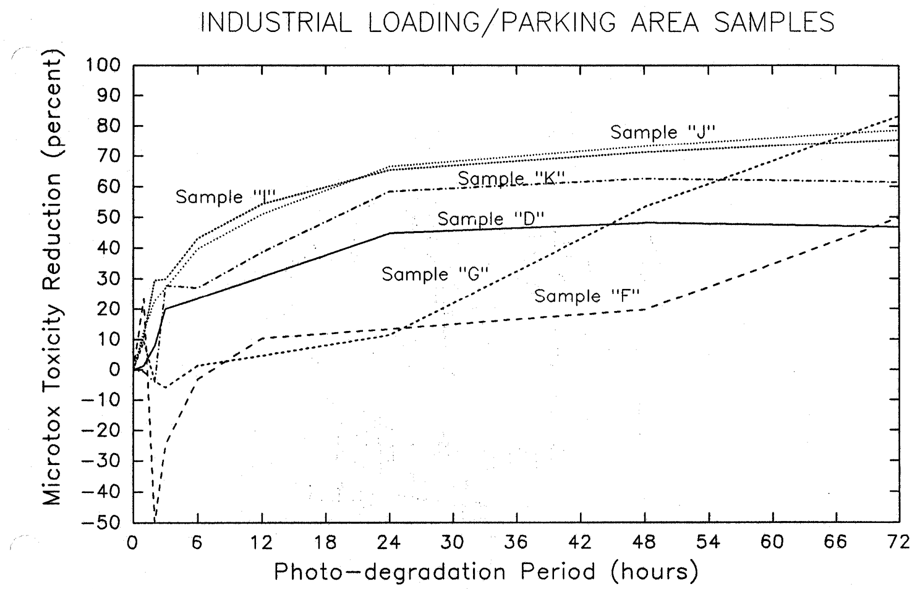


Figure B-13. Toxicity reduction from photo-degradation treatment - industrial loading and parking areas.

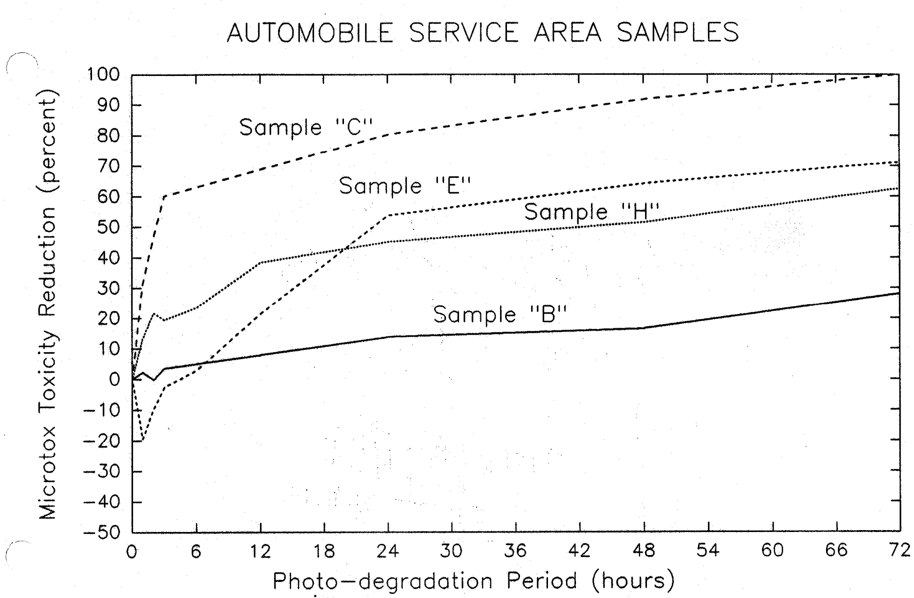


Figure B-14. Toxicity reduction from photo-degradation treatment - automobile service facilities.

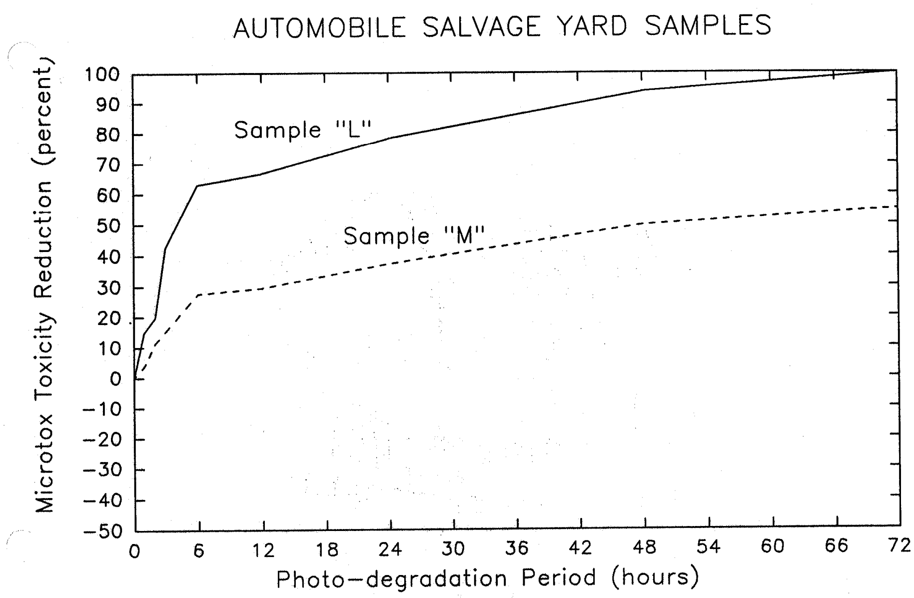


Figure B-15. Toxicity reduction from photo-degradation treatment - automobile salvage yards.

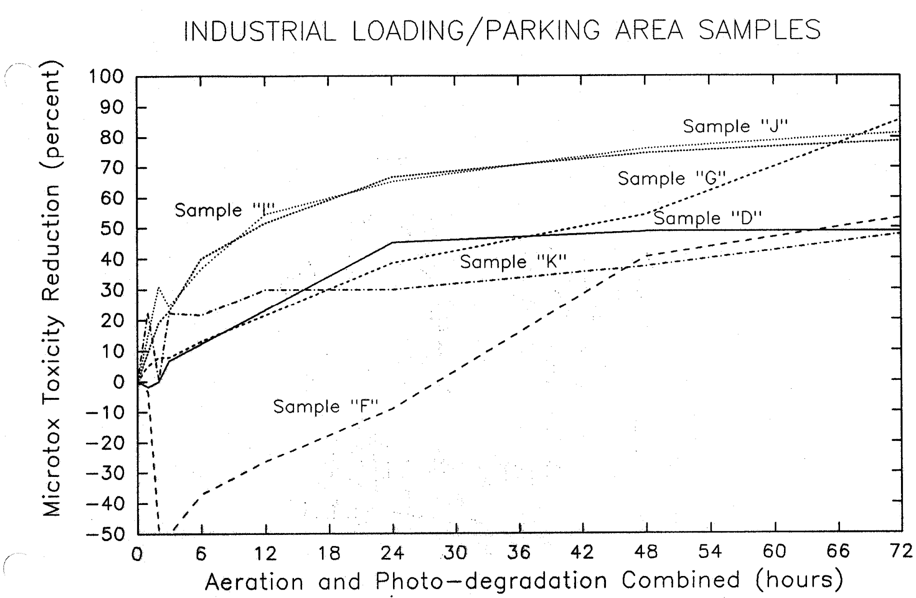


Figure B-16. Toxicity reduction from aeration and photo-degradation treatment - industrial loading and parking areas.

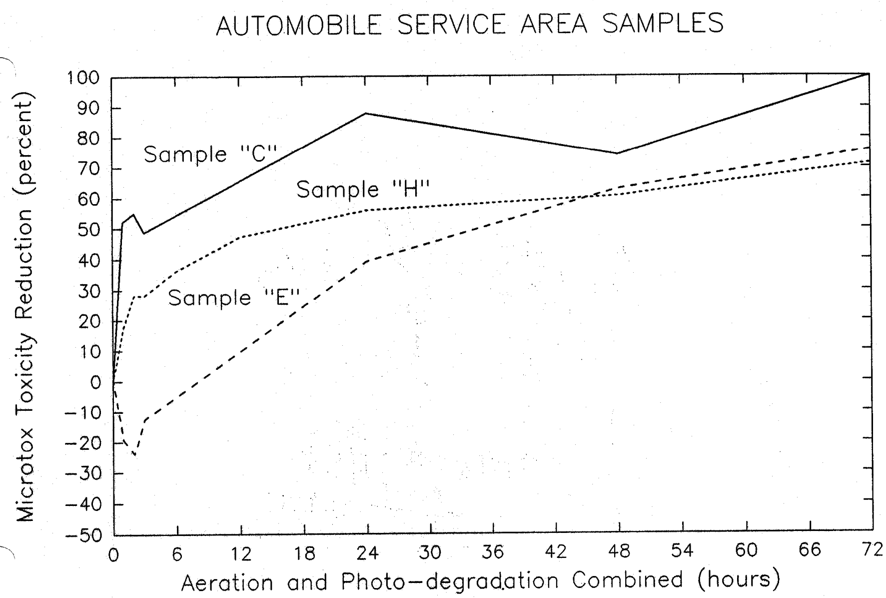


Figure B-17. Toxicity reduction from aeration and photo-degradation treatment - automobile service facilities.

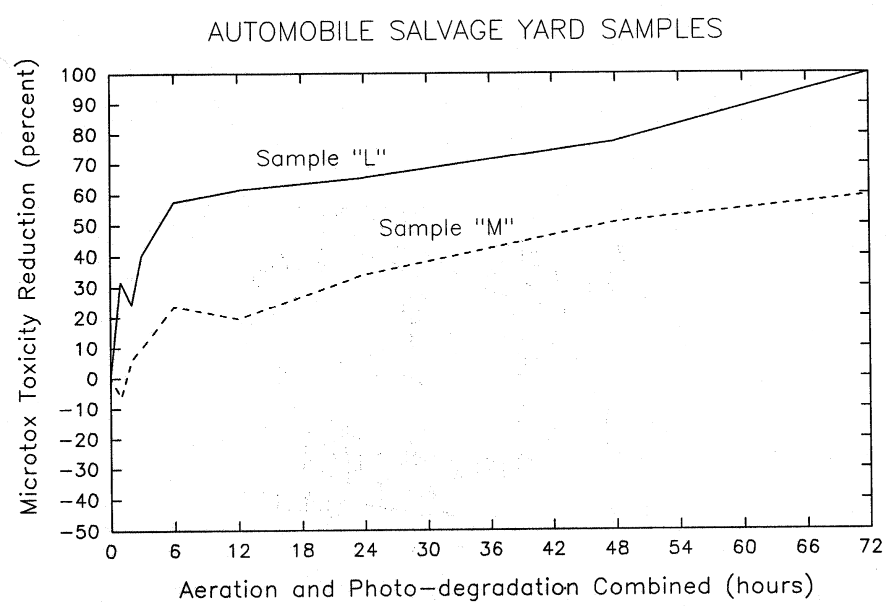


Figure B-18. Toxicity reduction from aeration and photo-degradation treatment - automobile salvage yards.

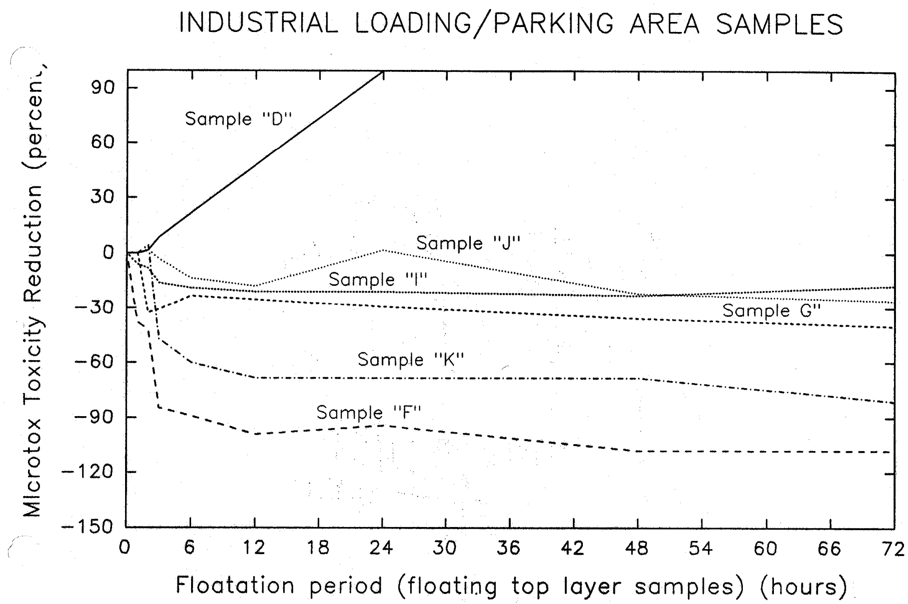


Figure B-19. Toxicity reduction from floatation treatment (top layer samples) - industrial loading and parking areas.

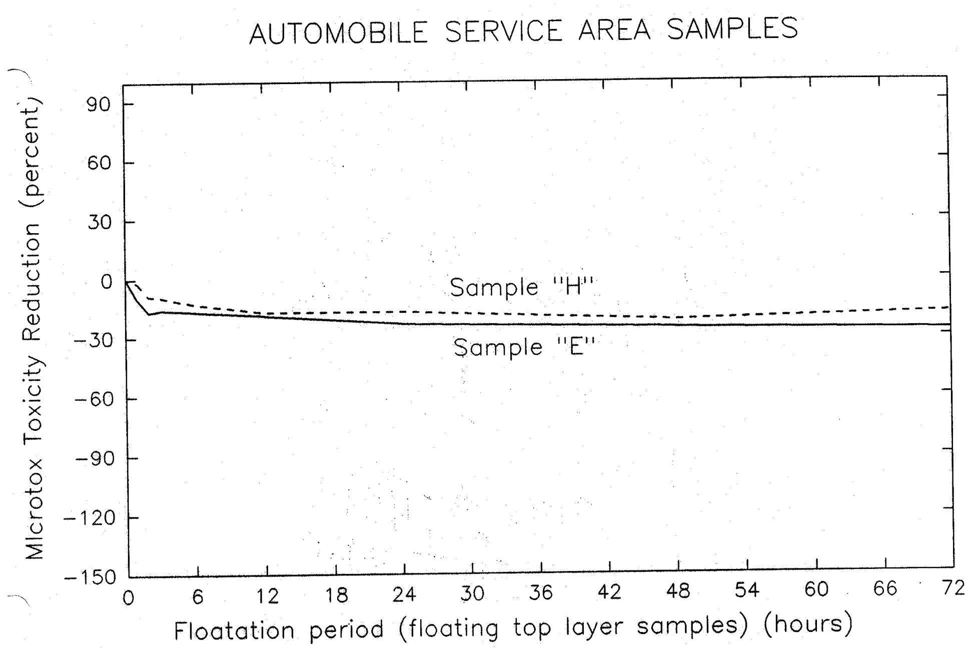


Figure B-20. Toxicity reduction from floatation treatment (top layer samples) - automobile service facilities.

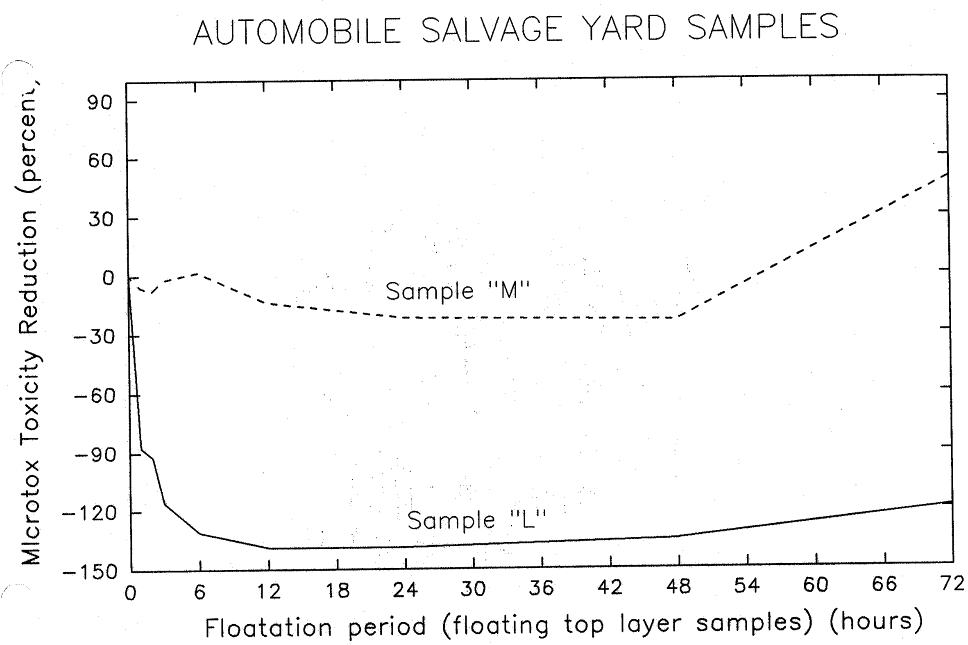


Figure B-21. Toxicity reduction from floatation treatment (top layer samples) - automobile salvage yards.

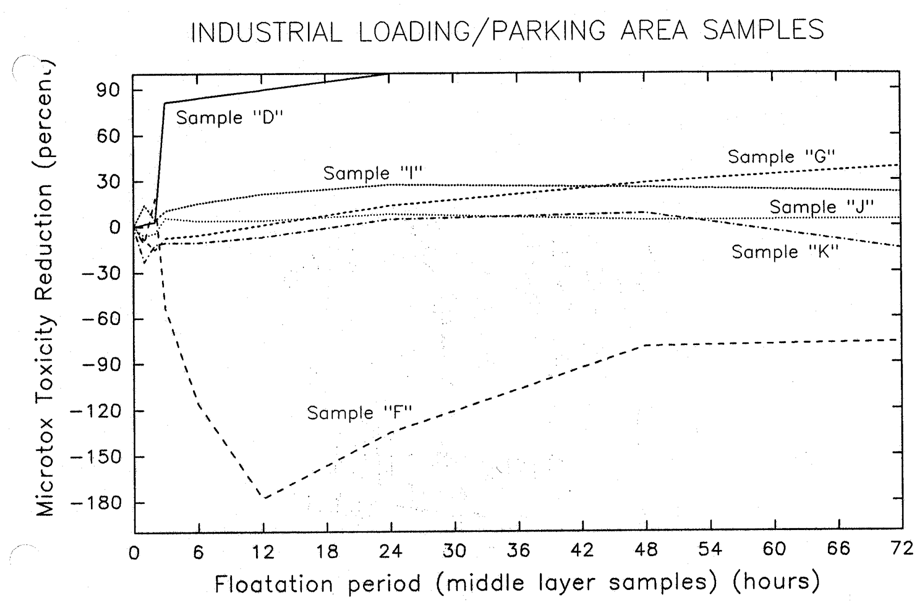


Figure B-22. Toxicity reduction from floatation treatment (middle layer samples) - industrial loading and parking areas.

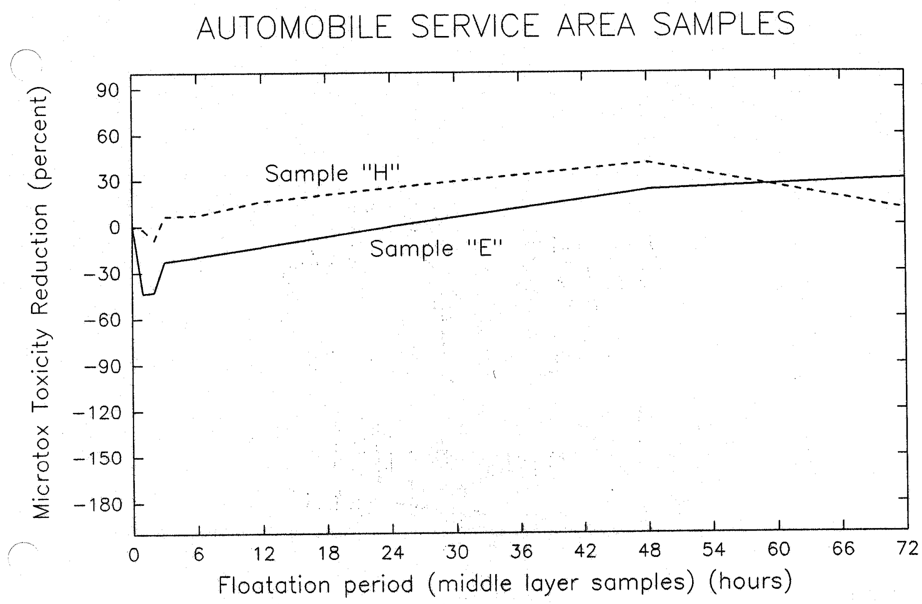


Figure B-23. Toxicity reduction from floatation treatment (middle layer samples) - automobile service facilities.

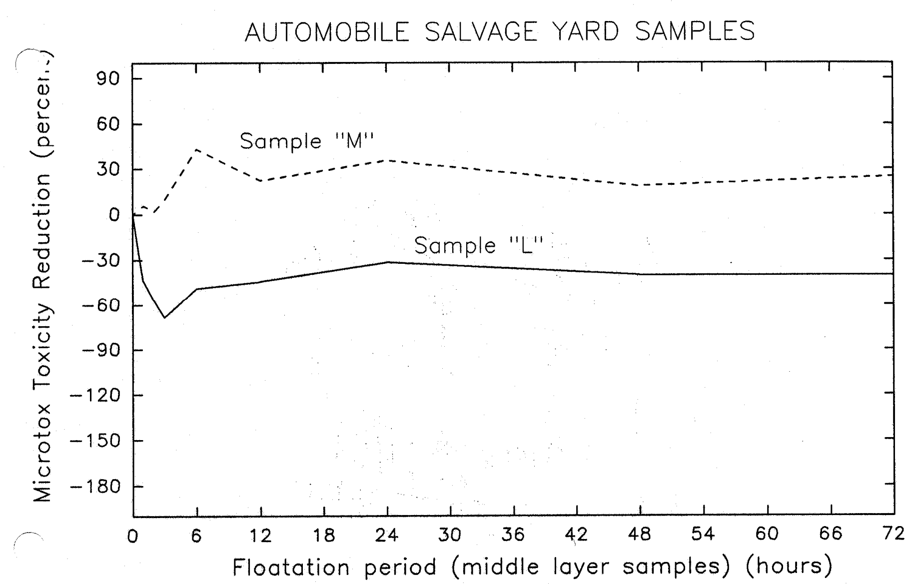


Figure B-24. Toxicity reduction from floatation treatment (middle layer samples) - automobile salvage yards.

Table B-2 summarizes results from the non-parametric Wilcoxon signed ranks test (using SYSTAT: The System for Statistics, Version 5, SYSTAT, Inc., Evanston, Ill.) for different treatment combinations. This statistical test indicates the two-sided probabilities that the sample groups are the same. A probability of 0.05, or less, is typically used to indicate significant differences in the data sets (indicated by bold italics in the table). As an example, Table

B-2 indicates that there were significant differences (probabilities of 0.02) for all of the treatment tests done on sample D (an extremely toxic sample), compared to the undisturbed control sample.

Table B-2. Two-sided Probabilities Comparing Different Treatment Tests with Control Samples

	Auto. Service Area				Industrial Loading & Parking Area						Auto. Salvage	
Undisturbed versus:	B	C	E	H	D	F	G	I	J	K	L	M
settling	n/a	0.25	***0.02***	0.41	***0.02***	0.12	0.09	0.07	***0.01***	***0.01***	***0.02***	***0.02***
aeration	n/a	0.31	0.25	0.07	***0.02***	***0.05***	0.06	***0.04***	***0.01***	***0.01***	***0.02***	***0.03***
photodegradation	n/a	0.12	0.06	0.16	***0.02***	***0.04***	***0.03***	0.07	***0.01***	***0.01***	***0.02***	0.16
aeration & photodegradation.	n/a	0.35	0.24	0.06	***0.02***	***0.05***	***0.03***	0.09	***0.01***	***0.01***	***0.02***	0.09
flotation - top layer	n/a	n/a	0.74	***0.02***	***0.02***	***0.05***	0.13	***0.01***	***0.03***	0.21	***0.01***	0.09
flotation - mid. layer	n/a	n/a	0.31	0.87	***0.02***	0.78	***0.02***	0.26	0.16	0.17	0.59	0.89

The aeration test provided the most samples that had significant probabilities of being significantly different from the control condition. Settling, photo-degradation, and aeration and photo-degradation combined, were similar in providing the next greatest number of samples that had significant probabilities of being different from the control condition. The floatation test had six samples that had significant differences in toxicity between the top floating layer and the control sample. However, the more important contrast between the middle sample layers (below the top floating layer) and the control sample, which would indicate a reduction in toxicity of post-treated water, had only two samples that were significantly different from the control sample.

The absolute magnitudes of toxicity reductions must also be considered. As an example, it may be significant, but unimportant, if a treatment test provided many (and therefore consistent) samples having statistically significant differences compared to the control sample, if the actual toxicity reductions were very small.

As shown on Figures B-1 to B-24, important reductions in toxicities were found during many of the treatment tests. The highest toxicant reductions were obtained by settling for at least 24 hours over a 0.5 m depth (providing at least 50 percent reductions for all but 2 samples), screening through at least a 40 μm screen (20-70 percent reductions), and aeration and/or photo-degradation for at least 24 hours (up to 80 percent reductions). Increased settling, aeration or photo-degradation times, and screening through finer meshes, all reduced sample toxicities further. The floatation tests produced floating sample layers that generally increased in toxicity with time and lower sample layers that generally decreased in toxicity with time, as expected; however, the benefits were quite small (less than 30 percent reductions). As shown on Table B-2, only about 40% of the floatation test toxicity changes were statistically different from the variations found in the control samples.

These tests indicate the wide-ranging behavior of these related samples for the different treatment tests. Some samples responded poorly to some tests, while other samples responded well to all of the treatment tests. Any practical application of these treatment unit processes would therefore require a treatment train approach, subjecting critical source area runoff to a combination of processes in order to obtain relatively consistent overall toxicant reduction benefits.

Physical-Chemical Bench-scale and Pilot-scale Treatability Tests

A physical-chemical treatment scheme was developed and tested to reduce toxic and pathogenic discharges of stormwater (Pitt, *et al.* 1998). These laboratory and pilot-scale experiments were developed for a special purpose (the rapid treatment of relatively small volumes of stormwater), but the results are expected to be indicative of general stormwater treatment behavior using similar treatment processes.

Both bench-scale and pilot-scale tests were used to examine many alternative treatment factors. The laboratory tests were comprised of two activities: jar tests of different coagulants, and filterability of water (after chemical addition) through 5 and 25 μm bag filter material. The purpose of these tests was to identify the most effective chemical coagulant (and dosage range) that could be used to treat stormwater, and the effectiveness of final filtration after the chemical addition. After these laboratory tests, a pilot-scale setup was developed and tested. The pilot-scale tests included the following unit processes assembled on a trailer:

1) initial 100 μm membrane filter bag
2) 25 μm membrane filter bag
3) 5 μm membrane filter bag
4) activated charcoal filter
5) UV disinfection unit
6) final cascade aerator

Bench-scale Chemical Treatability Tests

Numerous treatability tests were conducted by Pitt, *et al.* (1998) to identify the most effective coagulant, mixing time, and dosage, in addition to floc settling and capture and stability. These tests involved bench-scale jar tests using several categories of coagulants, with and without supplements. A range of conductivity and turbidity conditions were tested to examine the possible effects of these parameters on treatability.

Coagulation is a two-step process, the first step is destabilization of particles (by chemical addition), followed by amalgamation (by mixing) to form faster sinking particles. Added chemicals affect the surface charges on the particles so they will join into flocs. The flocs are larger than the initial primary particles, but have a much lower specific gravity (because of entrapped water). However, the increased size usually overcomes the decreased density and the flocs settle much more rapidly than the primary particles. In some cases and for some chemicals, the flocs will rise to the water surface where they are removed by skimming. In most water and wastewater treatment processes, initial flash mixing is followed by 20 to 60 minutes of slow mixing to help the particles collide to form the flocs. The floc is then allowed to settle for 2 to 8 hours in a sedimentation basin. These tests were conducted with much less time in an attempt to simulate field conditions where rapid treatment is desired. Chemicals that form heavy flocs quickly are needed, and filtration was investigated to replace slow sedimentation.

Hydrolyzing metal ions are a common family of coagulants used to destabilize the surface charges of the particles. These include alum (aluminum sulfate), the most commonly used coagulant used in the U.S., ferrous sulfate, ferric chloride, and mixtures of ferrous sulfate and ferric chloride (chlorinated copperas). Another common group of coagulants are organic polymers (polyelectrolytes) and synthetic polymers. Polymers are long chain molecules having many available active sites for adsorption. Besides coagulants and polymers, many coagulant aids are available to enhance coagulation and flocculation. In addition, many blends of chemicals are used. Because these chemicals react differently with different waters, bench-scale jar tests are needed to determine the most efficient dosages and pH conditions of the water being treated. The objective was to identify chemicals having relatively robust behavior for the range of water conditions likely found in stormwater.

Water can be grouped into four types according to turbidity and alkalinity conditions. Low turbidity is defined as <10 NTU and high turbidity is defined as >100 NTU. Turbidity is important because it has associated small particulates that form nuclei for the flocs. In most cases, stormwater is of intermediate turbidity, although locations affected by eroding areas can have extremely high turbidity values. Low alkalinity water has alkalinity levels < 50 mg/L, as $CaCO_3$, while high alkalinity water has alkalinity levels >250 mg/L. Alkalinity is important because the bicarbonates are important intermediate products in most coagulation processes. Most stormwaters likely have high alkalinity. Each water type has a preferred coagulant type:

• Type one water (high turbidity and high alkalinity): The easiest water to coagulate. Either alum (effective in pH range of 5 - 7), or ferric chloride (pH range of 6 - 7), or high molecular weight polymers work well. Cationic polymers are very effective, while anionic and non-ionic polymers may also be effective.

• Type two water (high turbidity, low alkalinity): The polymer recommendations are the same as for type one water, while alkalinity may need to be added for alum or ferric chloride, if the pH drops during water treatment.
• Type three water (low turbidity, high alkalinity): This is the likely category for many stormwaters. Polymers cannot work alone due to the low turbidity. Coagulant aids that increase the turbidity (such as clays) should be added before the polymer. Alum is needed in relatively large dosages, which forms a precipitate. Weighing agents may be needed to promote settling. Ferric chloride is also needed in relatively high dosages which also promotes hydroxide precipitates. Again, coagulant aids to weigh the floc is needed to improve settling.
• Type four water (low turbidity and low alkalinity): The most difficult water to coagulate. Must add alkalinity or turbidity to form type 2 or 3 water for either alum or ferric chloride. Polymers cannot work alone without added aids, such as clay, to increase the turbidity.

About 5% of the stormwaters collected by Pitt, *et al.* (1998) were type four waters, requiring additions of other chemicals besides the coagulants, while the majority of the waters sampled were type three. Few samples would be classified as type one or type two waters. Most were intermediate in turbidity. Therefore, the addition of coagulant aids to increase turbidity may be mandatory in some cases, and may improve the treatment in other cases. Microsand (silica sand with a nominal size range of about 75 to 150 μm in diameter) has recently been used to improve treatment of wastewaters. In most cases, the sand is separated from the floc after settling for recycling. This material is larger than clay material and may improve the weighting of the flocs for more rapid settling, while increasing turbidity.

Alum is commonly used in the U.S. for water treatment. However, during our preliminary tests on chemical treatment of stormwater, it was found to contribute toxicity to the finished waters (possibly due to dissolved aluminum at pH conditions encountered, or due to zinc contamination of the alum). The flocs formed with alum were also found to be more fragile and settle slower than with ferric chloride. Experience using full-scale ferric-chloride treatment of stormwater at European installations for phosphate control has been very successful, especially in low alkalinity waters where alum toxicity may be an issue. Ferric chloride also forms a floc that settles much more rapidly than alum flocs and does not add toxicity to the finished water. However, ferric chloride stock solutions are corrosive and must be handled carefully. The dilute solutions used for coagulation are not corrosive. Ferric chloride is also about twice as expensive as alum, on a weight basis.

These tests were evaluated by examining the reduction of turbidity, heavy metals, nutrients, and toxicity. A standard B-place jar test apparatus was used, with initial fast stirring followed by slow stirring. The first two chemical tests for the most commonly used coagulants (ferric chloride and alum) were conducted as a full-factorial test to examine the effects of varying concentrations of dissolved solids (measured as salinity) and particulate solids (measured as turbidity) of the chemical's ability to coagulate the desired pollutants. A microsand, made of selectively sieving local soil to obtain particles between 75 and 150 μm in diameter, was used to adjust the particulate solids content of the mixture. Typical additions of the microsand were 120 mg/L, which produced turbidity values of about 22 NTU. The microsand is an inert particulate which is sometimes used to enhance coagulation by weighting the chemical floc, resulting in increased settling rates. A concentrated solution of NaCl was also used to adjust the dissolved solids content of the test water to examine stormwater treatability when influenced by snowmelt. The final chemical tests were conducted using proprietary compounds, mostly blends of alum or iron compounds with organic polymers. These tests were only conducted with a single initial turbidity (microsand addition which produced 22 NTU turbidity) and salinity (about 1600 μS/cm) condition. The following lists the coagulant chemicals that were used in the tests:

Ferric chloride
DEPHOS-A (Aluminum sulfate (alum) and sodium bicarbonate buffer)
C-1015 Polyamine/alum blend
C-1325 Organic/Ferric sulfate blend
Accu-Clear (liquid alum slurry)
C-1150H Polyamine blend

C-318P Cationic Polymer
C-2238 Organic/aluminum chloride blend
C-1150 Polyamine blend
C-1025 Polyamine/alum blend

The measured amounts of clay and salt were added to each sample and initially stirred. The test chemicals were then added, with rapid mixing at 130 rpm for 2 minutes. Flocculation was encouraged by stirring at 30 rpm for 2 minutes. Setting was then allowed for 15 minute before the samples were withdrawn for analyses. The initial stormwater sample solution had the following concentrations for all of the tests: lead and copper (35 to 200 μg/L), copper (75 to 200 μg/L), phosphate (200 to 340 μg/L), and ammonia (60 to 1100 μg/L).

<u>Ferric chloride</u>. This test evaluated ferric chloride (purchased from HACH in a 40 gram/L stock solution). The recommended pH range of the water for ferric chloride coagulation is 6 to 9 pH. Before the tests, the pH of the water samples were 7.5. After the coagulation addition, the pH dropped to the range of 6.4 to 7.2. The design was divided into three subsets, each having the same ferric chloride concentration (either 25, 50, or 75 mg/L). Each set included 4 beakers, covering all combinations of high and low turbidity (3 and 24 NTU), plus high and low conductivity (500 and 1600 μS/cm), in a full 2^2 factorial experimental design.

There was good floc formation and fast setting was observed during this run for all concentrations of ferric chloride coagulant tested (25 to 75 mg/L). The percentage turbidity removals were very sensitive to the initial turbidities of the test waters. The high turbidity water (22 NTU) had the greatest turbidity reductions (87 to 95%), while the low turbidity test water (1.6 NTU) had minimal turbidity changes. The final turbidities (after the coagulation tests) were all less than 2 NTU. Initial turbidity conditions (affected by the presence of the microsand) did not affect the removal of any of the other pollutants examined. Salinity did not affect the removals of any of the pollutants examined during the ferric chloride tests. Figure B-25 shows the trends of some of the analyzed pollutants. The optimal dosage of ferric chloride was not clear during these tests, as all concentrations in the test range produced excellent metal control.

- pH decreased with increasing $FeCl_3$ dosage. The lowest pH of the test water was 6.4 (compared to an initial pH before the chemical addition of 7.5).
- Turbidity decreased as the dosage increased. Removals of 90+% were observed at ferric chloride dosages from 25 to 75 mg/L.
- lead, copper, and phosphate concentrations also decreased with increasing dosages. Lead was reduced by about 98%, copper by 90+% and phosphate by about 44% at dosages from 25 to 75 mg/L.

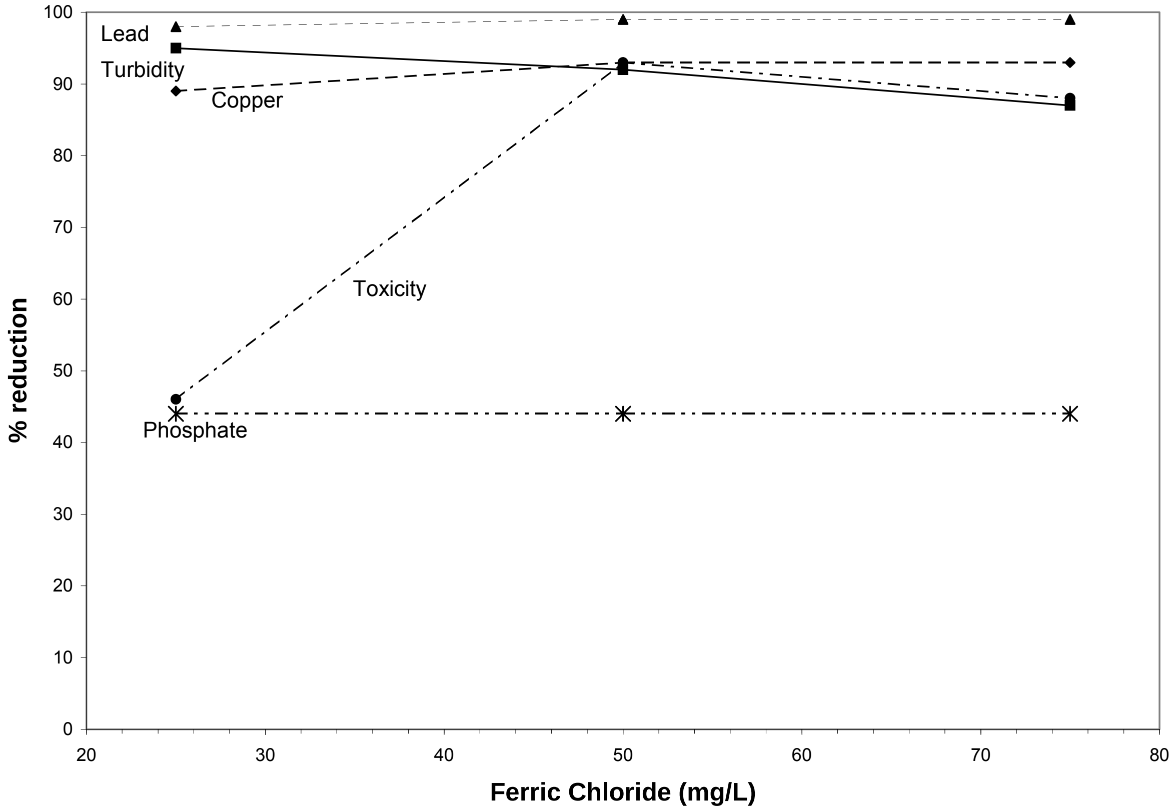


Figure B-25. Results using ferric chloride additions.

Buffered aluminum sulfate. These tests examined aluminum sulfate (alum) with a sodium bicarbonate buffer (added first). Both reagents were supplied as the commercial product DEPHOS-A (from Aquatic Eco-Systems, Inc., Apopka, FL). The buffer and alum were applied in equal concentrations, as recommended. These tests examined both low coagulant dosages (3.5 to 10.5 mg/L), and higher coagulant dosages (35 to 105 mg/L) during separate experiments.

A buffered alum was tested during these runs over a wide range of coagulant concentrations (3.5 to 105 mg/L). As for ferric chloride, the percentage turbidity removal was very sensitive to the initial turbidities of the test waters for these alum tests. The high turbidity water (22 NTU) had the greatest turbidity reductions (37 to 83%), while the low turbidity test water (1.5 NTU) had increases in turbidity (to about 3 to 9 NTU). Initial turbidity conditions (affected by the presence of the microsand) did not affect the removal of any of the other pollutants examined. Salinity also did not affect the removals of any of the pollutants examined during these buffered alum tests. Because of the lack of noticeable effects of turbidity and salinity on the removal of the pollutants, the further coagulant tests only examined single turbidity (about 22 NTU) and salinity (about 1600 µS/cm) conditions with more coagulant concentrations. Figure B-26 shows removal plots for this coagulant. The optimal dosage for this buffered alum coagulant is about 70 mg/L.

- pH remained relatively constant and did not change with the addition of the buffered alum.
- Turbidity reductions improved with increasing coagulant concentrations, up to about 83% reductions at 70 mg/L.
- Copper and lead removals improved with increasing coagulant dosages, up to about 70 mg/L alum. The removal rates were about the same for the highest dosage (105 mg/L) examined. The best copper removals were about 86%, and the best lead removals were about 97%.

• toxicity reductions were as high as 93% at 70 mg/L alum dosage, but the toxicity of the water increased at the highest alum dosage. All samples (except 1, 3 and 13) were transferred into Imhoff cones for longer settling periods. The supernatant from the Imhoff cones were then analyzed for toxicity screening (using the Microtox™ procedure) after 15 minutes and 24 hours of settling. The increased settling period significantly decreased the water toxicity.

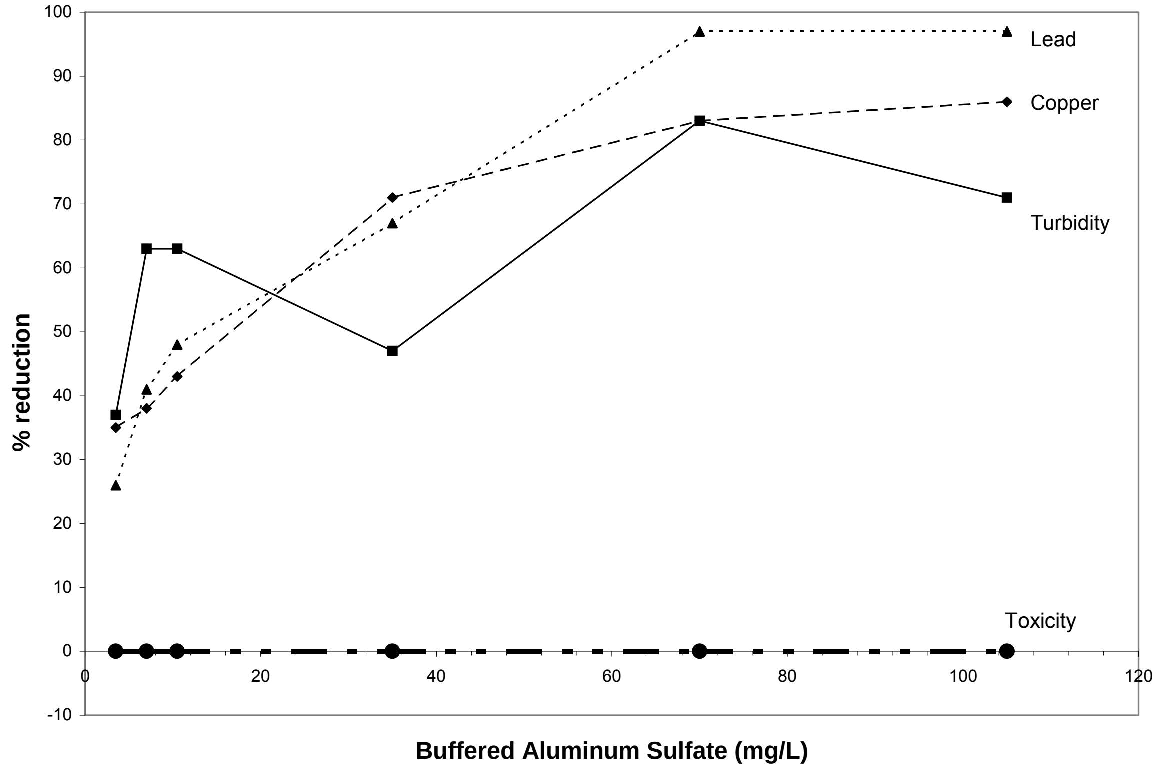


Figure B-26. Results using buffered aluminum sulfate (low and high conc.)

C-1015, polyamine/alum blend solution. This test examined a Polyamine/alum blend (C-1015 from Polydyne, Inc.). This material was advertised as being effective for color removal, phosphate removal, and for the treatment of oily waters. It is a combination of organic and inorganic coagulants. These coagulant tests were only conducted with a single turbidity and salinity. Turbidity, COD and phosphate all worsened with increasing dosages, associated with increasing coagulant concentrations. The pH remained the same during the tests (7.2 to 7.4). The optimal dosage for this coagulant was found to be about 15 mg/L.

• Turbidity was reduced by up to 27% at a dosage of 8 mg/L, after which the turbidity increased (up to 43 NTU) because of the large dosages of coagulant (up to 40 mg/L).
• Toxicity removal (67% reduction) was best at a 15 mg/L coagulant dosage, then the removal rate slightly decreased (to about 50%) at the highest coagulant dosage.
• Copper and lead were removed at rates of about 20 to 40% (copper) and 33 to 51% (lead), with slight increasing removals at higher coagulant concentrations.
• COD increased to 8 to 30 mg/L at higher coagulant dosages (after 15 mg/L).
• phosphate also increased to about 1.2 mg/L at the highest coagulant dosages (40 mg/L)

C-1325, organic/ferric sulfate blend solution. This test examined an organic ferric sulfate blend (C-1325, also from Polydyne, Inc.). All samples contained microsand and salt (high turbidity and high conductivity). Only high turbidity (22 NTU) and high salinity (1600 µS/cm) conditions were used for this test. The phosphate and COD levels also notably increased at high coagulant dosages. The pH remained the same during the tests (7.3 to 7.5). This coagulant was not found to be very suitable during these tests due to the generally low turbidity, toxicant, and metal removal rates observed, plus the large observed increases in phosphate and COD.

• turbidity was reduced by about 20 to 30% at low dosages, but the turbidity increased up to 82 NTU at a coagulant dosage of 40 mg/L.
• toxicity showed a general worsening (by about 20 to 30%) with this coagulant.
• copper decreased by about 30 to 40% and did not show any clear pattern with changing coagulant dosage.
• lead showed larger decreases (by 47 to 84%) with increasing dosages.
• phosphate increased to as high as 3.1 mg/L at the highest coagulant dosage (40 mg/L).
• COD also showed increases (up to 85 mg/L) with higher dosages.

Accu-Clear, an alum slurry. This test examined Accu-Clear (an alum slurry from Aquatic Eco-Systems). The Accu-Clear coagulant concentrations ranged from 0.01 to 0.16 mL/L. All samples contained microsand and salt (high turbidity and high conductivity). The pH generally remained the same during these tests (7.3 to 7.6). There was no noticeable change in phosphate (at 0.26 mg/L) or COD (not detected) with this material. The optimal Accu-Clear dosage was found to be about 0.08 mL/L during these tests.

• turbidity was reduced by 50 to 88%, increasing in removal with increasing dosages, but leveling off at about 0.08 mL/L.
• toxicity reductions were as high as 100% at 0.05 to 0.08 mL/L, but decreased to 66% at 0.16 mL/L coagulant dosage.
• copper reductions of 70 to 100% occurred between 0.02 and 0.16 mL/L.
• lead reductions were 80 to 100% also between 0.02 and 0.16 mL/L.

C-1150H, polyamine blend solution. This test examined a polyamine blend solution (C-1150H from Polydyne, Inc.). The C-1150H coagulant concentrations ranged from 3 to 40 mg/L. The C-1150H is a combination of organic and inorganic coagulants that performs better in low alkalinity water and in water with a high salt content. All samples contained microsand and salt (high turbidity and high conductivity). The pH dropped with increasing dosages (from 7.5 to 6.7 at 40 mg/L coagulant dosage). Both phosphate and COD had large increases with increasing coagulant dosages. The optimal C-1150H coagulant dosage was found to be about 15 mg/L during these tests.

• turbidity was reduced by 60 to 90%, with better results with higher concentrations (leveling off at about 15 mg/L).
• toxicity increased by about 40% with coagulant additions.
• copper was reduced by as much as 18% at the highest coagulant dosage.
• lead was reduced by about 95% with coagulant dosages greater than 15 mg/L.
• phosphate had large increases after dosages of about 15 mg/L (up to 2.4 mg/L at the highest coagulant dosage of 40 mg/L).
• COD had large increases after dosages of about 8 mg/L (up to 120 mg/L at the highest coagulant dosage of 40 mg/L).

C-318P, cationic polymer solution. This test examined a low molecular weight, highly cationic organic coagulant (C-318 from Polydyne, Inc.). All samples contained added microsand and salt (high turbidity, high conductivity). The pH slightly increased during these tests, from 6.3 to 7.2 before coagulant addition, to 7.5 to 7.7 after the coagulant test. Large increases in phosphate and COD were observed at all coagulant dosages tested (8 to 100 mg/L). No floc formation was observed with this coagulant. No metals were analyzed during these tests. This coagulant was not found to be very suitable during these tests due to the generally low turbidity and toxicant removal rates observed, plus the large observed increases in phosphate and COD.

• turbidity decreased by 50 to 60% at all dosages tested, with no apparent trend with coagulant dosage.
• toxicity had a slight improvement (by about 6 to 7%) at all dosages of 15 mg/L and greater.
• phosphate increased to as high as 2.3 mg/L during these tests (low of 1.1 mg/L phosphate at 8 mg/L coagulant dosage).
• COD increased to as high as >200 mg/L during these tests (low of 63 mg/L COD at 8 mg/L coagulant dosage).

<u>C-2238, polmethydiallyl ammonium chloride/aluminum chloride blend.</u> This test examined an organic/aluminum chloride blend (C-2238, from Polydyne, Inc.). The C-2238 is an organic/aluminum chloride blend that is advertised as being effective for color removal, phosphorus removal, and treatment of oily waters. All samples contained added sand and salt (high turbidity, high conductivity). The pH decreased during these tests, from 7.5 to 6.4 at the highest coagulant dosage (100 mg/L) and from 7.4 to 6.8 at a more modest dosage of 25 mg/L, for example. Large increases in phosphate and COD were observed at coagulant dosages greater than 15 mg/L. Large flocs and clear setting were observed with this coagulant. No metals were analyzed during this test. The optimal dosage for this coagulant was found to be about 8 to 15 mg/L.

• turbidity reductions were good (about 90%) at low coagulant dosages (8 to 40 mg/L). The turbidity then increased with increasing coagulant dosages.
• toxicity had very good removals (80 to 100%) at all dosages tested (8 to 100 mg/L).
• phosphates increased in concentration with increasing dosages, especially at dosages greater than 25 mg/L. At 100 mg/L coagulant, the phosphate concentration was 2.3 mg/L.
• COD also had increased concentrations with all dosages, from 10 mg/L at the lowest dosage (8 mg/L) to 86 mg/L at the highest dosage (100 mg/L).

<u>C-1150, polyamine blend solution.</u> A polyamine blend (C-1150 from Polydyne, Inc.) was also tested. The C-1150 is a polyamine blend solution and is a combination of organic and inorganic coagulants. It is supposed to perform better in waters having low alkalinity and with high salt contents. The concentrations of the coagulant ranged from 8 to 100 mg/L during the tests. The pH only changed slightly, from 7.7 to 7.8 before chemical addition, to 7.4 to 7.5 after chemical addition. No metals, nutrients, or COD were analyzed. The optimal coagulant dosage may be between 40 and 65 mg/L, depending on potential increases in other pollutants at these relatively high dosages.

• turbidity had good reductions (70 to 90% removals) at coagulant dosages between 8 to 65 mg/L.
• toxicity had moderate removals (40 to 50% reductions) at coagulant dosages between 40 and 100 mg/L.

<u>1025, polyamine/alum blend solution.</u> A polyamine/alum blend (C-1025, from Polydyne, Inc.) was tested. The C-1025 is a polyamine/alum blend for color removal, phosphate removal, and the treatment of oily waters. The concentrations of the coagulant ranged from 8 to 100 mg/L. The pH only changed slightly, from 7.7 to 7.8 before chemical addition, to 7.2 to 7.5 after chemical addition. No metals, nutrients, or COD were analyzed.

The optimal coagulant dosage may be between 15 and 40 mg/L, depending on potential increases in other pollutants at these relatively high dosages.

• turbidity had moderate reductions (20 to 45%) at low dosages (8 to 15 mg/L), but increased at high coagulant dosages (up to 47 NTU at 100 mg/L dosage).
• toxicity had good removals (50 to 65%) at moderate dosages (15 to 65 mg/L).

<u>Summary of Results of Chemical Addition Tests.</u> Ferric chloride, with a microsand additive, was the most successful coagulant and aid for rapid treatment over a wide range of water conditions. A wide range of ferric chloride levels produced excellent reductions of heavy metals (lead >99%, copper >90%) and phosphate (about

50%) within about 10 minutes. In addition, the toxicity (measured using the Microtox™ screening method) also indicated >80% reductions. No adverse changes were observed with ferric chloride. Other major chemicals tested included alum (with and without organic polymers), ferric sulfate, organic polymers, plus several priority mixtures (most likely mixtures of alum and polymers). None of these other chemicals produced results as good as the ferric chloride, and some added large amounts of phosphates and COD to the test water.

Pilot-Scale Treatability Tests

Based on the previous bench-scale chemical addition tests, Pitt, *et al*. (1998) further examined chemical coagulation and/or precipitation, followed by physical treatment to remove any pumped floc, and possibly ion exchange or sorption, and disinfection, as a treatment train during field pilot-scale testing. The pilot-scale tests used a specially constructed treatment train having the following components:

1) initial 100 µm membrane filter bag
2) 25 µm membrane filter bag
3) 5 µm membrane filter bag
4) activated charcoal filter
5) UV disinfection unit
6) final cascade aerator

Seven sampling locations were located on this treatment train, an initial sample before any treatment, and six sampling locations after each of the above six unit processes. In addition, chemical addition (using either ferric chloride or alum) was used before this treatment train, resulting in additional samples. The pilot-scale treatment train tests were performed at four locations in order to measure the performance of the different unit processes under a variety of actual conditions.

Mixing greatly increased the hydrocarbon concentrations in the water by apparently disturbing hydrocarbons in the sediment at the Bessemer test site. Only 45 grams of ferric chloride was added to the 8,200 L of water, resulting in a low concentration of only about 5.5 mg/L. Even at this low ferric chloride dosage, a yellowish floc was visible forming in the water. About 400 grams of ferric chloride should have been used to obtain a coagulant concentration of about 50 mg/L. This test site included two test series: the initial "unmixed, no coagulant treatment" and the "coagulant treatment."

Mixing of the water had minimal effects on water hydrocarbon concentrations, which were relatively uniform before any mixing, at the Oxmoor test site. A total of 90 grams of ferric chloride was added to the 8900 L of water, resulting in a low coagulant dosage of only about 10 mg/L. About 450 grams of ferric chloride would have been needed to obtain a ferric chloride coagulant dosage of 50 mg/L for this site.

Mixing of the water had no apparent effects on water hydrocarbon concentrations, which were relatively uniform before any mixing, at the Morris Ave. test site. About 150 mL of an alum slurry, Accu-Clear, was added to the 2800 L of water, resulting in a dosage of about 0.05 mL/L (the optimal Accu-Clear dosage has since been determined to be about 0.08 mL/L). After the Accu-Clear addition and mixing, a noticeable fine floc (about 50 to 150 µm in size) was visible, having a light gray color.

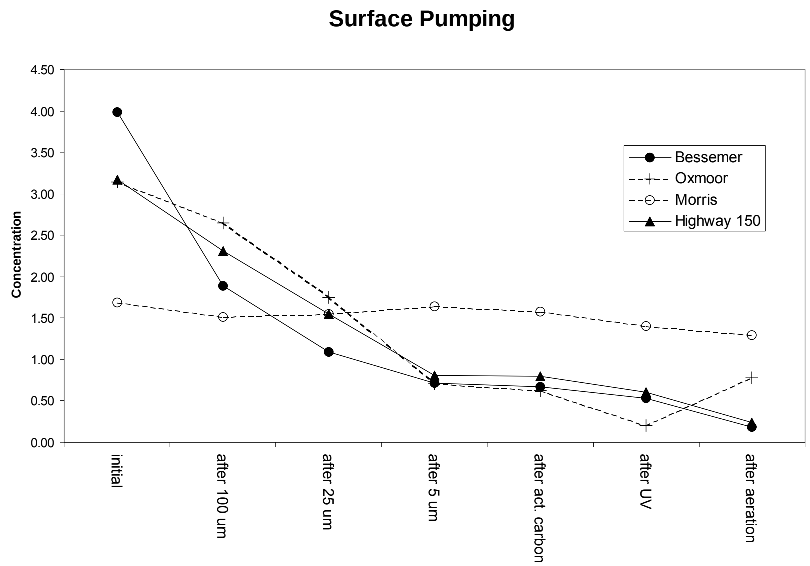
At the Hwy 150, about 300 mL of an alum slurry, Accu-Clear, was added to the 5900 L of water, resulting in a dosage of about 0.05 mL/L.

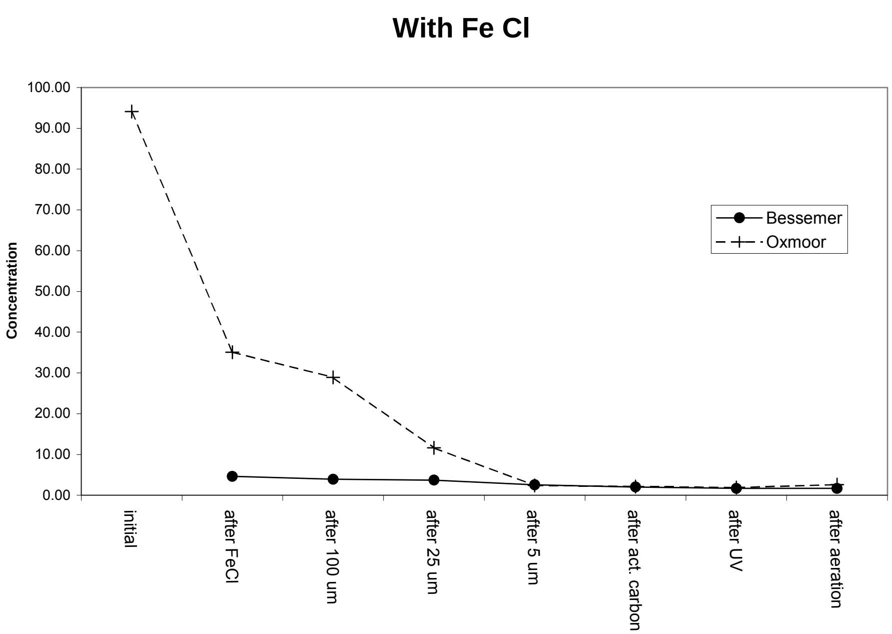
Pilot-Scale Treatment Performance Results. Figures B-27 through B-30 show the treatment train results for turbidity, copper, lead, and zinc. These figures are plots of concentration changes at different treatment steps. Each figure is divided into up to four plots, with appropriate data trends shown: surface pumping, mixed, with $FeCl_3$, and with alum.

The “mixed” plots were prepared using data from samples collected while treating the water during, or immediately after, complete mixing of the water, including re-suspending some of the settled sediment. The sampling pump was disconnected from the treatment train and the short hose was used to mix the water for 5 to 10 minutes. The mixed water was directed through the treatment train for 5 to 10 minutes before sampling started.

The “with $FeCl_3$” plots were prepared from data from samples collected during treatment tests that used ferric chloride additions to the water. After the “mixed” treatment sampling series was completed, ferric chloride was added to the water and completely mixed for another 5 minutes using the sampling pump. The water was then allowed to settle for 15 minutes before the pump was placed barely below the water surface to deliver water to the treatment train. The “initial” data point therefore is the same as the “initial” data point from the “mixed” plot and the “after $FeCl_3$” data point was obtained from the first sampling port on the treatment train. As before, the water was pumped through the treatment train for 5 to 10 minutes before the composite sampling started. The “with alum” plots were prepared from data collected identically as the above “with $FeCl_3$” plots, except that alum was added to the water in the manhole before the treatment train.

Figure B-27. Turbidity (NTU) Pilot-Scale Test Results





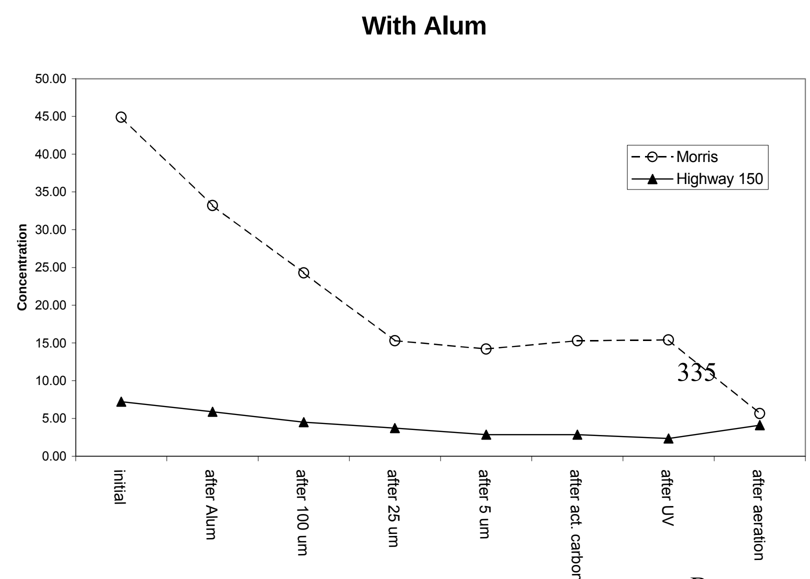
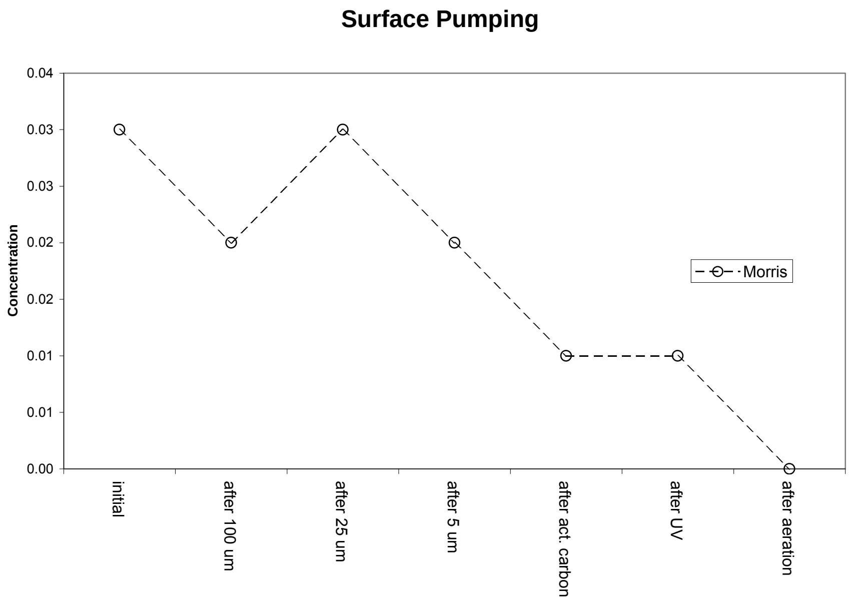
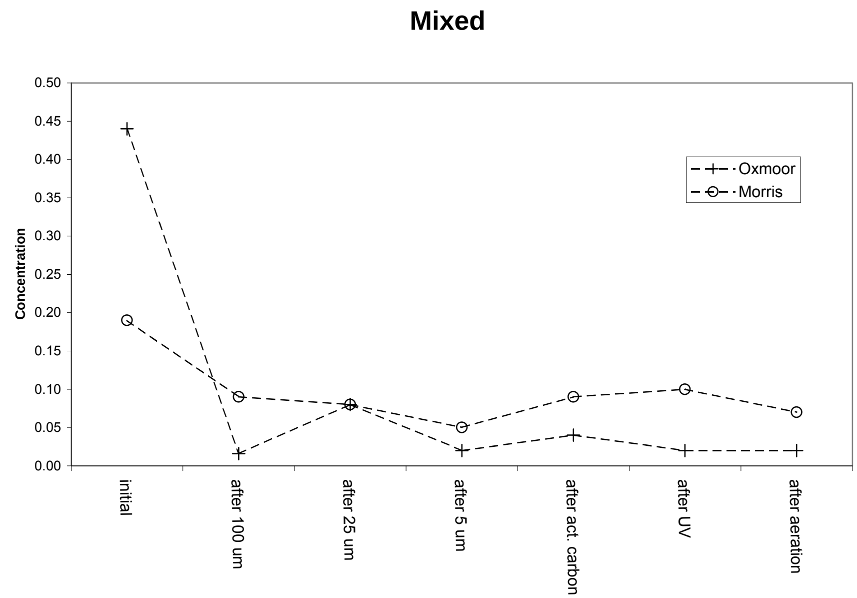
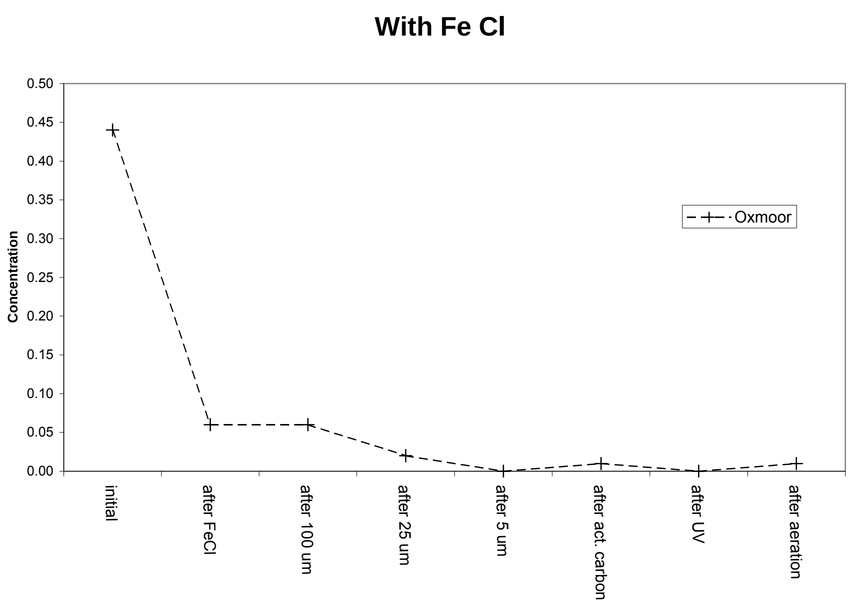


Figure B-28. Copper (mg/L) Pilot-Scale Test Results







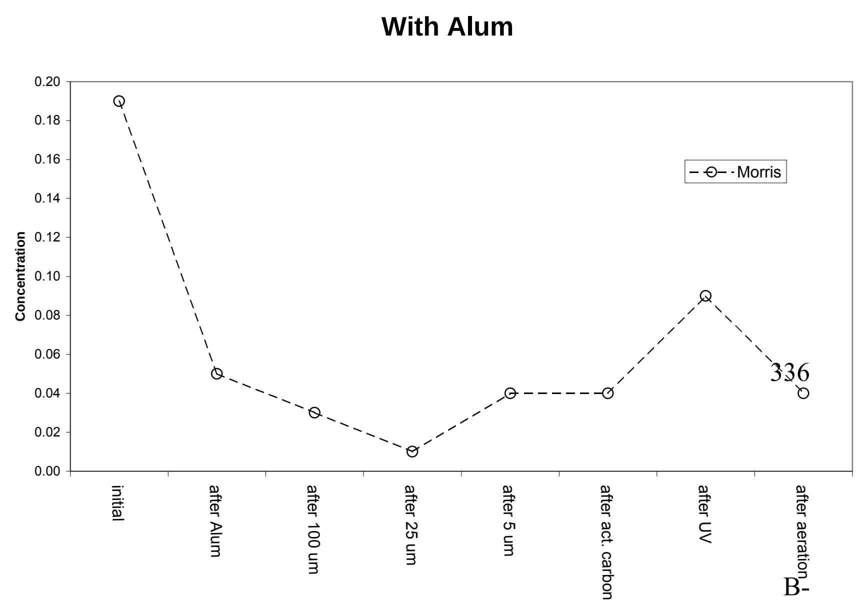
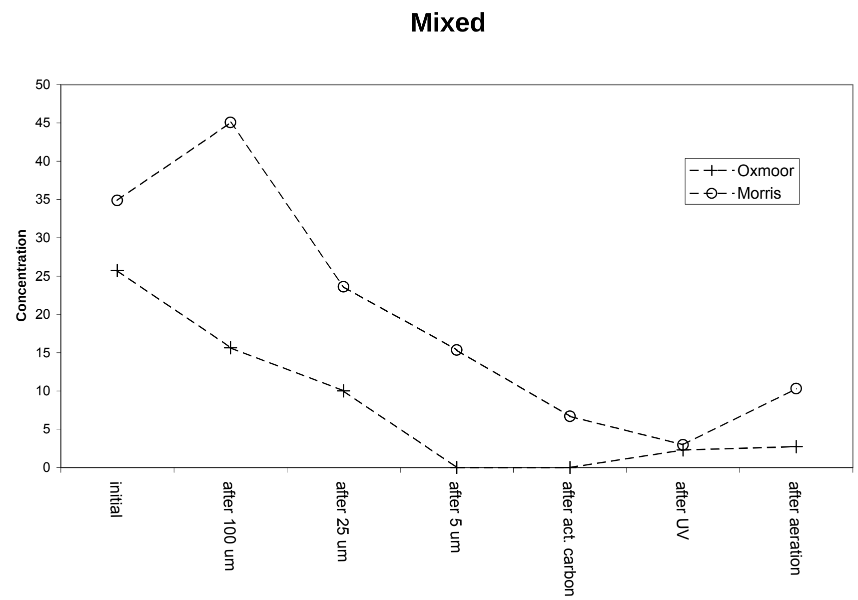
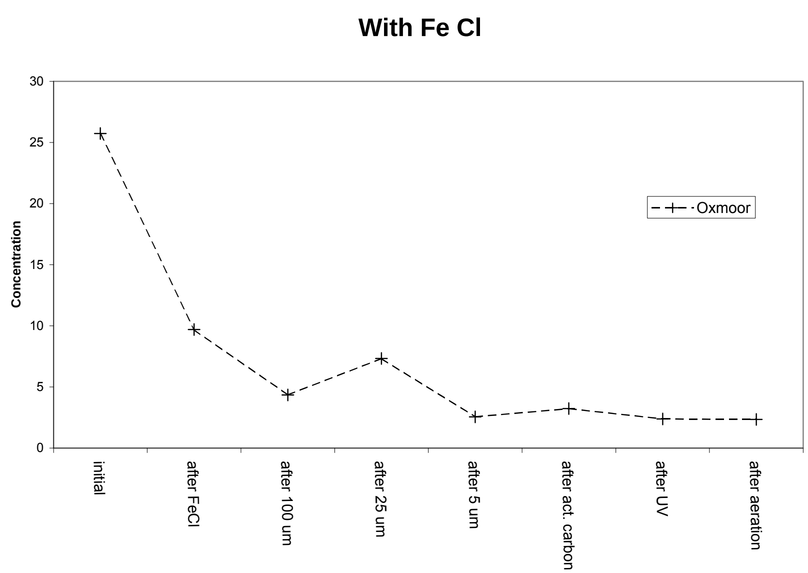


Figure B-29. Lead (µg/L) Pilot-Scale Test Results





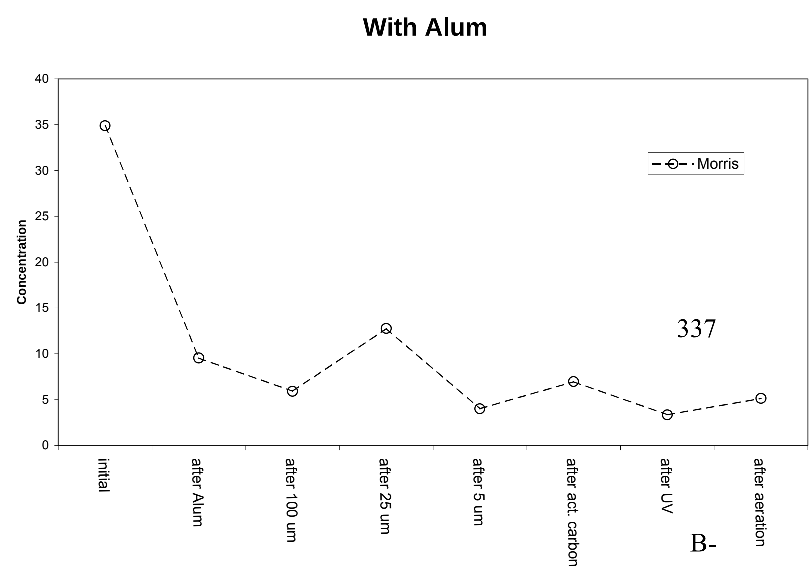
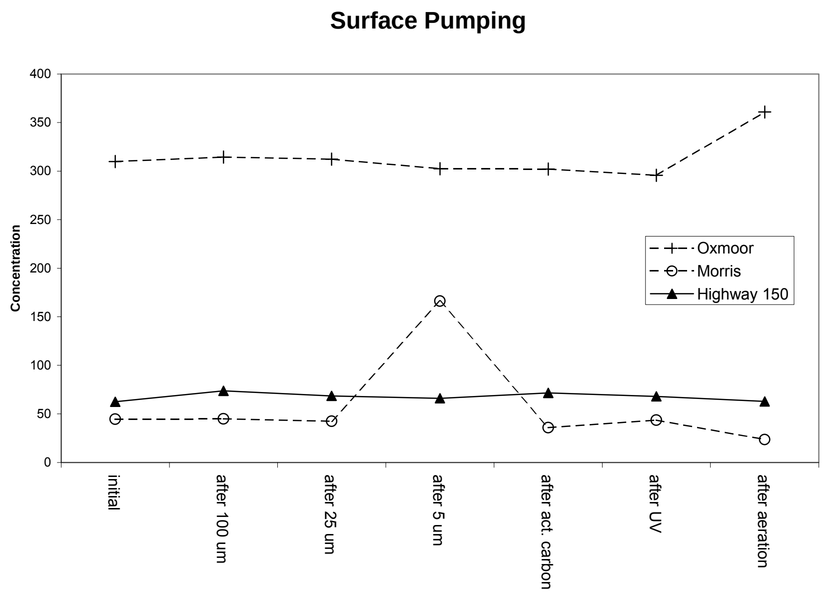
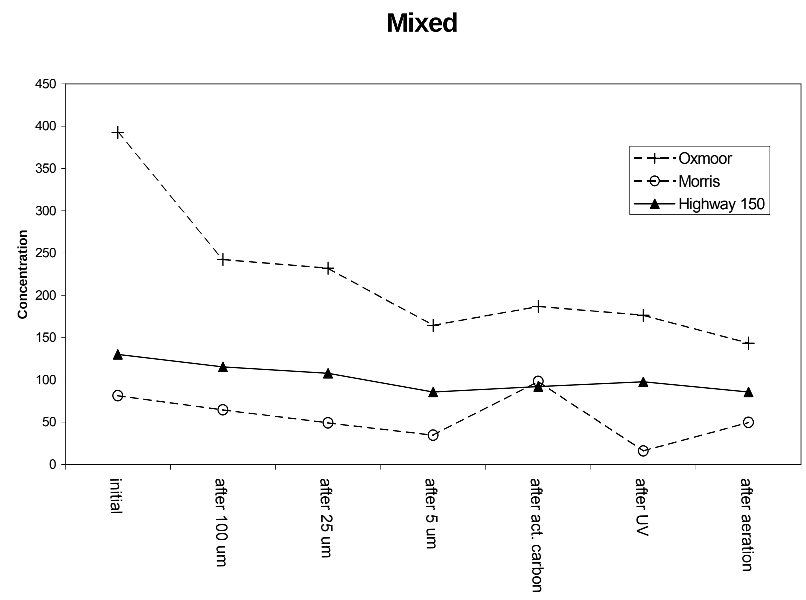
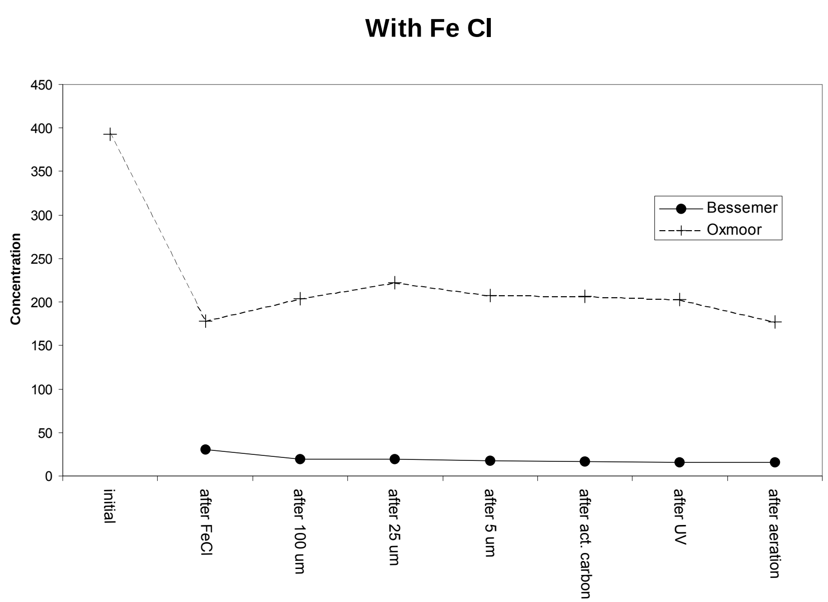


Figure B-30. Zinc (μg/L) Pilot-Scale Test Results







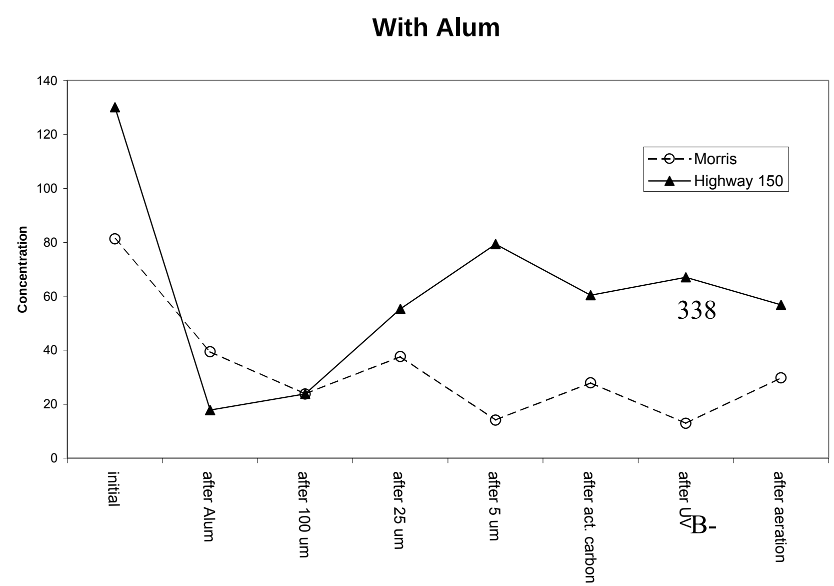


Table B-3 summarizes the levels of treatment obtained during these tests. Several pollutants significantly increased in concentration when the water was aggressively stirred with the pump during the "mixed" water tests due to re-suspension of sediment. The Oxmoor and Morris test sites showed the largest effects, with lead, copper, turbidity, and ammonia increasing by at least five times in concentration (some by more than 30 times). Color and COD also significantly increased in at least one of these two test locations during mixing. Zinc, conductivity, pH, fluoride, and phosphate, plus the bacteria, had much smaller changes during mixing. The particle size analyses showed significant decreases in particles as the treatment progresses in the treatment train from the initial samples through the 5 μm filter bag. It is interesting to note that the bag filters did not act as cut-off filters at their rated pore sizes (100, 25, and 5 μm), but appear to reduce particles in a broad range of sizes. This is most likely due to depth filtering in the felted bags, plus the effects of a filter cake that would build up on the inside of the bag filter, reducing the effective aperture size. These performance tests indicate that water treatment with ferric chloride and final filtering through a 5 μm filter offers a high level of heavy metal reductions. Any additional treatment only provided small marginal benefits.

Table B-3. Effective Treatment Levels (with concentrations and reductions)

Turbidity (NTU)	**Bessemer**	**Oxmoor**	**Morris**	**Hwy 150**
Surface pumping	after 5 μm (0.7 NTU, 82% reduction)	after 5 μm (0.7 NTU, 78% reduction)	little change (all 1 to 2 NTU)	after 5 μm (0.8 NTU, 75% reduction)
Mixed	na	after 5 μm (16 NTU, 83% reduction)	after 5 μm (15 NTU, 66% reduction)	after 5 μm (2.5 NTU, 65% reduction)
With $FeCl_3$	little change (all 2 to 5 NTU)	after 5 μm (2.4 NTU, 97% reduction)	na	na
With alum	na	na	after 25 μm (15 NTU, 66% reduction)	little change (all 4 to 8 NTU)
Copper (mg/L)	**Bessemer**	**Oxmoor**	**Morris**	**Hwy 150**
Surface pumping	na	na	after activated carbon (0.01 mg/L, 67% reduction)	na
Mixed	na	after 100 μm (0.02 mg/L, 96% reduction)	after 5 μm (0.05 mg/L, 74% reduction)	na
With $FeCl_3$	na	after $FeCl_3$ (0.06 mg/L, 86% reduction)	na	na
With alum	na	na	after alum (0.05 mg/L, 74% reduction)	na
Lead (μg/L)	**Bessemer**	**Oxmoor**	**Morris**	**Hwy 150**
Surface pumping	na	na	na	na
Mixed	na	after 5 μm (0 μg/L, 100 reduction)	After activated carbon (7 μg/L, 81% reduction)	na
With $FeCl_3$	na	after 100 μm (4 μg/L, 83% reduction)	na	na
With alum	na	na	after alum (10 μg/l, 73 % reduction)	na
Zinc (μg/L)	**Bessemer**	**Oxmoor**	**Morris**	**Hwy 150**
Surface pumping	na	increase (from 310 to 340 μg/L)	little change (all 24 to 166 μg/L)	little change (all 63 to 74 μg/L)
Mixed	na	after 5 μm (164 μg/L, 58 % reduction)	after 5 μm (35 μg/L, 57 % reduction)	after 5 μm (86 μg/L, 34 % reduction)
With $FeCl_3$	little change (all 16 to 30 μg/L)	after $FeCl_3$ (178 μg/L, 55 % reduction)	na	na
With alum	na	na	after 100 μm (24 μg/L, 71% reduction)	after alum (18 μg/L, 86% reduction)

Pilot-scale detention pond at Lincoln Creek side-stream toxicity test lab, Milwaukee, WI

Pilot-scale tests using different cartridge filters, along with sorption materials, disinfection, and aeration, Birmingham, AL

Pilot-scale filtration setup for post-sedimentation treatment tests

Figure B-31. Various pilot-scale treatability set-ups.

Appendix C: Use of the NRCS WinTR-55 Method

General Description of TR-55 for Small Watersheds

The complete User Guide for TR-55 (1986 version) can be downloaded from: http://www.wcc.nrcs.usda.gov/water/quality/common/tr55/tr55.pdf. According to the NRCS (2002), Technical Release 55 (TR-55) *Urban Hydrology for Small Watersheds* was first issued in January 1975 by the then named SCS (Soil Conservation Service) as a simplified procedure to calculate the storm runoff volume, peak rate of discharge, hydrographs and storage volumes required for stormwater management structures (SCS 1975). This initial version involved manual methods and assumed the Type II rainfall distribution for all calculations. In June 1986, major revisions were made in TR-55 by adding three additional rainfall distributions (Type I, IA and III) and programming the computations. Time of concentration was estimated by splitting the hydraulic flow path into separate flow phases (SCS 1986). This 1986 version is the last non-computerized version and has been widely used for drainage design in urban areas.

Even though the manual version of TR-55 is currently being phased out, its use may still be of interest for many situations, especially as WinTR-55 is still an official "beta" version. In addition, the User Guide for TR-55 (SCS 1986) contains a more through description of the basic processes included in the model. A later discussion presents a description and example of the Windows version of the program.

Only the following site characteristics are needed to use TR-55: drainage area, curve number (CN), and time of concentration (Tc). With this information, it is possible to develop a hydrograph for a specific design storm. If in a complex drainage area, the watershed can be subdivided into subwatersheds for routing the flows through the system. TR-55 and WinTR-55 handle watershed routing quite differently, with WinTR-55 conducting a more through routing approach, similar to the method used in TR-20. However, the WinTR-55 "structures" module (dealing with ponds) has some serious shortcomings in the available outlet structure descriptions. The basic TR-20 is therefore recommended by NRCS when more detailed analyses are needed. The following discussion summarizes many of the basic features and approaches of TR-55 that generally are shared with WinTR-55.

Selection of the Curve Number

The first part of using TR-55 is to select the curve number. The curve number is simply the single parameter that relates runoff to rainfall. This is illustrated in Figure C-1. The following equation shows how the CN is used to calculated the runoff depth, Q in inches, from the precipitation depth, P in inches, and the curve number, CN:

$$Q = \frac{\left[P - 0.2\left(\frac{1000}{CN} - 10\right)\right]^2}{P + 0.8\left(\frac{1000}{CN} - 10\right)}$$

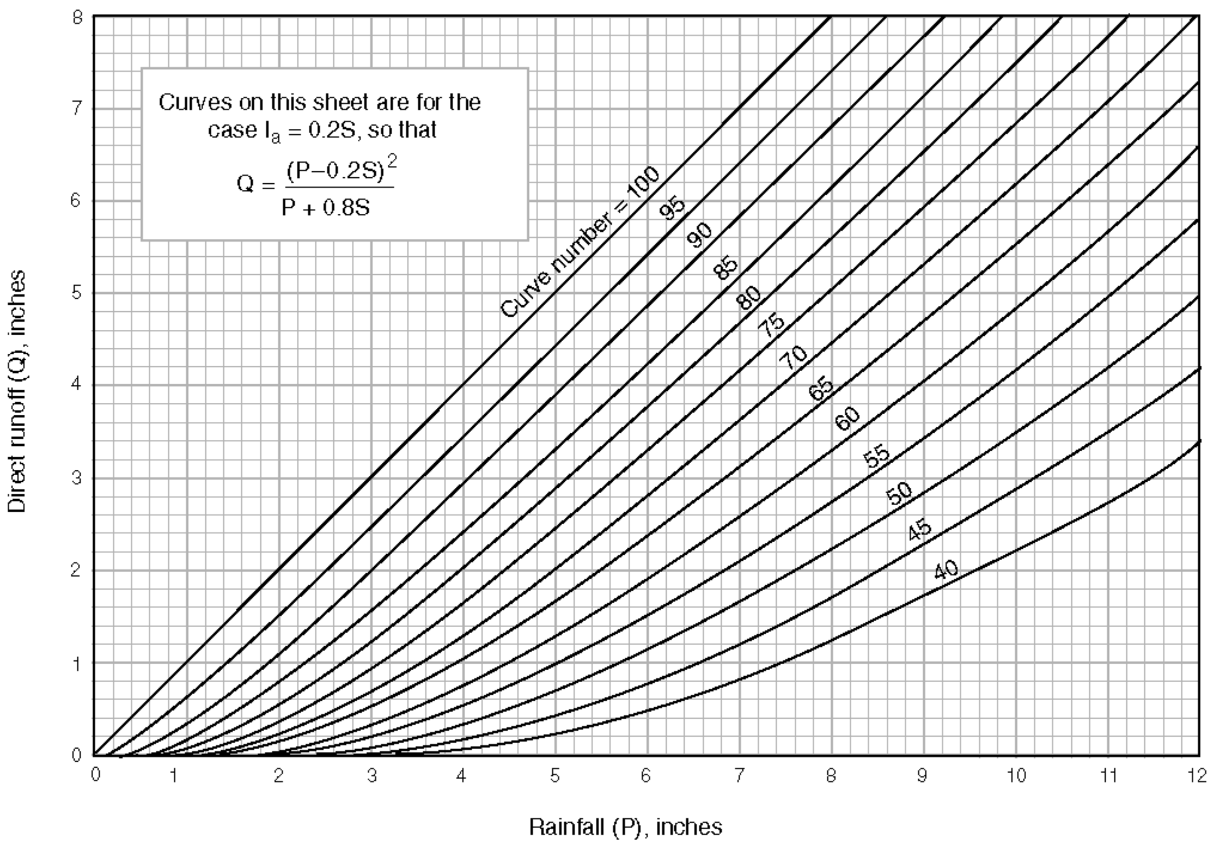


Figure C-1. Basic SCS rainfall-runoff relationship for different CN values (SCS 1986).

Tables C-1 and C-2 are used to select the most appropriate curve numbers for an area. As an example, Table C-1 shows that construction sites (newly graded areas) have curve numbers ranging from 77 for A type soils to 94 for D type soils. These are relatively high compared to typical pre-development conditions (woods ranging from 30 to 77), reflecting the increase in runoff volume during the period of construction, and the associated increased runoff rate.

Table C-1. Typical Curve Number Values for Urban Areas (SCS 1986)

Cover description: Cover type and hydrologic condition	Average percent impervious area [2]	Curve numbers for hydrologic soil group: A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

1 Average runoff condition, and $I_a = 0.2S$.

2 The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.

3 CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.

4 Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.

5 Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4 based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

Table C-2. Typical Curve Number Values for Pasture, Grassland, and Woods (SCS 1986)

Cover description: Cover type	Hydrologic condition	Curve numbers for hydrologic soil group: A	B	C	D
Pasture, grassland, or range—continuous forage for grazing. 2/	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow—continuous grass, protected from grazing and generally mowed for hay.	—	30	58	71	78
Brush—brush-weed-grass mixture with brush the major element. 3/	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30 4/	48	65	73
Woods—grass combination (orchard or tree farm). 5/	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods. 6/	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30 4/	55	70	77
Farmsteads—buildings, lanes, driveways, and surrounding lots.	—	59	74	82	86

1 Average runoff condition, and $I_a = 0.2S$.
2 *Poor:* <50%) ground cover or heavily grazed with no mulch.
Fair: 50 to 75% ground cover and not heavily grazed.
Good: > 75% ground cover and lightly or only occasionally grazed.
3 *Poor:* <50% ground cover.
Fair: 50 to 75% ground cover.
Good: >75% ground cover.
4 Actual curve number is less than 30; use CN = 30 for runoff computations.
5 CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture.
6 *Poor:* Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair: Woods are grazed but not burned, and some forest litter covers the soil.
Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

Time of Concentration Calculations

The time of concentration needs to be determined for each subwatershed in the study area. It is usually necessary to investigate several candidate flow paths in order to be relatively certain of the one that takes the longest time to reach the end of the subwatershed area. There are many different time of concentration formulas presented in hydrology textbooks, usually for different conditions and locations. The SCS/NRCS method has become relatively common recently and it is necessary to use this method when using TR-55 (and TR-20). This method separates the flow path into three segments: sheetflow, shallow concentrated flow, and channel flow. In some cases, especially for small sites, only sheetflow and possibly shallow concentrated flow may be evident. The candidate flow paths are drawn on a site topographic map; usually originate on the subwatershed boundary, and proceeding all the way to the bottom of the subwatershed. Sheetflow is usually the first element considered and normally is assumed to last for a maximum of 300ft, using a kinematic solution to Manning's equation. Some states limit its' use to even shorter lengths, such as 150 ft. WinTR-55 limits the sheetflow path length to 100ft. The flow path is then assumed to occur as shallow concentrated flow, until a designated channel on the topographic map is reached (usually taken as a designated creek or stream on a USGS quadrangle map). When several candidate flow paths are evaluated, the one with the longest travel time is assumed to represent the time of concentration for the subwatershed. If a rain lasts for that time period, runoff will therefore occur from the complete area, resulting in maximum runoff rates.

The following discussions show how the travel times are calcualted for each flow path element.

Sheetflow

The following equation (a kinematic solution to the Manning's equation) is used in the SCS procedures to calculate the travel time along the sheetflow path segment:

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5} S^{0.4}}$$

Where:

T_t = travel time (hr)
n = Manning roughness coefficient (for sheet flow)
L = flow length (ft) (maximum of 300 ft.)
P_2 = 2-year, 24-hour rainfall depth (in), and
s = slope of hydraulic grade line (land slope, ft/ft)

The sheetflow Manning's n roughness coefficient values are different from the channel lining roughness coefficients. Table C-3 lists these sheetflow values. These are all greater than the channel lining n values for the rougher surfaces, due to the shallow nature of the flows. As an example, a common channel lining n value for grass is 0.024, while the sheetflow n value for grass is 0.24, or 10 times higher. The grass has a much greater effect on flow when the flow is shallow than when the flow is deep. However, the smooth surface sheetflow n values (0.011) are very similar to the values that would be used for these surfaces in channels. This is because these smooth surfaces have a minimal effect on shallow and deeper flows due to their relatively low roughness heights. An important difference is the roughness coefficient of 0.011 for bare soils, compared to cultivated soils (with mulch covers of >20%) of 0.17, and dense grasses of 0.24. Natural woods can have n coefficients of 0.4 to 0.8, depending on the height of the underbrush. Figure C-2 includes graphs that can be used to estimate the travel time for different sheetflow conditions, calculated using the above SCS sheetflow formula, using a P_2 value of 4.2 inches (appropriate for Birmingham, AL). If the P2 ratio is not 4.2 inches, the Figure C-2 values can be adjusted using the above sheetflow equation.

Table C-3. Sheetflow Manning's Equation Roughness Coefficients (SCS 1986)

Surface Description	Sheetflow Roughness Factor, n
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤ 20%	0.06
Residue cover >20%	0.17
Grass:	
Short grass prairie	0.15
Dense grass[1]	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods[2]	
Light underbrush	0.40
Dense underbrush	0.80

[1] includes species such as weeping lovegrass, bluegrass, buffalo grass, blue gama grass, and native grass mixtures
[2] When selecting n for woods, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

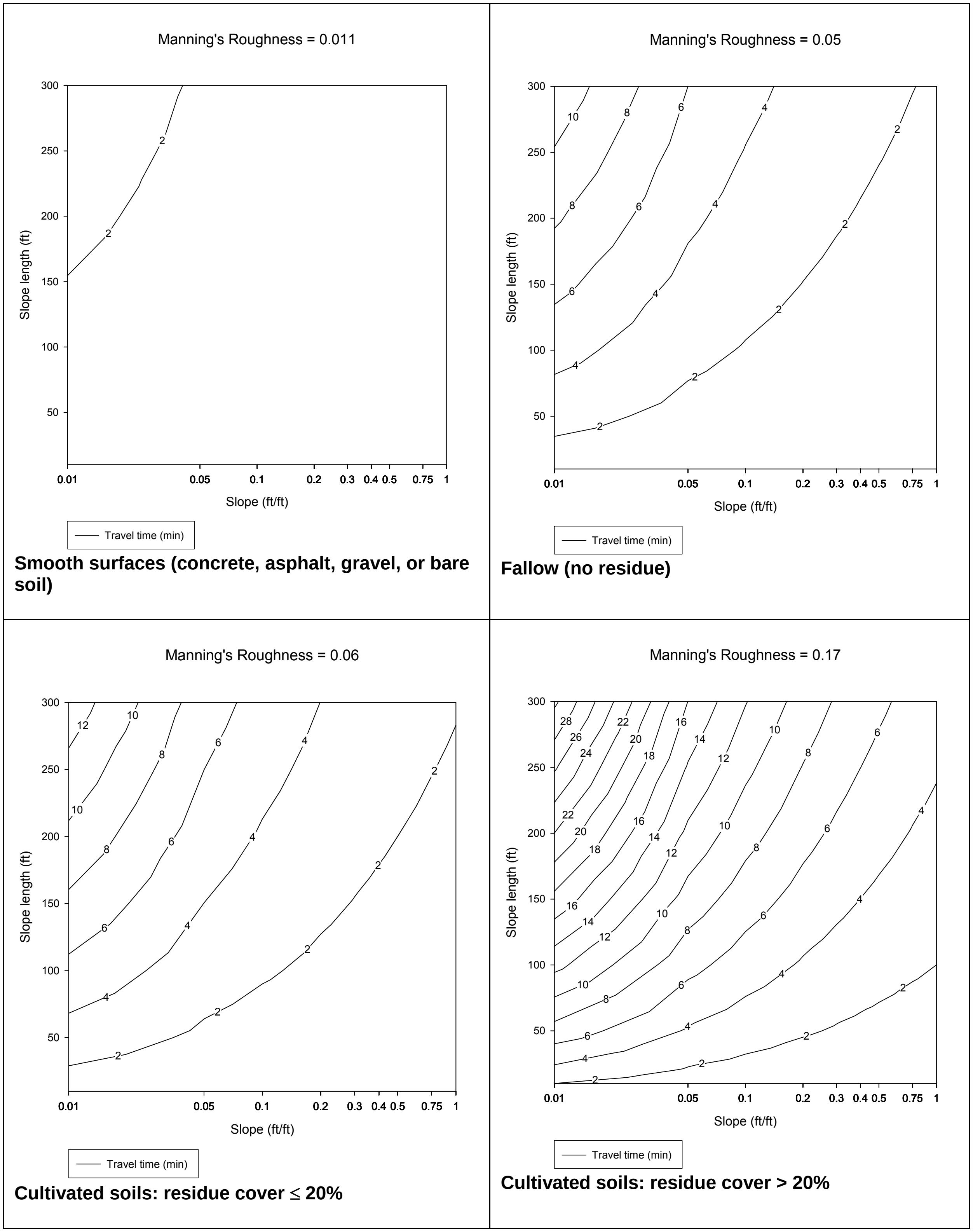
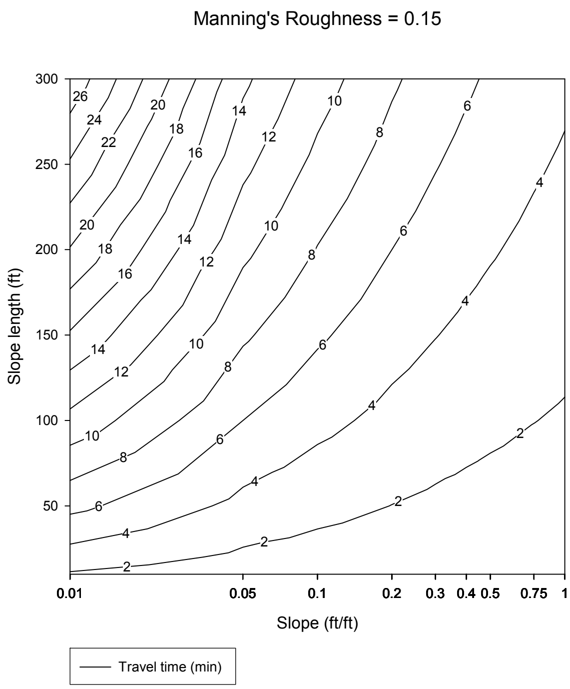
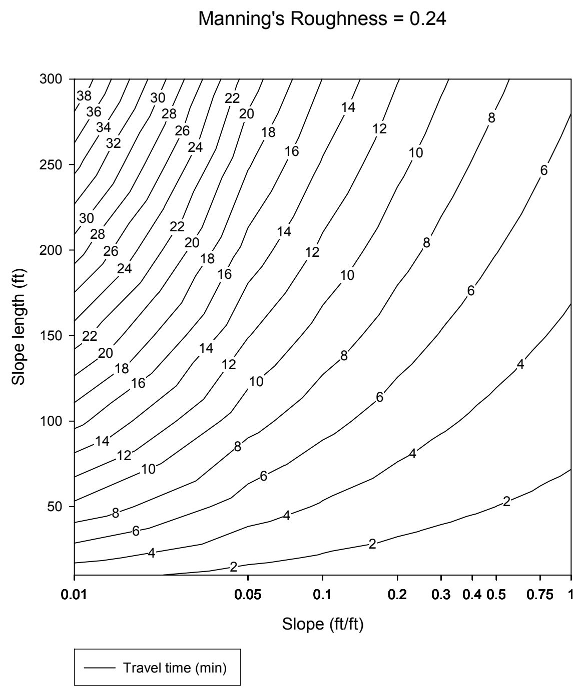


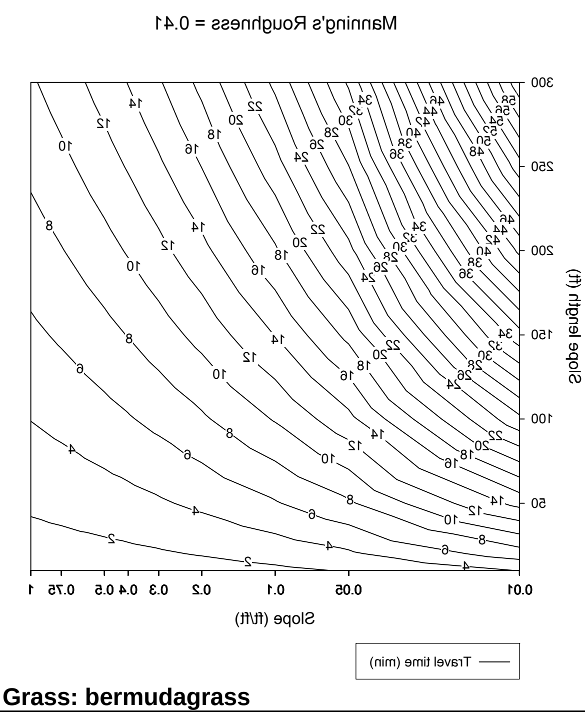
Figure C-2. Sheetflow travel times (using a P_2 value of 4.2 inches).



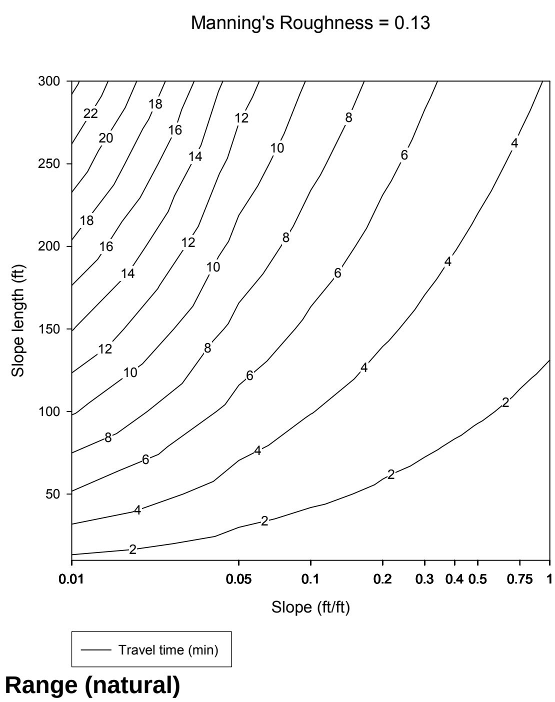
Grass: short grass prairie



Grass, dense (weeping lovegrass, bluegrass, buffalo grass, blue gama grass, and native grass mixtures

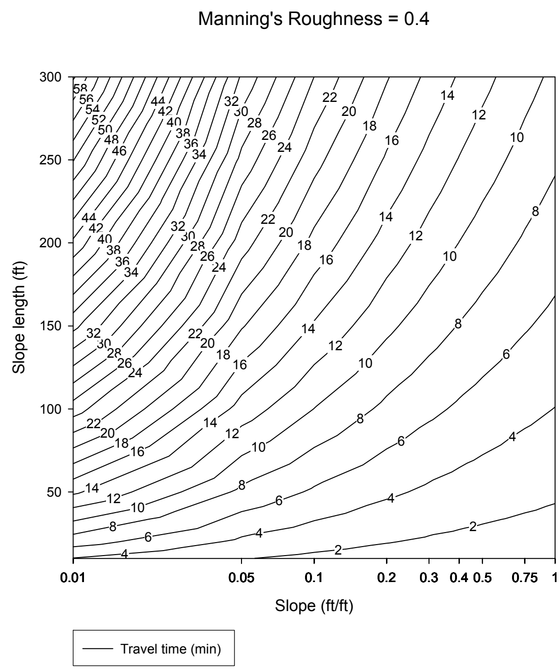


Grass: bermudagrass

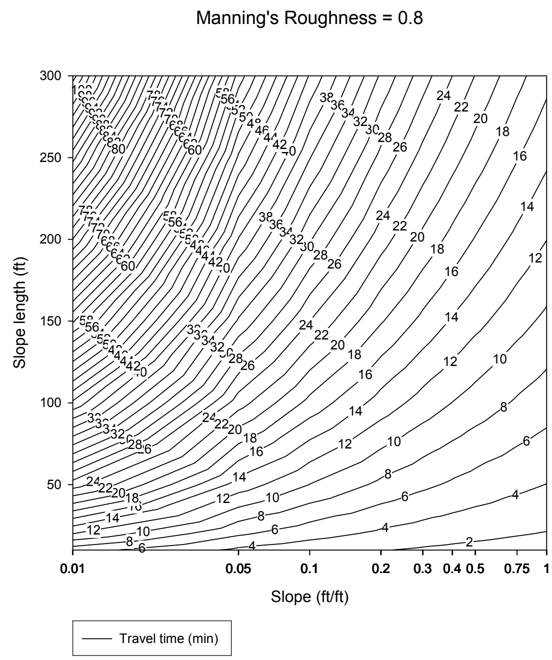


Range (natural)

Figure C-2. Sheetflow travel times (using a P_2 value of 4.2 inches) (cont).



Woods, light underbrush (considering cover to height of about 0.1 ft)



Woods: dense underbrush (considering cover to height of about 0.1 ft)

Figure C-2. Sheetflow travel times (using a P_2 value of 4.2 inches) (cont).

Shallow Concentrated Flow

After a maximum of 300 ft., sheetflow usually becomes shallow concentrated flow which is characterized by much narrower flow paths and faster flows. The following equations are used to calculate the velocities of this flow segment, based on the nature of the surface (paved or unpaved). Figure C-3 contains graphical solutions for these equations.

$$V = 16.1345\sqrt{s} \qquad \text{(Unpaved)}$$

$$V = 20.3282\sqrt{s} \qquad \text{(Paved)}$$

Where:
V = average velocity (ft/s), and
s = slope of hydraulic grade line (watercourse slope, ft/ft)

These two equations are based on a solution of the Manning equation with different assumptions for *n* (Manning roughness coefficient) and R (hydraulic radius, ft). For unpaved areas, *n* is 0.05 and R is 0.4 ft; for paved areas, *n* is 0.025 and R is 0.2 ft. The travel time associated with the shallow concentrated flow segment is calculated using this velocity and the flow path length.

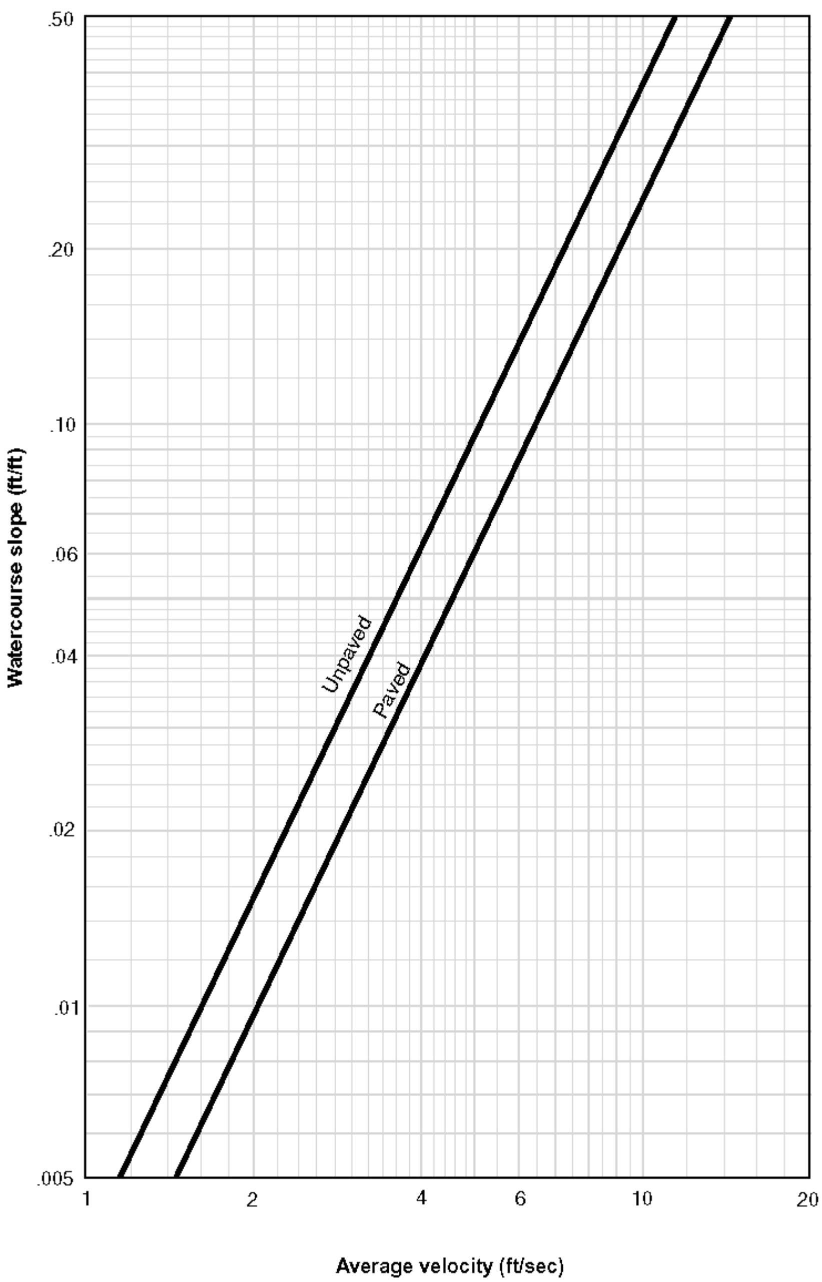


Figure C-3 . Shallow concentrated flow velocities (SCS 1986).

Channel Flow

If the flow path includes a designated channel shown on a USGS quadrangle map, the Manning's equation is used to calculate the velocity in the channel reach. The travel time in the reach is then calculated using this channel-full velocity and the length of the channel.

$$V = \frac{1.49 r^{2/3} \sqrt{s}}{n}$$

Where:

V = average velocity (ft/s), and
r = hydraulic radius (ft) and is equal to a/p_w
a = cross sectional flow area (ft^2)
p_w = wetted perimeter (ft)
s = slope of hydraulic grade line (channel slope, ft/ft)
n = Manning roughness coefficient (for open channel flow)

This is the conventional Manning's equation, and appropriate channel lining n coefficients are used. Hydrology texts list these, usually with modifiers for natural channels. The following list shows some of the values:

Bare earth, clean, recently completed:	0.016 to 0.018
Unlined open channel, not maintained, with weeds and brush:	0.08 to 0.12
Roadside channel, Bermuda grass 4 to 6 in (up to 0.7 ft deep flow):	0.05 to 0.09
Roadside channel, Bermuda grass 4 to 6 in (0.7 to 1.5 ft deep flow):	0.04 to 0.06
Natural stream channel, minor stream, some grass and weeds:	0.030 to 0.035
Natural stream channel, minor stream, irregular section with pools:	0.01 to 0.02
Heavy weeds and scatted brush floodplain:	0.05 to 0.07

Obviously, there is a large range in the values that can be used. Careful field surveys and comprehensive listings are therefore needed for the selection of the most appropriate value.

Example Travel Time Calculation
The TR-55 User Guide (SCS 1986) includes the following example. Figure C-4 shows a watershed in Dyer County, which is located in northwestern Tennessee. The problem is to compute Tc at the outlet of the watershed (point D). The 2-year 24-hour rainfall depth is 3.6 inches. All three types of flow occur from the hydraulically most distant point (A) to the point of interest (D). To compute Tc, first determine Tt for each segment using the following information:

Segment AB: Sheetflow; dense grass; slope (s) = 0.01 ft/ft; and length (L) = 100 ft.
Segment BC: Shallow concentrated flow; unpaved; s = 0.01 ft/ft; and L = 1400 ft.
Segment CD: Channel flow; Manning's n = 0.05; flow area (a) = 27 ft^2;
wetted perimeter (pw) = 28.2 ft; s = 0.005 ft/ft; and L=7300ft.

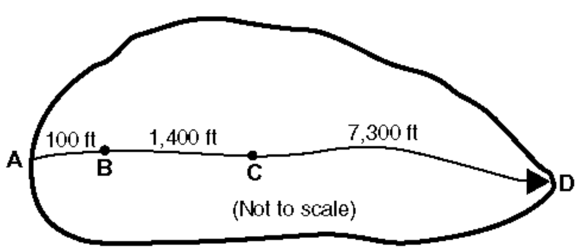


Figure C-4. Watershed for TR-55 Tt calculation example (SCS 1986).

Figure C-5 is the SCS worksheet showing the calculations for the above problem. In this case, each flow segment is comprised of a single condition of slope and cover. In many cases, the individual flow segments may need to be broken up into subunits to represent different slopes or roughness coefficients. The travel times for each of the segments are added. For the sheetflow segment, however, the travel length must still be less than 300 ft. (or the local "limit") in total, not for each calculation interval. Worksheet 3 has two columns to facilitate two segments for each flow portion. Additional segments may be needed. In this example, the total travel time for this flow path from A to

D is 1.53 hours, with almost 1 hour associated with the channel flow time. For small sites, including most urban areas, the sheetflow segment will likely comprise the largest portion of the total flow time.

Again, in order to determine the time of concentration for the watershed, several different candidate flow paths are usually needed to be evaluated and the one with the longest travel time is used as the time of concentration. This may not be the path with the longest travel distance, as shallower slopes and rougher ground covers can have a longer Tc, even with a shorter flow path.

Worksheet 3: Time of Concentration (T_c) or travel time (T_t)

Project: Heavenly Acres	By: DW	Date: 10/6/85
Location: Dyer County, Tennessee	Checked: NM	Date: 10/8/85

Check one: ☐ Present ☒ Developed

Check one: ☒ T_c ☐ T_t through subarea

Notes: Space for as many as two segments per flow type can be used for each worksheet. Include a map, schematic, or description of flow segments.

Sheet flow (Applicable to T_c only)

Segment ID	AB			
1. Surface description (table 3-1)	Dense Grass			
2. Manning's roughness coefficient, n (table 3-1)	0.24			
3. Flow length, L (total L ≤ 300 ft) ... ft	100			
4. Two-year 24-hour rainfall, P_2 ... in	3.6			
5. Land slope, s ... ft/ft	0.01			
6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5}\ s^{0.4}}$ Compute T_t ... hr	0.30	+		= 0.30

Shallow concentrated flow

Segment ID	BC			
7. Surface description (paved or unpaved)	Unpaved			
8. Flow length, L ... ft	1400			
9. Watercourse slope, s ... ft/ft	0.01			
10. Average velocity, V (figure 3-1) ... ft/s	1.6			
11. $T_t = \frac{L}{3600\ V}$ Compute T_t ... hr	0.24	+		= 0.24

Channel flow

Segement ID	CD			
12. Cross sectional flow area, a ... ft^2	27			
13. Wetted perimeter, p_w ... ft	28.2			
14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r ... ft	0.957			
15 Channel slope, s ... ft/ft	0.005			
16. Manning's roughness coefficient, n	0.05			
17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ... ft/s	2.05			
18. Flow length, L ... ft	7300			
19. $T_t = \frac{L}{3600\ V}$ Compute T_t ... hr	0.99	+		= 0.99
20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19) ... Hr				1.53

Figure C-5. Calculation example for travel time problem (SCS 1986).

Tabular Hydrograph Method

The SCS TR-55 tabular hydrograph method (SCS 1986) can be used to develop a hydrograph for each subwatershed area than can then be routed through the downstream project segments. This method will also produce the total runoff volume and the peak flow rate. This method is not used in WinTR-55; this computerized version uses the more complete routing procedures from TR-20. However, the following is still presented as an optional method and to illustrate the sensitivity of Tc and CN selections. Appendix C-1 includes all of the tabular hydrograph tables that can be used to calculate hydrographs for all locations in the US.

Example Tabular Hydrograph Calculation

The following example is from the TR-55 manual (SCS 1986) and illustrates how the Tc, CN, and other site characteristics are used to develop and route hydrographs for a complex watershed.

This example computes the 25-year frequency peak discharge at the downstream end of subarea 7 shown in Figure C-6. This example is for present conditions and uses the worksheets presented in SCS (1986). Calculate the present condition CN, Tc, and Tt for each subarea, using the procedures in TR-55 chapters 2 and 3. These values are entered on worksheet 5a (Figure C-7). Then, the tabular hydrograph tables are used to determine the normalized hydrograph for downstream locations.

The hydrograph tables are presented in SCS (1986) according to rain type (there are sections of tables for types I, Ia, II, and III rain distributions). The first step is to find the table section pertaining to the rain distribution for the study area. In this case, the area has type II rains. The type II rain hydrograph tables are further grouped according to the Tc for the subarea, ranging from 0.1 to 2 hours. In the case for subarea #1, the Tc is 1.5 hours, so pg 5-37 from SCS (1986) is used (Table C-5). Each page is further divided into three segments, corresponding to Ia/P ratios of 0.10, 0.30, and 0.50. The Ia is the initial abstractions for the area (not to be confused with rain distribution type Ia) and are a direct function of the CN value. These are given in the User Guide (SCS table 5-1), and on Table C-4. The P is the total rain depth being evaluated. The top set of values are used for Ia/P ratios of ≤ 0.2, the middle set for ratios from 0.2 to 0.4, while the bottom set is used for ratios of > 0.4 (interpolation is not used; WinTR-55 and TR-20 calculate more precise values based on actual site conditions). In this case, the #1 subarea Ia/P is 0.18, so the top set of values are used. Finally, each segment has 12 lines representing different travel times from the bottom of the subwatershed area to the location of interest.

The largest unit peak runoff rate values (csm/in, or cubic feet per second of runoff per square mile of drainage area, per inch of direct runoff) on each line start close to 12 hours for the top time, and shift to the right as the travel time increases. The shift between the largest values for each row is equal to the differences in the travel times between each line, representing routing of the hydrographs as they travel downstream. In this example, for the #1 subarea, the Tt is 2.5 hours. Therefore, the line near the bottom of the top segment, representing 2.5 hours, is used. The values in the table represent normalized hydrographs and are multiplied by AmQ (the factor of the watershed area, in mi^2 and the direct runoff in inches) to obtain the flow values in traditional units of ft^3/sec, or cfs. These final cfs values are written on worksheet 5b (Table C-6). As an example, the appropriate values for the peak discharge (q) for subarea 4 at 14.6 hr is:

$$q = qt(AmQ) = (274)(0.70) = 192 \text{ cfs}$$

Once all the prerouted subarea hydrographs have been tabulated on worksheet 5b, they are summed to obtain the composite hydrograph. The resulting 25-year frequency peak discharge is 720 cfs at 14.3 hr, as shown on Table C-6.

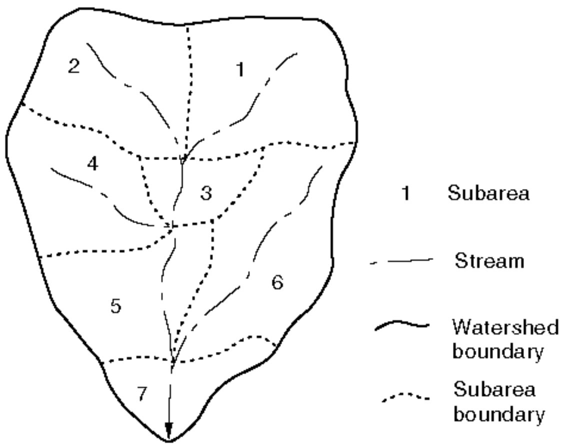


Figure C-6. Example watershed for tabular hydrograph calculations (SCS 1986).

Table C-4. I_a Values for Runoff Curve Numbers (SCS 1986)

Curve Number	I_a (inch)	Curve Number	I_a (inch)	Curve Number	I_a (inch)
40	3.000	60	1.333	80	0.500
41	2.878	61	1.279	81	0.469
42	2.762	62	1.226	82	0.439
43	2.651	63	1.175	83	0.410
44	2.545	64	1.125	84	0.381
45	2.444	65	1.077	85	0.353
46	2.348	66	1.030	86	0.326
47	2.255	67	0.985	87	0.299
48	2.167	68	0.941	88	0.273
49	2.082	69	0.899	89	0.247
50	2.000	70	0.857	90	0.222
51	1.922	71	0.817	91	0.198
52	1.846	72	0.778	92	0.174
53	1.774	73	0.740	93	0.151
54	1.704	74	0.703	94	0.128
55	1.636	75	0.667	95	0.105
56	1.571	76	0.632	96	0.083
57	1.509	77	0.597	97	0.062
58	1.448	78	0.564	98	0.041
59	1.390	79	0.532		

Worksheet 5a: Basic watershed data

Project: Fallswood | Location: Dyer County, Tennessee | By: DW | Date: 10/1/85

Check one: ☒ Present ☐ Developed | Frequency (yr): 25 | Checked: NM | Date: 10/3/85

Subarea name	Drainage area A_m (mi^2)	Time of concen-tration T_c (hr)	Travel time through subarea T_t (hr)	Downstream subarea names	Travel time summation to outlet ΣT_t (hr)	24-hr rain-fall P (in)	Runoff curve number CN	Runoff Q (in)	A_mQ (mi^2—in)	Initial abstraction I_a (in)	I_a/P
1	0.30	1.50	– –	3, 5, 7	2.50	6.0	65	2.35	0.71	1.077	0.18
2	0.20	1.25	– –	3, 5, 7	2.50	6.0	70	2.80	0.56	0.857	0.14
3	0.10	0.50	0.50	5, 7	2.00	6.0	75	3.28	0.33	0.667	0.11
4	0.25	0.75	– –	5, 7	2.00	6.0	70	2.80	0.70	0.857	0.14
5	0.20	1.50	1.25	7	0.75	6.0	75	3.28	0.66	0.667	0.11
6	0.40	1.50	– –	7	0.75	6.0	70	2.80	1.12	0.857	0.14
7	0.20	1.25	0.75	– –	0	6.0	75	3.28	0.66	0.667	0.11

From worksheet 3 | From worksheet 2 | From table 5-1

Figure C-7. Worksheet 5a for showing basic watershed data (SCS 1986).

Table C-5. Tabular Hydrograph Table for Example Problem (SCS 1986, pg 5-37)

Exhibit 5-II: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution—continued

HYDROGRAPH TIME (HOURS)

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
IA/P = 0.10												* * * TC = 1.5 HR * * *																IA/P = 0.10				
0.0	9	11	15	21	25	31	41	58	82	112	147	184	216	255	275	236	198	159	129	98	76	57	43	35	30	25	23	21	18	16	12	1
.10	8	10	13	18	20	23	28	37	51	72	98	131	166	226	265	254	226	187	151	113	86	63	46	37	31	26	23	21	19	16	13	2
.20	8	10	13	17	19	22	26	33	45	63	87	116	149	212	259	259	233	197	160	119	90	66	48	38	32	27	24	22	19	16	13	2
.30	7	9	12	16	18	21	24	30	40	55	76	103	134	197	244	255	238	206	169	125	95	68	49	38	32	27	24	22	19	17	13	2
.40	7	8	11	14	15	17	19	23	28	36	49	67	91	151	208	247	252	230	196	146	109	77	54	41	34	29	25	22	19	17	13	3
.50	6	8	10	13	15	16	18	21	26	33	43	59	80	136	194	238	249	235	204	154	115	81	56	42	34	29	25	23	20	17	13	3
.75	5	7	8	11	12	13	14	16	18	21	25	32	42	76	125	179	222	240	233	193	148	102	67	48	38	32	27	24	20	18	13	5
1.0	4	5	7	8	9	10	11	12	13	14	16	18	22	34	59	101	152	201	236	230	193	135	86	59	44	35	30	26	21	18	14	7
1.5	3	4	5	6	6	7	8	8	9	10	11	12	13	16	22	34	58	95	141	203	226	197	131	84	58	43	35	29	23	20	15	10
2.0	1	2	3	4	4	5	5	6	6	7	7	8	9	10	12	16	22	34	56	110	172	218	187	126	82	57	43	34	25	21	16	11
2.5	1	1	2	2	3	3	3	4	4	4	5	5	6	7	8	9	11	14	18	34	69	141	210	190	133	87	60	44	30	23	17	12
3.0	0	0	1	1	2	2	2	2	3	3	3	3	4	5	5	6	8	9	11	16	27	66	149	204	181	128	85	58	35	25	18	12
IA/P = 0.30												* * * TC = 1.5 HR * * *																IA/P = 0.30				
0.0	0	0	0	0	0	1	6	15	31	53	80	112	144	193	225	208	186	157	134	108	89	70	56	48	42	37	34	31	28	25	20	2
.10	0	0	0	0	0	0	1	4	12	25	43	68	97	157	198	219	203	178	151	120	98	77	60	50	44	38	35	32	28	25	20	3
.20	0	0	0	0	0	0	0	1	3	9	19	35	57	114	168	201	213	196	171	135	108	84	64	53	46	40	36	33	29	26	20	4
.30	0	0	0	0	0	0	0	1	2	7	15	29	48	100	155	193	210	200	177	140	113	87	66	54	46	41	36	33	29	26	20	5
.40	0	0	0	0	0	0	0	0	2	5	12	23	39	87	141	184	207	202	182	146	117	89	68	55	47	41	36	33	29	26	20	5
,50	0	0	0	0	0	0	0	0	0	1	4	9	18	51	101	153	190	205	197	164	131	99	73	58	49	43	38	34	30	26	20	7
.75	0	0	0	0	0	0	0	0	0	0	2	4	9	30	68	116	160	189	197	179	147	110	80	62	52	45	39	35	30	27	21	8
1.0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	20	49	92	138	175	195	178	137	97	72	57	48	42	37	31	28	21	12
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	21	47	85	145	187	178	133	95	71	57	48	42	34	29	23	16
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	13	45	97	162	180	138	99	74	58	49	38	32	25	18
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	31	89	161	174	133	97	72	58	42	34	26	18
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	29	98	160	169	129	95	71	48	37	28	19
IA/P = 0.50												* * * TC = 1.5 HR * * *																IA/P = 0.50				
.0	0	0	0	0	0	0	0	3	8	16	27	42	59	92	116	128	130	121	112	100	90	78	67	60	55	50	46	43	39	35	29	4
.10	0	0	0	0	0	0	0	2	6	12	22	35	51	84	110	125	128	123	114	102	91	79	68	61	55	50	46	43	39	35	29	4
.20	0	0	0	0	0	0	0	0	1	4	10	18	29	60	91	114	126	128	120	108	97	83	71	63	57	52	47	44	40	36	29	5
.30	0	0	0	0	0	0	0	0	1	3	8	14	24	52	83	108	123	126	122	110	98	85	72	63	57	52	48	44	40	36	29	6
.40	0	0	0	0	0	0	0	0	0	1	2	6	12	31	60	90	112	124	126	116	104	90	75	66	59	54	49	45	41	37	29	8
.50	0	0	0	0	0	0	0	0	0	0	2	4	9	26	53	83	106	121	125	118	106	91	77	67	60	54	49	46	41	37	29	8
.75	0	0	0	0	0	0	0	0	0	0	1	2	5	16	36	62	88	108	119	122	112	97	81	69	62	56	51	47	42	38	30	11
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	10	26	49	75	98	118	121	108	90	76	66	59	54	49	43	39	31	16
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	11	25	45	80	107	118	106	89	75	65	59	53	45	41	32	23
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	11	32	63	100	115	104	87	74	65	58	48	42	34	26
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	16	48	94	113	105	89	76	66	53	45	36	27
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	15	54	96	111	103	88	75	58	48	38	28

RAINFALL TYPE = II * * * TC = 1.5 HR * * * SHEET 9 OF 10

Table C-6. Worksheet 5b for Example Hydrograph Calculation (SCS 1986)

Worksheet 5b: Basic watershed data

Project Fallswood | Location Dyer County, Tennessee | By DW | Date 10/1/85

Check one: ☒ Present ☐ Developed | Frequency (yr) 25 | Checked NM | Date 10/3/85

Subarea name	Basic watershed data used 1/: Subarea T_c (hr)	ΣT_t to outlet (hr)	I_a/P	A_mQ (mi^2—in)	Select and enter hydrograph times in hours from exhibit 5-II 2/: 12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5
					Discharges at selected hydrograph times 3/ (cfs)											
1	1.50	2.50	0.10	0.71	4	4	5	6	6	8	10	113	24	49	100	149
2	1.25	2.50	0.10	0.56	3	4	4	6	7	8	11	16	32	64	110	127
3	0.50	2.00	0.10	0.33	5	5	6	8	12	21	41	67	98	92	60	29
4	0.75	2.00	0.10	0.70	8	9	11	14	20	34	62	106	172	192	149	81
5	1.50	0.75	0.10	0.66	21	28	50	83	118	147	158	154	127	98	67	44
6	1.50	0.75	0.10	1.12	36	47	85	140	200	249	269	261	216	166	114	75
7	1.25	0	0.10	0.66	169	187	205	176	140	108	85	69	51	40	31	24
Composite hydrograph at outlet					246	284	366	433	503	575	636	686	720	701	631	529

1/ Worksheet 5a. Rounded as needed for use with exhibit 5.
2/ Enter rainfall distribution type used.
3/ Hydrograph discharge for selected times is A_mQ multiplied by tabular discharge from appropriate exhibit 5.

Tabular Hydrograph Example for Urban Watershed
The following example is for a typical urban watershed, having four subareas that are quite different in their development characteristics. The following lists the procedure for evaluating this area:

1) subdivide the watershed into relatively homogeneous subareas (as shown in Figure C-8)

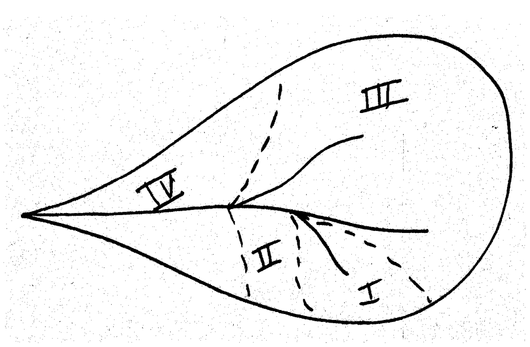


Figure C-8. Relatively homogeneous subareas in example urban watershed.

2) calculate the drainage for each subarea:

I	0.10 mi^2
II	0.08
III	0.6
IV	0.32
Total:	1.12

3) calculate the time of concentration (Tc) for each subarea (TR-55 chapter 3):

I	0.2 hrs
II	0.1
III	0.3
IV	0.1

4) calculate the travel time (Tt) from each subarea discharge location to the location of interest (outlet of total watershed in this example) (TR-55 chapter 3):

I	0.1 hrs
II	0.05
III	0.05
IV	0

5) select the curve number (CN) for each subarea:

I	Strip commercial, all directly connected	CN = 97
II	Medium density residential area, grass swales	CN = 46
III	Medium density residential area, curbs and gutters	CN = 72
IV	Low density residential area, grass swales	CN = 40

6) rainfall distribution: Type II for all areas

7) 24-hour rainfall depth for storm: 4.1 inches

8) calculate total runoff (inches) from CN and rain depth (from SCS fig. 2-1)

I	CN = 97	P = 4.1 in.	Q = 3.8 in.
II	CN = 46	P = 4.1 in.	Q = 0.25
III	CN = 72	P = 4.1 in.	Q = 1.5
IV	CN = 40	P = 4.1 in.	Q = 0.06

9) determine Ia for each subarea (assumes Ia = 0.2 S) (SCS table 5-1):

I	CN = 97	Ia = 0.062 in.
II	CN = 46	Ia = 2.348 in.
III	CN = 72	Ia = 0.778 in.
IV	CN = 40	Ia = 3.000 in.

10) calculate the ratio of Ia to P

I	Ia/P = 0.062/4.1 = 0.015
II	Ia/P = 2.348/4.1 = 0.57
III	Ia/P = 0.778/4.1 = 0.19
IV	Ia/P = 3.000/4.1 = 0.73

11) use worksheets SCS 5a and 5b to summarize above data and to calculate the composite hydrograph. These are shown in Tables C-7 and C-8.

Table C-7. SCS Worksheet 5a for Urban Example

Project: example urban | Location: Jefferson Co. | By | Date

Check one: ☐ Present ☑ Developed | Frequency (yr): 2 yr | Checked | Date

Subarea name	Drainage area A_m (mi^2)	Time of concentration T_c (hr)	Travel time through subarea T_t (hr)	Downstream subarea names	Travel time summation to outlet ΣT_t (hr)	24-hr rainfall P (in)	Runoff curve number CN	Runoff Q (in)	A_mQ (mi^2—in)	Initial abstraction I_a (in)	I_a/P
I	0.10	0.2	—	—	0.1	4.1	97	3.8	0.38	0.062	0.015
II	0.08	0.1	—	—	0.05	4.1	46	0.25	0.02	2.348	0.57
III	0.62	0.3	—	—	0.05	4.1	72	1.5	0.93	0.778	0.19
IV	0.32	0.1	—	—	0	4.1	40	0.06	0.019	3.000	0.73
Σ=	1.12										

Table C-8. SCS Worksheet 5b for Urban Example

Project				Location						By				Date		
example urban				Jefferson Co.												
Check one: ☐ Present ☑ Developed				Frequency (yr) 2-yr						Checked				Date		

Subarea name	Subarea T_c (hr)	ΣT_t to outlet (hr)	I_a/P	A_mQ (mi²–in)			11.0	11.6	12	12.2	12.3	13	14	16	18	26	
	Basic watershed data used 1/				Select and enter hydrograph times in hours from exhibit 5-II 2/ — Discharges at selected hydrograph times 3/ (cfs)												
I	0.2	0.1	~~0.15~~ 0.10	0.38	csm/in:		19	39	168	601	753	83	43	25	18	θ	pg 5-30
					cfs:		7.2	14.2	63.8	228	278	31.5	16.3	9.5	6.8	θ	
II	0.1	~~0.05~~ θ	~~0.37~~ 0.5	0.02	csm/in:		θ	θ	70	377	196	99	67	46	38	θ	pg 5-29
					cfs:		θ	θ	1.4	7.5	3.9	2.0	1.3	0.9	0.8	θ	
III	0.3	~~0.05~~ θ	~~0.19~~ 0.1	0.93	csm/in:		20	41	235	676	676	80	42	24	18	θ	pg 5-31
					cfs:		18.6	38.1	218	628	628	74.4	39.1	22.3	16.7	θ	
IV	0.1	θ	~~0.73~~ 0.5	0.019	too small												
Composite hydrograph at outlet							25.8	52.3	283	863	910	108	56.7	32.7	24.3	θ	

← peak flow

1/ Worksheet 5a. Rounded as needed for use with exhibit 5.
2/ Enter rainfall distribution type used.
3/ Hydrograph discharge for selected times is A_mQ multiplied by tabular discharge from appropriate exhibit 5.

The peak flow is seen to be 910 cfs, occurring at 12.3 hours. Figure C-9 is a plot of the 3 main components, plus the total hydrograph. Subarea III contributed most of the peak flow to the total hydrograph, while subareas II and IV contributed insignificant flows. The following discussion introduces WinTR-55 and presents this same example. The main differences is that WinTR-55 requires a description of the channel as it calculates the travel times and conducts the channel routing using a more precise procedure. In addition, the hydrograph development uses TR-20, instead of the tabular hydrograph method.

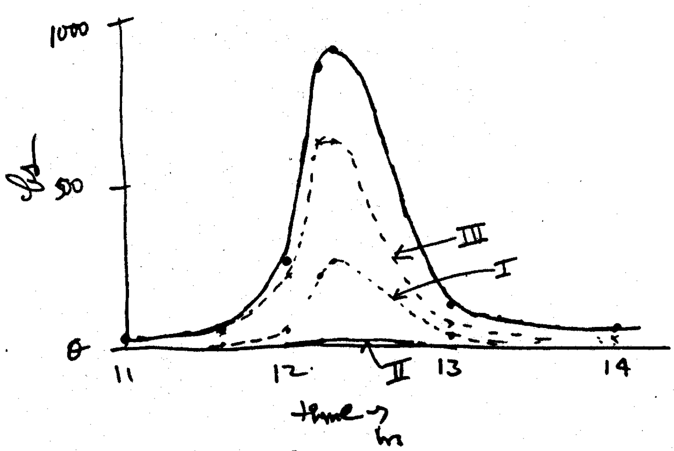


Figure C-9. Plot of individual and composite hydrograph for urban example.

Example use of WinTR55

The following discussion is summarized from the WinTR-55 user guide information, while the example uses the previously described information.

A WinTR-55 work group was formed in the spring of 1998 to modernize and revise TR-55 and the computer software. The current changes include: upgrading the source code to Visual Basic, changing the philosophy of data input, developing a Windows interface and output post-processor, enhancing the hydrograph-generation capability of the software and flood routing hydrographs through stream reaches and reservoirs.

The availability and technical capabilities of the personal computer have significantly changed the philosophy of problem solving for the engineer. Computer availability eliminated the need for TR-55 manual methods, thus the manual portions (graphs and tables) of the user document have been eliminated as official guidance. The WinTR-55 user manual (NRCS 2002a) covers the procedures used in and the operation of the WinTR-55 computer program. Part 630 of the Natural Resources Conservation Service (NRCS) National Engineering Handbook provides detailed information on NRCS hydrology and is the official technical reference for WinTR-55.

Program Description

WinTR-55 is a single-event rainfall-runoff small watershed hydrologic model. The model generates hydrographs from both urban and agricultural areas and at selected points along the stream system. Hydrographs are routed downstream through channels and/or reservoirs. Multiple sub-areas can be modeled within the watershed.

Model Overview

A watershed is composed of subareas (land areas) and reaches (major flow paths in the watershed). Each subarea has a hydrograph generated from the land area based on the land and climate characteristics provided. Reaches can be designated as either channel reaches where hydrographs are routed based on physical reach characteristics, or as storage reaches where hydrographs are routed through a reservoir based on temporary storage and outlet characteristics. Hydrographs from sub-areas and reaches are combined as needed to accumulate flow as water moves from the upland areas down through the watershed reach network. The accumulation of all runoff from the watershed is represented at the watershed outlet. Up to ten sub-areas and ten reaches may be included in the watershed.

WinTR-55 uses the TR-20 (NRCS 2002b) model for all of the hydrograph procedures: generation, channel routing, storage routing, and hydrograph summation. Figure C-10 is a diagram showing the WinTR-55 model, its relationship to TR-20, and the files associated with the model.

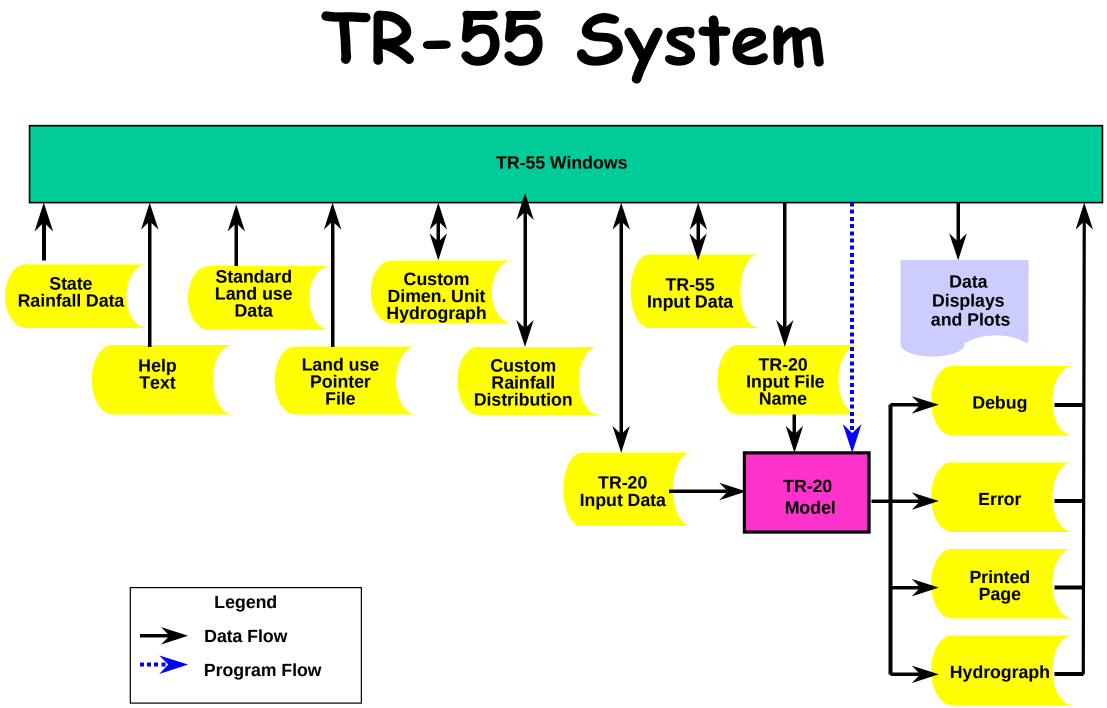


Figure C-10. WinTR-55 system schematic (NRCS 2002a).

Capabilities and Limitations

WinTR-55 hydrology has the capability to analyze watersheds that meet the criteria listed in Table C-9.

Table C-9. WinTR-55 Capabilities and Limitations (NRCS 2002a)

Variable	Limits
Minimum area	No absolute minimum is included in the software. However, carefully examine results from sub-areas less than 1 acre.
Maximum area	25 square miles (6,500 hectares)
Number of Subwatersheds	3-10
Time of concentration for any sub-area	0.1 hour $\leq T_c \leq$ 10 hour
Number of reaches	0-10
Types of reaches	Channel or Structure
Reach Routing	Muskingum-Cunge
Structure Routing	Storage-Indication
Structure Types	Pipe or Weir
Structure Trial Sizes	3-3
Rainfall Depth[1]	Default or user-defined 0 – 50 inches (0-1,270 mm)
Rainfall Distributions	NRCS Type I, IA, II, III, NM60, NM65, NM70, NM75, or user-defined
Rainfall Duration	24-hour
Dimensionless Unit Hydrograph	Standard peak rate factor 484, or user-defined (e.g. Delmarva—see Example 3)
Antecedent Moisture Condition	2 (average)

[1] Although no minimum rain depth is listed by the NRCS in the above table, it must be recognized that the original SCS curve number methods, incorporated in this newer version, are not accurate for small storms. In most cases, larger storms used for drainage design are reasonably well suited to this method. Pitt (1987) and Pitt, *et al.* (2002) showed that rain depths less than 2 or 3 inches can have significant errors when using the CN approach.

Model Input

The various data used in the WinTR-55 procedures are user entered via a series of input windows in the model. A description of each of the input windows follows the figure. Data entry is needed only on the windows that are applicable to the watershed being evaluated.

Minimum Data Requirements. While WinTR-55 can be used for watersheds with up to ten sub-areas and up to ten reaches, the simplest run involves only a single sub-area. Data required for a single sub-area run can be entered on the TR-55 Main Window. These data include**: Identification Data**-User, -State, -County, -Project, and -Subtitle; **Dimensionless Unit Hydrograph**; **Storm Data; Rainfall Distribution**; and **Subarea Data.** The subarea data can be entered directly into the Subarea Entry and Summary table**: Subarea name, subarea description, subarea flows to reach/outlet**, **area**, **runoff curve number** (CN), and **time of concentration** (T_c). Detailed information for the subarea CN and T_c can be entered here or on other windows; if detailed information is entered elsewhere the computational results are displayed in this window.

Watershed Subareas and Reaches. To properly route stream flow to the watershed outlet, the user must understand how WinTR-55 relates watershed subareas and stream reaches. Figure C-11 and Table C-10 show a typical watershed with multiple sub-areas and reaches.

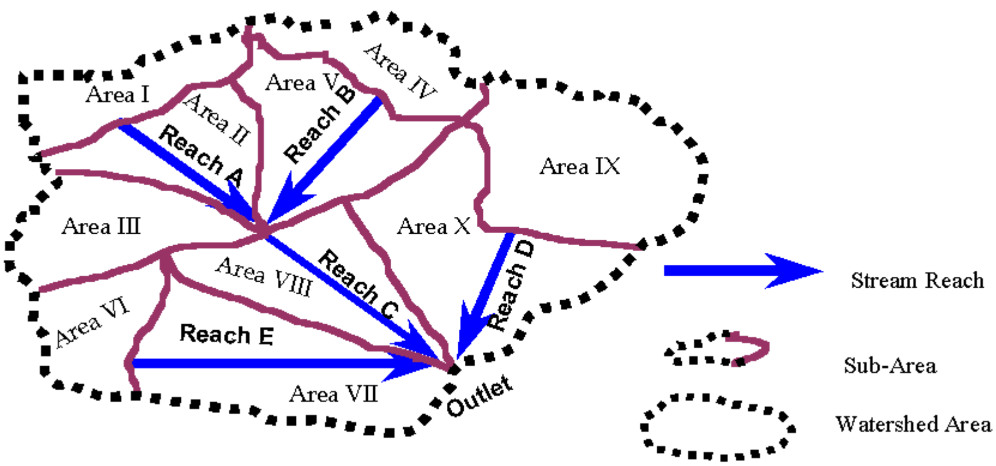


Figure C-11. Sample Watershed Schematic (NRCS 2002a)

Table C-10. Sample Watershed Flows (NRCS 2002a)

Subarea	Flows into Upstream End of
Area I	Reach A
Area II	Reach C
Area III	Reach C
Area IV	Reach B
Area V	Reach C
Area VI	Reach E
Area VII	OUTLET
Area VIII	OUTLET
Area IX	Reach D
Area X	OUTLET

Reach	Flows into
Reach A	Reach C
Reach B	Reach C
Reach C	OUTLET
Reach D	OUTLET
Reach E	OUTLET

Reaches define flow paths through the watershed to its outlet. Each subarea and reach contributes flow to the upstream end of a receiving reach or to the Outlet. Accumulated runoff from all sub-areas routed through the watershed reach system, by definition, is flow at the watershed outlet.

Processes
WinTR-55 relies on the TR-20 model for all hydrograph processes. These include: hydrograph generation, combining hydrographs, channel routing, and structure routing. The program now uses a Muskingum-Cunge method of channel routing (Chow, *et al.* 1988; Maidment 1993; Ponce 1989). The storage-indication method (NRCS NEH Part 630, Chapter 17) is used to route structure hydrographs.

Example WinTR-55 Setup and Operation
An application using WinTR-55 and the previously presented urban watershed example is shown on Figures C-12 through C-21. Figures C-22 and C-23 are other screens available in WinTR-55 that can be used to aid in the calculation of some of the site data, while Figure C-24 is used for detention facilities (structures).

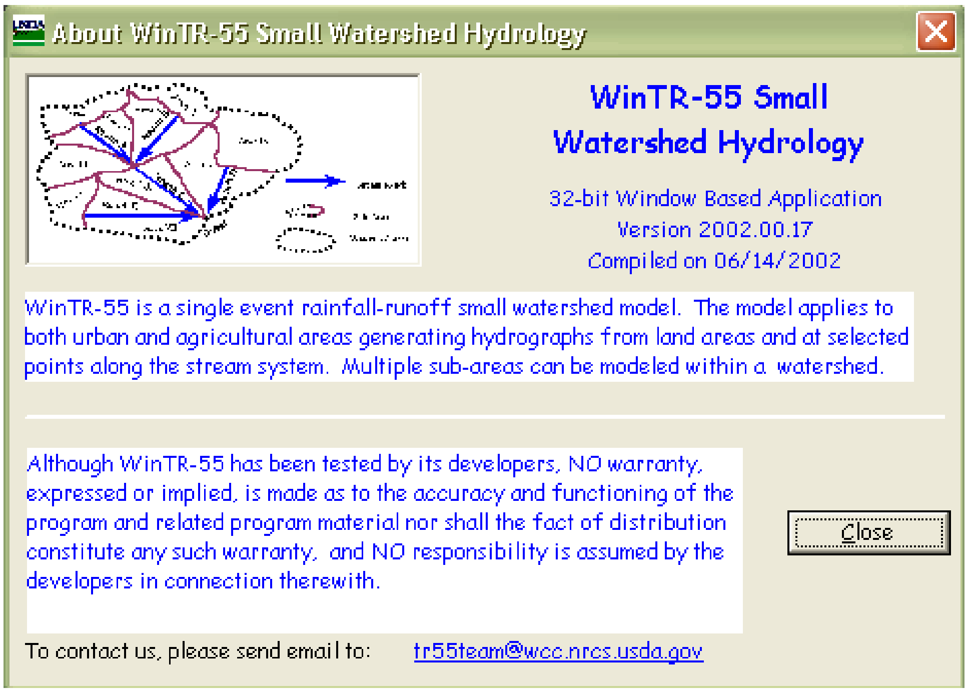


Figure C-12. WinTR-55 opening screen.

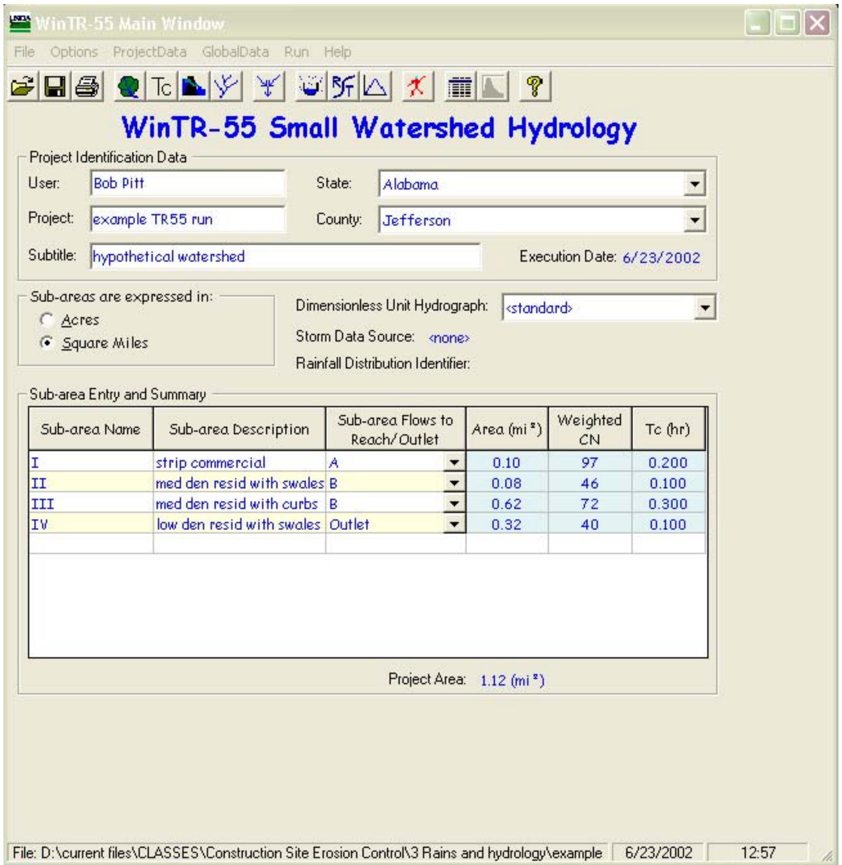


Figure C-13. WinTR-55 small watershed basic information screen.

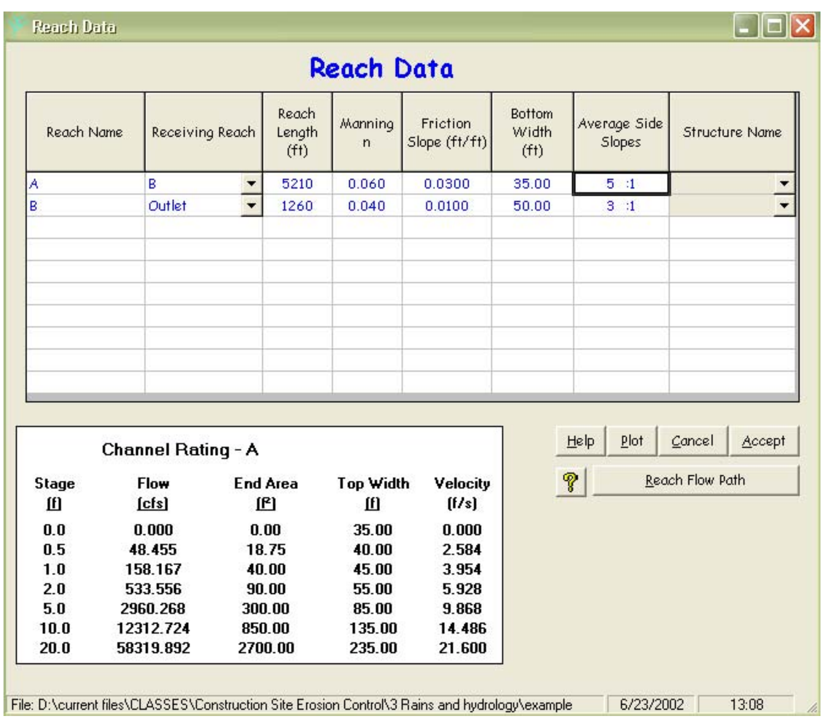


Figure C-14. WinTR-55 reach data screen.

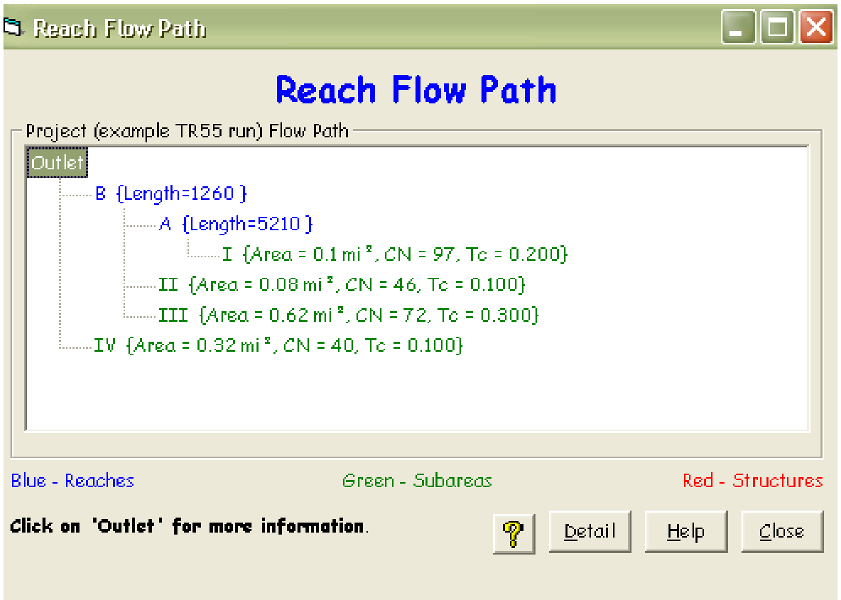


Figure C-15. WinTR-55 reach flow path screen.

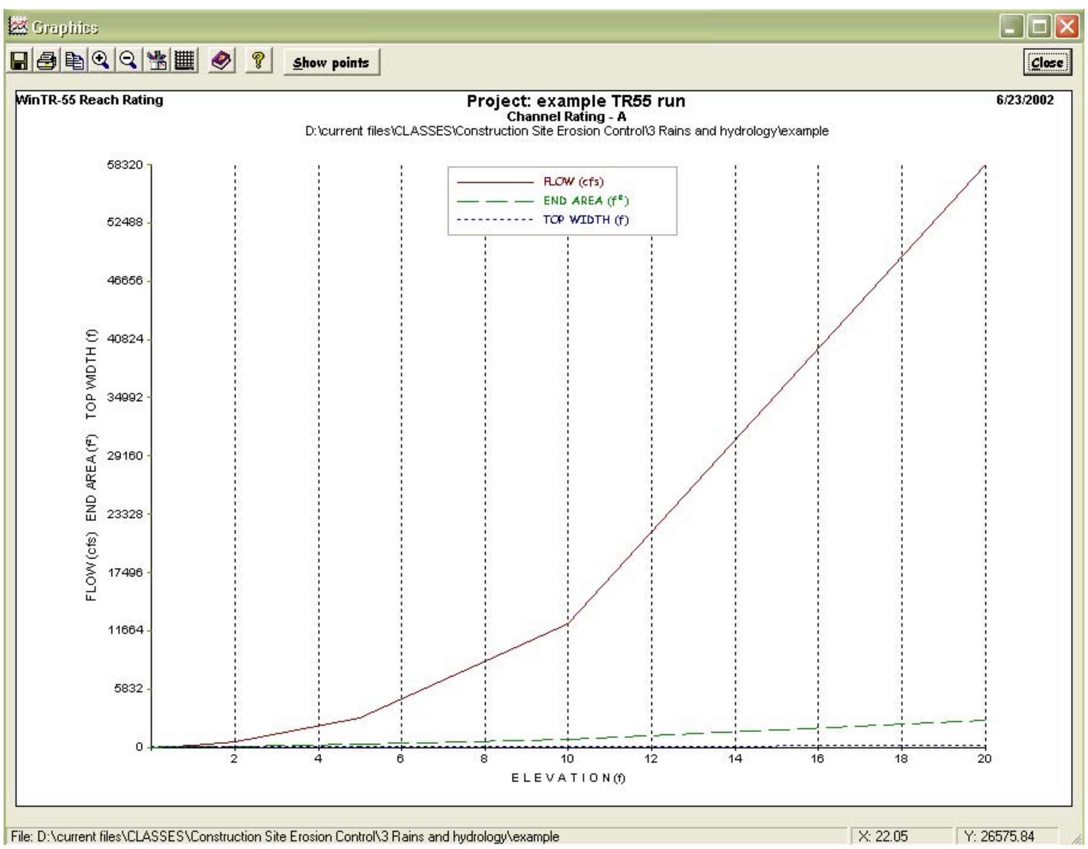


Figure C-16. WinTR-55 reach routing screen.

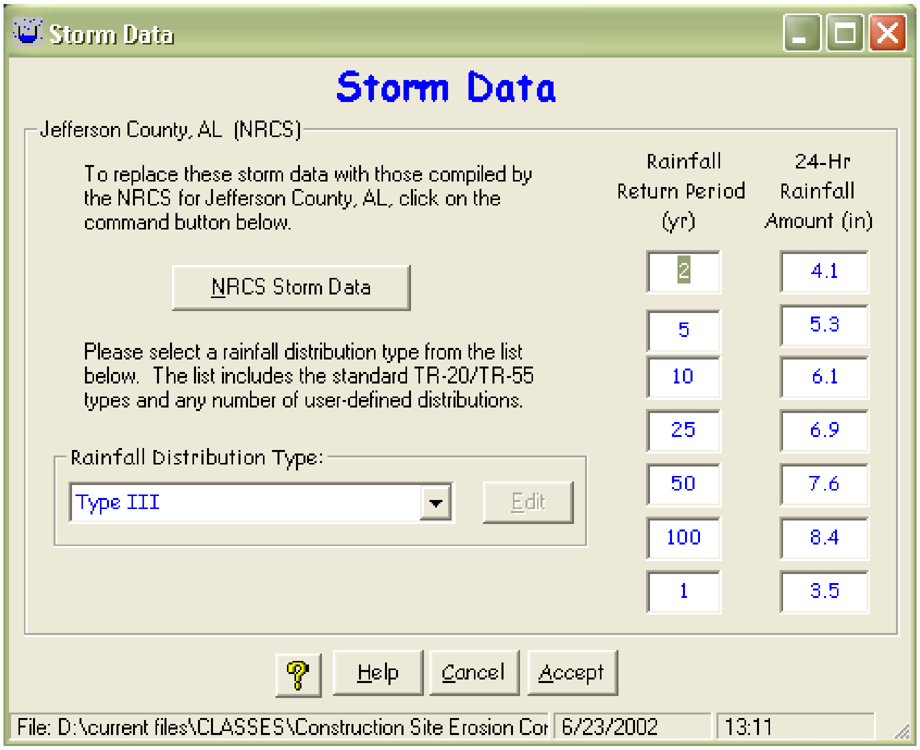
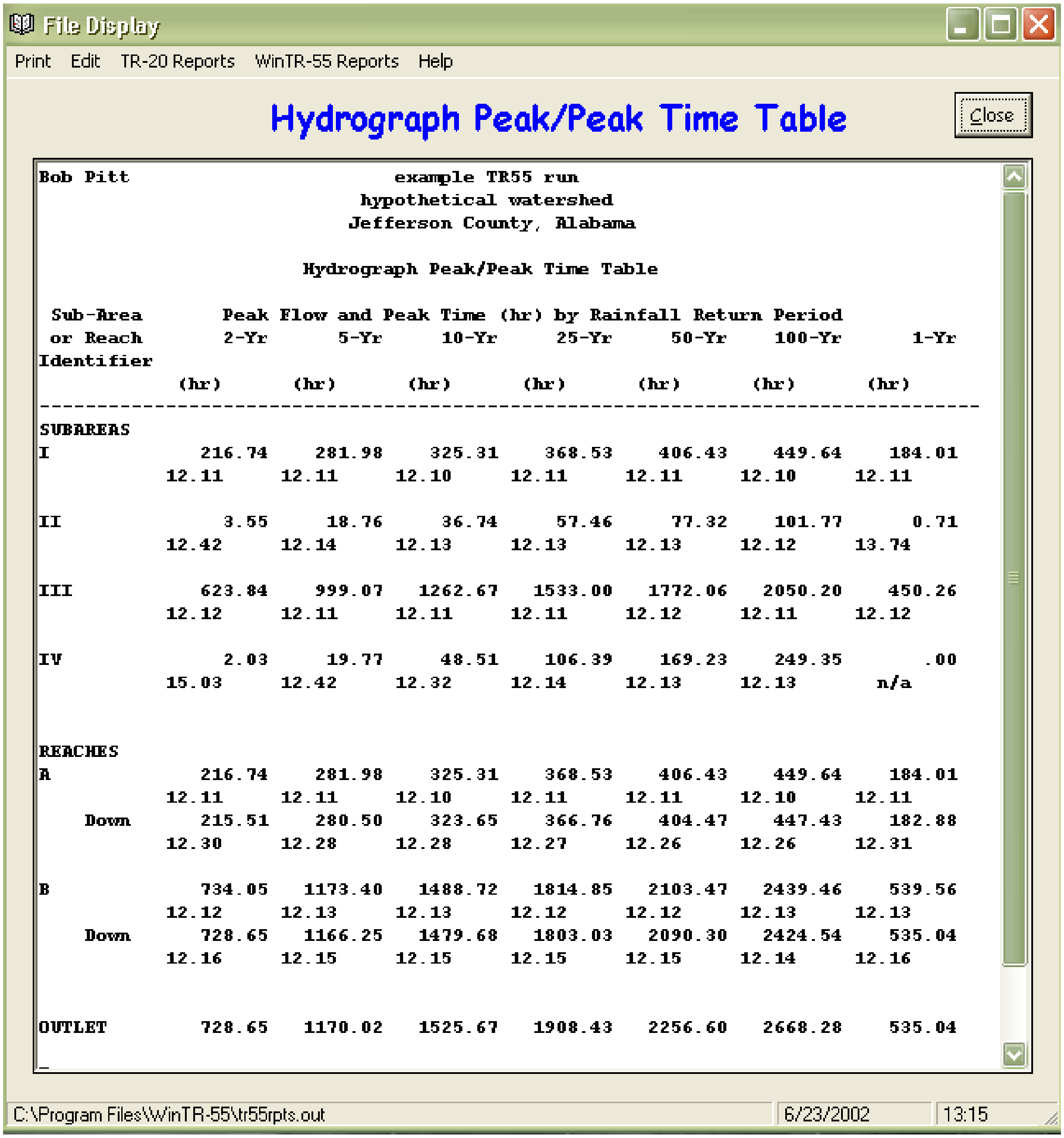


Figure C-17. WinTR-55 storm data screen (information automatically determined by location).

Figure C-18. WinTR-55 event selection/run screen.



Bob Pitt

example TR55 run
hypothetical watershed
Jefferson County, Alabama

Hydrograph Peak/Peak Time Table

Sub-Area or Reach Identifier	Peak Flow and Peak Time (hr) by Rainfall Return Period 2-Yr (hr)	5-Yr (hr)	10-Yr (hr)	25-Yr (hr)	50-Yr (hr)	100-Yr (hr)	1-Yr (hr)
SUBAREAS							
I	216.74	281.98	325.31	368.53	406.43	449.64	184.01
	12.11	12.11	12.10	12.11	12.11	12.10	12.11
II	3.55	18.76	36.74	57.46	77.32	101.77	0.71
	12.42	12.14	12.13	12.13	12.13	12.12	13.74
III	623.84	999.07	1262.67	1533.00	1772.06	2050.20	450.26
	12.12	12.11	12.11	12.11	12.12	12.11	12.12
IV	2.03	19.77	48.51	106.39	169.23	249.35	.00
	15.03	12.42	12.32	12.14	12.13	12.13	n/a
REACHES							
A	216.74	281.98	325.31	368.53	406.43	449.64	184.01
	12.11	12.11	12.10	12.11	12.11	12.10	12.11
Down	215.51	280.50	323.65	366.76	404.47	447.43	182.88
	12.30	12.28	12.28	12.27	12.26	12.26	12.31
B	734.05	1173.40	1488.72	1814.85	2103.47	2439.46	539.56
	12.12	12.13	12.13	12.12	12.12	12.13	12.13
Down	728.65	1166.25	1479.68	1803.03	2090.30	2424.54	535.04
	12.16	12.15	12.15	12.15	12.15	12.14	12.16
OUTLET	728.65	1170.02	1525.67	1908.43	2256.60	2668.28	535.04

Figure C-19. WinTR-55 calculated hydrograph summary screen.

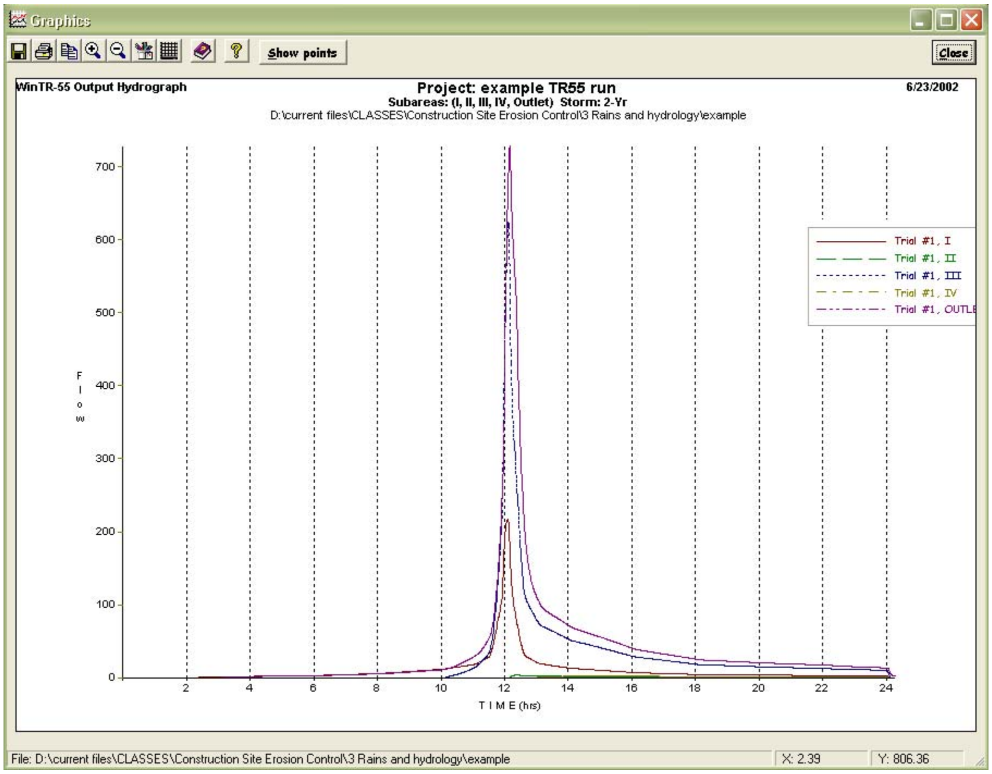


Figure C-20. WinTR-55 hydrograph plot screen.

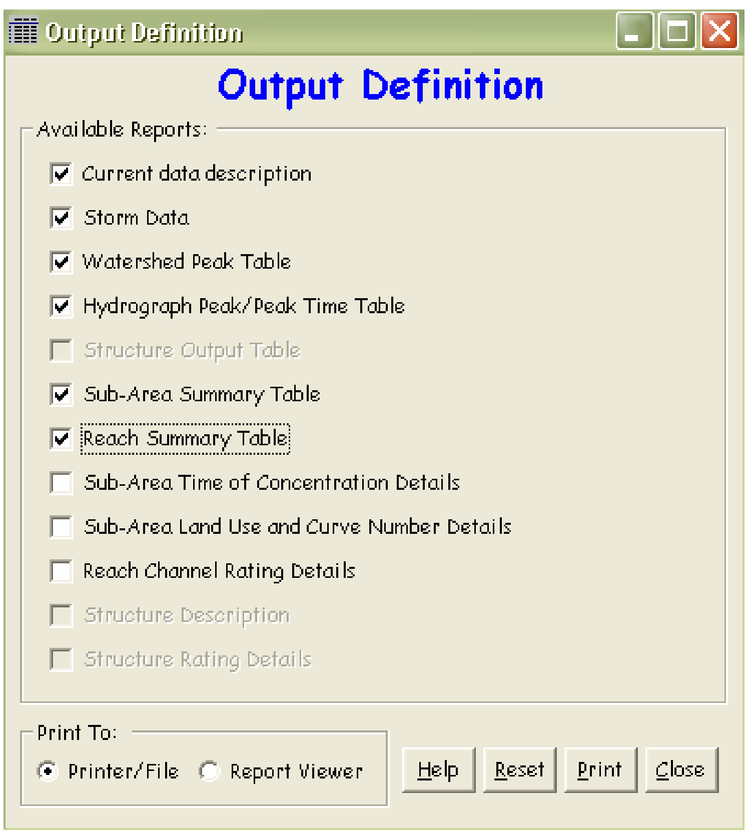


Figure C-21. WinTR-55 report generation screen.

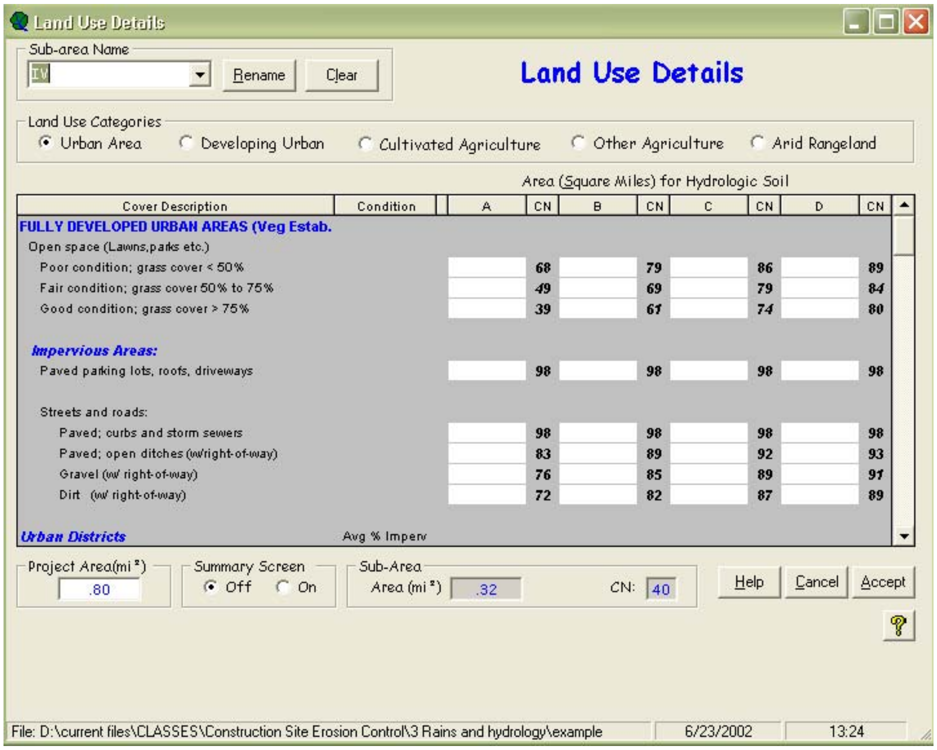


Figure C-22. WinTR-55 land use details screen (if data not directly entered).

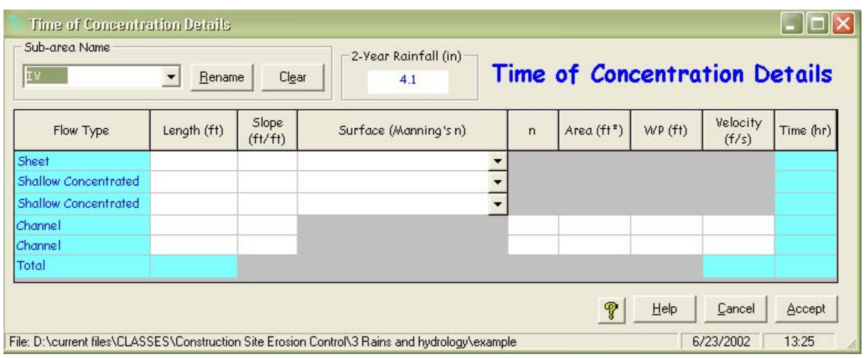


Figure C-23. WinTR-55 time of concentration details screen/calculator (if data not directly entered).

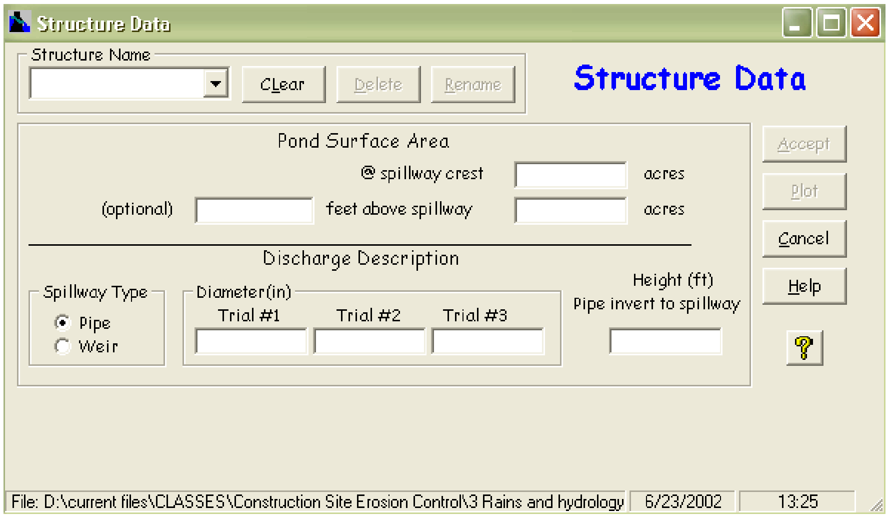


Figure C-24. WinTR-55 structure data screen for detention facilities.

This WinTR-55 example resulted in a peak flow for the 2-yr storm of about 730 cfs, compared to the previously calculated value of 910 cfs. This difference is due to the different routing procedure used, plus the more precise hydrograph development procedure in the updated WinTR-55 version compared to the tabular hydrograph method.

Example Applications to Construction Sites

The following example outlines the hydrographic information needs and how they can be determined using WinTR-55 for a small urban area, in case, a construction site. There are a number of situations where WinTR-55 (or TR-55) can be used to advantage when evaluating construction sites, including the design of erosion and sediment controls. These may include:

- Determination of flows going away from the site affecting downstream areas. Downstream erosion controls may include filter fencing along the project perimeter, or sediment ponds, depending on flow conditions. These controls must be completed before any on-site construction is started.

- Determination of upland flows coming towards the disturbed areas. These flows must be diverted by swales or dikes, or safely carried through the construction sites. Channel design will be based on the expected flow conditions. These controls must be completed after the downstream controls, and before any on-site controls are started.

- Determination of on-site flows on slopes going towards filter fencing, sediment ponds, or other controls. Needed to also evaluate shear stress on channels and on slopes.

Figure C-25 is an example site regional map (drawn on a USGS quadrangle) showing a construction site, and associated upland and downslope drainages. The previous discussion illustrated how it is possible to easily calculate the runoff characteristics affecting the site and downslope areas for different rain conditions. In addition, detailed site conditions for different project phases can also be evaluated for the design of appropriate erosion and sediment controls.

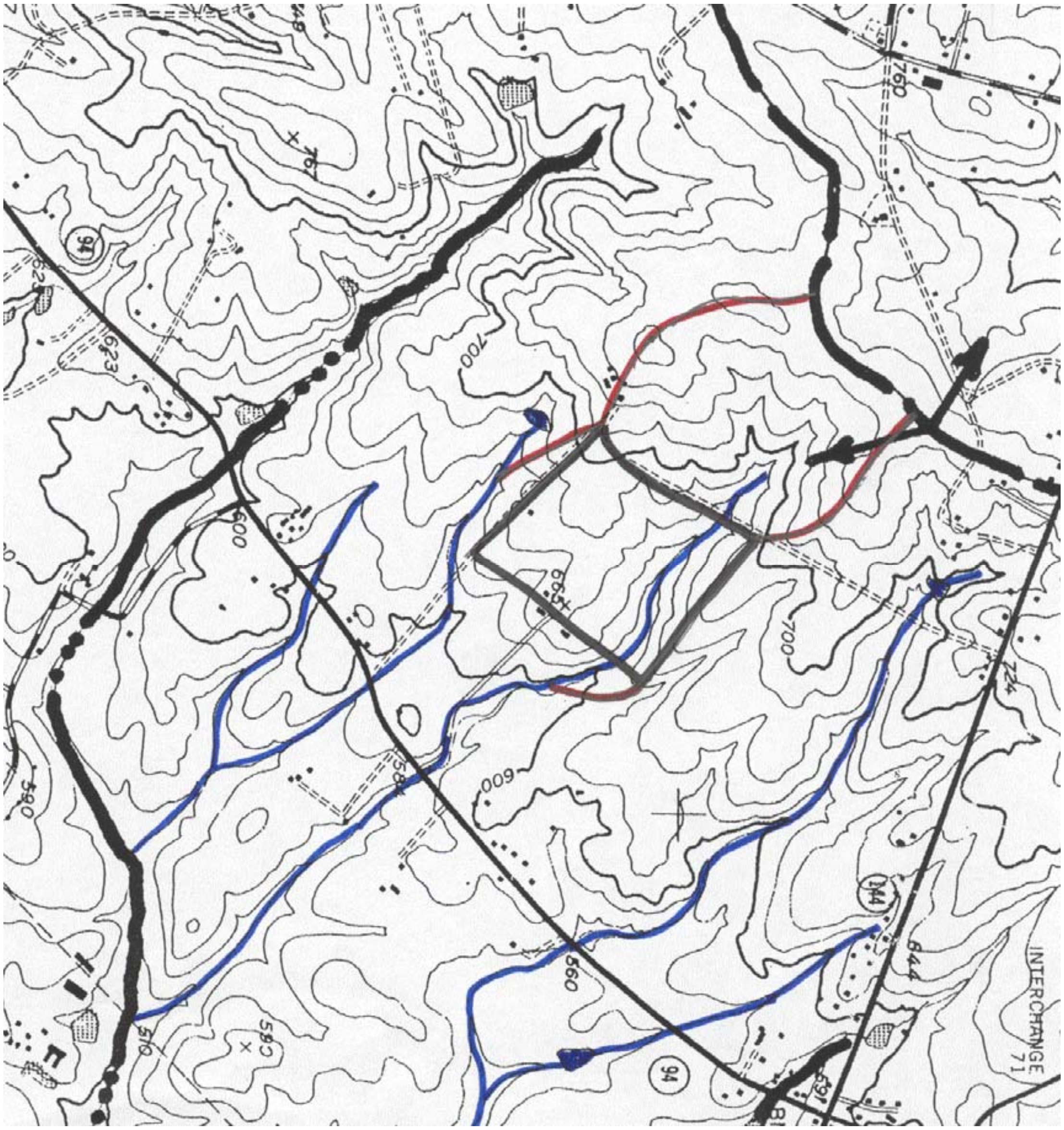


Figure C-25. Determination of general upslope and downslope drainage areas from hypothetical construction site.

Figure C-26 shows subdrainages for the upslope, downslope, and on-site areas for this example construction site. Table C-11 summarizes the characteristics of these areas, along with the hydrologic information needs for each area. Most of the site will be cleared and graded, except for the two small areas near the downslope edge. The upslope diversions will carry the upslope water to the main channel. The runoff from the O1 and O2 on-site areas will be controlled by slope mulches and filter fences, before the runoff drains to the on-site main channel. A sediment pond will be constructed at the downslope property boundary before this main channel leaves the site.

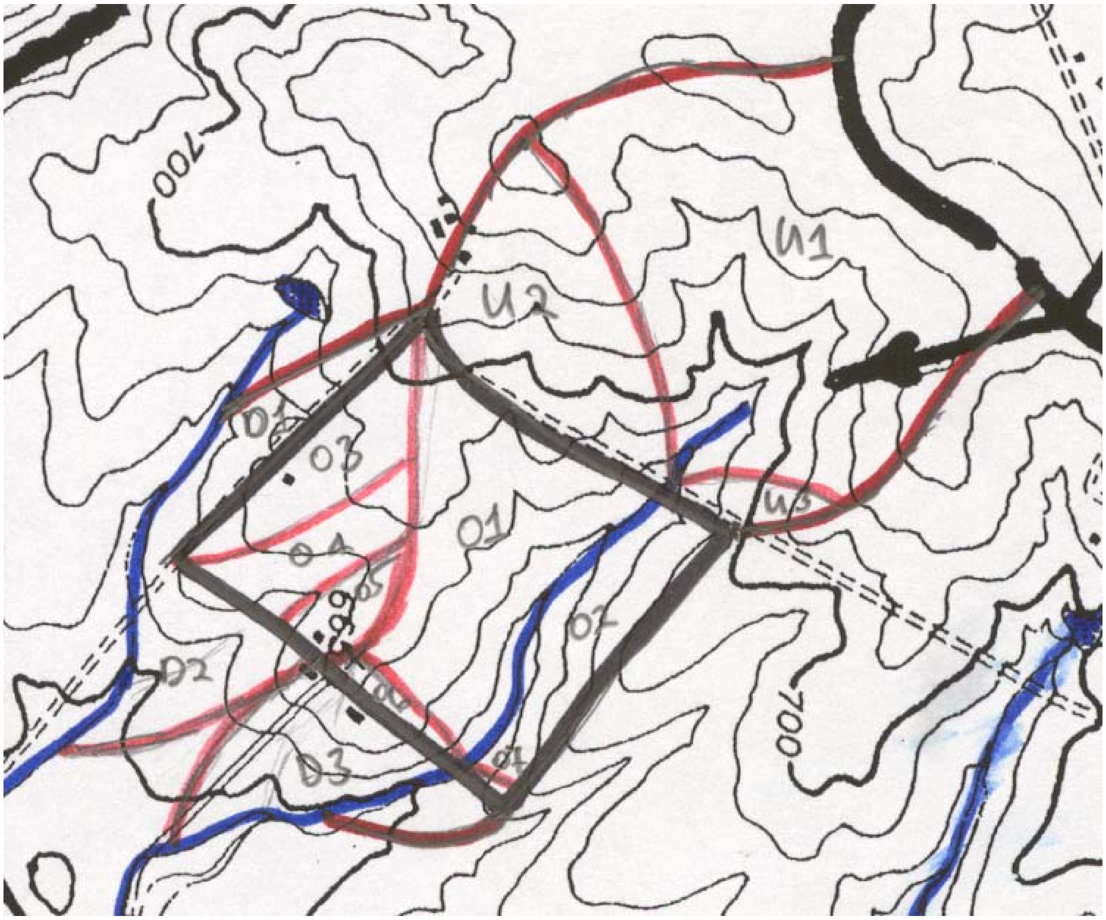


Figure C-26. Subdrainage areas on and near construction site.

Table C-11. Upslope and On-Site Subdrainage Area Characteristics for Construction Site

Area Notation	Location	Objective	Area (acres)	Cover n	Average flow path slope	CN (all "C" soils)	Tc (min)
U1	Upslope – direct to on site stream	Hydrograph (to be combined with U2 and U3)	37.4	0.4	8%	73	29
U2	Upslope – diversion to on site stream	Peak flow rate and hydrograph (to be combined with U1 and U3)	14.6	0.4	11.5	73	25
U3	Upslope – diversion to on site stream	Peak flow rate and hydrograph (to be combined with U1 and U2)	2.4	0.4	12.7	73	20.7
O1	On site – drainage to sediment pond and main site stream (also slope protection needed)	Peak flow rate and hydrograph	12.6	0.011	10	91	3.5
O2	On site – drainage to filter fence and main site stream (also slope protection needed)	Peak flow rate and hydrograph	7.1	0.011	10.5	91	1.6
O3	On site – towards perimeter filter fence (also slope protection needed)	Peak flow rate and hydrograph	6.1	0.011	5	91	4.1
O4	On site – towards perimeter filter fence (also slope protection needed)	Peak flow rate and hydrograph	3.1	0.011	6.7	91	3.3
O5	On site – towards perimeter filter fence (also slope protection needed)	Peak flow rate and hydrograph	1.8	0.011	11.3	91	1.5
O6	On site – nothing (will remain undisturbed)	na	1.3	0.24	6.7	na	na
O7	On site – nothing (will remain undisturbed)	na	0.3	0.24	10	na	na

Design Storms for Different Site Controls

All of the information needed to calculate the expected flows from these upslope and on-site areas is shown on Table C-12, except for the design storm. The area has a SCS type III rain distribution and the construction period will be one year. The different site features will require different design storms due to the different levels of protection that are appropriate. Table C-12 lists the features and the (assumed) acceptable failure rates during this one year period, along with the corresponding design storm frequency and associated 24 hr rain total appropriate for the area. The design storms range from 4.0 to 8.4 inches in depth and the times of concentration range from 1.5 to 30 minutes. The design rain intensities could be very large for some of these design elements.

Table C-12. Acceptable Levels of Protection for Different Site Activities

Site Construction Control	Acceptable Failure Rate during Site Construction Activities	Design Storm Return Period (years)	24-hr Rain Depth Associated with this Design Storm Return Period
Diversion channels	25%	6.5	5.5
Main site channel	5%	20	6.6
Site slopes	10%	10	6.0
Site filter fences	50%	1.9	4.0
Sediment pond	5% and 1%	20 and 100	6.6 and 8.4
Downslope perimeter filter fences	10%	10	6.0

The Use of WinTR-55 for Detention Pond Analyses

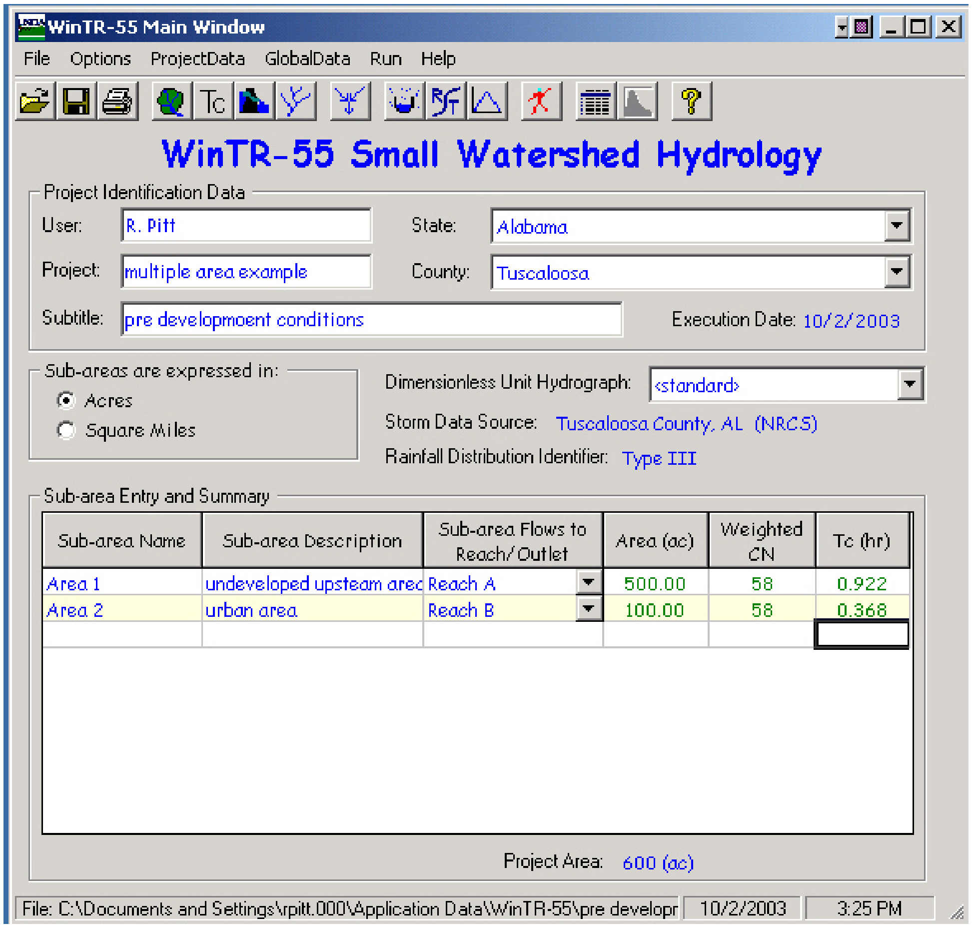
This example application of WinTR-55 demonstrates the use of this new program in evaluating detention ponds in a watershed with multiple subdrainage areas. As noted in the User Guide, WinTR-55 has some limitations compared to the more comprehensive TR-20 program. In the analysis of detention ponds ("structures"), the most important limitation is the availability of only 3 types of outlet structures (a broad-crested weir, a 90^o weir, and a pipe outlet). The greatest concern is how the pipe outlet is considered (a short-tube approximation approach). The USDA WinTR-55 team explained this as follows: This approximation uses the pipe diameter and the head on the pipe as the total head (permanent pool elevation to outlet invert plus 1/2 diameter). This estimation of head, coupled with a slightly different orifice flow coefficient (0.6 instead of 0.8), essentially cancel each other out and the result is a higher discharge estimate for one type of pipe material versus a slightly lower discharge estimate for another. The pipe materials checked were reinforced concrete and corrugated metal. Overall, the estimated differences were very small. The future version of the User Manual will be rewritten to reflect this short tube flow assumption with a disclaimer that if the user needs a more exact estimate they should use a different tool (SITES or a user-estimated rating in TR-20). They also stated that Version 1.0 of WinTR-55 will keep the existing short-tube approximation for pipe outlets of structures. A future version 2.0 of WinTR-55 will likely have the ability to enter a user-provided stage-storage-discharge rating curve or more complete pipe rating curves.

WinTR-55 is a great improvement over the older TR-55 in that more accurate channel and reservoir routing is provided. This Windows version of the program is also very easy to use and the provided graphical output options enable efficient and rapid evaluations.

This simple example is comprised of two subwatersheds, a 500 undeveloped area and an adjacent 100 developing area. Specific characteristics of these areas (soils, land use breakdowns, channel characteristics, etc.) are provided in the following discussion. Initially, the pre-development conditions are examined, followed by developed conditions. A preliminary design of a detention pond is then evaluated to attempt to provide similar discharge peak flows from the developed watershed portion after development as before development.

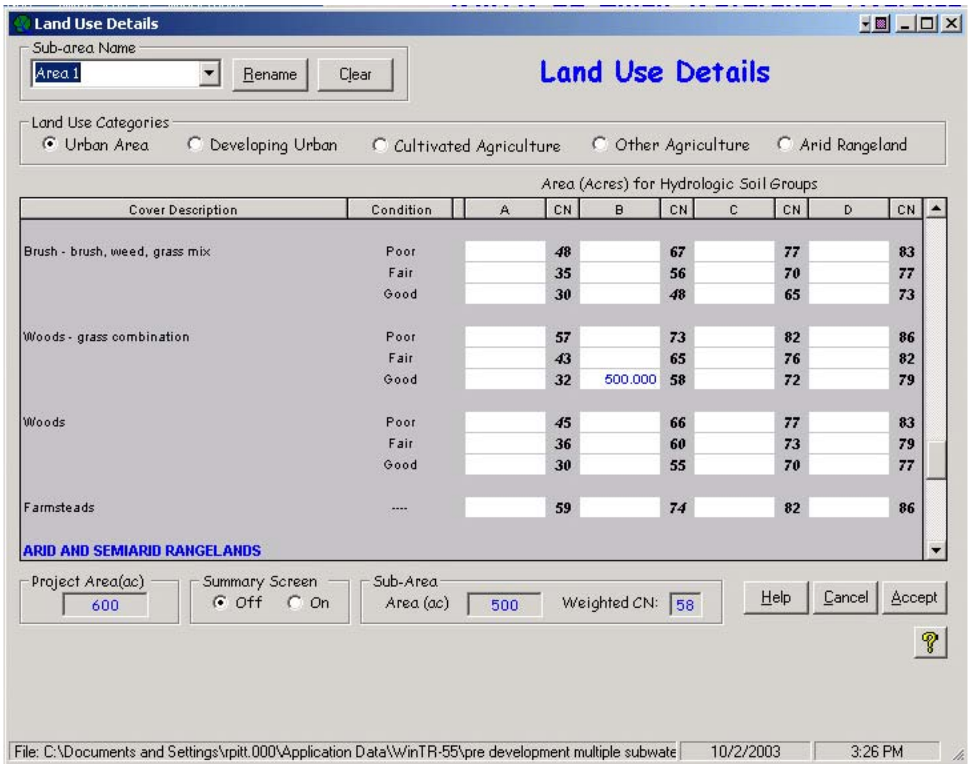
Predevelopment Conditions

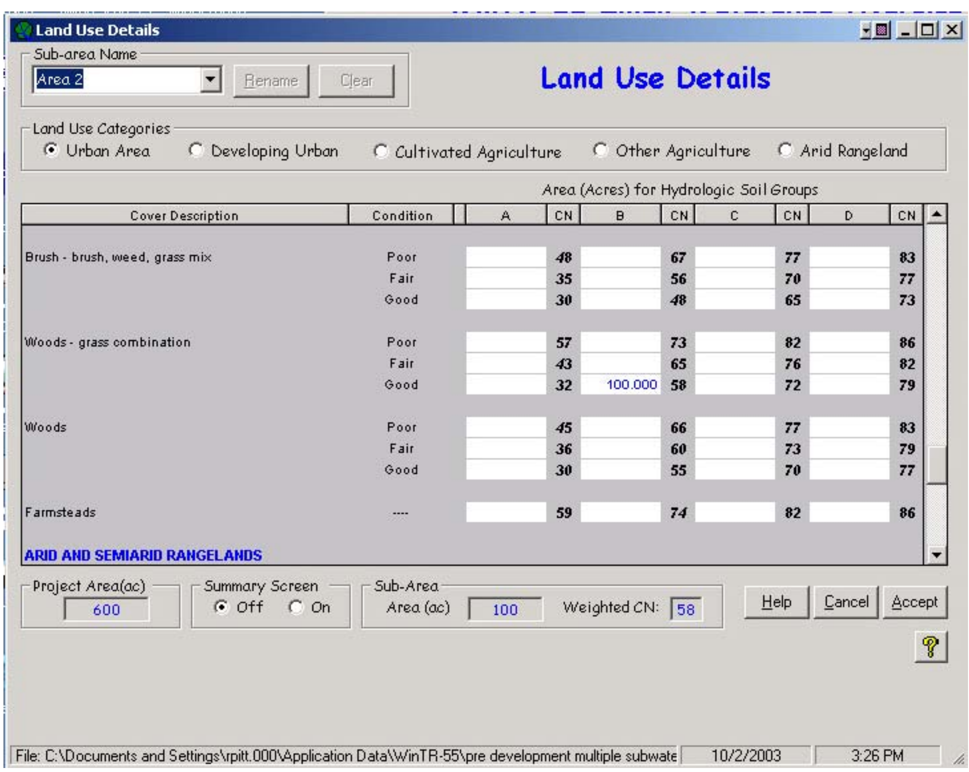
The following screen shows the basic site conditions. The screen also shows the location of the area (Tuscaloosa County, Alabama), the selection of the standard dimensionless hydrograph, the selection of the area units, and labels. The drop-down "options" menu was also used to select "English" units (actually US customary units). The area, CN, and Tc values area entered and calculated in other screens and the information was automatically transferred to this screen.



In order to enter the area, CN, and Tc screens, double-click on one of the cells in the columns under the desired label. The following screen is opened when either the area or CN column is selected. The screen shows the complete listing of available land uses and surface covers for each of the 4 hydrological soil groups (scrolling is needed to see all the options). Type in the area associated with each condition for each area. In this example, the pre-development condition is woods-grass combination in good condition, with B soils. Area 1 is 500 acres in size, while Area 2 (the developing area) is 100 acres in size. These pre-development conditions are the same throughout the sub-areas, but it is possible to select a variety of conditions and have the program automatically weight the overall CN. If desired, it is possible to directly enter the CN value without using the calculator.

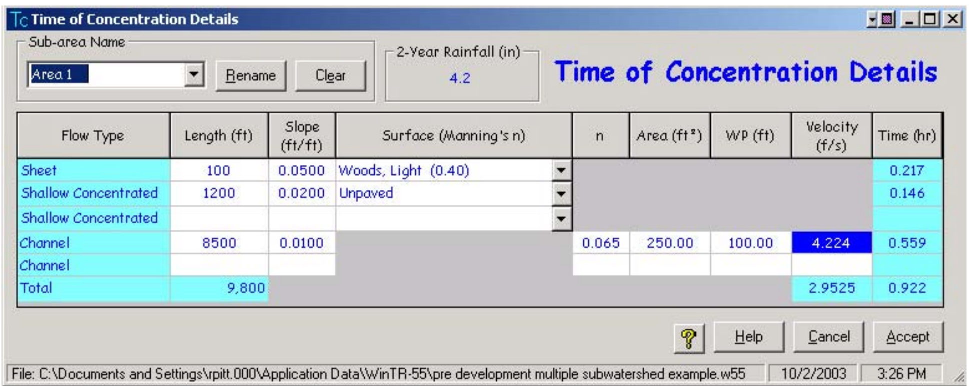
Although not noted in the WinTR-55 User Guide, the prior TR-55 guidance recommended that the range of CNs for one area should be relatively narrow, with no more than an extreme difference of 5 in the CNs for any area. If the CN values varied by more than 5, it was recommended that the sub-area be further divided to place the extreme values in separate sub-areas. This was recommended to enable more accurate routing of sub-area flows compared to using a composite CN based on a wide range of individual values.

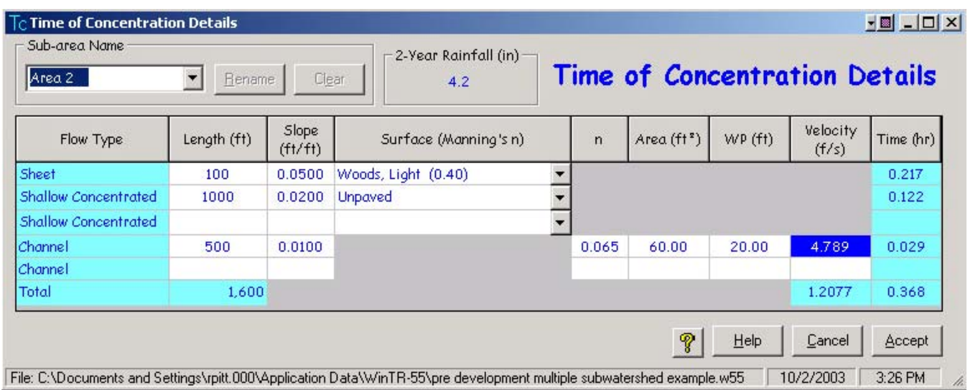




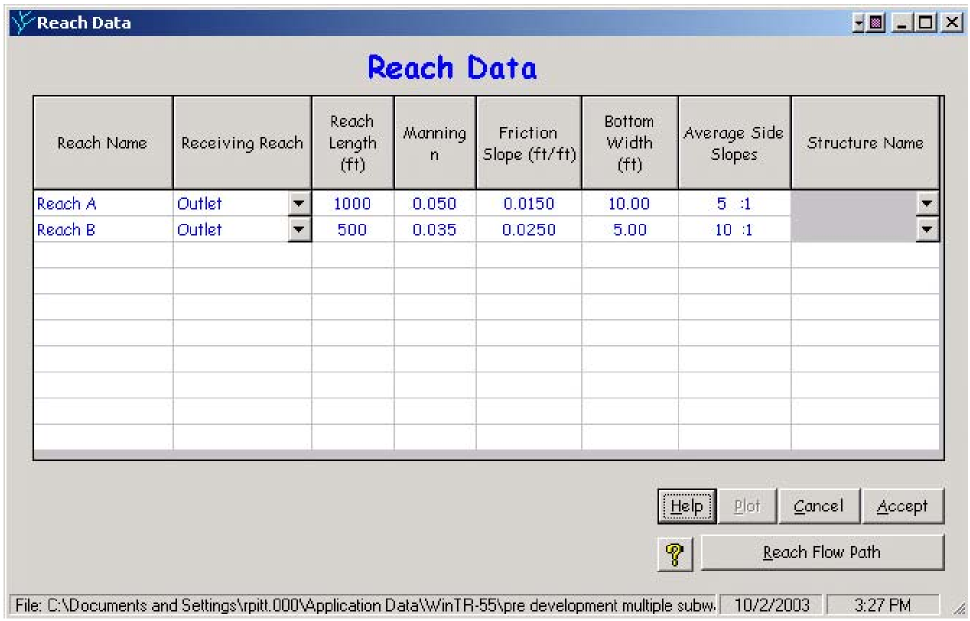
The Tc calculation screen can be opened in the same way, by double-clicking on any cell under the Tc column. If available, the Tc can be directly entered without using the calculator. The following screens show the examples for sub-areas 1 and 2 (selected by using the drop-down option under “sub-area name”). The flow path described on the screens needs to be pre-determined to be the critical Tc flow path (the path that requires the longest time for water drainage, not the physically longest flow path necessarily).

As in TR-55, the Tc can be comprised of three components. The sheet flow length is now restricted to a maximum length of 100 ft. Prior TR-55 guidance allowed a maximum length of 300 ft, but this was thought to be excessive by the WinTR-55 development team. The “Surface (Manning’s n)” menu lists the available sheetflow roughness values. These are substantially different than what would be appropriate for channel flow conditions for rougher material. Smooth surfaces have similar values. The shallow concentrated flow surface drop-down options are restricted to “paved” and “unpaved.” Two shallow concentrated flow segments are allowed. There are also two channel segments allowed. These are usually designated as streams on USGS topographic maps.

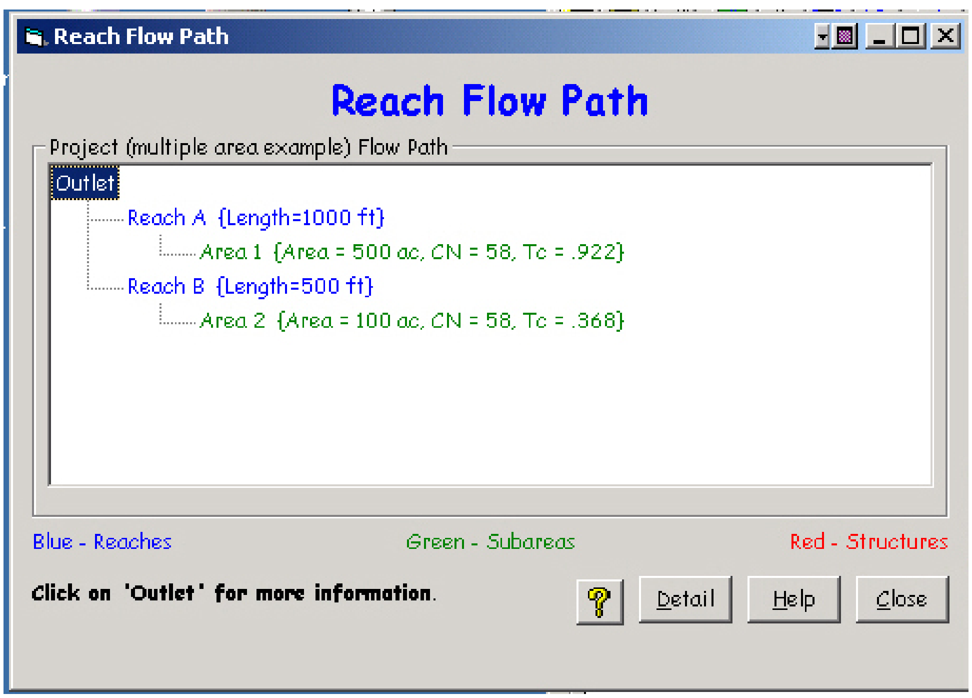




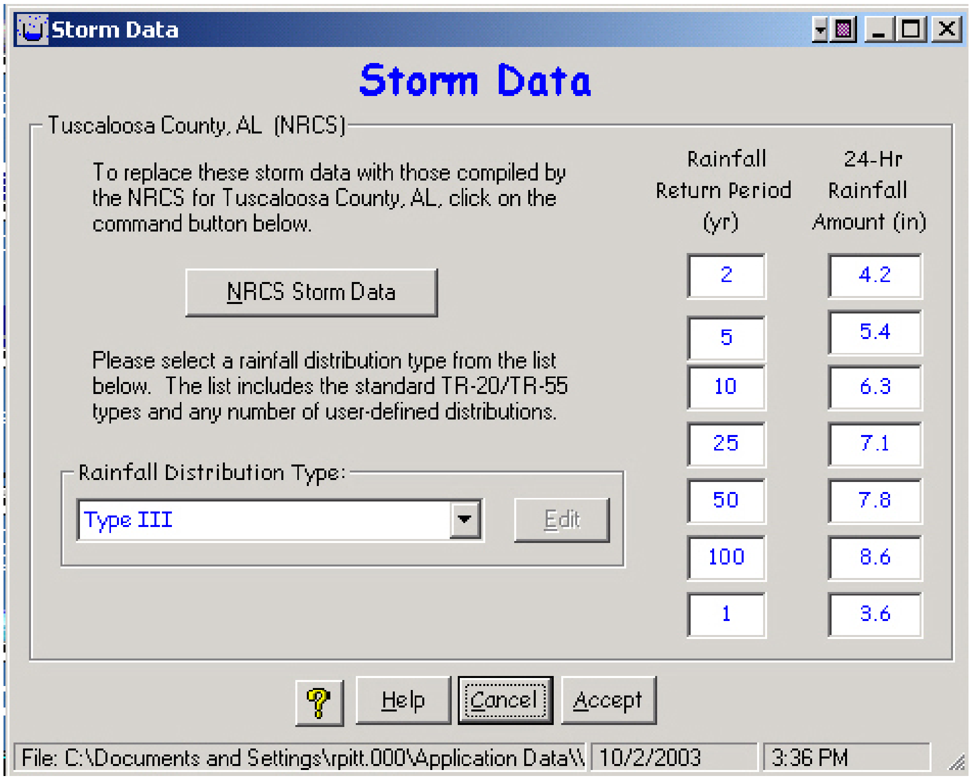
The “Reach Data” also needs to be entered. These are not the channels described on the Tc screen. The Tc channels are located within the sub-areas. The Reach channels are the channels into which the sub-areas discharge (and as noted on the opening screen). This screen also asks for the receiving reach into which each reach discharges. It is also possible to designate the outlet as the receiving reach, as in this example. This screen is also used to designate a reach as a structure (“reaches” can be either channels or detention ponds, with the appropriate routing procedure used). If the structure has already been described, then the structure name will appear on the structure name drop-down menu.



The "Reach Flow Path" should be selected to confirm that the model interpreted the entered the area and reach connections correctly. This screen shows the basic watershed area conditions, plus shows the reaches each sub area flows into, plus shows how the reaches are combined as they flow downstream. It is possible to construct and evaluate a very complex set of sub-areas for evaluation. This example is about as simple as possible and still show pond and sub area hydrographs can be combined.

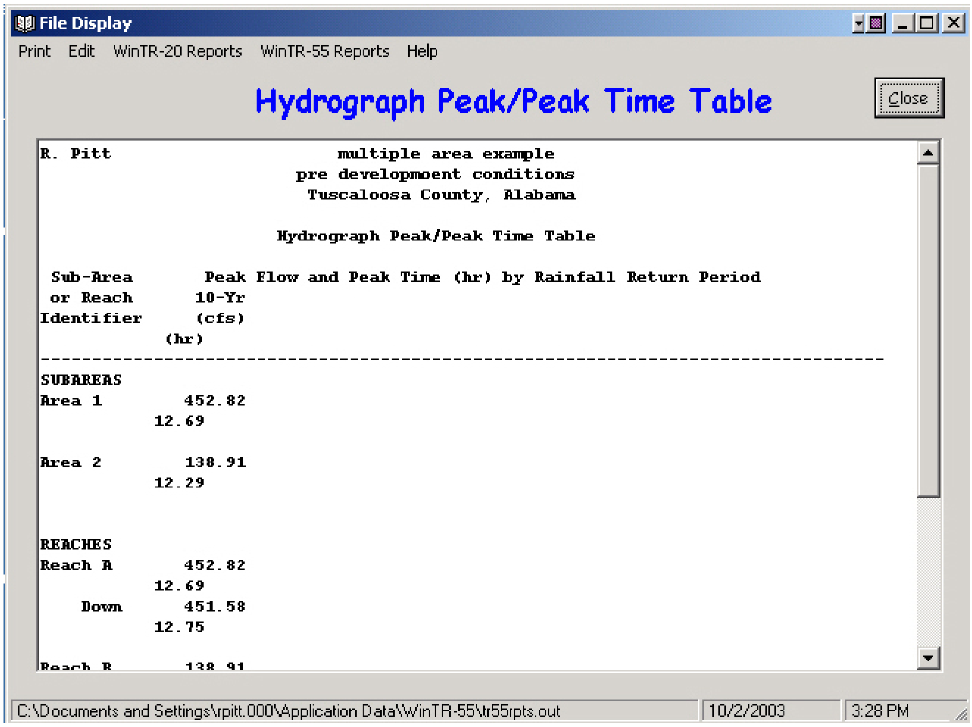


Finally, the "Storm Data" must be selected, or entered. The following screen is available under the "GlobalData" drop down main menu. If the "NRCS Storm Data" button is selected, the standard 24-hr rainfall amounts and appropriate Rainfall Distribution Type are used, corresponding to the county selected on the first program screen. It is possible to enter other rainfall amounts. WinTR-55 is an event model that is used for individual design storms, although WinTR-55 can examine the entire set, or a sub-set, of the standard storms. Although the 24-hr rainfall amounts are used, the critical rain intensity corresponding to the Tc is actually used to normalize the dimensionless hydrograph.

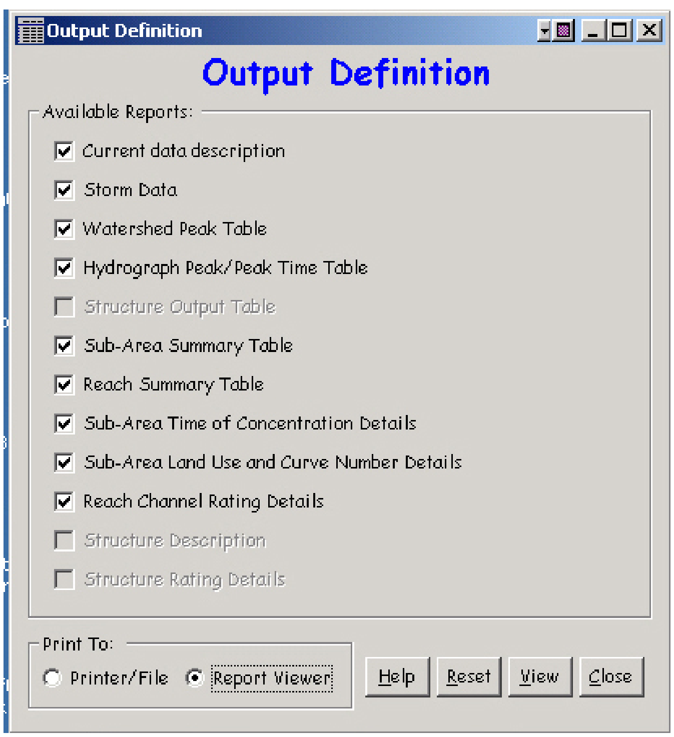


The "Run" icon is then selected and the following screen appears. This screen is used to select which event(s) are to be evaluated.

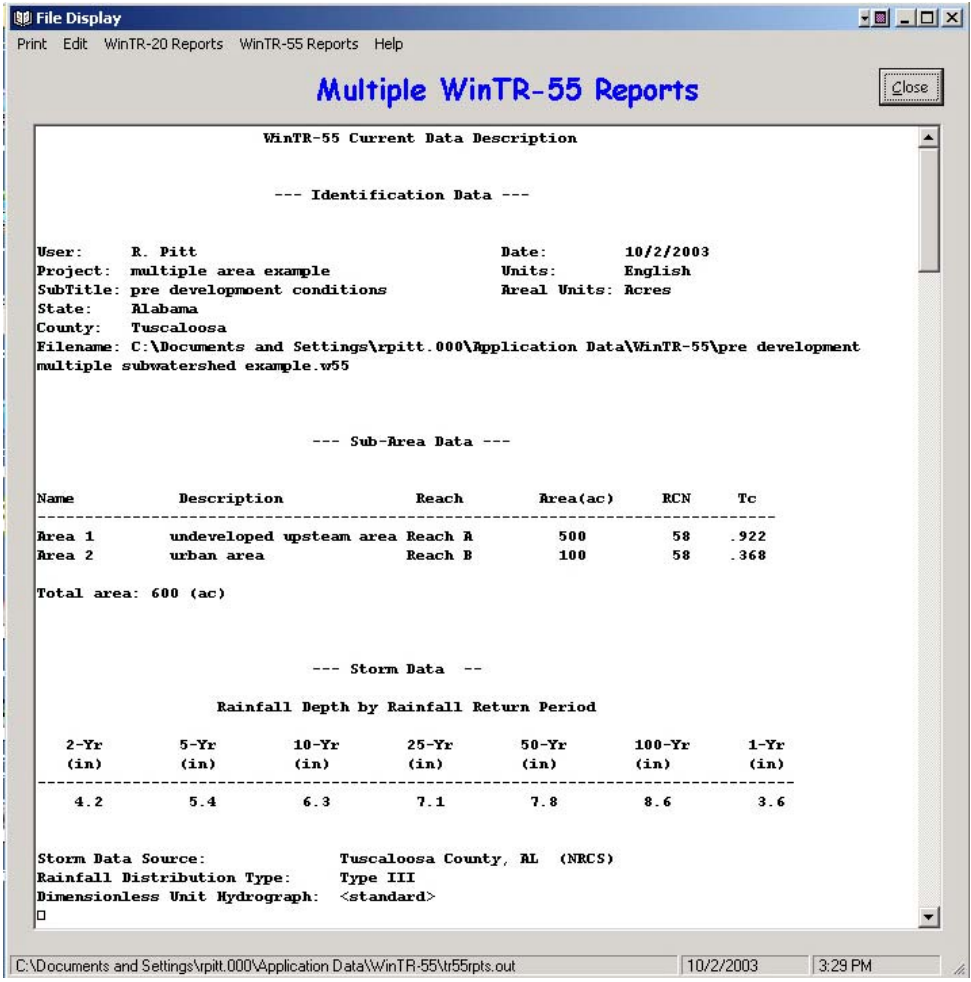
When the "Run" button is selected, after clicking on each desired rain, the program calculates the site runoff and routes it through each reach. An embedded version of TR-20 is actually used to conduct the analyses, being much better than the prior manual TR-55 procedures which required rather crude increments of important site factors. The following screen is then automatically displayed after a run. This screen displays the TR-20 output screen, showing the peak runoff conditions and times. It is also possible to select "WinTR-20 Reports" for more detailed output information.



The “Output Definition”, or report writer, icon displays the following screen. This allows specific information to the produced in a written report, or displayed on the computer screen.



The following is an on-screen report.


File Display

Print Edit WinTR-20 Reports WinTR-55 Reports Help

Multiple WinTR-55 Reports

Close

WinTR-55 Current Data Description

--- Identification Data ---

User: R. Pitt Date: 10/2/2003
Project: multiple area example Units: English
SubTitle: pre developmoent conditions Areal Units: Acres
State: Alabama
County: Tuscaloosa
Filename: C:\Documents and Settings\rpitt.000\Application Data\WinTR-55\pre development multiple subwatershed example.w55

--- Sub-Area Data ---

Name	Description	Reach	Area(ac)	RCN	Tc
Area 1	undeveloped upsteam area	Reach A	500	58	.922
Area 2	urban area	Reach B	100	58	.368

Total area: 600 (ac)

--- Storm Data --

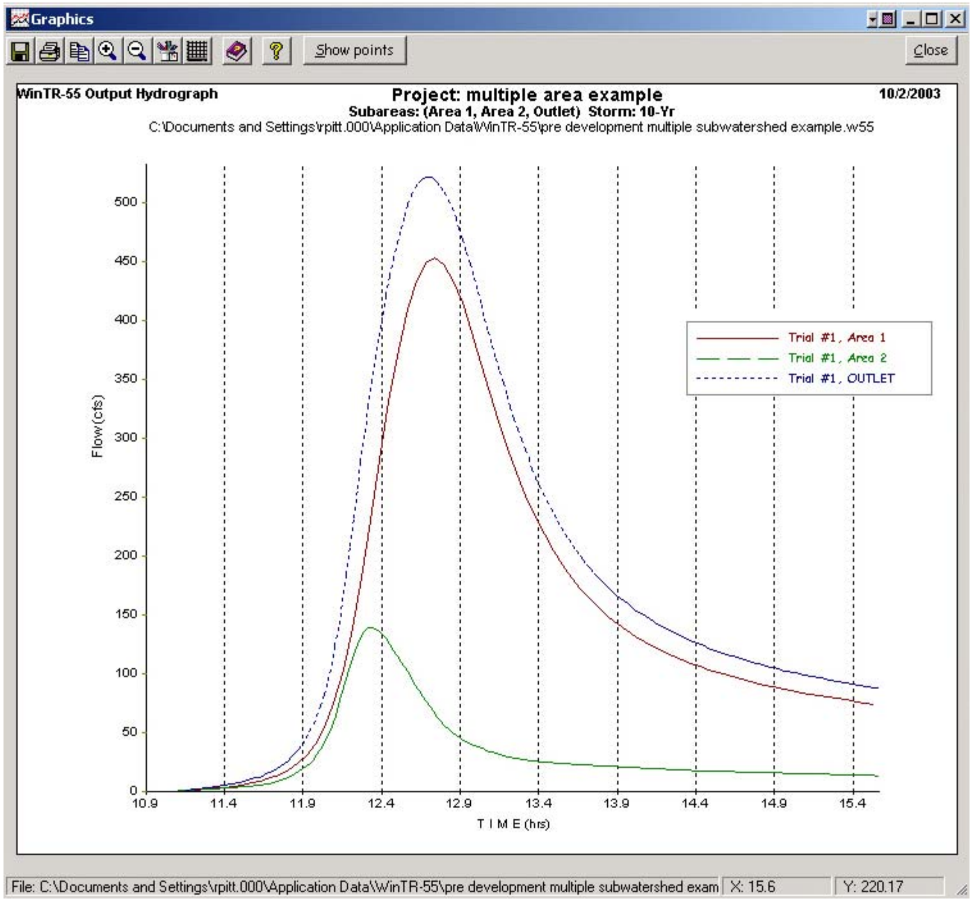
Rainfall Depth by Rainfall Return Period

2-Yr (in)	5-Yr (in)	10-Yr (in)	25-Yr (in)	50-Yr (in)	100-Yr (in)	1-Yr (in)
4.2	5.4	6.3	7.1	7.8	8.6	3.6

Storm Data Source: Tuscaloosa County, AL (NRCS)
Rainfall Distribution Type: Type III
Dimensionless Unit Hydrograph: <standard>

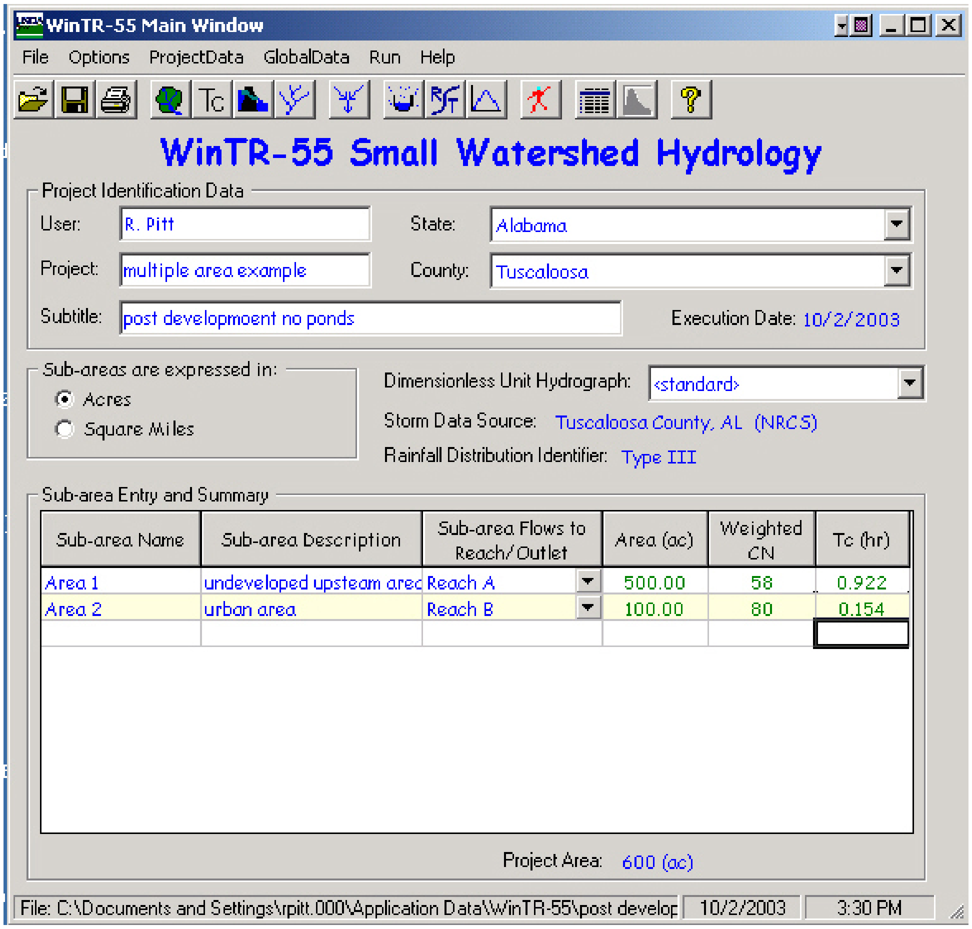
C:\Documents and Settings\rpitt.000\Application Data\WinTR-55\tr55rpts.out 10/2/2003 3:29 PM

The following is the hydrograph that can be plotted by selecting the next to last icon on the top tool bar. The selection screen allows different hydrographs to be displayed. This plot shows how the pre-development hydrographs from the two sub-areas join for the complete hydrograph. The 10-year storm (having a 10% chance of occurring in any one year) produces a peak flow of about 139 cfs in the developing watershed. The upland sub-area peak flow was about 453 cfs, while these combined to create a total basin peak flow of about 522 cfs.

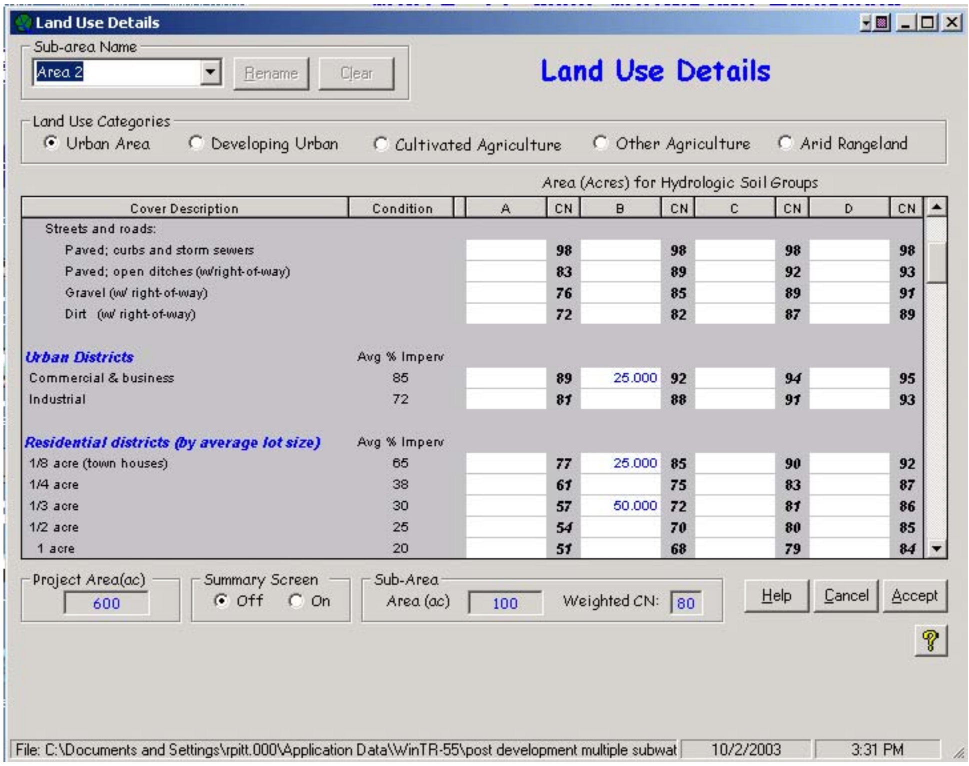


Post Development Conditions

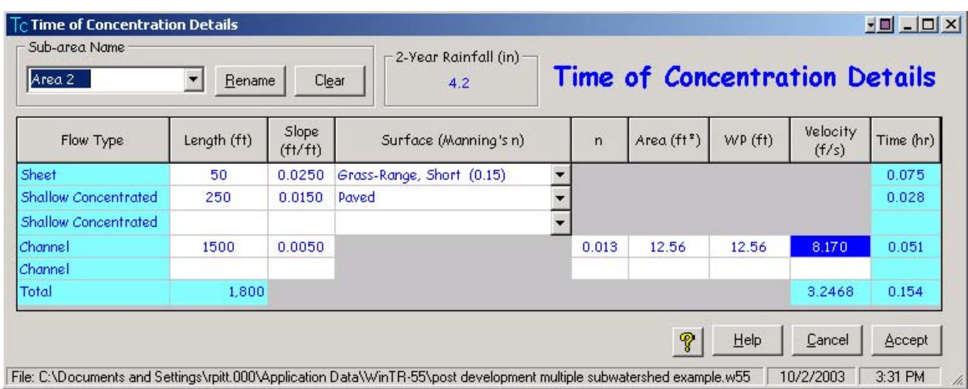
The pre-development file was edited and re-saved (using the "save as" option under the file drop-down menu) to reflect developed conditions in sub-area 2, as shown on the following screens:



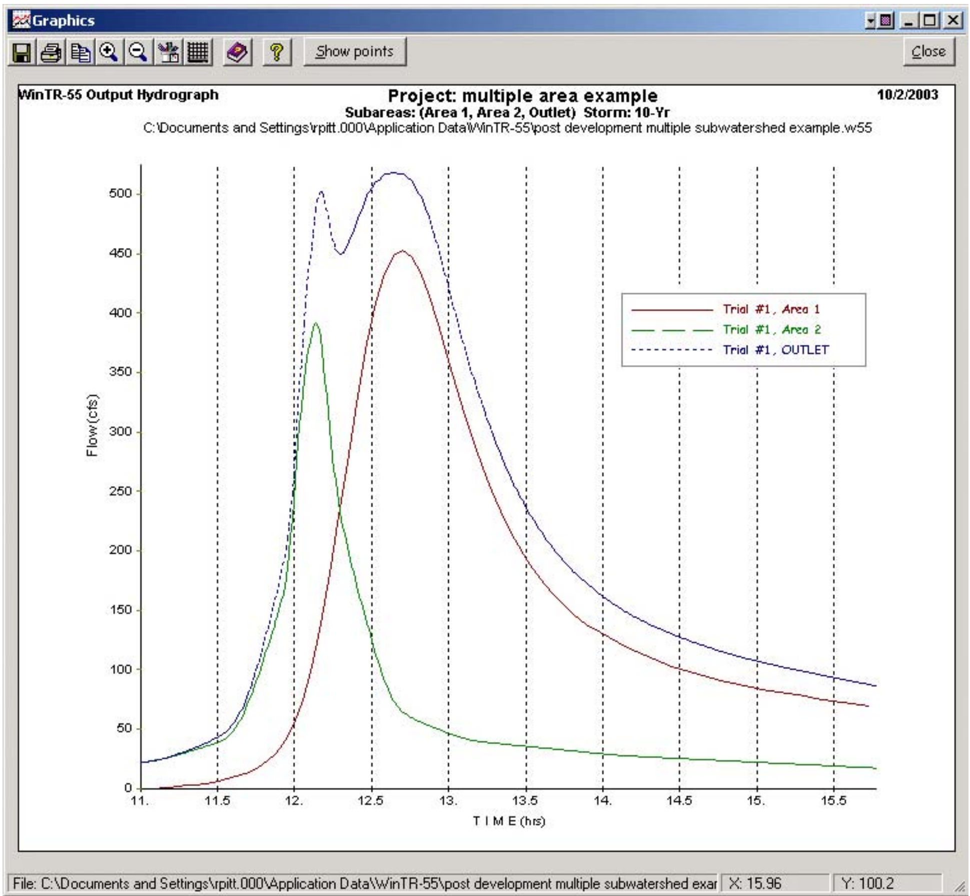
The developed 100 acre sub-area is comprised of 25 acres of commercial, 25 acres of town houses, and 50 acres of 1/3 acre lot residential areas (notice that the individual CNs range from 72 to 92, much broader than a difference of 5. Therefore, this area should be further sub-divided to separate the individual land uses, if possible. They were not in this example though).



The Tc factors also changed substantially for sub-area 2 after development:



When the same 10-year storm was evaluated, the following hydrograph was produced:



The upper sub-area (#1) had the same hydrograph characteristics, but the urbanized sub-area (#2) had a substantial increase in runoff volume and peak flow rate. The above composite hydrographs also show that the peaks are much more separated after development, with the hydrograph of the developed area to develop and recede much faster than the slower responding upper area sub-area. The developed area now has a peak flow rate of 391 cfs, but because the hydrograph components are more separated than for pre-developed conditions, the overall total peak hydrograph actually decreases slightly, to about 518 cfs.

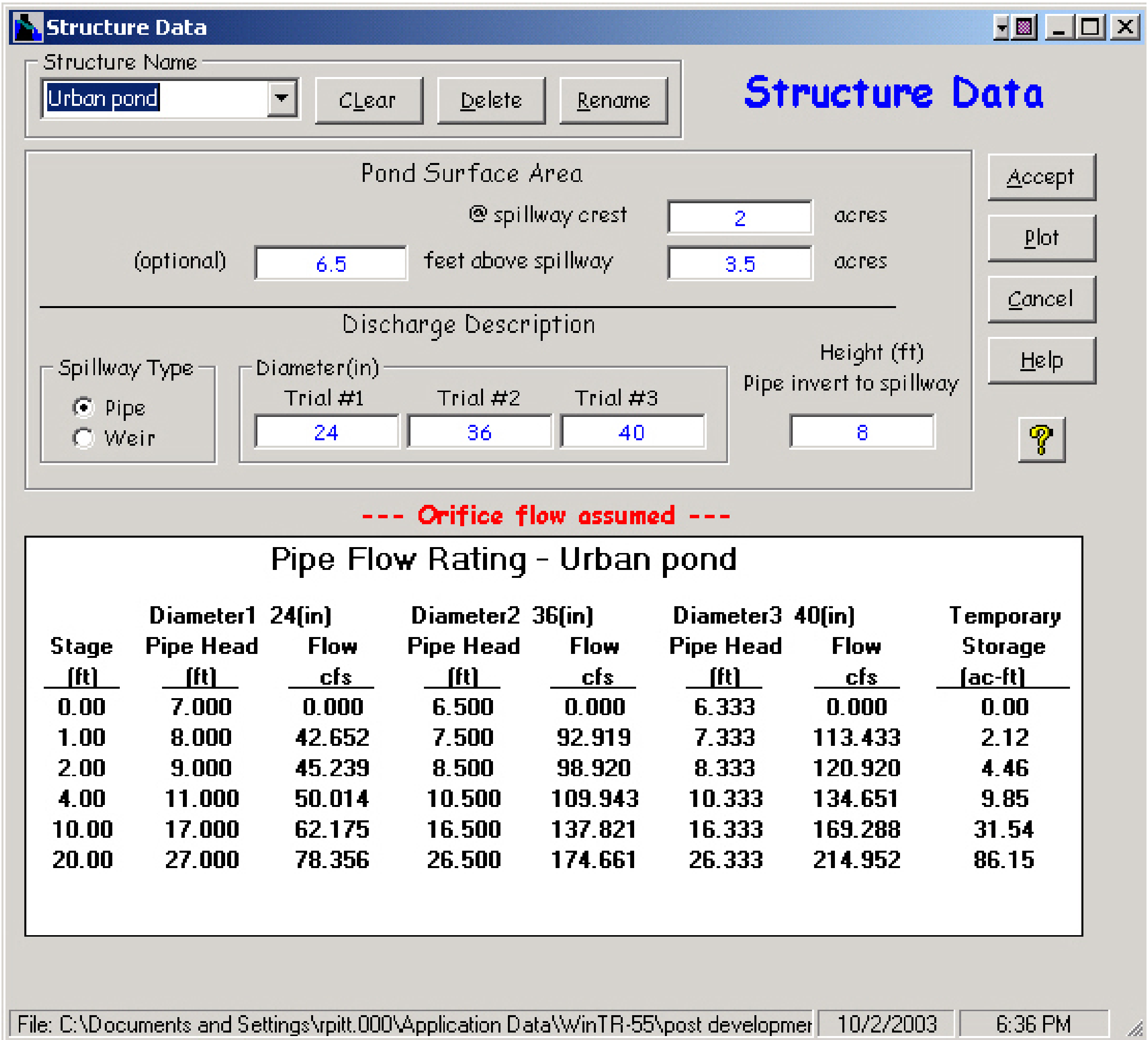
Post Development with Pond

Even though the total area peak flows are actually less after development with no pond, the site development standards still required a detention pond to reduce the post-development peak flow to the pre-development levels for the area undergoing development. The WinTR-55 suggests a simplified approach to size the needed pond based on the difference in the runoff volumes for pre and post-development conditions, and restricting the pond outlet device to the pre-development flow.

The “WinTR-20 Reports” lists the runoff depth, in watershed inches. The pre-development runoff was reported to be 1.95 inches (over 100 acres). This corresponds to about 16.2 acre-feet. The post-development runoff depth was about 4.05 inches (also over the same 100 acres), corresponding to about 33.8 acre-feet. The difference (and “required” pond storage) is therefore 17.6 acre-feet. The maximum pond discharge was the pre-development peak flow (for the 10-year storm for this example) of 139 cfs.

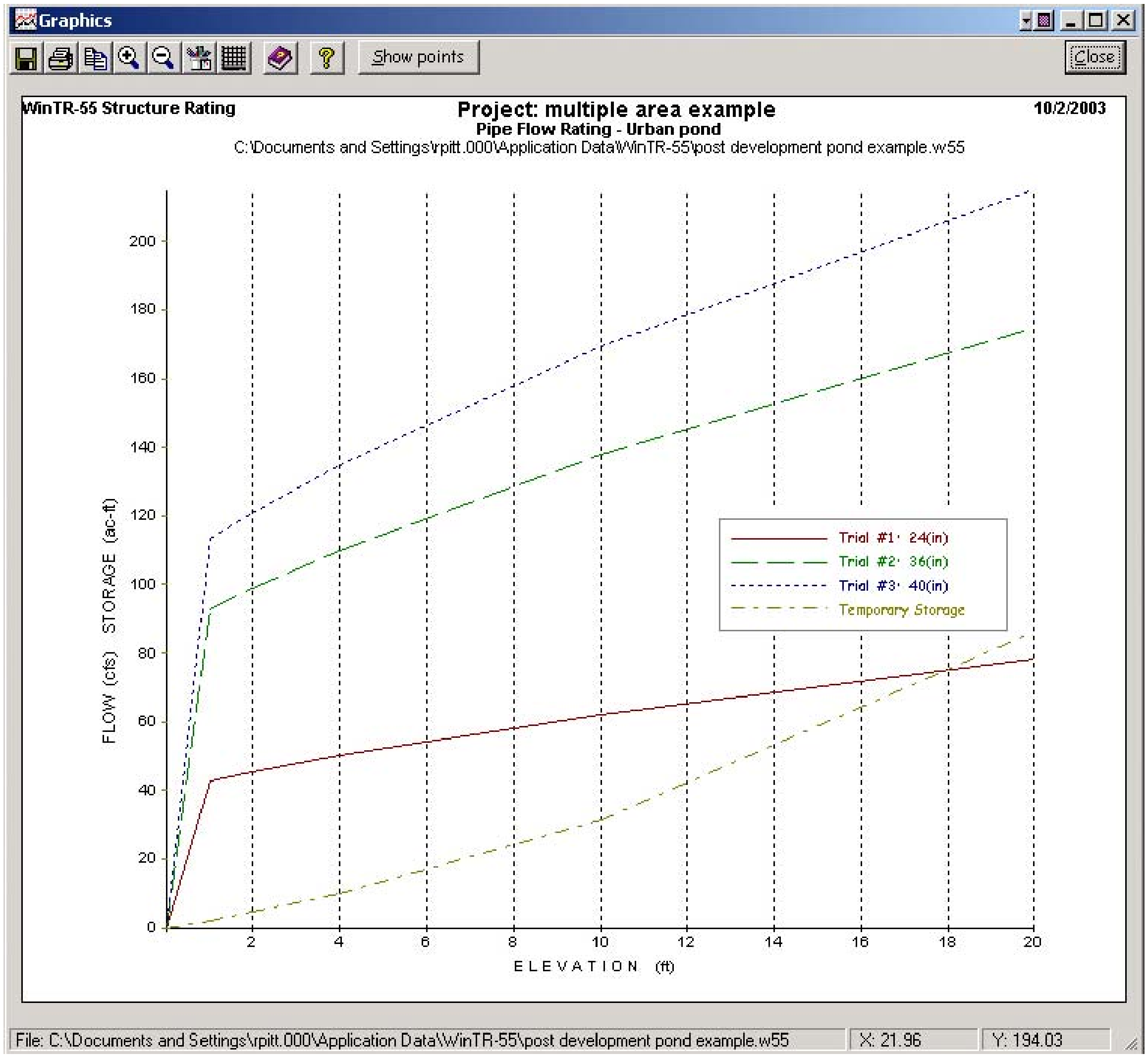
The pond size can then be crudely sized using these values. However, this was to be a multi-purpose pond, also providing water quality benefits. A rough guide for the pond surface area (the bottom of the storage layer) for water quality benefits can be estimated to be about 3% of the watershed paved area, plus 0.5% of the watershed pervious area. The CN menu presented the watershed % imperiousness areas for each development category. The commercial area is assumed to have 85% impervious area, the high density residential area to have 65% imperviousness, and the low density residential area to have 30% imperviousness. A simple calculation resulted in a pond bottom area (the actual surface of the permanent pool, which needs to be at least 3 feet deep), of 1.78 acres. A value of 2 acres will therefore be used. If this portion of the pond is 6.5 feet deep, and the top area is 3.5 acres, the pond side slopes would be about 7.3:1 (H:V), a reasonable value, to provide about 17.6 acre-feet of storage.

The first step was to describe the pond and to edit the post-development file to change Reach B from a channel to a pond. The following is the description of the pond "structure" using the "Structure Data" top menu bar option. The pond surface areas are described using the above calculated estimates. The area is 2 acres at the depth where the discharge begins, and is 3.5 acres in area 6.5 feet above this spillway elevation. WinTR-55 will assume a deeper pond as needed (above 6.5 feet) but will use this side slope. If the upper area was not entered (it is an optional value), the pond is assumed to then have vertical side slopes (not a good idea). The Discharge Description" is based on the spillway type selected, either a pipe (using the pipe approach previously described), or a weir. If a weir is selected, it can be a broad-crested weir and the weir length entered. If a 0 value is entered for the weir length, the model will assume a 90^{o} V-notch weir. If a pipe spillway is selected (as in this example), the pipe diameter (in inches) is given, ranging from 6 to 60 inches. When a pipe is selected, the height from the invert of the discharge end of the pipe to the spillway elevation is also needed for the simplified equation. This height must be at least twice the diameter of the pipe. Up to three pipe diameters (or weir lengths) can be entered. The model will evaluate all three options, making the selection of the choice easier. As the dimensions are entered, the rating curves (flow vs. height) and storage below the elevations are displayed. This is a good indication of the correct spillway size, as the maximum discharge close to the desired pond depth can be observed. In this case, the 40 inch pipe has the desired discharge of 139 cfs at a stage slightly above 4 feet, and well under 10 feet. The 36 inch pipe option would need about 10 feet of stage (greater than planned), while the 24 inch pipe would require even more (more than 20 ft). Therefore, it is expected that the 3rd pipe option, the 40 inch pipe would work best.

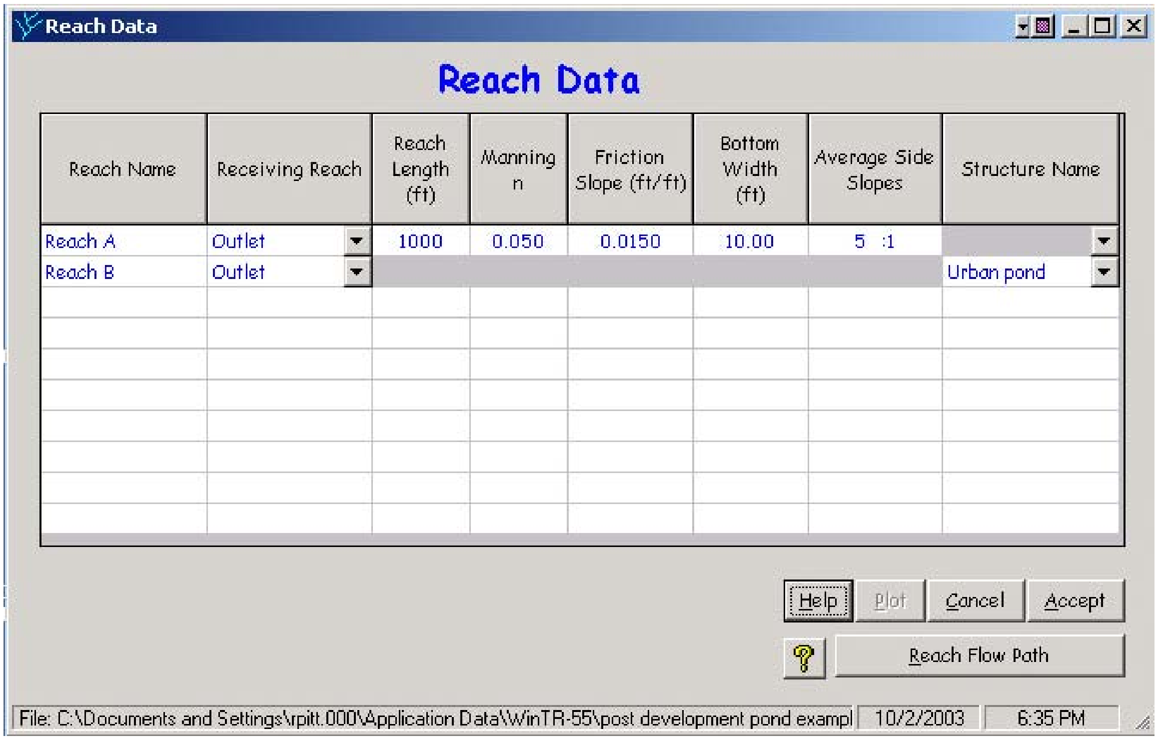


	Diameter1 24(in)		Diameter2 36(in)		Diameter3 40(in)		Temporary
Stage (ft)	Pipe Head (ft)	Flow cfs	Pipe Head (ft)	Flow cfs	Pipe Head (ft)	Flow cfs	Storage (ac-ft)
0.00	7.000	0.000	6.500	0.000	6.333	0.000	0.00
1.00	8.000	42.652	7.500	92.919	7.333	113.433	2.12
2.00	9.000	45.239	8.500	98.920	8.333	120.920	4.46
4.00	11.000	50.014	10.500	109.943	10.333	134.651	9.85
10.00	17.000	62.175	16.500	137.821	16.333	169.288	31.54
20.00	27.000	78.356	26.500	174.661	26.333	214.952	86.15

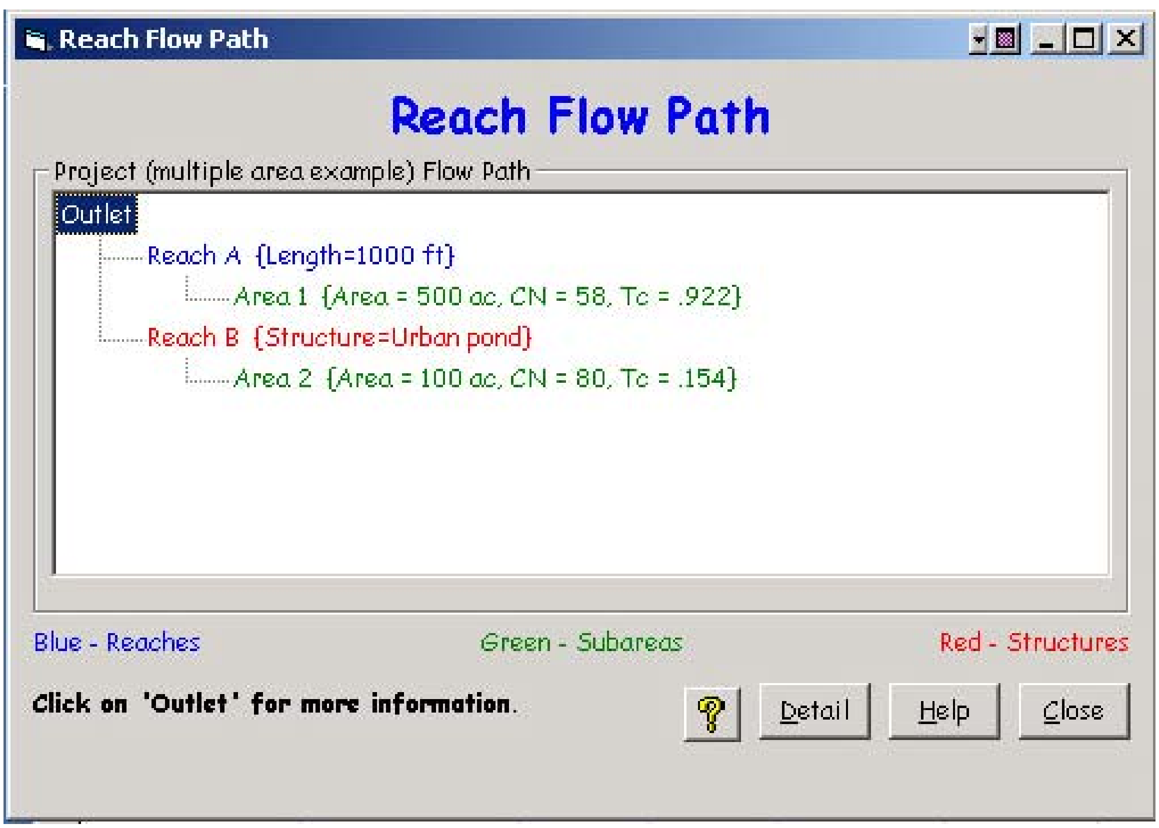
A rating curve can also be plotted for each outlet option if the “Plot” option is selected on the structure screen. This plot confirms that the 40 inch pipe discharge would require about 5 feet of the available pond stage.



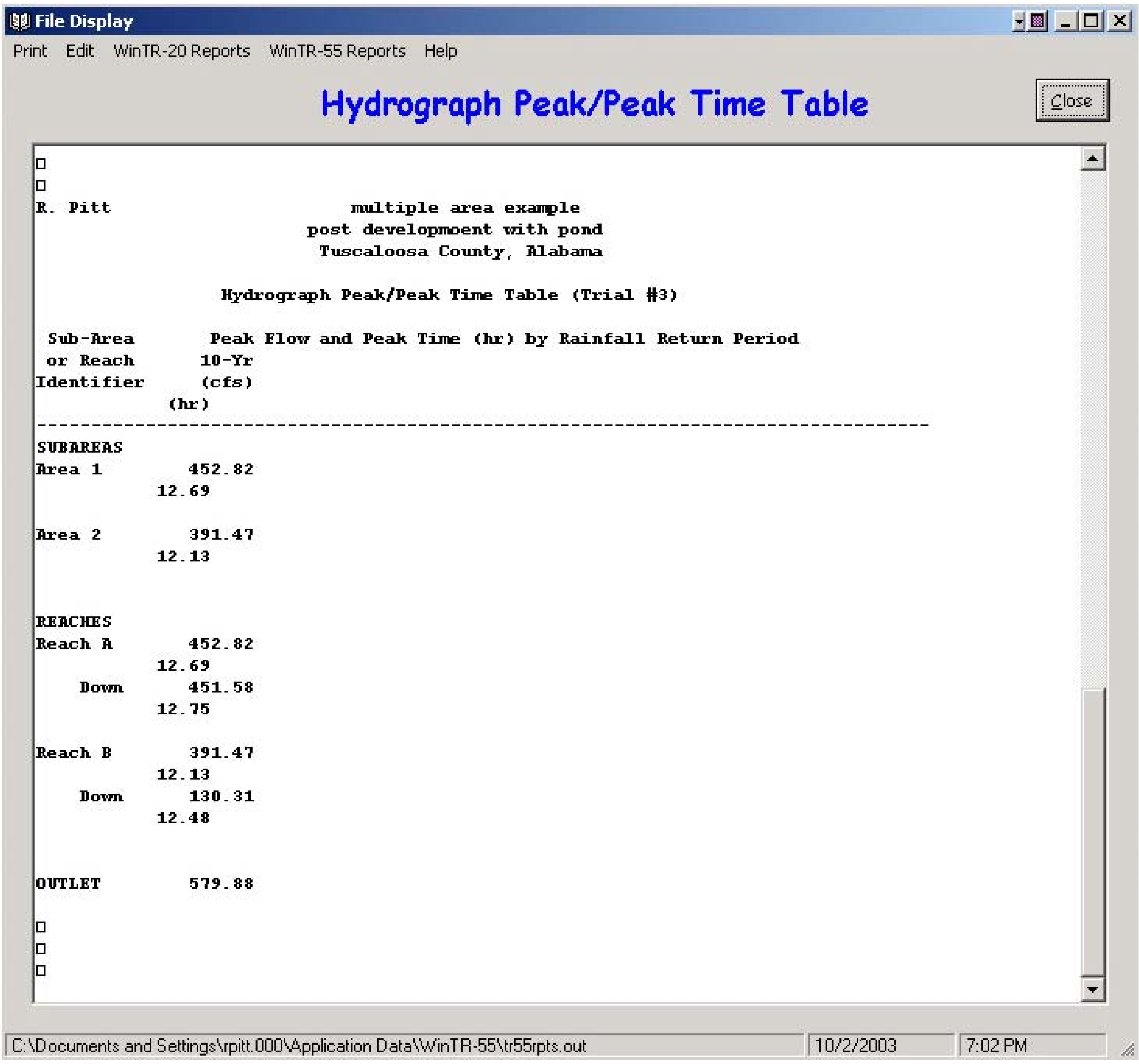
The reach is then modified to be a pond instead of a creek. The "Structure Name" drop-down menu in the appropriate cell is used to select the available pond name (available after the "accept" button on the pond menu is clicked). The creek data, if previously on the reach data menu row for the named reach that is now a pond, needs to be deleted.



The "Reach Flow Path" screen (selected from the "Project Data" drop down menu) can also be selected to ensure that the model has the outfall, reaches and areas correctly connected:

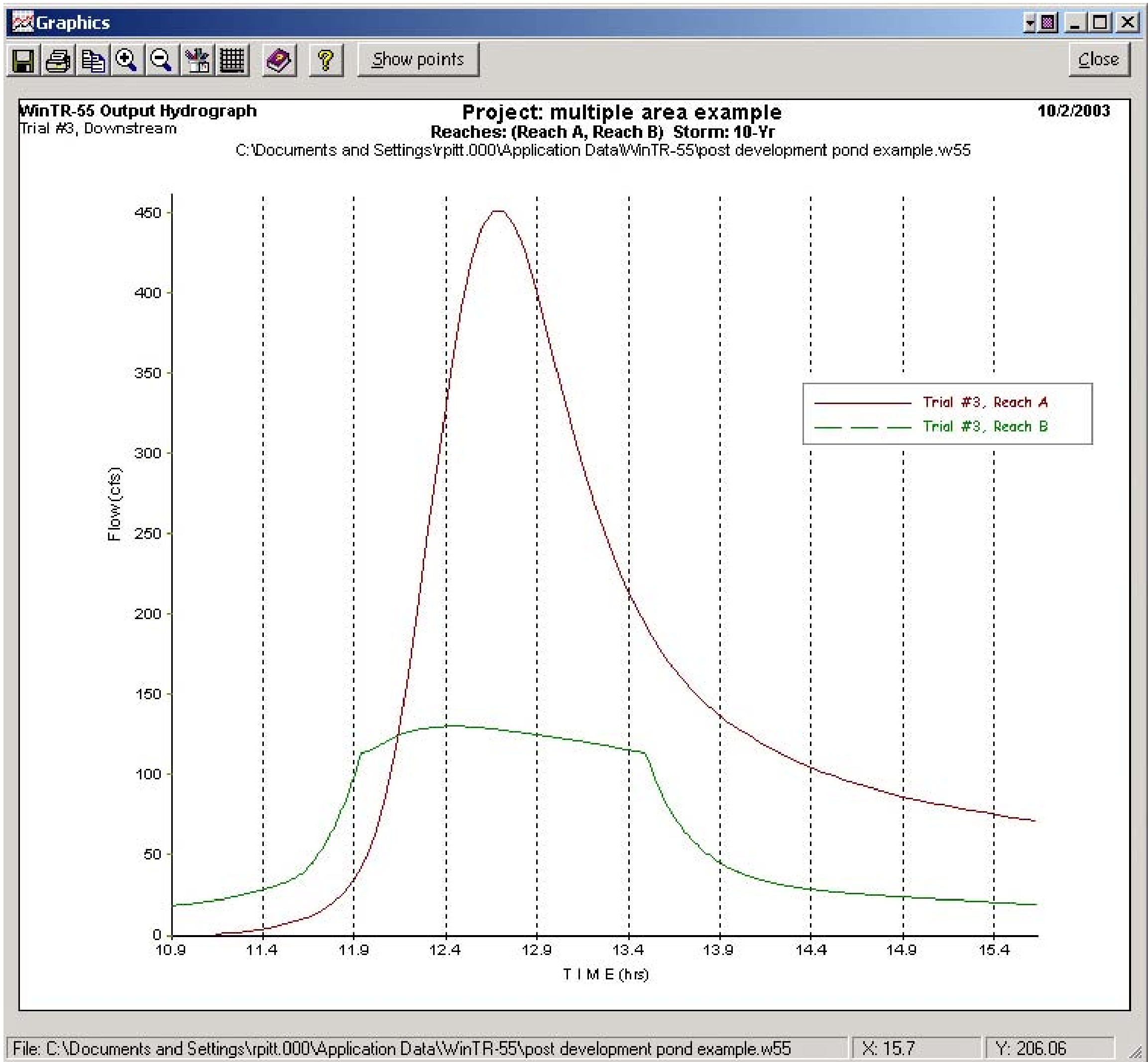


Upon program execution, the data can be reviewed to verify if any of the spillway options were suitable. The following table shows that trial #3 (the 40 inch pipe) reduces the reach B influent flow (391 cfs) down to about 130 cfs, close enough to the desired maximum peak flow. Unfortunately, the outfall peak flow is shown to be about 580 cfs, substantially greater than the predevelopment peak flow of 521 cfs and the post development peak flow, with no pond, of 518 cfs.




```
R. Pitt                      multiple area example
                          post developmoent with pond
                           Tuscaloosa County, Alabama

                 Hydrograph Peak/Peak Time Table (Trial #3)

 Sub-Area       Peak Flow and Peak Time (hr) by Rainfall Return Period
 or Reach      10-Yr
Identifier     (cfs)
             (hr)
----------------------------------------------------------------------------------
SUBAREAS
Area 1        452.82
           12.69

Area 2        391.47
           12.13


REACHES
Reach A       452.82
           12.69
    Down      451.58
           12.75

Reach B       391.47
           12.13
    Down      130.31
           12.48


OUTLET        579.88
```


The following plot of the reach hydrographs indicate how this occurred. The water from subarea 2 was delayed in the detention pond (Reach B) and was discharged so that its peak rate closely coincided in time with the undeveloped hydrograph from subarea 1 (Reach A), causing a larger peak flow than if the water was not detained.



This example illustrated how a detention pond can be evaluated for a developing area, how it can be designed for multiple objectives, and how these objectives may, or may not, be realized in a watershed. The simple application of detention pond standards may not always provide the desired downstream benefits. A basin-wide hydrologic analysis (the above example was a crude and simple example) is needed to ensure that ponds area sized and located correctly to provide the desired benefits. Obviously, the above example was a set-up to illustrate this issue. However, it would be relatively easy to modify the pond to still provide the desired water quality benefits, while not exasperating the flood control objective. A change in the pond spillway device to allow the pond to empty more rapidly would solve this problem. In most cases, detention ponds providing large amounts of storage for flood control should be located in upper reaches of watersheds to lessen these problems.

Summary

WinTR-55 is probably the simplest (and cheapest!) model that can be used to examine basin-wide hydraulic issues. It is relatively simple to use and is based on conventional drainage design procedures. Future improvements in the spillway options will make it more accurate. If more precise analyses are needed, TR-20, or more sophisticated models should be used. It must also be emphasized that WinTR-55 (and TR-20) are not suitable models for water quality evaluations. The curve number approach is not applicable for the moderate-sized events that are responsible for the vast majority of pollutant discharges, continuous simulations for long periods are needed to understand the complex behavior of pollutant discharges under a wide range of environmental conditions, and particle routing (including scour from shallow and dry ponds) is needed to predict the level of pollutant control that may be achieved in detention ponds. However, multiple tools can be used together to better understand how multiple (and often times, conflicting) objectives can be met.

Important Internet Links

Alabama Rainfall Atlas:
http://bama.ua.edu/~rain/

WinTR-55 computer program (windows beta version):
http://www.wcc.nrcs.usda.gov/hydro/hydro-tools-models-wintr55.html

TR-55 1986 documentation:
ftp://ftp.wcc.nrcs.usda.gov/downloads/hydrology_hydraulics/tr55/tr55.pdf

TR-20 computer program (new windows beta version):
http://www.wcc.nrcs.usda.gov/hydro/hydro-tools-models-wintr20.html

National Engineering Handbook, Part 630 HYDROLOGY
http://www.wcc.nrcs.usda.gov/hydro/hydro-techref-neh-630.html

US Army Corps of Engineers, Hydrologic Management System User Guide (HEC HMS) (replacement for HEC-1):
http://www.hec.usace.army.mil/software/hec-hms/hechms-hechms.html

US Army Corps of Engineers, River Analysis System User Guide for water surface profile calculations (HEC RAS) (replacement for HEC-2):
http://www.hec.usace.army.mil/software/hec-ras/hecras-hecras.html

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Pitt, R. and S.R. Durrans. *Drainage of Water from Pavement Structures*. Alabama Dept. of Transportation. 253 pgs. September 1995.

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Pitt, R., M. Lilburn, S. Nix, S.R. Durrans, S. Burian, J. Voorhees, and J. Martinson *Guidance Manual for Integrated Wet Weather Flow (WWF) Collection and Treatment Systems for Newly Urbanized Areas (New WWF Systems).* U.S. Environmental Protection Agency. 612 pgs. Expected final publication in 2002.Ponce, V.M., *Engineering Hydrology*, Prentice Hall, 640 pages. 1989.

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Appendix C-1. Tabular Hydrograph Unit Discharges (from TR-55, SCS 1986)

Exhibit 5-I: Tabular hydrograph unit discharges (csm/in) for type I rainfall distribution

HYDROGRAPH TIME(HOURS)

TRVL TIME (hr)	9.0	9.3	9.6	9.9	10.0	10.1	10.2	10.3	10.4	10.5	10.6	10.7	10.8	11.0	11.2	11.4	11.6	11.8	12.0	12.3	12.6	13.0	13.5	14.0	14.5	15.0	15.5	16.0	17.0	18.0	20.0	24.0
IA/P = 0.10													* * * TC = 0.1 HR * * *																IA/P = 0.10			
0.0	30	40	56	183	337	504	326	155	122	107	93	81	73	66	60	56	54	52	49	46	44	40	36	32	30	29	28	27	26	24	20	13
.10	26	35	48	93	153	276	428	360	223	156	123	103	83	72	65	59	56	54	51	47	45	42	37	33	30	29	28	28	26	24	21	13
.20	23	30	41	60	82	129	227	361	360	269	194	147	118	85	71	63	58	55	53	49	46	43	39	35	31	29	29	28	26	24	21	14
.30	22	29	39	56	73	111	188	303	341	293	227	173	136	94	75	65	60	56	54	50	47	43	39	35	31	29	29	28	26	25	21	14
.40	18	25	34	46	53	66	96	157	255	312	300	251	199	126	90	73	64	59	56	52	48	44	40	36	32	30	29	28	26	25	21	14
.50	18	24	32	44	50	61	84	133	214	280	293	265	221	144	99	77	66	60	56	52	49	45	41	37	32	30	29	28	27	25	21	14
.75	14	19	25	34	38	43	49	62	88	134	190	234	252	221	162	115	87	71	63	56	52	47	43	39	35	31	29	29	27	25	22	15
1.0	11	14	19	26	28	31	34	38	44	52	68	98	141	222	238	191	139	101	79	63	56	51	45	41	37	33	30	29	27	26	22	15
1.5	9	10	13	17	18	20	22	25	27	30	34	38	44	74	132	191	211	190	151	101	73	58	50	45	41	37	33	30	28	27	23	16
2.0	6	7	9	11	12	13	14	15	16	18	20	22	24	29	38	58	97	148	193	193	141	89	61	51	46	41	37	33	29	27	24	17
2.5	4	5	7	8	9	9	10	11	11	12	13	14	16	19	23	29	39	58	93	154	181	147	87	61	51	45	41	37	30	28	25	18
3.0	2	3	5	6	6	7	7	8	9	9	10	10	11	13	15	19	23	28	39	72	124	170	138	86	61	50	45	40	33	29	26	19
IA/P = 0.30													* * * TC = 0.1 HR * * *																IA/P = 0.30			
0.0	0	0	0	61	195	343	232	129	113	103	91	81	76	71	66	64	62	61	59	56	54	51	47	43	40	40	39	39	37	36	31	21
.10	0	0	0	12	45	145	277	247	169	131	112	98	87	76	70	65	64	62	60	57	55	53	49	44	40	40	39	39	38	36	32	21
.20	0	0	0	9	33	107	220	238	192	151	125	107	94	79	71	66	64	62	61	58	56	53	49	45	41	40	40	39	38	36	32	21
.30	0	0	0	1	6	24	79	173	216	200	168	139	118	90	77	70	66	64	62	59	57	54	50	46	41	40	40	39	38	36	32	22
.40	0	0	0	1	4	17	59	135	189	196	177	152	129	97	81	72	67	64	63	60	57	54	51	46	42	40	40	39	38	36	32	22
.50	0	0	0	0	0	3	12	43	104	161	185	180	161	121	93	79	71	66	64	61	58	55	52	48	43	40	40	39	38	37	33	22
.75	0	0	0	0	0	1	5	18	49	92	130	153	159	142	114	92	79	71	66	63	60	56	53	49	44	41	40	40	38	37	33	23
1.0	0	0	0	0	0	0	0	0	2	9	27	56	92	144	152	128	103	86	75	67	63	59	55	51	47	43	40	40	39	37	34	24
1.5	0	0	0	0	0	0	0	0	0	0	0	2	6	32	79	121	136	127	109	85	72	64	58	55	51	47	43	40	39	38	34	25
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	33	70	105	126	118	97	75	63	58	54	50	46	42	40	39	35	26
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	11	32	63	105	118	100	75	63	58	54	50	46	40	39	36	27
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	12	43	84	112	96	74	63	58	54	50	42	40	37	28
IA/P = 0.50													* * * TC = 0.1 HR * * *																IA/P = 0.50			
0.0	0	0	0	0	0	3	13	28	39	44	45	46	46	48	49	51	53	54	54	55	56	55	53	50	50	49	49	49	49	48	45	32
.10	0	0	0	0	0	2	9	22	33	41	44	45	46	48	49	50	52	54	54	54	56	55	53	50	50	49	49	49	49	48	45	32
.20	0	0	0	0	0	0	1	7	17	28	36	41	44	46	48	49	51	53	54	54	56	56	54	51	50	50	49	49	49	48	45	32
.30	0	0	0	0	0	0	0	1	5	13	23	32	38	44	47	48	50	51	53	54	54	55	54	52	50	50	49	49	49	48	46	33
.40	0	0	0	0	0	0	0	0	3	10	18	27	34	43	46	48	49	51	53	54	54	55	54	52	50	50	49	49	49	48	46	33
.50	0	0	0	0	0	0	0	0	0	2	7	15	23	36	43	46	48	50	51	53	54	55	55	53	50	50	49	49	49	48	46	34
.75	0	0	0	0	0	0	0	0	0	1	3	7	13	27	37	43	46	48	50	52	54	55	55	54	51	50	50	49	49	48	46	34
1.0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	20	32	40	45	47	50	52	54	55	54	52	50	50	49	49	49	47	36
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	13	23	33	42	47	51	54	55	54	53	50	50	49	49	48	37
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	12	25	37	46	51	54	55	54	52	50	49	49	48	39
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	15	31	45	51	53	55	54	52	50	49	48	41
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	14	32	45	51	53	54	54	51	49	48	42

RAINFALL TYPE = I * * * TC = 0.1 HR * * * SHEET 1 OF 10

Exhibit 5-I: Tabular hydrograph unit discharges (csm/in) for type I rainfall distribution—continued

HYDROGRAPH TIME (HOURS)

IA/P = 0.10 * * * TC = 0.2 HR * * * IA/P = 0.10

TRVL TIME (hr)	9.0	9.3	9.6	9.9	10.0	10.1	10.2	10.3	10.4	10.5	10.6	10.7	10.8	11.0	11.2	11.4	11.6	11.8	12.0	12.3	12.6	13.0	13.5	14.0	14.5	15.0	15.5	16.0	17.0	18.0	20.0	24.0
0.0	28	37	52	126	220	379	405	267	168	129	108	92	80	69	62	58	55	53	50	47	44	41	37	33	30	29	28	27	26	24	21	13
.10	24	32	44	71	108	182	313	375	303	213	157	124	103	78	68	61	57	55	52	49	46	42	38	34	30	29	29	28	26	24	21	14
.20	21	28	38	53	65	94	153	260	336	314	247	187	145	97	77	67	60	57	54	50	47	44	39	35	31	30	29	28	26	25	21	14
.30	20	27	36	50	60	82	129	216	296	308	267	214	168	110	82	69	62	58	55	51	47	44	40	36	32	30	29	28	26	25	21	14
.40	17	23	31	42	47	56	74	111	181	258	291	275	234	152	103	79	68	61	57	53	49	45	41	37	33	30	29	28	27	25	22	14
.50	16	22	30	40	45	52	66	96	153	223	269	273	247	171	115	86	71	63	58	54	50	46	42	37	33	30	29	28	27	25	22	14
.75	13	17	24	32	35	39	44	52	68	99	145	194	229	240	183	132	97	77	66	58	53	48	44	39	35	31	30	29	27	26	22	15
1.0	11	13	17	24	26	29	32	35	39	45	56	75	107	189	229	206	158	115	88	67	58	52	46	42	38	34	31	29	28	26	23	15
1.5	8	10	12	15	17	19	21	23	25	28	31	35	39	60	106	166	204	197	165	112	79	60	51	46	42	37	33	30	28	27	23	16
2.0	5	7	9	10	11	12	13	14	15	17	18	20	22	27	34	49	79	126	169	188	152	97	64	52	46	42	38	34	29	28	24	17
2.5	3	5	6	8	8	9	10	10	11	12	13	14	15	18	22	26	34	49	77	136	176	155	95	64	52	46	42	37	31	28	25	18
3.0	2	3	4	5	6	6	7	7	8	8	9	9	10	11	13	16	19	23	29	49	91	154	167	102	67	53	47	42	34	29	26	19

IA/P = 0.30 * * * TC = 0.2 HR * * * IA/P = 0.30

TRVL TIME (hr)	9.0	9.3	9.6	9.9	10.0	10.1	10.2	10.3	10.4	10.5	10.6	10.7	10.8	11.0	11.2	11.4	11.6	11.8	12.0	12.3	12.6	13.0	13.5	14.0	14.5	15.0	15.5	16.0	17.0	18.0	20.0	24.0
0.0	0	0	0	22	76	206	258	207	144	119	104	92	82	74	68	65	63	62	60	57	55	52	48	44	40	40	39	39	38	36	32	21
.10	0	0	0	3	16	56	156	224	213	167	135	114	99	81	73	67	65	63	61	58	56	53	50	45	41	40	40	39	38	36	32	22
.20	0	0	0	2	11	41	116	189	205	179	150	126	108	85	75	69	65	63	62	59	56	54	50	46	41	40	40	39	38	36	32	22
.30	0	0	0	0	2	8	30	58	155	188	182	161	138	103	83	74	68	65	63	60	57	55	51	47	42	40	40	39	38	36	32	22
.40	0	0	0	0	1	6	22	66	126	167	177	166	147	111	88	76	69	66	63	61	58	55	51	47	43	40	40	39	38	37	33	22
.50	0	0	0	0	0	1	4	16	50	100	145	167	166	136	105	85	75	69	65	62	59	56	53	49	44	41	40	40	38	37	33	23
.75	0	0	0	0	0	0	2	7	22	50	87	119	140	148	124	101	85	74	69	64	61	57	54	50	45	42	40	40	38	37	33	23
1.0	0	0	0	0	0	0	0	0	1	3	11	28	55	114	142	135	113	93	80	69	64	60	56	52	48	44	41	40	39	37	34	24
1.5	0	0	0	0	0	0	0	0	0	0	0	1	2	17	52	97	124	129	115	91	75	65	59	55	52	48	43	41	39	38	35	25
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	19	49	84	111	120	102	79	65	59	55	51	47	43	40	39	36	26
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	20	45	89	113	104	78	65	59	55	51	47	41	39	36	28
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	7	30	68	108	99	77	65	58	54	50	43	40	37	29

IA/P = 0.50 * * * TC = 0.2 HR * * * IA/P = 0.50

TRVL TIME (hr)	9.0	9.3	9.6	9.9	10.0	10.1	10.2	10.3	10.4	10.5	10.6	10.7	10.8	11.0	11.2	11.4	11.6	11.8	12.0	12.3	12.6	13.0	13.5	14.0	14.5	15.0	15.5	16.0	17.0	18.0	20.0	24.0
0.0	0	0	0	0	0	0	4	15	28	38	43	45	45	48	49	50	52	53	53	53	53	53	53	50	50	50	49	49	49	48	45	32
.10	0	0	0	0	0	0	3	11	22	32	40	43	45	47	48	50	52	53	53	53	53	53	53	50	50	50	49	49	49	48	45	32
.20	0	0	0	0	0	0	2	8	17	27	36	41	43	46	48	49	51	53	53	53	53	53	53	51	50	50	49	49	49	48	45	33
.30	0	0	0	0	0	0	1	6	13	23	31	38	41	45	48	49	51	52	53	53	53	53	53	51	50	50	49	49	49	48	46	33
.40	0	0	0	0	0	0	1	4	10	19	27	34	39	44	47	49	50	52	53	53	53	53	53	51	50	50	49	49	49	49	46	33
.50	0	0	0	0	0	0	1	3	8	15	23	30	36	43	46	48	50	51	53	53	53	53	53	52	50	50	49	49	49	48	46	33
.75	0	0	0	0	0	0	0	0	3	8	14	20	27	37	43	46	48	50	52	53	53	53	53	52	50	50	50	49	49	48	46	34
1.0	0	0	0	0	0	0	0	0	0	0	0	2	4	14	26	36	42	46	48	51	52	53	53	53	52	50	50	49	49	49	47	35
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	13	23	33	40	46	49	52	53	53	53	51	50	50	49	49	48	37
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	8	16	26	37	44	50	52	53	53	53	51	50	49	49	48	38
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	8	19	31	43	50	52	53	53	52	51	50	49	48	39
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	11	25	41	49	52	53	53	52	50	49	48	41

RAINFALL TYPE = I * * * TC = 0.2 HR * * * SHEET 2 OF 10

Exhibit 5-I: Tabular hydrograph unit discharges (csm/in) for type I rainfall distribution—continued

TRVL TIME (hr)	9.0	9.3	9.6	9.9	10.0	10.1	10.2	10.3	10.4	10.5	10.6	10.7	10.8	11.0	11.2	11.4	11.6	11.8	12.0	12.3	12.6	13.0	13.5	14.0	14.5	15.0	15.5	16.0	17.0	18.0	20.0	24.0
IA/P = 0.10												* * * TC = 0.3 HR * * *																IA/P = 0.10				
0.0	25	34	46	87	138	242	349	346	256	182	141	115	97	76	66	60	57	54	52	48	45	42	38	34	30	29	28	28	26	24	21	13
.10	24	32	44	77	118	200	299	330	281	215	166	132	109	82	69	62	58	55	52	49	46	42	38	34	30	29	29	28	26	24	21	14
.20	21	28	38	54	69	102	167	255	305	289	240	190	152	103	79	67	61	57	54	50	47	44	40	35	31	30	29	28	26	25	21	14
.30	20	27	36	51	63	89	141	217	276	285	255	212	172	115	85	71	63	58	55	51	48	44	40	36	32	30	29	28	26	25	21	14
.40	17	23	31	42	48	58	79	120	185	246	272	260	228	156	108	82	69	62	57	53	49	45	41	37	33	30	29	28	27	25	22	14
.50	16	22	30	40	45	54	70	104	158	216	253	258	238	173	120	89	72	63	58	54	50	46	42	37	33	30	29	28	27	25	22	14
.75	13	17	24	32	35	39	45	54	72	104	147	189	219	231	182	135	100	79	67	58	53	48	44	40	35	32	30	29	27	26	22	15
1.0	11	13	17	24	26	29	32	35	40	46	58	79	110	184	221	202	158	118	90	68	58	52	46	42	38	34	31	29	28	26	23	15
1.5	8	10	12	15	17	19	21	23	25	28	31	35	40	62	107	163	199	193	165	114	81	61	52	46	42	37	33	31	28	27	23	16
2.0	5	7	9	10	11	12	13	14	15	17	18	20	22	27	35	50	80	125	165	184	152	99	64	53	47	42	38	34	29	28	24	17
2.5	3	5	6	8	8	9	10	10	11	12	13	14	15	18	22	27	35	50	78	134	173	154	96	64	52	46	42	37	31	28	25	18
3.0	2	3	4	6	6	7	7	8	8	9	9	10	11	12	14	17	21	26	34	61	107	164	145	94	64	52	46	41	33	29	26	19
IA/P = 0.30												* * * TC = 0.3 HR * * *																IA/P = 0.30				
0.0	0	0	0	8	32	98	198	217	192	148	125	108	95	79	72	67	64	63	61	58	56	53	49	45	41	40	40	39	38	36	32	22
.10	0	0	0	1	6	23	73	156	197	194	164	138	118	91	77	70	66	64	62	59	57	54	50	46	41	40	40	39	38	36	32	22
.20	0	0	0	1	4	17	54	122	172	187	171	149	128	98	81	72	67	64	63	60	57	54	51	46	42	40	40	39	38	36	32	22
.30	0	0	0	0	0	3	12	40	95	146	173	172	157	120	34	79	71	67	64	61	58	55	52	48	43	40	40	39	38	37	33	23
.40	0	0	0	0	0	2	9	30	73	122	156	167	160	128	100	83	73	68	65	62	59	56	52	48	44	41	40	39	38	37	33	23
.50	0	0	0	0	0	1	6	22	56	100	137	157	159	135	107	87	76	69	65	62	59	56	53	49	44	41	40	40	38	37	33	23
.75	0	0	0	0	0	0	1	3	9	26	53	86	115	143	134	113	93	80	72	65	62	58	54	51	46	42	40	40	39	37	33	23
1.0	0	0	0	0	0	0	0	0	1	5	14	32	57	111	138	132	113	94	81	69	64	60	56	52	48	44	41	40	39	37	34	24
1.5	0	0	0	0	0	0	0	0	0	0	0	0	1	8	34	74	111	126	121	99	80	67	61	56	53	49	44	41	39	38	35	26
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	11	33	66	97	118	108	84	67	60	56	52	48	44	40	39	36	27
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	12	32	74	106	112	83	67	60	56	52	48	41	39	37	28
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	21	54	98	107	81	67	60	55	51	43	40	37	29
IA/P = 0.50												* * * TC = 0.3 HR * * *																IA/P = 0.50				
0.0	0	0	0	0	0	0	1	6	16	27	36	41	44	46	48	49	51	52	52	52	52	52	52	51	50	50	49	49	49	48	46	33
.10	0	0	0	0	0	0	1	4	12	22	31	38	42	45	48	49	51	52	52	52	52	52	52	51	50	50	49	49	49	48	46	33
.20	0	0	0	0	0	0	0	3	9	18	27	34	39	44	47	49	50	51	52	52	52	52	52	51	50	50	49	49	49	48	46	33
.30	0	0	0	0	0	0	0	2	7	14	23	30	36	43	46	48	50	51	52	52	52	52	52	51	50	50	49	49	49	48	46	33
.40	0	0	0	0	0	0	0	1	5	11	19	26	33	41	46	48	49	51	52	52	52	52	52	52	50	50	49	49	49	48	46	33
.50	0	0	0	0	0	0	0	1	4	9	15	23	30	39	44	47	49	50	51	52	52	52	52	52	50	50	49	49	49	48	46	33
.75	0	0	0	0	0	0	0	0	2	4	8	14	20	31	40	44	47	49	51	52	52	52	52	52	50	50	50	49	49	48	46	34
1.0	0	0	0	0	0	0	0	0	0	0	0	1	2	9	20	31	39	44	47	50	51	52	52	52	52	50	50	50	49	49	47	36
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	9	18	28	36	44	48	51	52	52	52	51	50	50	49	49	48	37
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	12	21	33	42	48	51	52	52	52	51	50	49	49	48	38
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	15	27	40	48	51	52	52	52	51	50	49	48	40
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	11	25	41	48	51	52	52	51	50	49	48	41

RAINFALL TYPE = I * * * TC = 0.3 HR * * * SHEET 3 OF 10

Exhibit 5-I: Tabular hydrograph unit discharges (csm/in) for type I rainfall distribution—continued

TRVL TIME (hr)	9.0	9.3	9.6	9.9	10.0	10.1	10.2	10.3	10.4	10.5	10.6	10.7	10.8	11.0	11.2	11.4	11.6	11.8	12.0	12.3	12.6	13.0	13.5	14.0	14.5	15.0	15.5	16.0	17.0	18.0	20.0	24.0
IA/P = 0.10														* * * TC = 0.4 HR * * *														IA/P = 0.10				
0.0	23	31	42	66	96	157	250	310	304	244	186	149	122	89	73	64	59	56	53	49	46	43	39	35	31	29	29	28	26	25	21	14
.10	22	29	40	61	84	133	211	277	295	261	211	170	138	98	78	66	60	56	54	50	47	43	39	35	31	29	29	28	26	25	21	14
.20	19	26	34	47	56	75	114	178	244	278	267	230	190	128	93	75	65	59	56	52	48	44	40	36	32	30	29	28	27	25	21	14
.30	18	24	33	45	53	67	98	152	213	257	263	241	207	143	102	80	68	61	57	52	49	45	41	37	32	30	29	28	27	25	21	14
.40	15	21	28	38	43	49	61	86	130	185	233	253	245	188	132	97	77	66	60	55	51	46	42	38	34	30	29	28	27	25	22	14
.50	15	20	27	36	41	46	56	76	112	161	209	238	243	201	146	106	82	69	61	55	51	47	42	38	34	31	29	29	27	25	22	14
.75	12	16	21	29	32	35	39	46	57	77	108	147	184	220	200	157	118	91	74	61	55	50	45	40	36	32	30	29	27	26	22	15
1.0	10	12	16	21	24	26	29	32	36	40	48	61	83	147	202	212	178	138	105	75	62	54	47	43	39	35	31	29	28	26	23	16
1.5	8	9	11	14	16	17	19	21	23	25	28	31	35	50	83	134	179	193	177	131	92	65	53	47	42	38	34	31	29	27	24	16
2.0	5	6	8	10	10	11	12	13	14	15	17	18	20	25	31	42	64	102	144	179	163	111	70	55	48	43	39	35	30	28	25	17
2.5	3	4	6	7	8	8	9	9	10	11	12	13	14	16	20	24	30	42	63	114	160	170	107	70	54	47	43	38	31	29	25	18
3.0	2	3	4	5	6	6	7	7	8	8	9	9	10	11	13	16	19	23	30	51	90	148	161	104	69	54	47	42	34	29	26	19
IA/P = 0.30														* * * TC = 0.4 HR * * *														IA/P = 0.30				
0.0	0	0	0	3	14	48	115	184	192	178	148	127	111	88	77	70	66	64	62	59	56	54	50	46	41	40	40	39	38	36	32	22
.10	0	0	0	0	2	10	35	89	152	179	178	158	137	105	85	75	69	65	63	60	58	55	51	47	42	40	40	39	38	36	32	22
.20	0	0	0	0	2	7	26	68	124	161	172	163	146	113	90	78	70	66	64	61	58	55	52	47	43	40	40	39	38	37	33	22
.30	0	0	0	0	1	5	19	52	100	140	162	162	151	120	96	81	72	67	64	61	59	55	52	48	43	41	40	39	38	37	33	23
.40	0	0	0	0	0	1	4	14	39	80	120	148	158	142	114	92	79	71	67	63	60	57	53	49	45	41	40	40	38	37	33	23
.50	0	0	0	0	0	1	3	10	29	63	101	132	152	145	120	97	82	73	68	63	60	57	53	50	45	41	40	40	38	37	33	23
.75	0	0	0	0	0	0	0	1	4	13	31	58	87	130	138	123	103	87	77	68	63	59	55	51	47	43	41	40	39	37	34	24
1.0	0	0	0	0	0	0	0	0	0	2	7	18	36	86	125	134	122	104	88	73	66	61	57	53	49	45	41	40	39	38	34	24
1.5	0	0	0	0	0	0	0	0	0	0	0	0	1	10	36	75	109	124	120	100	81	68	61	56	53	49	44	41	40	38	35	26
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	22	50	83	116	116	91	70	61	57	53	49	45	40	39	36	27
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	8	22	60	97	111	88	70	61	56	53	48	42	39	37	28
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	14	43	89	106	86	69	61	56	52	44	40	37	29
IA/P = 0.50														* * * TC = 0.4 HR * * *														IA/P = 0.50				
0.0	0	0	0	0	0	0	0	2	8	17	27	34	40	44	47	49	50	51	51	51	51	51	51	51	50	50	49	49	49	48	46	33
.10	0	0	0	0	0	0	0	0	2	6	13	22	30	40	45	47	49	50	51	51	51	51	51	51	50	50	50	49	49	48	46	34
.20	0	0	0	0	0	0	0	0	0	1	4	10	18	33	41	45	48	49	51	51	51	51	51	51	50	50	50	49	49	49	46	34
.30	0	0	0	0	0	0	0	0	0	0	1	3	8	22	35	42	46	48	50	51	51	51	51	51	51	50	50	49	49	49	47	35
.40	0	0	0	0	0	0	0	0	0	0	1	2	6	19	32	40	45	47	49	51	51	51	51	51	51	50	50	49	49	49	47	35
.50	0	0	0	0	0	0	0	0	0	0	0	0	2	9	23	34	41	45	48	50	51	51	51	51	51	50	50	49	49	49	47	35
.75	0	0	0	0	0	0	0	0	0	0	0	0	1	5	14	26	35	42	46	49	50	51	51	51	51	50	50	50	49	49	47	36
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	14	25	35	44	48	50	51	51	51	51	50	50	49	49	48	37
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	6	13	27	39	47	50	51	51	51	51	50	49	49	48	39
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	9	21	36	47	50	51	51	51	51	50	49	48	40
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	19	37	47	50	51	51	51	50	40	48	42
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	25	40	48	50	51	51	50	50	49	43

RAINFALL TYPE = I   * * * TC = 0.4 HR * * *   SHEET 4 OF 10

Exhibit 5-I: Tabular hydrograph unit discharges (csm/in) for type I rainfall distribution—continued

TRVL TIME (hr)	9.0	9.3	9.6	9.9	10.0	10.1	10.2	10.3	10.4	10.5	10.6	10.7	10.8	11.0	11.2	11.4	11.6	11.8	12.0	12.3	12.6	13.0	13.5	14.0	14.5	15.0	15.5	16.0	17.0	18.0	20.0	24.0
IA/P = 0.10												* * * TC = 0.5 HR * * *																IA/P = 0.10				
0.0	21	23	38	56	73	111	175	247	282	271	229	186	152	108	84	71	63	58	55	51	47	44	40	35	31	30	29	28	26	25	21	14
.10	20	27	36	52	66	96	149	214	259	267	242	204	169	119	90	74	65	59	55	51	48	44	40	36	32	30	29	28	26	25	21	14
.20	17	23	31	42	49	60	84	127	185	234	256	247	219	156	112	86	72	63	58	53	49	45	41	37	33	30	29	28	27	25	22	14
.30	16	22	30	40	46	55	74	110	160	210	241	245	227	171	123	93	76	66	59	54	50	46	42	38	33	30	29	28	27	25	22	14
.40	14	19	26	35	38	43	51	67	95	138	186	222	237	210	157	115	89	73	64	57	52	47	43	39	35	31	29	29	27	25	22	15
.50	14	18	25	33	37	41	48	61	84	120	164	203	229	216	170	126	96	77	66	58	53	48	43	39	35	31	30	29	27	25	22	15
.75	11	14	19	26	29	32	35	40	48	61	83	114	148	202	210	176	137	106	84	66	57	51	46	41	37	33	30	29	28	26	22	15
1.0	10	11	15	19	21	24	26	29	32	36	41	50	65	116	176	203	190	156	122	85	67	56	49	44	40	35	32	30	28	26	23	16
1.5	7	9	11	13	14	16	17	19	21	23	25	28	31	42	67	110	157	187	183	146	104	72	55	48	43	39	35	32	29	27	24	17
2.0	4	6	7	9	10	10	11	12	13	14	15	17	18	22	27	36	53	83	123	175	175	124	77	57	49	44	40	35	30	28	25	18
2.5	3	4	5	7	7	8	8	9	9	10	11	12	13	15	18	22	27	36	53	97	146	166	119	76	57	49	44	39	32	29	25	18
3.0	1	2	3	5	5	6	6	6	7	8	8	9	9	11	12	15	17	21	27	43	76	135	158	114	75	57	48	43	35	30	26	19
IA/P = 0.30												* * * TC = 0.5 HR * * *																IA/P = 0.30				
0.0	0	0	0	1	6	22	64	123	168	174	165	143	126	100	84	74	68	65	63	60	57	54	51	47	42	40	40	39	38	36	32	22
.10	0	0	0	1	5	16	48	98	144	164	165	150	134	106	87	77	70	66	63	60	58	55	51	47	42	40	40	39	38	36	32	22
.20	0	0	0	0	1	3	12	36	77	122	150	160	154	126	101	85	75	69	65	62	59	56	52	48	44	41	40	39	38	37	33	23
.30	0	0	0	0	0	2	9	27	60	101	134	151	153	132	107	89	77	70	66	63	60	56	53	49	44	41	40	40	38	37	33	23
.40	0	0	0	0	0	0	2	6	20	47	83	117	140	148	125	102	86	76	69	64	61	57	54	50	46	42	40	40	39	37	33	23
.50	0	0	0	0	0	0	1	5	15	36	68	101	127	144	130	108	90	78	71	65	62	58	54	50	46	42	40	40	39	37	33	23
.75	0	0	0	0	0	0	0	2	6	17	36	60	87	125	133	121	103	88	78	68	63	59	55	52	47	43	41	40	39	37	34	24
1.0	0	0	0	0	0	0	0	0	0	1	3	9	21	62	106	130	126	112	96	78	69	62	57	54	50	46	42	40	39	38	34	25
1,5	0	0	0	0	0	0	0	0	0	0	0	0	1	6	23	56	92	115	120	106	87	71	62	57	53	49	45	42	40	38	35	26
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	14	36	67	104	113	96	74	63	58	54	50	46	40	39	36	27
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	15	47	85	109	93	73	63	57	53	49	42	40	37	28
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	32	77	105	90	72	62	57	53	45	40	38	30
IA/P = 0.50												* * * TC = 0.5 HR * * *																IA/P = 0.50				
0.0	0	0	0	0	0	0	0	1	4	10	18	26	33	41	45	47	49	50	50	50	50	50	50	50	50	50	50	49	49	48	46	33
.10	0	0	0	0	0	0	0	1	3	8	15	22	30	39	44	47	49	50	50	50	50	50	50	50	50	50	50	49	49	48	46	34
.20	0	0	0	0	0	0	0	0	2	6	12	19	26	37	43	46	48	50	50	50	50	50	50	50	50	50	50	49	49	48	46	34
.30	0	0	0	0	0	0	0	0	1	4	9	16	23	34	41	45	48	49	50	50	50	50	50	50	50	50	50	49	49	49	46	34
.40	0	0	0	0	0	0	0	0	1	3	7	13	19	31	39	44	47	49	50	50	50	50	50	50	50	50	50	49	49	49	46	34
.50	0	0	0	0	0	0	0	0	1	2	6	10	16	28	37	43	46	48	50	50	50	50	50	50	50	50	50	49	49	49	46	34
.75	0	0	0	0	0	0	0	0	0	1	3	6	10	20	30	38	43	46	48	50	50	50	50	50	50	50	50	49	49	49	47	35
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	10	20	30	38	43	47	49	50	50	50	50	50	50	50	49	49	47	36
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	6	14	23	31	41	46	49	50	50	50	50	50	50	49	49	48	38
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	6	13	25	36	45	49	50	50	50	50	50	49	49	48	39
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	12	23	37	46	49	50	50	50	50	50	49	48	40
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	8	21	37	46	49	50	50	50	50	49	48	42

RAINFALL TYPE = I * * * TC = 0.5 HR * * * SHEET 5 OF 10

Exhibit 5-I: Tabular hydrograph unit discharges (csm/in) for type I rainfall distribution—continued

IA/P = 0.10 * * * TC = 0.75 HR * * *

TRVL TIME (hr)	9.0	9.3	9.6	9.9	10.0	10.1	10.2	10.3	10.4	10.5	10.6	10.7	10.8	11.0	11.2	11.4	11.6	11.8	12.0	12.3	12.6	13.0	13.5	14.0	14.5	15.0	15.5	16.0	17.0	18.0	20.0	24.0
0.0	16	22	30	41	47	59	82	119	167	209	229	233	212	161	122	97	80	69	62	56	51	46	42	38	33	31	29	28	27	25	22	14
.10	14	19	26	35	39	44	54	72	104	146	188	215	227	197	150	116	93	77	68	59	53	48	43	39	35	31	30	29	27	25	22	15
.20	13	16	22	30	33	37	42	50	65	91	128	168	200	218	183	142	110	89	75	63	56	49	44	40	36	32	30	29	27	26	22	15
.30	12	16	21	28	31	35	39	46	59	80	112	149	183	214	191	151	118	94	79	65	57	50	45	41	36	32	30	29	27	26	22	15
.40	12	15	20	27	30	33	37	43	54	71	98	132	166	207	197	161	126	100	83	67	58	51	45	41	37	33	30	29	27	26	22	15
.50	11	13	18	24	26	29	32	36	41	49	64	87	117	178	204	186	152	120	96	74	62	53	47	42	38	34	31	29	28	26	23	15
.75	10	12	15	21	23	25	28	31	35	40	49	63	83	136	180	190	171	142	115	85	69	57	49	44	39	35	32	30	28	26	23	16
1.0	8	10	12	16	17	19	21	23	25	28	31	35	42	66	110	158	186	180	157	116	86	65	53	46	42	38	34	31	28	27	23	16
1.5	5	7	8	10	11	12	13	14	15	17	18	20	22	27	35	52	83	124	159	174	147	102	69	54	47	42	38	34	30	28	24	17
2.0	3	5	6	8	8	9	9	10	11	12	13	14	15	18	22	27	36	52	80	131	165	148	99	68	54	47	42	38	31	28	25	18
2.5	2	3	4	5	6	6	7	7	8	9	9	10	11	12	14	17	21	26	35	63	107	157	141	96	67	54	46	42	34	29	26	19
3.0	1	1	2	3	4	4	4	5	5	6	6	7	7	8	10	11	13	15	19	26	42	86	151	141	101	71	55	47	38	31	27	20

IA/P = 0.30 * * * TC = 0.75 HR * * *

TRVL TIME (hr)	9.0	9.3	9.6	9.9	10.0	10.1	10.2	10.3	10.4	10.5	10.6	10.7	10.8	11.0	11.2	11.4	11.6	11.8	12.0	12.3	12.6	13.0	13.5	14.0	14.5	15.0	15.5	16.0	17.0	18.0	20.0	24.0
0.0	0	0	0	0	1	4	14	35	69	105	131	143	146	124	104	89	79	72	68	64	60	56	53	49	44	41	40	40	38	37	33	23
.10	0	0	0	0	0	1	3	10	27	55	88	117	134	142	118	100	87	77	71	66	62	58	54	50	46	42	40	40	39	37	33	23
.20	0	0	0	0	0	0	2	7	20	43	73	102	123	137	123	105	90	80	73	66	62	58	54	50	46	42	40	40	39	37	33	23
.30	0	0	0	0	0	0	0	1	5	15	34	60	88	128	134	118	101	88	78	69	64	60	55	52	48	43	41	40	39	37	34	24
.40	0	0	0	0	0	0	0	1	4	12	27	49	75	118	131	121	105	91	80	71	65	60	56	52	48	44	41	40	39	37	34	24
.50	0	0	0	0	0	0	0	0	1	3	9	21	39	87	122	128	116	101	88	75	68	62	57	53	49	45	42	40	39	38	34	24
.75	0	0	0	0	0	0	0	0	0	1	4	10	21	56	95	118	121	110	98	82	72	64	58	54	50	46	42	41	39	38	34	25
1.0	0	0	0	0	0	0	0	0	0	0	0	1	2	12	39	76	106	119	115	98	82	69	61	57	53	49	44	42	40	38	35	26
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	14	37	67	95	112	105	85	69	61	56	52	48	44	40	39	36	27
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	15	35	74	102	107	84	69	61	56	52	48	41	39	37	28
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	23	55	94	103	82	68	60	56	52	44	40	37	29
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	15	50	93	100	81	68	60	55	47	41	38	30

IA/P = 0.50 * * * TC = 0.75 HR * * *

TRVL TIME (hr)	9.0	9.3	9.6	9.9	10.0	10.1	10.2	10.3	10.4	10.5	10.6	10.7	10.8	11.0	11.2	11.4	11.6	11.8	12.0	12.3	12.6	13.0	13.5	14.0	14.5	15.0	15.5	16.0	17.0	18.0	20.0	24.0
0.0	0	0	0	0	0	0	0	0	1	2	5	10	17	29	37	42	45	48	49	49	49	49	49	49	49	49	49	49	49	49	47	34
.10	0	0	0	0	0	0	0	0	0	2	4	8	14	26	35	41	45	47	49	49	49	49	49	49	49	49	49	49	49	49	47	35
.20	0	0	0	0	0	0	0	0	0	1	3	7	11	23	33	39	44	47	48	49	49	49	49	49	49	49	49	49	49	49	47	35
.30	0	0	0	0	0	0	0	0	0	1	2	5	9	20	30	38	43	46	48	49	49	49	49	49	49	49	49	49	49	49	47	35
.40	0	0	0	0	0	0	0	0	0	1	2	4	8	17	28	36	41	45	47	49	49	49	49	49	49	49	49	49	49	49	47	35
.50	0	0	0	0	0	0	0	0	0	0	1	3	6	15	25	34	40	44	47	48	49	49	49	49	49	49	49	49	49	49	47	35
.75	o	0	0	0	0	0	0	0	0	0	1	1	3	9	18	27	35	40	44	47	48	49	49	49	49	49	49	49	49	49	47	36
1.0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	10	18	27	35	40	45	48	49	49	49	49	49	49	49	49	49	48	37
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	6	13	21	29	38	44	48	49	49	49	49	49	49	49	49	48	38
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	9	15	26	36	44	48	49	49	49	49	49	49	49	48	39
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	11	21	34	44	48	48	48	48	48	48	48	48	40
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	8	19	35	44	48	48	48	48	48	48	48	42

RAINFALL TYPE = I * * * TC = 0.75 HR * * * SHEET 6 OF 10

Exhibit 5-I: Tabular hydrograph unit discharges (csm/in) for type I rainfall distribution—continued

HYDROGRAPH TIME(HOURS)

TRVL TIME (hr)	9.0	9.3	9.6	9.9	10.0	10.1	10.2	10.3	10.4	10.5	10.6	10.7	10.8	11.0	11.2	11.4	11.6	11.8	12.0	12.3	12.6	13.0	13.5	14.0	14.5	15.0	15.5	16.0	17.0	18.0	20.0	24.0
IA/P = 0.10												* * * TC = 1.0 HR * * *																IA/P = 0.10				
0.0	14	19	25	34	38	46	58	79	106	136	165	186	202	185	153	122	101	85	74	63	56	50	44	40	35	32	30	29	27	26	22	15
.10	13	18	24	32	36	42	53	70	94	122	151	174	194	188	160	130	106	89	77	65	57	50	44	40	36	32	30	29	27	26	22	15
.20	12	15	21	28	31	34	40	49	63	83	109	137	162	190	180	152	124	102	86	70	61	53	46	41	37	33	31	29	27	26	22	15
.30	12	15	20	26	29	33	37	45	57	75	98	124	149	187	183	159	131	107	90	73	62	53	47	42	37	33	31	29	28	26	23	15
.40	10	13	17	23	25	28	31	35	42	52	67	88	112	160	183	176	151	125	103	81	67	56	48	43	39	35	31	30	28	26	23	15
.50	10	13	16	22	24	27	30	33	39	48	61	79	101	148	181	181	157	131	108	84	69	58	49	43	39	35	32	30	28	26	23	16
.75	9	11	14	19	21	23	26	29	33	38	47	59	75	115	153	172	167	148	125	96	77	62	51	45	40	36	33	30	28	27	23	16
1.0	8	9	11	15	16	17	19	21	23	26	29	33	40	61	95	134	162	169	157	126	97	72	57	49	43	39	35	32	29	27	24	16
1.5	5	6	8	10	10	11	12	13	14	16	17	19	21	25	33	48	74	107	138	160	148	111	76	59	50	44	39	35	30	28	25	17
2.0	3	4	6	7	8	8	9	9	10	11	12	13	14	17	20	25	33	48	71	115	153	153	108	75	58	49	43	39	32	29	25	18
2.5	2	3	4	5	5	6	6	7	7	8	9	9	10	12	14	16	20	25	33	56	94	139	147	104	74	58	49	43	34	30	26	19
3.0	1	1	2	3	3	4	4	4	5	5	6	6	7	8	9	10	12	14	17	24	38	76	131	143	108	78	60	50	39	32	27	20
IA/P = 0.30												* * * TC = 1.0 HR * * *																IA/P = 0.30				
0.0	0	0	0	0	1	2	7	17	34	55	79	99	114	128	114	100	89	80	74	68	63	59	54	51	46	43	41	40	39	37	33	23
.10	0	0	0	0	0	0	2	5	13	27	46	68	89	116	124	110	98	87	79	71	65	60	56	52	48	43	41	40	39	37	34	24
.20	0	0	0	0	0	0	1	4	10	21	37	58	78	109	121	113	101	90	81	72	66	61	56	52	48	44	41	40	39	37	34	24
.30	0	0	0	0	0	0	0	1	3	8	17	31	49	87	113	118	109	98	87	76	69	63	57	53	49	45	42	40	39	38	34	24
.40	0	0	0	0	0	0	0	1	2	6	13	25	41	78	107	117	111	101	90	78	70	63	58	54	50	45	42	41	39	38	34	25
.50	0	0	0	0	0	0	0	0	2	5	10	20	34	69	100	115	113	103	93	80	71	64	58	54	50	46	42	41	39	38	34	25
.75	0	0	0	0	0	0	0	0	0	1	2	5	10	31	61	90	107	110	104	90	79	69	61	56	52	48	44	41	40	38	35	25
1.0	0	0	0	0	0	0	0	0	0	0	0	1	3	12	33	61	89	105	109	99	86	73	64	58	54	50	45	42	40	38	35	26
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	13	31	55	80	104	104	88	73	63	58	53	49	45	41	39	36	27
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	8	20	51	82	100	90	74	64	58	54	50	42	40	37	28
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	13	37	76	98	88	73	64	58	53	45	41	38	30
3,0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	9	34	77	95	86	73	63	58	49	42	38	31
IA/P = 0.50												* * * TC = 1.0 HR * * *																IA/P = 0.50				
0.0	0	0	0	0	0	0	0	0	0	1	3	5	9	18	28	35	40	44	47	48	48	48	48	48	48	48	48	48	48	48	47	35
.10	0	0	0	0	0	0	0	0	0	1	2	4	7	16	25	33	39	43	46	48	48	48	48	48	48	48	48	48	48	48	47	35
.20	0	0	0	0	0	0	0	0	0	1	2	3	6	14	23	31	38	42	45	47	48	48	48	48	48	48	48	48	48	48	47	36
.30	0	0	0	0	0	0	0	0	0	0	1	3	5	12	21	29	36	41	44	47	48	48	48	48	48	48	48	48	48	48	47	36
.40	0	0	0	0	0	0	0	0	0	0	1	2	4	10	19	27	34	40	44	47	48	48	48	48	48	48	48	48	48	48	47	36
.50	0	0	0	0	0	0	0	0	0	0	1	2	3	8	16	25	33	38	43	46	48	48	48	48	4&	48	48	48	48	48	47	36
.75	0	0	0	0	0	0	0	0	0	0	1	2	3	8	15	23	30	36	41	45	47	48	48	48	48	48	48	48	48	48	47	36
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	11	19	27	33	41	45	47	48	48	48	48	48	48	48	48	48	37
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	5	11	17	28	37	44	47	48	48	48	48	48	48	48	48	39
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	7	16	26	38	45	47	48	48	48	48	48	48	48	40
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	12	24	38	45	47	47	47	47	47	47	47	41
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	14	29	40	46	47	47	47	47	47	47	42

RAINFALL TYPE = I   * * * TC = 1.0 HR * * *   SHEET 7 OF 10

Exhibit 5-I: Tabular hydrograph unit discharges (csm/in) for type I rainfall distribution—continued

IA/P = 0.10 * * * TC = 1.25 HR * * * IA/P = 0.10

TRVL TIME (hr)	9.0	9.3	9.6	9.9	10.0	10.1	10.2	10.3	10.4	10.5	10.6	10.7	10.8	11.0	11.2	11.4	11.6	11.8	12.0	12.3	12.6	13.0	13.5	14.0	14.5	15.0	15.5	16.0	17.0	18.0	20.0	24.0
0.0	12	16	22	29	33	38	47	61	80	103	127	149	164	180	163	138	115	98	85	71	62	53	47	41	37	33	31	29	27	26	22	15
.10	11	14	19	25	28	31	36	44	55	72	92	116	138	167	175	156	132	111	95	77	66	56	48	43	38	34	31	30	28	26	23	15
.20	11	14	18	24	27	30	34	40	50	65	83	105	127	160	172	160	138	116	99	80	68	57	49	43	39	34	31	30	28	26	23	15
.30	10	12	16	21	23	25	28	32	38	46	58	75	95	136	164	169	154	132	112	89	74	61	51	45	40	36	32	30	28	26	23	16
.40	9	12	15	20	22	24	27	30	35	42	53	68	86	126	157	167	157	137	117	92	76	62	52	46	40	36	33	30	28	26	23	16
.50	9	11	13	17	19	21	23	26	29	33	39	49	61	97	134	160	165	152	132	104	84	67	55	47	42	37	33	31	28	27	23	16
.75	7	9	11	14	15	17	18	20	22	25	28	32	38	59	90	124	150	159	152	126	101	77	60	51	45	40	35	32	29	27	24	17
1.0	6	8	10	12	13	14	15	17	19	20	23	25	29	40	60	91	123	148	157	144	118	88	66	54	47	41	37	33	29	27	24	17
1.5	4	6	7	9	9	10	11	12	13	14	15	16	18	22	28	40	59	85	114	145	150	121	86	65	54	46	41	37	31	28	25	13
2.0	2	3	5	6	6	7	7	8	9	9	10	11	11	13	16	19	24	33	47	80	119	144	124	90	68	55	47	42	33	29	26	19
2.5	1	2	3	4	4	5	5	6	6	7	7	8	8	10	11	13	16	19	24	38	65	111	140	120	88	67	54	47	37	31	27	20
3.0	1	1	2	2	3	3	3	4	4	5	5	5	6	7	8	9	11	13	15	21	32	62	114	136	116	86	66	54	41	33	27	20

IA/P = 0.30 * * * TC = 1.25 HR * * * IA/P = 0.30

TRVL TIME (hr)	9.0	9.3	9.6	9.9	10.0	10.1	10.2	10.3	10.4	10.5	10.6	10.7	10.8	11.0	11.2	11.4	11.6	11.8	12.0	12.3	12.6	13.0	13.5	14.0	14.5	15.0	15.5	16.0	17.0	18.0	20.0	24.0
0.0	0	0	0	0	0	1	4	10	21	35	53	71	86	106	115	104	94	86	79	72	66	61	56	52	48	44	41	40	39	37	34	24
.10	0	0	0	0	0	0	1	3	8	16	29	45	62	92	107	113	101	92	84	75	69	63	58	53	49	45	42	41	39	38	34	24
.20	0	0	0	0	0	0	1	2	6	13	23	37	54	84	104	110	104	94	86	77	70	64	58	54	49	45	42	41	39	38	34	24
.30	0	0	0	0	0	0	0	0	2	4	10	19	31	62	90	106	109	101	92	81	73	66	60	55	51	46	43	41	39	38	34	25
.40	0	0	0	0	0	0	0	0	1	3	8	15	26	55	83	102	107	103	94	83	74	67	60	56	51	47	43	41	39	38	35	25
.50	0	0	0	0	0	0	0	0	0	1	3	6	12	33	62	38	103	106	100	88	78	69	62	57	53	48	44	42	40	38	35	25
.75	0	0	0	0	0	0	0	0	0	0	0	1	3	12	31	56	80	97	103	97	87	75	65	59	55	50	46	43	40	39	35	26
1.0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	13	32	56	79	95	101	94	80	69	61	56	52	48	44	40	39	36	27
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	13	30	51	81	98	94	79	68	61	56	52	47	42	39	36	28
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	13	37	66	95	95	78	68	61	56	51	44	40	37	29
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	9	26	61	93	89	77	67	60	55	47	41	38	30
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	24	64	91	87	76	66	60	50	43	39	31

IA/P = 0.50 * * * TC = 1.25 HR * * * IA/P = 0.50

TRVL TIME (hr)	9.0	9.3	9.6	9.9	10.0	10.1	10.2	10.3	10.4	10.5	10.6	10.7	10.8	11.0	11.2	11.4	11.6	11.8	12.0	12.3	12.6	13.0	13.5	14.0	14.5	15.0	15.5	16.0	17.0	18.0	20.0	24.0
0.0	0	0	0	0	0	0	0	0	0	1	2	3	6	13	21	29	35	40	43	47	47	47	47	47	47	47	47	47	47	47	47	36
.10	0	0	0	0	0	0	0	0	0	0	1	3	5	11	19	27	34	39	42	47	47	47	47	47	47	47	47	47	47	47	47	36
.20	0	0	0	0	0	0	0	0	0	0	1	2	4	9	17	25	32	37	41	46	47	47	47	47	47	47	47	47	47	47	47	36
.30	0	0	0	0	0	0	0	0	0	0	1	2	3	8	15	23	30	36	40	45	47	47	47	47	47	47	47	47	47	47	47	36
.40	0	0	0	0	0	0	0	0	0	0	0	1	2	7	13	21	28	35	39	45	47	47	47	47	47	47	47	47	47	47	47	37
.50	0	0	0	0	0	0	0	0	0	0	0	1	2	6	12	19	27	33	38	44	46	47	47	47	47	47	47	47	47	47	47	37
.75	0	0	0	0	0	0	0	0	0	0	0	0	1	3	8	14	21	28	34	41	45	47	47	47	47	47	47	47	47	47	47	37
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	5	11	17	24	34	41	46	47	47	47	47	47	47	47	47	47	38
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	5	10	19	29	39	45	47	47	47	47	47	47	47	47	40
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	9	18	30	41	46	47	47	47	47	47	47	47	41
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	7	17	31	41	46	47	47	47	47	47	47	42
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	18	32	42	46	47	47	47	47	47	43

RAINFALL TYPE = I * * * TC = 1.25 HR * * * SHEET 8 OF 10

Exhibit 5-I: Tabular hydrograph unit discharges (csm/in) for type I rainfall distribution—continued

TRVL TIME (hr)	9.0	9.3	9.6	9.9	10.0	10.1	10.2	10.3	10.4	10.5	10.6	10.7	10.8	11.0	11.2	11.4	11.6	11.8	12.0	12.3	12.6	13.0	13.5	14.0	14.5	15.0	15.5	16.0	17.0	18.0	20.0	24.0
IA/P = 0.10 * * * TC = 1.5 HR * * * IA/P = 0.10																																
0.0	11	14	19	25	28	32	39	48	60	76	94	112	129	151	163	147	130	112	97	81	69	59	50	44	39	35	32	30	28	26	23	15
.10	10	12	16	22	24	27	30	36	44	55	69	86	103	135	152	159	143	125	108	88	75	62	52	46	41	36	32	30	28	26	23	16
.20	9	11	14	19	21	23	25	29	34	41	50	63	78	112	140	155	152	138	121	98	81	67	55	43	42	37	33	31	28	27	23	16
.30	9	11	14	18	20	22	24	27	31	38	46	57	71	104	133	151	154	141	125	101	84	68	56	48	43	38	34	31	29	27	23	16
.40	8	10	13	17	19	21	23	26	30	35	42	52	65	96	126	147	152	144	129	105	87	70	57	49	43	38	34	31	29	27	23	16
.50	8	9	12	15	17	18	20	22	25	28	33	39	48	73	104	131	148	151	140	117	95	75	60	51	45	40	35	32	29	27	24	17
.75	7	8	11	13	15	16	18	19	21	24	27	32	38	56	82	110	133	146	144	127	106	83	65	54	47	41	36	33	29	27	24	17
1.0	5	7	9	11	11	12	13	15	16	18	19	22	24	32	47	69	96	122	139	145	127	99	74	60	51	44	39	35	30	28	24	17
1.5	4	5	6	8	8	9	10	10	11	12	13	14	16	19	24	32	46	66	90	124	139	128	97	73	59	50	44	39	32	29	25	18
2.0	2	3	4	5	5	6	6	7	7	8	9	9	10	12	14	17	21	27	38	63	96	135	129	100	76	61	51	44	35	30	26	19
2.5	1	2	2	3	4	4	4	5	5	6	6	7	7	8	10	11	14	16	20	31	51	91	132	124	98	75	60	51	39	32	27	20
3.0	0	1	1	2	2	2	2	3	3	3	4	4	5	6	7	8	9	10	12	16	23	42	86	123	128	101	77	62	45	35	28	21
IA/P = 0.30 * * * TC = 1.5 HR * * * IA/P = 0.30																																
0.0	0	0	0	0	0	1	3	7	13	21	33	46	60	83	97	106	97	90	84	76	70	64	59	54	50	45	43	41	39	38	34	25
.10	0	0	0	0	0	0	1	2	5	10	18	28	40	66	86	98	103	95	88	80	73	66	60	55	51	47	43	41	39	38	35	25
.20	0	0	0	0	0	0	0	1	4	8	14	23	34	60	81	95	101	97	90	81	74	67	61	56	52	47	44	41	40	38	35	25
.30	0	0	0	0	0	0	0	0	1	3	6	12	19	41	65	85	98	100	95	85	77	69	62	57	53	48	44	42	40	38	35	25
.40	0	0	0	0	0	0	0	0	1	2	5	9	16	36	59	80	94	99	96	87	79	70	63	58	53	49	45	42	40	38	35	26
.50	0	0	0	0	0	0	0	0	1	2	4	8	13	31	54	75	91	98	97	88	80	71	64	58	54	49	45	42	40	38	35	26
.75	0	0	0	0	0	0	0	0	0	0	1	2	4	13	28	49	69	85	95	95	87	76	67	61	56	51	47	44	40	39	36	26
1.0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	13	29	49	69	84	95	92	81	71	63	58	53	49	45	41	39	36	27
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	9	20	35	63	84	92	83	72	64	58	54	49	43	40	37	28
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	9	25	50	79	90	81	71	63	58	53	45	41	38	29
2.s	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	18	46	79	88	80	70	63	57	49	42	38	31
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	17	50	79	87	79	70	62	52	45	39	32
IA/P = 0.50 * * * TC = 1.5 HR * * * IA/P = 0.50																																
0.0	0	0	0	0	0	0	0	0	0	0	1	2	3	8	15	22	29	34	39	44	46	46	46	46	46	46	46	46	46	46	46	37
.10	0	0	0	0	0	0	0	0	0	0	0	1	2	5	10	17	24	30	35	41	46	46	46	46	46	46	46	46	46	46	46	37
.20	0	0	0	0	0	0	0	0	0	0	0	1	1	4	9	15	22	28	34	40	45	46	46	46	46	46	46	46	46	46	46	37
.30	0	0	0	0	0	0	0	0	0	0	0	0	1	3	7	13	20	27	33	39	44	46	46	46	46	46	46	46	46	46	46	38
.40	0	0	0	0	0	0	0	0	0	0	0	0	1	3	6	12	18	25	31	38	44	46	46	46	46	46	46	46	46	46	46	38
.50	0	0	0	0	0	0	0	0	0	0	0	0	1	2	5	10	17	23	30	37	43	46	46	46	46	46	46	46	46	46	46	38
.75	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	7	12	19	25	33	40	45	46	46	46	46	46	46	46	46	46	38
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	5	10	15	25	33	42	46	46	46	46	46	46	46	46	46	39
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	7	14	23	34	43	46	46	46	46	46	46	46	46	40
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	11	22	35	43	46	46	46	46	46	46	46	42
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	10	23	36	43	46	46	46	46	46	46	43
3.0	0	n	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	14	27	38	44	46	46	46	46	46	44

RAINFALL TYPE = I * * * TC = 1.5 HR * * * SHEET 9 OF 10

Exhibit 5-I: Tabular hydrograph unit discharges (csm/in) for type I rainfall distribution—continued

TRVL TIME (hr)	9.0	9.3	9.6	9.9	10.0	10.1	10.2	10.3	10.4	10.5	10.6	10.7	10.8	11.0	11.2	11.4	11.6	11.8	12.0	12.3	12.6	13.0	13.5	14.0	14.5	15.0	15.5	16.0	17.0	18.0	20.0	24.0
IA/P = 0.10														* * * TC = 2.0 HR * * *														IA/P = 0.10				
0.0	9	11	14	19	21	24	28	33	40	49	59	70	82	106	123	138	138	125	115	97	83	70	59	50	44	39	35	32	29	27	23	16
.10	8	10	13	17	18	20	23	26	31	37	45	54	65	89	110	125	135	130	123	106	90	75	62	53	46	40	36	33	29	27	24	16
.20	8	10	12	16	17	19	22	25	29	34	41	50	60	83	105	121	132	131	125	109	92	77	63	53	46	41	36	33	29	27	24	17
.30	7	9	11	14	15	17	18	21	23	27	32	38	46	66	89	109	123	131	129	117	100	82	66	56	48	42	38	34	30	28	24	17
.40	7	8	10	13	15	16	18	20	22	25	30	35	43	61	83	104	120	131	131	119	103	84	68	57	49	43	38	34	30	28	24	17
.50	6	8	10	13	14	15	17	19	21	24	28	33	39	56	78	99	116	127	130	121	106	86	69	58	50	44	39	35	30	28	24	17
.75	5	7	8	11	12	13	14	15	16	18	20	23	27	38	53	73	93	111	123	127	118	98	77	63	54	47	41	37	31	28	25	17
1.0	4	5	7	8	9	10	11	12	13	14	15	17	18	24	32	45	63	83	102	122	126	113	89	71	59	51	44	39	32	29	25	18
1.5	3	4	5	6	7	7	8	8	9	10	10	11	12	15	18	23	32	44	60	88	111	123	110	87	70	58	50	44	35	30	26	19
2.0	1	2	3	4	5	5	5	6	6	7	7	8	9	10	12	14	18	23	31	49	74	106	121	107	86	69	58	49	38	32	27	20
2.5	1	1	2	2	3	3	3	4	4	4	5	5	6	7	8	9	11	13	16	22	35	62	101	118	108	88	71	59	44	35	28	21
3.0	0	0	1	1	1	2	2	2	3	3	3	3	4	5	5	6	8	9	10	14	19	34	67	103	116	105	86	70	50	39	29	21
IA/P = 0.30														* * * TC = 2.0 HR * * *														IA/P = 0.30				
0.0	0	0	0	0	0	0	1	3	6	10	16	23	31	49	65	77	84	92	86	80	75	69	63	58	53	49	45	43	40	38	35	26
.10	0	0	0	0	0	0	0	1	2	5	9	13	19	35	53	68	79	85	90	83	77	71	64	59	55	50	46	43	40	39	35	26
.20	0	0	0	0	0	0	0	0	1	2	4	7	11	24	40	57	71	81	89	89	80	73	66	61	56	51	47	44	41	39	35	26
.30	0	0	0	0	0	0	0	0	1	1	3	6	9	20	36	53	68	78	84	88	81	74	67	61	56	52	48	44	41	39	36	26
.40	0	0	0	0	0	0	0	0	0	0	1	2	5	12	24	40	57	70	80	87	84	76	69	63	58	53	49	45	41	39	36	27
.50	0	0	0	0	0	0	0	0	0	0	1	2	4	10	21	36	53	67	77	87	85	77	69	63	58	54	49	46	41	39	36	27
.75	0	0	0	0	0	0	0	0	0	0	0	1	2	6	14	26	41	56	69	82	85	80	72	65	60	55	51	47	42	40	36	27
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	10	20	34	49	68	81	85	77	69	63	58	53	49	43	40	37	28
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	10	19	38	58	78	83	76	68	62	57	53	45	41	37	29
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	4	13	29	55	77	82	75	68	62	57	48	43	38	30
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	6	22	51	75	81	75	68	62	53	45	39	32
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	25	54	75	80	75	68	57	48	43	33
IA/P = 0.50														* * * TC = 2.0 HR * * *														IA/P = 0.50				
0.0	0	0	0	0	0	0	0	0	0	0	0	1	2	4	8	13	18	24	29	36	41	46	46	46	46	46	46	46	46	46	46	38
.10	0	0	0	0	0	0	0	0	0	0	0	1	1	3	7	12	17	22	28	35	40	45	46	46	46	46	46	46	46	46	46	38
.20	0	0	0	0	0	0	0	0	0	0	0	0	1	2	4	8	13	18	24	31	38	44	46	46	46	46	46	46	46	46	46	39
.30	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	7	12	17	22	30	37	43	45	46	46	46	46	46	46	46	46	39
.40	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	6	11	16	21	29	36	42	45	46	46	46	46	46	46	46	46	39
.50	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	5	9	14	20	28	35	42	45	45	45	45	45	45	45	45	45	39
.75	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	5	9	13	18	26	33	41	45	45	45	45	45	45	45	45	45	39
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	5	9	16	24	33	42	45	45	45	45	45	45	45	45	40
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	4	8	15	25	36	43	45	45	45	45	45	45	45	42
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	6	14	26	37	43	45	45	45	45	45	45	43
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	6	15	27	37	43	45	45	45	45	45	44
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	3	19	30	39	44	45	45	45	45	45

RAINFALL TYPE = I * * * TC = 2.0 HR * * * SHEET 10 OF 10

Exhibit 5-IA: Tabular hydrograph unit discharges (csm/in) for type IA rainfall distribution

IA/P = 0.10 — * * * TC = 0.1 HR * * *

TRVL TIME (hr)	7.0	7.3	7.6	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.3	10.6	11.0	11.5	12.0	12.5	13.0	13.5	14.0	15.0	16.0	18.0	22.0
0.0	28	36	50	143	154	163	140	103	87	76	68	67	65	61	54	49	45	44	44	41	40	39	36	33	32	32	31	30	30	29	26	21
.10	27	32	43	104	130	146	157	145	117	97	83	73	69	65	59	53	48	45	45	42	40	39	37	34	33	32	32	31	30	29	27	22
.20	26	29	37	59	89	116	136	150	147	127	107	91	79	68	63	57	52	47	45	44	41	39	37	35	33	32	32	31	30	29	27	22
.30	25	28	35	53	77	103	125	142	145	133	116	99	86	71	65	59	53	48	46	44	41	39	38	35	33	32	32	31	30	29	27	22
.40	24	26	31	41	49	68	91	114	132	141	136	122	107	82	70	63	57	52	48	45	43	40	38	36	33	32	32	31	30	29	27	22
.50	24	26	30	39	46	60	81	103	123	135	135	127	114	88	73	65	59	53	49	45	43	40	39	36	33	33	32	31	30	29	27	22
.75	20	24	27	32	35	39	47	60	76	94	111	122	125	114	94	79	68	61	55	48	45	42	39	37	35	33	32	32	30	30	27	22
1.0	16	20	24	27	28	30	32	36	41	49	62	77	94	118	122	104	86	74	65	55	49	44	41	39	36	34	33	32	31	30	28	23
1.5	12	15	18	22	23	24	25	27	28	30	32	36	42	61	86	107	112	104	91	73	60	50	44	41	38	36	34	33	31	30	28	23
2.0	7	10	12	15	16	18	19	20	21	22	24	25	26	30	36	50	69	90	106	103	87	67	52	45	41	39	36	34	32	31	29	24
2.5	4	6	8	11	12	13	14	15	16	17	18	19	21	23	26	30	36	49	66	91	101	89	66	51	44	41	38	36	33	31	29	25
3.0	2	3	5	7	8	9	10	11	11	12	13	14	16	18	20	23	25	29	36	55	79	98	86	65	51	44	41	38	34	32	30	25

IA/P = 0.30 — * * * TC = 0.1 HR * * *

TRVL TIME (hr)	7.0	7.3	7.6	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.3	10.6	11.0	11.5	12.0	12.5	13.0	13.5	14.0	15.0	16.0	18.0	22.0
0.0	0	0	1	45	65	89	78	64	59	54	54	53	51	50	48	45	44	43	43	43	42	42	41	40	40	40	40	40	39	39	38	33
.10	0	0	0	18	36	55	78	78	69	62	57	55	53	51	50	47	44	44	43	43	42	42	41	40	40	40	40	40	39	39	38	34
.20	0	0	0	13	29	46	67	74	71	65	60	56	54	52	50	48	45	44	43	43	43	42	41	41	40	40	40	40	39	39	38	34
.30	0	0	0	10	22	38	58	69	70	67	62	58	56	52	51	48	46	44	44	43	43	42	41	41	40	40	40	40	39	39	38	34
,40	0	0	0	2	7	17	31	49	62	67	67	64	60	55	52	50	48	45	44	43	43	42	42	41	40	40	40	40	39	39	38	34
,50	0	0	0	1	5	13	25	41	55	63	66	64	61	56	53	51	48	46	44	43	43	42	42	41	40	40	40	40	39	39	38	34
.75	0	0	0	0	0	2	6	13	24	36	47	55	61	61	57	54	51	49	47	44	43	43	42	41	41	40	40	40	40	39	38	34
1.0	0	0	0	0	0	1	3	8	15	25	36	46	53	60	59	55	53	50	48	45	44	43	42	42	41	40	40	40	40	39	38	35
1.5	0	0	0	0	0	0	0	0	1	2	5	9	15	31	47	55	57	56	53	50	46	44	43	42	41	41	40	40	40	39	39	35
2.0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	15	28	42	51	55	54	51	47	44	43	42	41	41	40	40	39	39	36
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	6	14	26	38	50	54	52	47	44	43	42	41	40	40	40	39	36
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	6	13	29	43	53	51	47	44	43	42	41	40	40	39	37

IA/P = 0.50 — * * * TC = 0.1 HR * * *

TRVL TIME (hr)	7.0	7.3	7.6	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.3	10.6	11.0	11.5	12.0	12.5	13.0	13.5	14.0	15.0	16.0	18.0	22.0
0,0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	7	12	14	18	21	23	26	30	32	33	37	38	41	42	46	48	53	49
.10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	9	12	15	19	22	25	29	32	32	35	39	41	42	45	48	53	49
.20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	7	11	14	18	22	24	28	32	32	35	39	41	42	45	48	53	49
.30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	8	12	15	20	23	27	31	31	34	39	40	41	45	48	53	49
.40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	9	13	18	22	25	30	31	33	37	39	41	44	47	53	49
.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	6	10	tS	20	24	29	31	32	36	39	41	44	47	53	53
.75	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	6	11	16	21	26	30	31	34	38	40	43	46	53	53
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	11	17	23	28	31	32	35	39	42	45	53	53
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	8	17	23	28	31	32	35	40	43	49	52
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	9	17	23	28	31	32	38	42	48	52
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	10	18	24	28	31	36	40	47	52
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	11	18	24	28	33	39	45	52

RAINFALL TYPE = IA — * * * TC = 0.1 HR * * * — SHEET 1 OF 10

(210-VI-TR-55, Second Ed., June 1986)

Exhibit 5-IA: Tabular hydrograph unit discharges (csm/in) for type IA rainfall distribution—continued

IA/P = 0.10 — * * * TC = 0.2 HR * * * — IA/P = 0.10

TRVL TIME (hr)	7.0	7.3	7.6	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.3	10.6	11.0	11.5	12.0	12.5	13.0	13.5	14.0	15.0	16.0	18.0	22.0
0.0	27	34	46	124	143	153	153	127	103	87	76	68	67	63	57	51	47	45	44	42	40	39	36	33	33	32	32	31	30	29	27	21
.10	27	33	44	108	131	146	151	135	113	96	83	73	69	65	59	53	48	45	45	42	40	39	37	33	33	32	32	31	30	29	27	22
.20	26	29	38	67	95	119	137	146	139	122	105	90	79	69	63	57	52	47	45	44	41	39	37	34	33	32	32	31	30	29	27	22
.30	25	27	33	45	60	83	107	127	140	139	128	112	97	76	68	62	56	50	47	45	42	40	38	35	33	33	32	31	30	29	27	22
.40	24	27	32	43	54	73	96	117	132	137	131	118	104	81	70	63	57	52	48	45	43	40	38	36	33	33	32	32	30	29	27	22
.50	23	25	29	37	41	50	65	86	106	124	132	131	123	98	79	69	62	56	51	46	44	41	39	37	34	33	32	32	30	29	27	22
.75	20	24	27	33	36	41	50	63	80	97	112	121	123	111	93	78	68	61	55	48	45	42	39	37	34	33	32	32	30	30	27	22
1.0	16	20	24	27	29	31	33	37	43	52	65	81	96	120	120	102	85	73	64	55	49	44	41	39	36	34	33	32	31	30	28	23
1.5	12	15	19	22	23	24	26	27	28	30	33	37	44	63	88	107	111	102	89	72	60	50	44	41	38	36	34	33	32	30	28	23
2.0	7	10	12	16	17	18	19	20	22	23	24	25	27	30	38	52	71	91	105	101	86	66	52	45	41	39	36	34	32	31	29	24
2.5	4	6	9	11	12	13	14	15	16	17	19	20	21	23	26	30	38	50	68	92	101	88	65	51	44	41	38	36	33	31	29	25
3.0	2	3	5	7	7	8	9	10	11	12	13	14	15	17	19	22	24	28	33	49	72	97	89	68	53	45	41	39	34	32	30	25

IA/P = 0.30 — * * * TC = 0.2 HR * * * — IA/P = 0.30

TRVL TIME (hr)	7.0	7.3	7.6	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.3	10.6	11.0	11.5	12.0	12.5	13.0	13.5	14.0	15.0	16.0	18.0	22.0
0.0	0	0	0	26	45	64	76	74	65	59	55	53	52	51	49	46	44	44	43	43	42	42	41	40	40	40	40	40	39	39	38	33
.10	0	0	0	20	37	55	69	72	67	62	57	54	53	52	50	47	44	44	43	43	42	42	41	40	40	40	40	40	39	39	38	34
.20	0	0	0	5	15	30	47	61	68	68	64	59	56	53	52	49	46	44	44	43	43	42	42	41	40	40	40	40	39	39	38	34
.30	0	0	0	4	11	23	39	54	64	66	65	61	58	54	52	50	47	45	44	43	43	42	42	41	40	40	40	40	39	39	38	34
.40	0	0	0	3	8	18	32	47	58	64	64	62	59	55	52	50	48	45	44	43	43	42	42	41	40	40	40	40	40	39	38	34
.50	0	0	0	0	2	6	14	26	40	52	60	63	62	58	54	52	50	47	45	44	43	42	42	41	40	40	40	40	40	39	38	34
.75	0	0	0	0	1	3	7	14	24	35	45	53	57	59	56	54	51	49	47	44	43	43	42	41	41	40	40	40	40	39	38	34
1.0	0	0	0	0	0	0	1	4	8	15	25	35	44	56	58	56	54	51	49	46	44	43	42	42	41	40	40	40	40	39	39	35
1.5	0	0	0	0	0	0	0	0	0	1	2	5	9	23	38	50	55	56	54	51	47	44	43	42	42	41	40	40	40	39	39	35
2.0	0	0	0	0	0	0	0	0	0	0	0	1	1	6	15	27	40	49	54	54	51	47	44	43	42	41	41	40	40	40	39	36
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	6	14	25	36	49	53	51	47	44	43	42	41	41	40	40	39	36
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	6	13	28	42	52	51	47	44	43	42	41	40	40	39	37

IA/P = 0.50 — * * * TC = 0.2 HR * * * — IA/P = 0.50

TRVL TIME (hr)	7.0	7.3	7.6	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.3	10.6	11.0	11.5	12.0	12.5	13.0	13.5	14.0	15.0	16.0	18.0	22.0
0.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	5	10	13	16	19	23	25	29	32	33	36	39	41	42	46	48	50	49
.10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	6	10	14	17	21	24	28	31	32	35	40	41	42	45	48	50	49
.20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	7	11	15	19	23	26	31	31	34	39	40	41	45	48	50	49
.30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	8	12	17	21	25	30	31	33	37	39	41	44	47	50	50
.40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	9	15	19	23	28	31	32	35	39	41	44	47	50	50
.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	6	12	17	22	27	31	32	34	39	40	43	46	50	50
.75	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	9	14	20	25	30	31	33	37	39	42	46	50	50
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	9	16	22	27	31	32	35	38	42	45	49	50
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	8	17	23	28	31	32	35	40	43	49	50
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	9	17	23	28	31	32	38	42	48	50
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	9	16	22	27	30	35	40	46	50
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	9	17	23	27	33	38	45	50

RAINFALL TYPE = IA — * * * TC = 0.2 HR * * * — SHEET 2 OF 10

Exhibit 5-IA: Tabular hydrograph unit discharges (csm/in) for type IA rainfall distribution—continued

HYDROGRAPH TIME (HOURS)

IA/P = 0.10 — * * * TC = 0.3 HR * * *

TRVL TIME (hr)	7.0	7.3	7.6	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.3	10.6	11.0	11.5	12.0	12.5	13.0	13.5	14.0	15.0	16.0	18.0	22.0
0.0	26	31	41	92	120	138	145	144	125	105	90	78	71	66	60	54	49	46	45	43	40	39	37	34	33	32	32	31	30	29	27	22
.10	26	30	39	80	106	128	139	142	131	113	98	85	76	68	62	56	51	47	45	43	41	39	37	34	33	32	32	31	30	29	27	22
.20	25	28	34	51	70	94	117	131	139	133	120	105	92	74	67	60	54	49	46	44	42	40	38	35	33	33	32	31	30	29	27	22
.30	24	26	31	40	47	62	84	106	123	133	133	124	112	87	73	65	59	53	49	45	43	40	39	36	33	33	32	32	30	29	27	22
.40	23	26	30	38	44	56	75	95	114	127	131	127	117	93	76	67	60	55	50	46	43	40	39	36	34	33	32	32	30	29	27	22
.50	21	25	28	34	37	42	51	67	86	104	119	127	127	110	88	74	66	59	53	47	45	41	39	37	34	33	32	32	30	30	27	22
.75	19	23	26	31	33	36	42	51	64	80	96	109	119	116	102	85	73	65	58	50	46	43	40	38	35	33	33	32	31	30	28	23
1.0	15	19	23	26	27	29	31	33	37	44	53	66	80	106	116	109	93	79	69	58	51	45	41	39	37	34	33	32	31	30	28	23
1.5	11	14	17	21	22	23	24	26	27	29	31	34	38	53	75	97	109	106	96	78	64	52	45	41	39	36	34	33	32	30	28	24
2.0	6	9	11	14	16	17	18	19	20	21	23	24	25	28	34	44	61	81	97	103	91	71	54	46	42	39	37	34	32	31	29	24
2.5	4	5	8	10	11	12	13	14	15	16	17	19	20	22	25	28	34	44	58	84	59	92	70	54	46	41	39	36	33	32	29	25
3.0	2	3	5	7	7	8	9	10	11	12	13	14	15	17	19	22	24	28	33	49	71	96	89	68	53	45	41	39	34	32	30	25

IA/P = 0.30 — * * * TC = 0.3 HR * * *

TRVL TIME (hr)	7.0	7.3	7.6	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.3	10.6	11.0	11.5	12.0	12.5	13.0	13.5	14.0	15.0	16.0	18.0	22.0
0.0	0	0	0	13	28	45	63	69	69	65	60	55	53	53	51	48	45	44	43	43	42	42	42	40	40	40	40	40	39	39	38	34
.10	0	0	0	9	21	37	54	64	67	65	62	57	54	53	51	49	46	44	43	43	42	42	42	41	40	40	40	40	40	39	38	34
.20	0	0	0	7	17	30	46	58	64	65	63	59	56	53	52	49	46	44	44	43	42	42	42	41	40	40	40	40	40	39	38	34
.30	0	0	0	1	5	13	25	39	52	60	63	63	60	55	53	51	49	46	44	43	43	42	42	41	40	40	40	40	40	39	38	34
.40	0	0	0	1	4	10	20	33	45	55	61	62	61	56	54	52	49	46	45	43	43	42	42	41	40	40	40	40	40	39	38	34
.50	0	0	0	1	3	7	15	27	39	50	57	60	61	57	54	52	50	47	45	44	43	42	42	41	40	40	40	40	40	39	38	34
.75	0	0	0	0	1	3	8	15	24	34	43	51	55	58	56	54	52	49	47	44	43	43	42	41	41	40	40	40	40	39	38	34
1.0	0	0	0	0	0	0	0	2	4	9	16	25	34	49	56	57	55	53	50	47	44	43	42	42	41	40	40	40	40	39	39	35
1.5	0	0	0	0	0	0	0	0	0	1	3	5	10	23	37	49	54	55	54	51	47	44	43	42	42	41	40	40	40	39	39	35
2.0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	10	21	33	44	51	54	52	48	44	43	42	41	41	40	40	40	39	36
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	10	19	30	45	53	53	48	44	43	42	41	41	40	40	39	37
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	9	23	37	50	52	47	44	43	42	41	40	40	39	37

IA/P = 0.50 — * * * TC = 0.3 HR * * *

TRVL TIME (hr)	7.0	7.3	7.6	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.3	10.6	11.0	11.5	12.0	12.5	13.0	13.5	14.0	15.0	16.0	18.0	22.0
0.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	7	11	14	18	22	24	28	32	32	35	40	41	42	45	48	48	48
.10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	8	12	15	20	23	27	31	31	34	39	40	41	45	48	48	48
.20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	9	13	18	21	25	30	31	33	37	39	41	44	47	48	48
.30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	8	12	17	21	25	30	31	33	37	39	41	44	47	48	48
.40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	9	14	19	23	28	31	32	35	39	41	44	47	48	48
,50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	10	15	21	26	30	31	33	38	40	43	46	48	48
.75	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	7	13	19	24	29	31	33	36	39	42	46	48	48
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	9	16	22	27	31	32	35	38	42	45	48	48
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	8	16	23	27	31	32	35	40	43	48	48
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	8	16	22	27	30	32	38	42	47	48
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	9	16	22	27	30	35	40	46	48
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	9	17	23	27	33	38	45	48

RAINFALL TYPE = IA — * * * TC = 0.3 HR * * * — SHEET 3 OF 10

Exhibit 5-IA: Tabular hydrograph unit discharges (csm/in) for type IA rainfall distribution—continued

HYDROGRAPH TIME (HOURS)

IA/P = 0.10 — * * * TC = 0.4 HR * * *

TRVL TIME (hr)	7.0	7.3	7.6	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.3	10.6	11.0	11.5	12.0	12.5	13.0	13.5	14.0	15.0	16.0	18.0	22.0
0.0	26	29	37	67	93	117	134	137	136	121	105	91	81	70	64	58	52	48	46	44	41	40	37	34	33	33	32	31	30	29	27	22
.10	25	29	36	60	82	105	124	133	135	126	112	98	87	73	66	59	53	49	46	44	41	40	38	34	33	33	32	31	30	29	27	22
.20	24	27	32	43	55	73	94	114	127	132	128	117	104	83	71	64	58	52	48	45	43	40	38	35	33	33	32	32	30	29	27	22
.30	24	26	31	41	50	65	85	104	119	128	128	121	110	88	74	66	59	54	49	46	43	40	39	35	34	33	32	32	30	29	27	22
.40	22	25	28	35	39	46	59	76	95	111	122	126	122	104	85	72	64	58	52	47	44	41	39	37	34	33	32	32	30	29	27	22
.50	20	23	26	31	34	37	43	54	69	86	103	116	123	117	98	82	71	63	56	49	46	42	40	38	34	33	33	32	30	30	27	22
.75	16	20	24	27	29	31	33	37	43	53	65	80	94	116	116	101	86	74	65	56	49	44	41	39	36	34	33	32	31	30	28	23
1.0	14	18	22	25	26	27	29	31	34	38	45	54	67	93	111	113	101	87	75	62	53	47	42	40	37	34	33	33	31	30	28	23
1.5	10	13	16	20	21	22	23	24	26	27	29	31	34	45	64	86	102	107	101	84	69	55	46	42	39	37	34	33	32	30	28	24
2.0	6	8	11	13	14	16	17	18	19	20	21	23	24	27	31	39	52	71	88	102	96	76	57	47	42	40	37	35	33	31	29	24
2.5	3	5	7	9	10	11	12	13	14	15	16	17	18	21	23	26	31	38	51	75	94	98	74	57	47	42	39	37	33	32	30	25
3.0	1	2	4	6	7	7	8	9	10	11	12	13	14	16	18	20	23	26	30	43	64	89	95	72	56	47	42	39	34	33	30	25

IA/P = 0.30 — * * * TC = 0.4 HR * * *

TRVL TIME (hr)	7.0	7.3	7.6	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.3	10.6	11.0	11.5	12.0	12.5	13.0	13.5	14.0	15.0	16.0	18.0	22.0
0.0	0	0	0	6	15	29	46	60	65	65	63	59	56	53	52	49	47	44	44	43	42	42	42	41	40	40	40	40	40	39	38	34
.10	0	0	0	4	12	23	38	53	61	63	63	60	57	54	53	50	47	45	44	43	42	42	42	41	40	40	40	40	40	39	38	34
.20	0	0	0	3	9	19	32	46	56	61	62	61	59	55	53	51	48	45	44	43	43	42	42	41	40	40	40	40	40	39	38	34
.30	0	0	0	2	7	15	26	39	50	57	61	61	59	56	53	51	49	46	44	44	43	42	42	41	40	40	40	40	40	39	38	34
.40	0	0	0	0	2	5	11	21	33	44	53	58	60	58	55	53	51	48	45	44	43	42	42	41	40	40	40	40	40	39	38	34
.50	0	0	0	0	1	4	9	17	28	39	48	55	59	59	56	53	51	49	46	44	43	42	42	41	40	40	40	40	40	39	38	34
.75	0	0	0	0	0	2	4	9	16	25	34	43	49	57	57	55	53	50	48	45	44	43	42	41	41	40	40	40	40	39	39	35
1.0	0	0	0	0	0	0	1	2	5	10	17	25	34	48	55	56	55	53	50	47	45	43	42	42	41	40	40	40	40	39	39	35
1.5	0	0	0	0	0	0	0	0	0	0	1	3	6	16	30	43	51	55	55	52	49	45	43	42	42	41	40	40	40	40	39	36
2.0	3	0	0	0	0	0	0	0	0	0	0	0	0	2	6	15	27	39	47	53	53	49	45	43	42	42	41	40	40	40	39	36
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	7	14	25	40	50	52	49	45	43	42	41	41	40	40	39	37
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	7	18	32	47	52	48	45	43	42	41	40	40	39	37

IA/P = 0.50 — * * * TC = 0.4 HR * * *

TRVL TIME (hr)	7.0	7.3	7.6	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.3	10.6	11.0	11.5	12.0	12.5	13.0	13.5	14.0	15.0	16.0	18.0	22.0
0.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	9	13	16	20	23	27	31	32	34	39	40	42	45	47	47	47
.10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	6	10	13	18	22	26	30	31	33	38	39	41	45	47	47	47
.20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	7	11	16	20	24	29	31	33	36	39	41	44	47	47	47
.30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	8	13	18	23	28	31	32	35	39	41	43	47	47	47
.40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	11	16	21	26	30	32	34	38	40	43	46	47	47
.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	8	13	19	25	29	31	33	37	39	42	46	47	47
.75	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	6	11	17	23	28	31	32	35	39	42	45	47	47
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	6	12	20	25	29	31	33	37	41	44	47	47
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	13	20	26	30	31	34	40	43	47	47
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	14	21	26	30	32	37	41	47	47
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	7	15	21	26	30	34	40	46	47
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	8	15	22	27	32	38	45	47

RAINFALL TYPE = IA — * * * TC = 0.4 HR * * * — SHEET 4 OF 10

(210-VI-TR-55, Second Ed., June 1986)

Exhibit 5-IA: Tabular hydrograph unit discharges (csm/in) for type IA rainfall distribution—continued

HYDROGRAPH TIME (HOURS)

IA/P = 0.10 * * * TC = 0.5 HR * * * IA/P = 0.10

TRVL TIME (hr)	7.0	7.3	7.6	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.3	10.6	11.0	11.5	12.0	12.5	13.0	13.5	14.0	15.0	16.0	18.0	22.0
0.0	25	28	34	53	72	94	115	129	130	129	117	104	92	76	68	61	55	50	47	44	42	40	38	34	33	33	32	31	30	29	27	22
.10	24	27	33	49	64	84	105	121	127	128	121	109	98	80	70	63	57	51	48	45	42	40	38	34	33	33	32	32	30	29	27	22
.20	23	26	30	39	46	58	75	95	112	122	126	123	114	93	78	69	61	55	50	46	44	41	39	36	34	33	32	32	30	29	27	22
.30	21	24	27	34	37	43	53	68	86	103	116	123	123	108	89	76	67	60	54	48	45	42	39	37	34	33	33	32	30	30	27	22
.40	20	24	27	32	35	40	49	61	78	95	109	118	121	111	93	79	69	62	55	49	45	42	40	37	34	33	33	32	30	30	27	22
.50	18	22	25	29	31	34	38	45	56	70	87	101	113	118	106	89	77	67	60	52	47	43	40	38	35	33	33	32	31	30	28	23
.75	15	19	23	26	27	29	31	34	38	45	54	67	80	104	112	106	93	80	70	59	51	46	41	39	36	34	33	33	31	30	28	23
1.0	13	16	20	24	25	26	27	29	31	34	39	46	56	80	102	110	105	93	81	66	56	48	43	40	38	35	33	33	31	30	28	23
1.5	9	12	15	18	20	21	22	23	24	26	27	29	31	39	55	75	94	104	103	89	74	58	48	43	40	37	35	33	32	31	29	24
2.0	5	7	10	12	13	14	15	17	18	19	20	21	23	25	29	35	45	62	79	100	100	81	61	49	43	40	38	35	33	31	29	25
2.5	3	4	6	8	9	10	11	12	13	14	15	16	17	20	22	25	28	34	45	67	88	96	79	60	49	43	40	37	33	32	30	25
3.0	1	2	3	5	6	7	7	8	9	10	11	12	13	15	17	19	21	24	28	38	56	83	93	77	59	48	43	40	35	33	30	26

IA/P = 0.30 * * * TC = 0.5 HR * * * IA/P = 0.30

TRVL TIME (hr)	7.0	7.3	7.6	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.3	10.6	11.0	11.5	12.0	12.5	13.0	13.5	14.0	15.0	16.0	18.0	22.0
0.0	0	0	0	2	8	18	31	46	57	61	61	61	58	54	53	51	48	45	44	44	43	42	42	41	40	40	40	40	40	39	38	34
.10	0	0	0	2	6	14	25	39	51	58	60	61	59	55	53	51	49	46	44	44	43	42	42	41	40	40	40	40	40	39	38	34
.20	0	0	0	0	1	4	11	21	33	45	54	58	60	58	55	53	51	48	46	44	43	42	42	41	40	40	40	40	40	39	38	34
.30	0	0	0	0	1	3	8	16	27	39	49	55	59	59	55	53	51	49	46	44	43	42	42	41	40	40	40	40	40	39	39	34
.40	0	0	0	0	1	2	6	13	23	34	44	51	56	58	56	54	52	49	47	44	43	42	42	41	40	40	40	40	40	39	39	34
.50	0	0	0	0	0	0	2	5	10	18	29	39	47	56	57	55	53	51	49	45	44	43	42	41	41	40	40	40	40	39	39	35
.75	0	0	0	0	0	0	1	2	5	10	17	25	34	48	54	56	55	53	50	47	45	43	42	42	41	40	40	40	40	39	39	35
1.0	0	0	0	0	0	0	0	0	0	1	3	6	11	26	41	51	55	55	54	50	47	44	43	42	41	41	40	40	40	40	39	35
1.5	0	0	0	0	0	0	0	0	0	0	0	0	1	4	11	23	36	47	52	54	52	48	44	43	42	41	40	40	40	40	39	36
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	7	16	27	38	49	53	51	47	44	42	42	41	40	40	40	39	36
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	7	15	30	43	52	50	46	44	42	42	41	40	40	39	37
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	10	23	40	51	50	46	44	42	42	40	40	39	37

IA/P = 0.50 * * * TC = 0.5 HR * * * IA/P = 0.50

TRVL TIME (hr)	7.0	7.3	7.6	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.3	10.6	11.0	11.5	12.0	12.5	13.0	13.5	14.0	15.0	16.0	18.0	22.0
0.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	7	11	14	19	22	26	30	31	34	38	40	41	45	46	46	46
.10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	8	12	17	21	25	29	31	33	37	39	41	44	46	46	46
.20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	9	14	19	23	28	31	32	35	39	41	43	46	46	46
.30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	6	12	17	22	27	31	32	34	38	40	43	46	46	46
.40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	11	16	21	26	30	32	34	38	40	43	46	46	46
.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	8	13	19	25	29	31	33	37	39	42	46	46	46
.75	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	6	11	17	23	28	31	32	35	39	42	45	46	46
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	11	18	24	29	31	33	36	41	44	46	46
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	11	19	24	29	31	33	39	42	46	46
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	12	19	25	29	31	37	41	46	46
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	13	20	25	29	34	39	45	46
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	7	14	20	26	32	37	44	46

RAINFALL TYPE = IA * * * TC = 0.5 HR * * * SHEET 5 OF 10

Exhibit 5-IA: Tabular hydrograph unit discharges (csm/in) for type IA rainfall distribution—continued

TC = 0.75 HR

TRVL TIME (hr)	7.0	7.3	7.6	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.3	10.6	11.0	11.5	12.0	12.5	13.0	13.5	14.0	15.0	16.0	18.0	22.0
IA/P = 0.10																																
0.0	22	25	29	38	45	56	71	87	103	114	117	117	111	95	81	72	65	58	53	48	45	41	39	36	33	33	33	32	30	29	27	22
.10	21	25	28	36	42	51	64	79	95	108	114	116	113	99	85	74	66	60	54	48	45	42	39	37	34	33	33	32	30	30	27	22
.20	19	23	26	32	35	39	47	58	72	87	101	110	114	109	95	82	72	65	58	51	47	43	40	38	34	33	33	32	31	30	27	23
.30	19	22	26	31	33	37	44	54	66	80	94	104	112	110	98	85	75	67	60	52	47	43	40	38	34	33	33	32	31	30	28	23
.40	17	21	24	28	30	32	36	41	49	61	74	87	99	111	106	94	82	73	65	56	49	45	41	39	35	33	33	33	31	30	28	23
.50	16	20	23	27	29	31	34	39	46	56	68	81	93	109	107	97	85	75	67	57	50	45	41	39	36	33	33	33	31	30	28	23
.75	15	18	22	25	27	28	31	34	38	45	54	64	75	95	104	102	93	83	74	62	54	47	42	40	37	34	33	33	31	30	28	23
1.0	12	15	18	22	23	24	25	27	29	31	34	39	46	65	85	100	103	98	88	74	62	52	45	41	39	35	33	33	32	30	28	23
1.5	7	10	12	15	17	18	19	20	21	22	24	25	26	31	39	53	71	87	99	99	85	68	53	46	41	39	36	34	33	31	29	24
2.0	4	6	8	11	12	13	14	15	16	17	18	19	21	23	26	31	39	51	67	88	95	86	67	53	45	41	38	36	33	32	29	25
2.5	2	3	5	7	8	9	10	11	11	12	13	14	16	18	20	23	26	30	38	56	77	93	84	66	52	45	41	38	34	33	30	25
3.0	0	1	2	4	4	5	6	6	7	8	8	9	10	12	14	16	18	21	24	30	42	67	90	84	68	54	46	42	36	33	30	26
IA/P = 0.30																																
0.0	0	0	0	0	2	4	10	19	29	39	48	53	56	56	54	53	51	49	46	44	44	42	42	40	40	40	40	40	40	39	39	34
.10	0	0	0	0	1	3	8	15	24	34	43	50	54	56	54	53	52	49	47	45	44	42	42	41	40	40	40	40	40	39	39	34
.20	0	0	0	0	0	1	2	6	12	20	30	39	46	54	55	54	53	51	49	46	44	43	42	41	40	40	40	40	40	39	39	35
.30	0	0	0	0	0	1	2	5	10	17	25	34	42	52	55	54	53	51	49	46	44	43	42	42	40	40	40	40	40	39	39	35
.40	0	0	0	0	0	0	1	4	8	14	21	30	38	50	55	55	53	52	50	47	45	43	42	42	40	40	40	40	40	39	39	35
.50	0	0	0	0	0	0	1	3	6	11	18	26	34	47	53	54	54	52	50	47	45	43	42	42	40	40	40	40	40	39	39	35
.75	0	0	0	0	0	0	0	1	3	6	10	16	23	37	47	52	53	53	52	49	46	44	42	42	41	40	40	40	40	40	39	35
1.0	0	0	0	0	0	0	0	0	0	1	2	4	7	17	30	42	50	53	53	51	49	45	43	42	41	40	40	40	40	40	39	36
1.5	0	0	0	0	0	0	0	0	0	0	0	0	1	4	11	21	33	43	49	52	51	48	45	43	42	41	40	40	40	40	39	36
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	11	20	30	43	50	51	48	45	43	42	41	40	40	40	39	37
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	10	23	36	48	51	47	44	43	42	41	40	40	39	37
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	7	17	34	48	50	47	44	43	42	40	40	39	38
IA/P = 0.50																																
0.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	6	10	15	19	23	28	31	32	36	39	41	44	44	44	44
.10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	7	12	17	22	27	30	32	34	38	40	43	44	44	44
.20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	5	10	15	20	25	30	31	33	37	39	43	44	44	44
.30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	7	12	18	24	29	31	33	36	39	42	44	44	44
.40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	10	16	22	27	30	32	35	38	42	44	44	44
.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	7	14	21	26	30	31	34	38	41	44	44	44
.75	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	11	19	24	29	31	33	36	41	44	44	44
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	8	16	22	27	30	32	35	40	43	44	44
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	9	16	23	27	30	32	38	42	44	44
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	10	17	23	28	30	36	40	44	44
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	11	18	23	28	33	39	44	44
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	10	17	23	30	36	43	44

RAINFALL TYPE = IA * * * TC = 0.75 HR * * * SHEET 6 OF 10

Exhibit 5-IA: Tabular hydrograph unit discharges (csm/in) for type IA rainfall distribution—continued

HYDROGRAPH TIME (HOURS)

IA/P = 0.10   * * * TC = 1.0 HR * * *

TRVL TIME (hr)	7.0	7.3	7.6	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.3	10.6	11.0	11.5	12.0	12.5	13.0	13.5	14.0	15.0	16.0	18.0	22.0
0.0	19	23	26	33	37	43	52	63	76	88	97	104	108	102	91	81	72	65	59	52	48	43	40	38	34	33	33	32	31	30	28	23
.10	19	22	26	32	35	41	48	58	70	82	92	100	105	103	94	83	75	67	61	53	48	44	40	38	35	33	33	32	31	30	28	23
.20	17	21	24	28	30	34	38	45	54	65	76	87	96	104	100	91	81	73	66	57	51	45	41	39	36	33	33	33	31	30	28	23
.30	16	20	23	27	29	32	36	42	50	60	71	81	91	103	101	93	84	75	67	58	52	46	42	39	36	34	33	33	31	30	28	23
.40	15	18	22	25	27	29	31	35	40	46	55	65	76	94	102	99	91	81	73	63	55	48	43	40	37	34	33	33	31	30	28	23
.50	14	18	21	25	26	28	30	33	37	43	51	61	71	90	101	101	93	84	75	64	56	49	43	40	37	34	33	33	31	30	28	23
.75	13	16	19	23	24	25	27	29	32	37	42	49	58	76	91	98	96	89	81	70	60	51	45	41	38	35	34	33	32	30	28	23
1.0	10	13	16	19	20	22	23	24	26	27	30	33	37	50	67	83	94	97	93	81	70	58	48	43	40	37	34	33	32	31	29	24
1.5	6	8	10	13	14	15	16	18	19	20	21	22	24	27	33	42	56	71	84	93	89	75	59	49	44	40	37	35	33	31	29	24
2.0	3	5	7	9	10	11	12	13	14	15	16	17	18	21	23	27	32	41	53	74	88	91	74	59	49	43	40	37	34	32	30	25
2.5	1	2	4	6	7	7	8	9	10	11	11	12	13	16	18	20	23	26	32	45	64	84	89	72	58	48	43	40	34	33	30	26
3.0	0	1	2	3	3	4	4	5	6	6	7	8	9	10	12	14	16	18	21	26	35	55	81	87	73	59	50	44	37	34	31	26

IA/P = 0.30   * * * TC = 1.0 HR * * *

TRVL TIME (hr)	7.0	7.3	7.6	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.3	10.6	11.0	11.5	12.0	12.5	13.0	13.5	14.0	15.0	16.0	18.0	22.0
0.0	0	0	0	0	1	2	5	10	16	23	31	38	44	51	53	52	52	50	48	46	44	43	42	41	40	40	40	40	40	40	39	35
.10	0	0	0	0	0	1	2	4	8	13	20	27	35	46	51	53	52	51	50	47	45	44	42	42	40	40	40	40	40	40	39	35
.20	0	0	0	0	0	0	1	3	6	11	17	24	31	43	50	52	52	51	50	47	45	44	42	42	40	40	40	40	40	40	39	35
.30	0	0	0	0	0	0	1	2	5	9	14	21	28	40	48	52	52	52	50	48	46	44	43	42	40	40	40	40	40	40	39	35
.40	0	0	0	0	0	0	0	1	2	4	7	12	18	31	42	49	52	52	51	49	47	45	43	42	41	40	40	40	40	40	39	35
.50	0	0	0	0	0	0	0	1	1	3	6	10	15	28	39	47	51	52	51	49	47	45	43	42	41	40	40	40	40	40	39	35
.75	0	0	0	0	0	0	0	0	1	1	3	6	9	19	31	41	47	50	51	50	48	46	43	42	41	40	40	40	40	40	39	36
1.0	0	0	0	0	0	0	0	0	0	0	0	1	2	6	14	25	36	44	49	51	50	47	45	43	42	41	40	40	40	40	39	36
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	6	13	23	33	44	49	50	47	44	43	42	41	40	40	40	39	37
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	6	13	25	38	48	50	47	44	43	42	41	40	40	39	37
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	9	19	36	48	49	47	44	43	42	40	40	39	38
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	6	18	37	47	49	46	44	43	41	40	40	38

IA/P = 0.50   * * * TC = 1.0 HR * * *

TRVL TIME (hr)	7.0	7.3	7.6	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.3	10.6	11.0	11.5	12.0	12.5	13.0	13.5	14.0	15.0	16.0	18.0	22.0
0.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	4	7	11	16	21	26	30	32	34	38	40	43	43	43	43
.10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	5	9	14	19	25	29	31	33	37	39	42	43	43	43
.20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	7	11	17	23	28	30	32	36	39	42	43	43	43
.30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	9	15	21	27	30	32	35	38	41	43	43	43
.40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	7	13	20	25	29	31	34	37	41	43	43	43
.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	11	18	24	28	31	33	36	41	43	43	43
.75	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	8	16	22	27	30	32	35	40	43	43	43
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	6	13	20	25	29	31	34	39	42	43	43
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	7	14	20	25	29	31	37	41	43	43
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	7	14	20	26	29	34	39	43	43
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	8	15	21	26	32	37	43	43
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	8	14	20	29	34	42	43

RAINFALL TYPE = IA   * * * TC = 1.0 HR * * *   SHEET 7 OF 10

Exhibit 5-IA: Tabular hydrograph unit discharges (csm/in) for type IA rainfall distribution—continued

TRVL TIME (hr)	7.0	7.3	7.6	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.3	10.6	11.0	11.5	12.0	12.5	13.0	13.5	14.0	15.0	16.0	18.0	22.0
IA/P = 0.10														* * * TC = 1.25 HR * * *														IA/P = 0.10				
0.0	17	21	24	29	33	37	43	52	61	71	81	89	95	100	94	86	78	71	64	57	51	46	42	39	35	34	33	33	31	30	28	23
.10	16	19	23	27	29	31	35	41	48	57	66	76	85	96	99	92	84	76	69	60	54	47	43	40	36	34	34	33	31	30	28	23
.20	15	19	22	26	28	30	33	38	45	53	62	71	80	93	97	93	86	78	71	62	55	48	43	40	37	34	34	33	31	30	28	23
.30	14	17	20	24	25	27	29	32	36	42	49	58	67	84	95	96	91	84	76	66	58	50	44	41	38	35	34	33	31	30	28	23
.40	13	16	20	23	25	26	28	31	34	39	46	54	62	80	92	96	93	86	78	68	59	51	45	41	38	35	34	33	31	30	28	23
.50	12	15	18	21	23	24	25	27	29	33	37	43	50	67	83	93	95	91	84	73	63	54	46	42	39	36	34	33	32	30	28	23
.75	10	13	16	20	21	22	23	25	27	29	32	36	42	55	71	84	93	93	88	78	68	57	49	43	40	37	34	34	32	31	28	24
1.0	8	10	13	16	17	19	20	21	22	23	25	27	29	37	49	63	78	88	92	88	78	65	53	46	42	39	36	34	33	31	29	24
1.5	5	7	9	12	13	14	15	16	17	18	19	20	22	25	29	36	47	60	73	87	89	79	64	53	46	42	38	36	33	32	29	25
2.0	2	4	6	8	9	10	10	11	12	13	14	15	16	19	21	24	29	36	45	64	80	87	77	63	52	46	41	38	34	32	30	25
2.5	1	2	3	4	5	6	6	7	8	8	9	10	11	13	15	17	19	22	26	35	49	72	85	78	65	53	46	42	36	33	30	26
3.0	0	1	1	2	3	3	4	4	5	5	6	7	7	9	11	12	14	17	19	23	31	47	73	84	76	63	53	46	38	34	31	26
IA/P = 0.30														* * * TC = 1.25 HR * * *														IA/P = 0.30				
0.0	0	0	0	0	0	1	3	6	10	15	22	28	34	43	48	51	51	51	49	47	45	44	43	42	40	40	40	40	40	40	39	35
.10	0	0	0	0	0	0	1	2	5	8	13	19	25	36	45	49	51	50	50	48	46	44	43	42	40	40	40	40	40	40	39	35
.20	0	0	0	0	0	0	1	2	4	7	11	16	22	34	43	48	50	50	50	48	46	44	43	42	41	40	40	40	40	40	39	35
.30	0	0	0	0	0	0	1	1	3	5	9	14	19	31	40	46	49	50	50	48	46	45	43	42	41	40	40	40	40	40	39	35
.40	0	0	0	0	0	0	0	1	2	4	8	12	17	28	38	45	49	50	50	49	47	45	43	42	41	40	40	40	40	40	39	35
.50	0	0	0	0	0	0	0	1	2	4	6	10	14	25	36	43	48	50	50	49	47	45	43	42	41	40	40	40	40	40	39	35
.75	0	0	0	0	0	0	0	0	0	1	2	3	6	13	23	33	41	46	49	50	48	46	44	43	42	40	40	40	40	40	39	36
1.0	0	0	0	0	0	0	0	0	0	0	0	1	2	6	14	23	32	40	46	49	49	47	45	43	42	41	40	40	40	40	39	36
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	6	13	21	30	41	47	49	47	45	43	42	41	40	40	40	39	37
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	6	12	23	35	45	48	47	45	43	42	41	40	40	40	37
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	8	18	33	45	48	46	44	43	42	40	40	40	38
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	6	17	34	45	48	46	44	43	41	40	40	38
IA/P = 0.50														* * * TC = 1.25 HR * * *														IA/P = 0.50				
0.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	5	9	14	19	24	28	31	33	37	39	42	42	42	42
.10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	7	11	17	23	27	30	32	36	38	42	42	42	42
.20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	5	9	15	21	26	29	32	35	38	41	42	42	42
.30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	7	13	19	25	29	31	34	37	41	42	42	42
.40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	11	17	23	28	30	33	36	40	42	42	42
.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	8	15	22	26	30	32	35	40	42	42	42
.75	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	7	13	20	25	29	31	34	39	42	42	42
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	11	17	23	27	30	33	38	42	42	42
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	11	18	23	28	30	36	40	42	42
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	12	18	24	28	33	39	42	42
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	6	13	19	24	31	37	42	42
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	6	12	18	28	33	42	42

RAINFALL TYPE = IA   * * * TC = 1.25 HR * * *   SHEET 8 OF 10

Exhibit 5-IA: Tabular hydrograph unit discharges (csm/in) for type IA rainfall distribution—continued

IA/P = 0.10 * * * TC = 1.5 HR * * * IA/P = 0.10

TRVL TIME (hr)	7.0	7.3	7.6	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.3	10.6	11.0	11.5	12.0	12.5	13.0	13.5	14.0	15.0	16.0	18.0	22.0
0.0	15	19	22	27	29	32	37	42	49	57	65	73	80	89	94	89	83	76	70	62	55	49	44	40	37	35	34	33	31	30	28	23
.10	14	17	20	24	26	28	31	35	40	46	53	61	69	82	90	92	87	81	74	65	58	51	45	41	38	35	34	33	31	30	28	23
.20	13	16	20	23	25	27	29	33	37	43	50	57	65	79	88	91	88	82	76	67	59	52	46	42	38	35	34	33	32	30	28	23
.30	12	15	18	22	23	24	26	28	31	35	41	47	54	69	81	89	90	86	81	71	63	54	47	43	39	36	34	34	32	30	28	24
.40	11	14	18	21	22	24	25	27	30	34	38	44	51	65	78	87	90	87	82	73	64	55	48	43	40	36	34	34	32	30	28	24
.50	10	13	16	19	20	22	23	24	26	29	32	36	41	54	69	81	88	89	86	77	68	58	50	44	41	37	35	34	32	31	29	24
.75	9	11	14	18	19	20	21	22	24	26	28	31	35	46	58	71	82	88	88	81	73	62	52	46	42	38	35	34	32	31	29	24
1.0	7	9	11	14	15	16	18	19	20	21	22	24	26	32	40	52	65	76	84	87	81	70	58	49	44	40	37	35	33	31	29	24
1.5	4	6	8	10	11	12	13	14	15	16	17	18	19	22	26	31	39	50	62	77	85	81	69	57	49	44	40	37	34	32	30	25
2.0	2	3	5	7	7	8	9	10	11	12	12	13	14	17	19	22	25	31	38	53	69	83	80	68	56	48	43	40	35	33	30	25
2.5	1	1	2	3	4	5	5	6	6	7	8	9	9	11	13	15	17	20	23	30	41	61	82	82	69	58	49	44	37	34	31	26
3.0	0	0	1	2	2	2	3	3	4	4	5	5	6	8	9	11	13	15	17	21	27	40	63	80	78	68	57	49	40	35	31	27

IA/P = 0.30 * * * TC = 1.5 HR * * * IA/P = 0.30

TRVL TIME (hr)	7.0	7.3	7.6	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.3	10.6	11.0	11.5	12.0	12.5	13.0	13.5	14.0	15.0	16.0	18.0	22.0
0.0	0	0	0	0	0	1	2	4	6	10	14	19	24	34	41	46	48	49	48	47	46	44	43	42	41	40	40	40	40	40	39	35
.10	0	0	0	0	0	0	1	1	3	5	8	12	17	27	36	42	46	48	49	48	46	45	43	42	41	40	40	40	40	40	39	36
.20	0	0	0	0	0	0	0	1	2	4	7	10	15	24	33	41	45	48	49	48	47	45	43	43	41	40	40	40	40	40	39	36
.30	0	0	0	0	0	0	0	1	2	3	6	9	13	22	31	39	44	47	48	48	47	45	44	43	41	40	40	40	40	40	39	36
.40	0	0	0	0	0	0	0	0	1	1	3	5	7	15	24	33	40	45	47	48	47	46	44	43	42	40	40	40	40	40	39	36
.50	0	0	0	0	0	0	0	0	0	1	2	4	6	13	22	31	38	44	47	48	48	46	44	43	42	40	40	40	40	40	39	36
.75	0	0	0	0	0	0	0	0	0	1	1	2	4	9	16	24	33	39	44	47	48	47	45	43	42	41	40	40	40	40	39	36
1.0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	6	12	20	29	36	44	47	48	46	44	43	42	40	40	40	40	40	36
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	6	12	19	30	39	46	47	46	44	43	42	40	40	40	40	37
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	4	8	17	28	40	46	47	45	44	42	41	40	40	40	37
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	6	13	26	40	47	47	45	43	42	40	40	40	38
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	12	28	41	46	46	45	43	41	40	40	38

IA/P = 0.50 * * * TC = 1.5 HR * * * IA/P = 0.50

TRVL TIME (hr)	7.0	7.3	7.6	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.3	10.6	11.0	11.5	12.0	12.5	13.0	13.5	14.0	15.0	16.0	18.0	22.0
0.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	3	7	11	16	22	27	30	32	35	38	42	42	42	42
.10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	4	7	13	19	24	28	31	34	37	41	42	42	42
.20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	7	12	19	24	28	31	33	36	41	42	42	42
.30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	6	12	18	23	28	30	33	36	40	42	42	42
.40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	6	11	17	23	27	30	33	36	40	42	42	42
.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	5	10	17	22	27	30	32	36	40	41	41	41
.75	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	5	10	16	22	26	29	32	35	40	41	41	41
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	11	17	23	27	30	33	38	41	41	41
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	12	18	23	27	30	36	40	41	41
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	6	12	18	24	28	33	39	41	41
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	7	13	19	24	31	36	41	41
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	7	13	19	28	34	41	41

RAINFALL TYPE = IA * * * TC = 1.5 HR * * * SHEET 9 OF 10

Exhibit 5-IA: Tabular hydrograph unit discharges (csm/in) for type IA rainfall distribution—continued

HYDROGRAPH TIME (HOURS)

IA/P = 0.10 * * * TC = 2.0 HR * * * IA/P = 0.10

TRVL TIME (hr)	7.0	7.3	7.6	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.3	10.6	11.0	11.5	12.0	12.5	13.0	13.5	14.0	15.0	16.0	18.0	22.0
0.0	12	15	18	22	24	26	28	32	36	41	46	52	58	68	76	80	84	79	76	68	62	55	48	44	40	37	35	34	32	31	28	24
.10	11	13	16	20	21	23	25	27	30	34	38	43	49	60	70	77	83	83	78	72	65	57	50	45	41	38	36	34	32	31	29	24
.20	9	12	15	18	19	21	22	24	26	29	32	36	41	52	63	72	78	82	81	75	68	60	52	46	42	39	36	35	32	31	29	24
.30	9	12	14	18	19	20	21	23	25	28	31	34	39	49	60	70	77	81	81	76	70	61	53	47	43	39	36	35	33	31	29	24
.40	9	11	14	17	18	19	21	22	24	26	29	33	37	47	57	67	75	80	81	77	71	62	53	47	43	39	37	35	33	31	29	24
.50	7	10	12	15	17	18	19	20	21	23	25	28	31	39	49	60	69	76	81	79	74	65	56	49	44	40	37	35	33	31	29	24
.75	6	9	11	14	15	16	17	18	20	21	23	25	27	34	42	52	62	70	76	79	76	68	59	51	46	41	38	36	33	31	29	24
1.0	5	6	9	11	12	13	14	15	16	17	18	20	21	25	30	38	47	57	66	76	79	75	64	55	49	44	40	37	34	32	29	25
1.5	3	4	6	8	8	9	10	11	12	13	14	15	16	18	21	24	30	37	45	59	71	78	73	63	55	48	43	40	35	33	30	25
2.0	1	2	3	4	5	5	6	7	7	8	9	10	10	12	14	16	19	22	26	36	48	65	77	73	64	56	49	44	37	34	31	26
2.5	0	1	1	2	3	3	4	4	5	5	6	6	7	8	10	12	14	16	18	23	31	46	66	76	72	63	55	48	40	35	31	27
3.0	0	0	1	1	1	2	2	2	3	3	3	4	4	5	7	8	10	11	13	17	21	30	49	67	75	71	63	54	43	37	32	27

IA/P = 0.30 * * * TC = 2.0 HR * * * IA/P = 0.30

TRVL TIME (hr)	7.0	7.3	7.6	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.3	10.6	11.0	11.5	12.0	12.5	13.0	13.5	14.0	15.0	16.0	18.0	22.0
0.0	0	0	0	0	0	0	1	2	3	5	7	10	13	20	27	34	39	42	44	47	45	45	44	43	42	40	40	40	40	40	39	36
.10	0	0	0	0	0	0	0	1	1	2	4	6	8	15	22	29	35	40	43	45	46	45	44	43	42	41	40	40	40	40	40	36
.20	0	0	0	0	0	0	0	0	1	1	2	3	5	10	17	24	31	36	41	44	46	45	44	43	42	41	40	40	40	40	40	36
.30	0	0	0	0	0	0	0	0	0	1	2	3	4	9	15	22	29	35	39	44	46	45	44	43	42	41	40	40	40	40	40	36
.40	0	0	0	0	0	0	0	0	0	1	1	2	4	8	14	20	27	33	38	43	46	46	44	43	42	41	40	40	40	40	40	36
.50	0	0	0	0	0	0	0	0	0	0	1	1	2	5	9	15	22	29	35	41	44	45	45	44	43	42	40	40	40	40	40	36
.75	0	0	0	0	0	0	0	0	0	0	0	1	1	3	6	11	17	24	30	38	42	45	45	44	43	42	41	40	40	40	40	37
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	5	9	14	21	30	38	43	45	44	44	43	41	40	40	40	40	37
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	4	8	16	25	36	43	45	44	43	42	41	40	40	40	38
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	6	12	24	36	43	45	44	43	42	40	40	40	38
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	12	25	37	43	45	44	43	41	40	40	38
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	13	26	37	43	44	44	42	40	40	39

IA/P = 0.50 * * * TC = 2.0 HR * * * IA/P = 0.50

TRVL TIME (hr)	7.0	7.3	7.6	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.3	10.6	11.0	11.5	12.0	12.5	13.0	13.5	14.0	15.0	16.0	18.0	22.0
0.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	4	7	12	18	23	27	30	33	36	40	40	40	40
.10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	5	10	16	21	26	29	32	35	40	40	40	40
.20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	4	8	14	20	25	28	31	34	39	40	40	40
.30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	6	12	18	23	27	30	33	38	40	40	40
.40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	5	11	16	22	26	29	32	38	40	40	40
.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	9	15	20	25	28	31	37	40	40	40
.75	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	7	13	19	24	27	30	36	40	40	40
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	8	14	20	24	28	34	39	40	40
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	10	16	21	25	32	37	40	40
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	11	16	21	29	35	40	40
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	6	11	17	26	32	40	40
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	6	11	21	29	39	40

RAINFALL TYPE = IA * * * TC = 2.0 HR * * * SHEET 10 OF 10

Exhibit 5-II: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution

IA/P = 0.10 * * * TC = 0.1 HR * * * IA/P = 0.10

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
0.0	24	34	53	334	647	1010	623	217	147	123	104	86	76	66	57	51	46	42	38	34	32	29	26	23	21	20	19	18	15	13	12	0
.10	21	29	43	134	267	520	847	701	378	224	157	122	98	75	64	56	50	45	41	36	33	30	27	24	21	20	19	18	16	13	12	0
.20	18	25	35	61	110	215	418	704	702	486	312	209	151	94	73	62	54	49	44	38	34	31	28	25	22	21	19	18	16	14	12	0
.30	17	23	33	56	92	174	337	582	662	545	389	269	190	109	79	65	56	50	45	39	35	32	29	25	22	21	20	18	16	14	12	0
.40	15	20	28	41	51	78	142	272	478	601	563	447	328	172	104	76	63	55	49	42	37	33	29	26	23	21	20	19	17	14	12	0
.50	14	19	26	39	47	68	117	220	392	531	553	482	380	209	121	84	67	57	51	43	38	33	30	27	23	21	20	19	17	14	12	0
.75	12	15	21	29	33	38	49	73	126	224	343	432	464	385	252	156	103	76	62	50	43	36	31	28	25	22	21	19	17	15	12	0
1.0	9	12	15	21	23	26	29	33	40	55	86	148	238	406	434	317	205	130	89	62	50	41	34	30	27	24	22	20	18	16	12	0
1.5	7	8	10	14	15	16	18	20	22	25	29	34	45	101	220	339	373	320	234	131	80	53	40	34	30	27	24	21	19	17	12	2
2.0	4	6	7	9	9	10	11	12	13	15	16	18	20	25	37	72	150	252	336	312	216	109	58	42	34	30	27	24	20	18	13	8
2.5	3	4	5	6	7	7	8	8	9	10	11	12	13	16	19	25	39	75	142	262	308	229	108	58	41	34	30	27	22	19	14	11
3.0	1	2	3	4	4	5	5	6	6	7	7	8	8	10	12	14	17	22	31	76	169	288	236	122	64	43	35	30	24	20	16	11

IA/P = 0.30 * * * TC = 0.1 HR * * * IA/P = 0.30

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
0.0	0	0	0	154	568	936	524	217	172	149	126	107	97	86	76	69	63	58	53	48	46	42	38	34	31	30	28	27	24	20	19	0
.10	0	0	0	19	109	415	762	603	346	230	176	143	119	96	84	74	68	62	57	50	47	44	40	35	32	30	29	27	24	21	19	0
.20	0	0	0	0	13	77	302	609	605	432	297	217	167	115	94	81	73	66	60	53	48	45	41	37	33	31	29	28	25	21	19	0
.30	0	0	0	0	9	54	219	479	563	476	357	263	199	129	99	85	75	68	62	54	49	45	41	37	33	31	29	28	25	21	19	0
.40	0	0	0	0	0	6	38	159	372	500	484	399	309	183	123	96	82	73	66	58	51	46	42	38	34	31	30	28	25	22	19	0
.50	0	0	0	0	0	4	27	115	287	429	465	421	346	213	138	103	86	76	68	59	52	47	43	39	34	32	30	29	25	22	19	0
.75	0	0	0	0	0	0	1	10	46	132	246	338	381	341	243	165	119	94	80	67	58	50	45	41	37	33	31	29	26	23	19	0
1.0	0	0	0	0	0	0	0	1	4	22	69	149	241	357	331	246	170	122	96	76	64	54	47	42	38	34	32	30	27	24	19	0
1.5	0	0	0	0	0	0	0	0	0	0	0	1	4	41	142	258	310	285	224	142	97	71	55	47	43	39	35	32	29	25	20	4
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	10	49	130	221	279	255	182	108	70	55	47	42	38	34	30	27	20	11
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	14	52	119	224	256	193	107	70	55	47	42	38	32	28	22	17
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	52	141	240	199	117	74	56	48	43	35	30	24	18

IA/P = 0.50 * * * TC = 0.1 HR * * * IA/P = 0.50

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
0.0	0	0	0	0	70	539	377	196	171	154	134	117	108	99	89	83	77	72	67	61	59	56	51	46	43	42	40	38	34	30	28	0
.10	0	0	0	0	47	375	376	256	199	169	146	126	114	102	92	85	79	73	68	62	59	56	52	47	43	42	40	38	34	30	28	0
.20	0	0	0	0	0	31	260	338	283	227	189	160	138	112	99	90	83	77	72	64	60	57	53	48	44	42	41	39	35	30	28	0
.30	0	0	0	0	0	0	21	180	285	284	246	208	176	131	110	97	88	82	76	68	62	59	54	50	45	43	41	39	36	31	28	0
.40	0	0	0	0	0	0	14	125	232	266	253	223	192	142	115	100	91	83	77	69	63	59	55	50	45	43	41	40	36	31	28	0
.50	0	0	0	0	0	0	9	86	183	239	248	231	205	154	122	104	93	85	79	71	64	59	55	51	46	43	41	40	36	32	28	0
.75	0	0	0	0	0	0	3	31	87	147	190	211	213	184	147	121	103	92	84	75	67	61	57	52	47	44	42	40	37	32	28	0
1.0	0	0	0	0	0	0	0	0	1	13	45	92	141	205	197	165	134	112	98	84	75	65	59	55	50	46	43	41	38	34	28	0
1.5	0	0	0	0	0	0	0	0	0	0	0	2	9	51	118	170	183	167	143	111	92	77	65	59	54	50	45	43	39	35	28	2
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	15	51	103	148	168	156	127	96	76	65	58	54	49	45	41	37	29	12
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	31	69	131	159	140	101	78	66	59	54	50	43	39	31	24
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	11	46	101	151	134	99	77	65	59	54	45	41	33	26

RAINFALL TYPE = II * * * TC = 0.1 HR * * * SHEET 1 OF 10

Exhibit 5-II: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution—continued

HYDROGRAPH TIME(HOURS)

IA/P = 0.10 * * * TC = 0.2 HR * * * IA/P = 0.10

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
0.0	23	31	47	209	403	739	800	481	250	166	128	102	86	70	61	54	49	44	40	35	33	30	27	24	21	20	19	18	16	13	12	0
.10	19	26	39	86	168	325	601	733	565	355	229	161	122	83	69	59	53	47	43	37	34	31	28	25	22	21	19	18	16	14	12	0
.20	17	23	32	49	74	136	262	488	652	594	435	298	207	115	81	67	58	51	46	40	35	32	29	26	23	21	20	19	16	14	12	0
.30	16	22	30	46	64	112	212	396	566	585	485	360	258	139	90	71	60	53	48	41	36	32	29	26	23	21	20	19	16	14	12	0
.40	14	19	25	37	43	57	94	173	322	485	551	507	409	227	129	87	68	58	52	44	38	33	30	27	24	21	20	19	17	14	12	0
.50	13	18	24	35	40	52	80	142	262	410	504	506	441	269	153	98	73	61	53	45	39	34	30	27	24	22	20	19	17	15	12	0
.75	10	13	17	23	26	30	34	40	55	86	150	247	349	438	360	240	151	101	75	57	47	39	33	29	26	23	21	20	18	15	12	0
1.0	9	11	14	19	21	24	26	30	35	44	62	101	167	337	413	353	245	157	104	68	53	42	35	31	28	24	22	20	18	16	12	0
1.5	6	8	10	13	14	15	17	19	21	23	26	30	37	73	166	288	356	337	264	154	91	57	42	35	30	27	24	22	19	17	13	3
2.0	4	5	7	8	9	10	10	11	12	14	15	16	18	23	31	55	114	206	291	324	239	125	63	44	35	31	28	24	20	18	14	9
2.5	3	4	5	6	6	7	7	8	9	9	10	11	12	15	18	22	32	58	111	227	298	246	122	63	43	35	31	27	22	19	15	11
3.0	1	2	3	4	4	4	5	5	6	6	7	7	8	9	11	13	16	19	27	59	138	280	248	137	70	46	36	31	25	21	16	11

IA/P = 0.30 * * * TC = 0.2 HR * * * IA/P = 0.30

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
0.0	0	0	0	39	180	545	697	497	276	198	158	130	110	93	81	73	67	61	56	49	46	43	39	35	32	30	29	27	24	21	19	0
.10	0	0	0	2	27	129	407	600	532	361	252	190	150	108	90	79	71	65	59	52	48	44	41	36	32	31	29	28	25	21	19	0
.20	0	0	0	2	19	92	302	501	521	415	306	228	176	119	95	82	73	67	61	53	48	45	41	37	33	31	29	28	25	21	19	0
.30	0	0	0	0	1	13	66	223	408	484	438	350	269	163	114	93	80	72	65	57	51	46	42	38	34	31	30	28	25	22	19	0
.40	0	0	0	0	1	9	47	164	327	431	436	379	306	189	127	98	83	74	67	58	52	47	43	38	34	31	30	28	25	22	19	0
.50	0	0	0	0	0	0	6	33	120	258	374	415	391	271	173	121	95	81	72	62	55	48	44	40	35	32	30	29	26	22	19	0
.75	0	0	0	0	0	0	2	13	50	126	221	302	348	323	240	167	121	96	81	68	59	50	45	41	37	33	31	29	26	23	19	0
1.0	0	0	0	0	0	0	0	0	1	6	24	69	139	285	331	280	204	145	109	82	68	56	48	43	39	35	32	30	27	24	19	0
1.5	0	0	0	0	0	0	0	0	0	0	0	0	1	16	79	186	271	288	247	165	110	76	58	49	44	40	35	32	29	26	20	5
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	24	80	163	235	262	202	123	76	58	49	43	39	35	30	27	21	13
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	28	77	179	242	207	120	75	57	48	43	39	32	29	22	17
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	30	101	207	227	130	80	59	49	44	35	30	24	18

IA/P = 0.50 * * * TC = 0.2 HR * * * IA/P = 0.50

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
0.0	0	0	0	0	7	98	371	322	221	182	158	137	120	104	94	86	80	74	69	62	60	57	52	47	44	42	40	39	35	30	28	0
.10	0	0	0	0	4	67	270	305	249	204	174	149	130	108	97	88	82	76	71	64	60	57	53	48	44	42	41	39	35	30	28	0
.20	0	0	0	0	0	3	45	195	268	255	221	189	163	125	106	95	87	80	75	67	62	58	54	49	45	43	41	39	35	31	28	0
.30	0	0	0	0	0	2	31	140	226	245	229	203	176	134	111	98	89	82	76	68	62	59	55	50	45	43	41	39	36	31	28	0
.40	0	0	0	0	0	1	21	101	184	225	228	211	188	144	117	101	91	84	78	69	63	59	55	50	45	43	41	40	36	31	28	0
.50	0	0	0	0	0	0	1	14	72	146	199	218	213	175	137	113	99	89	82	73	66	60	56	52	47	43	42	40	36	32	28	0
.75	0	0	0	0	0	0	0	5	28	71	121	162	186	193	161	133	112	98	88	78	70	62	57	53	48	44	42	41	37	33	28	0
1.0	0	0	0	0	0	0	0	0	0	2	13	38	77	154	186	174	147	122	105	89	78	68	60	56	51	46	43	42	38	34	28	0
1.5	0	0	0	0	0	0	0	0	0	0	0	0	2	22	71	129	163	168	150	120	98	80	67	60	55	51	46	43	40	36	28	4
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	5	25	65	112	146	157	134	103	79	67	60	55	50	46	41	38	29	14
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	26	60	117	148	136	101	79	66	59	54	50	43	39	31	24
3,0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	9	40	90	142	130	99	78	66	59	54	45	41	33	26

RAINFALL TYPE = II * * * TC = 0.2 HR * * * SHEET 2 OF 10

Exhibit 5-II: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution—continued

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
IA/P = 0.10													* * * TC = 0.3 HR * * *															IA/P = 0.10				
0.0	20	28	41	118	235	447	676	676	459	283	196	146	114	80	66	57	51	46	42	37	33	31	28	24	22	20	19	18	16	13	12	0
.10	19	26	39	99	189	361	571	641	520	362	251	181	136	89	70	60	53	48	43	37	34	31	28	25	22	21	19	18	16	14	12	0
.20	17	23	32	53	83	154	292	478	587	542	422	308	223	127	86	68	58	52	46	40	35	32	29	26	23	21	20	19	16	14	12	0
.30	16	22	30	49	72	127	237	398	524	536	460	359	268	151	97	73	61	53	48	41	36	32	29	26	23	21	20	19	16	14	12	0
.40	14	19	25	37	45	63	105	193	330	459	510	477	398	237	139	92	70	59	52	44	38	34	30	27	24	21	20	19	17	14	12	0
.50	13	18	24	35	42	56	89	158	272	397	472	475	424	274	163	104	76	62	54	46	39	34	30	27	24	22	20	19	17	15	12	0
.75	11	14	19	26	30	34	42	59	95	160	250	339	417	398	299	196	128	89	69	54	45	37	32	29	26	23	21	20	17	15	12	0
1.0	9	11	14	19	21	24	27	30	36	46	68	109	174	328	396	346	248	163	109	70	54	43	35	31	28	24	22	20	18	16	12	0
1.5	6	8	10	13	14	15	17	19	21	23	26	31	38	77	169	282	347	330	264	158	94	58	42	35	31	27	24	22	19	17	13	3
2.0	4	5	7	8	9	10	10	11	12	14	15	16	18	23	32	57	116	205	285	317	239	128	64	44	36	31	28	25	20	18	14	9
2.5	2	4	5	6	6	7	7	8	9	9	10	11	12	15	18	23	33	60	113	223	293	245	125	65	44	35	31	27	22	19	15	11
3.0	1	2	3	4	4	4	5	5	6	6	7	7	8	9	11	13	16	20	27	61	138	275	246	139	72	46	36	31	25	21	16	11
IA/P = 0.30													* * * TC = 0.3 HR * * *															IA/P = 0.30				
0.0	0	0	0	11	64	251	525	574	454	303	221	173	140	104	88	77	70	64	58	51	47	44	40	36	32	31	29	28	24	21	19	0
.10	0	0	0	0	7	45	183	411	520	476	360	268	205	133	101	85	76	69	62	55	49	45	41	37	33	31	30	28	25	21	19	0
.20	0	0	0	0	5	32	132	318	452	468	396	310	240	151	109	90	78	70	64	56	50	46	42	38	33	31	30	28	25	22	19	0
.30	0	0	0	0	0	3	22	96	244	383	440	411	344	217	142	105	87	76	69	60	53	47	43	39	35	32	30	29	26	22	19	0
.40	0	0	0	0	0	2	16	69	186	317	399	407	365	246	160	115	92	79	71	61	54	48	43	39	35	32	30	29	26	22	19	0
,50	0	0	0	0	0	0	2	11	50	140	258	352	389	327	223	149	110	89	77	66	57	50	45	41	36	33	31	29	26	23	19	0
.75	0	0	0	0	0	0	1	4	20	63	135	219	290	335	281	205	146	110	89	72	62	52	46	42	38	34	31	30	27	23	19	0
1.0	0	0	0	0	0	0	0	0	0	2	9	32	78	216	320	306	243	176	128	90	72	59	49	44	40	36	33	31	28	24	19	1
1.5	0	0	0	0	0	0	0	0	0	0	0	0	2	20	84	185	264	281	246	168	112	77	58	49	44	40	36	32	29	26	20	5
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	12	50	121	200	257	224	141	83	61	50	44	40	36	31	28	21	14
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	16	51	145	239	223	137	82	60	50	44	40	33	29	22	17
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	19	74	184	224	146	89	63	51	45	36	31	24	18
IA/P = 0.50													* * * TC = 0.3 HR * * *															IA/P = 0.50				
0.0	0	0	0	0	1	25	151	299	277	219	187	162	141	113	100	90	84	78	72	65	61	58	53	48	44	42	41	39	35	31	28	0
.10	0	0	0	0	1	17	106	235	263	234	202	175	152	120	104	93	85	79	73	66	61	58	54	49	44	42	41	39	35	31	28	0
.20	0	0	0	0	0	0	12	75	182	236	234	213	188	144	116	101	91	84	78	70	63	59	55	50	45	43	41	40	36	31	28	0
.30	0	0	0	0	0	0	8	52	138	203	224	217	197	154	123	105	94	86	79	71	64	59	55	51	46	43	42	40	36	32	28	0
.40	0	0	0	0	0	0	5	37	105	170	206	213	203	164	131	110	97	88	81	72	65	60	56	51	46	43	42	40	36	32	28	0
.50	0	0	0	0	0	0	0	4	26	78	140	184	203	191	155	126	107	95	86	76	69	62	57	53	48	44	42	41	37	33	28	0
.75	0	0	0	0	0	0	0	1	10	34	73	117	153	184	173	146	122	105	94	82	73	64	58	54	49	45	43	41	37	33	28	0
1.0	0	0	0	0	0	0	0	0	0	0	4	17	42	114	168	178	159	134	114	94	82	70	61	57	52	47	44	42	39	35	28	0
1.5	0	0	0	0	0	0	0	0	0	0	0	0	1	10	44	98	144	163	157	130	105	84	69	61	56	52	47	44	40	36	29	6
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	14	44	87	127	153	141	110	83	69	61	56	51	47	42	38	30	17
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	16	42	97	138	145	107	82	68	60	55	51	43	40	32	25
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	27	71	127	139	105	81	68	60	55	46	41	33	27

RAINFALL TYPE = II * * * TC = 0.3 HR * * * SHEET 3 OF 10

Exhibit 5-II: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution—continued

IA/P = 0.10 * * * TC = 0.4 HR * * * IA/P = 0.10

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
0.0	18	25	36	77	141	271	468	592	574	431	298	216	163	104	77	63	55	49	44	38	34	31	28	25	22	21	20	18	16	14	12	0
.10	18	24	34	67	116	219	385	523	557	473	357	263	196	119	84	67	57	51	46	39	35	32	29	25	22	21	20	19	16	14	12	0
.20	15	20	28	44	59	97	179	316	454	523	489	401	309	178	112	81	65	56	49	42	37	33	30	26	23	21	20	19	17	14	12	0
.30	15	20	27	41	53	82	147	260	389	478	486	429	349	210	129	89	69	58	51	43	38	33	30	27	24	21	20	19	17	14	12	0
.40	13	17	23	33	38	48	71	121	214	331	429	467	442	308	189	120	85	66	56	47	41	35	31	28	24	22	20	19	17	15	12	0
.50	12	16	22	31	36	44	62	102	176	279	379	438	440	339	218	137	94	71	59	49	42	35	31	28	25	22	21	19	17	15	12	0
.75	10	13	17	24	26	30	35	45	65	106	170	251	326	393	341	245	164	112	81	59	48	39	33	30	26	23	21	20	18	15	12	0
1.0	8	10	13	17	19	21	24	27	31	37	50	75	118	251	360	376	292	205	138	83	60	45	36	32	28	25	22	21	18	16	12	1
1.5	6	7	9	12	13	14	15	17	19	21	23	26	31	56	121	224	311	333	293	192	115	66	45	36	31	28	25	22	19	17	13	4
2.0	4	5	6	8	8	9	10	10	11	12	14	15	16	20	27	43	85	159	243	306	264	154	74	47	37	32	28	25	21	18	14	9
2.5	2	3	4	5	6	6	7	7	8	9	9	10	11	13	16	20	27	46	85	184	285	262	147	74	47	37	32	28	22	19	15	11
3.0	1	2	2	3	4	4	4	5	5	6	6	7	7	8	10	12	14	17	23	47	109	227	268	160	83	50	38	32	25	21	16	11

IA/P = 0.30 * * * TC = 0.4 HR * * * IA/P = 0.30

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
0.0	0	0	0	4	26	113	296	480	495	413	306	234	186	127	100	84	74	67	61	54	49	45	41	37	33	31	29	28	25	21	19	0
.10	0	0	0	0	2	18	81	224	395	462	430	347	272	172	121	96	82	73	66	57	51	46	42	38	34	31	30	28	25	22	19	0
.20	0	0	0	0	2	13	59	169	320	414	424	373	305	196	134	103	85	75	67	59	52	47	43	39	34	32	30	29	25	22	19	0
.30	0	0	0	0	0	1	9	42	127	255	361	403	383	274	181	127	99	83	73	63	55	48	44	40	36	32	30	29	26	23	19	0
.40	0	0	0	0	0	1	6	30	94	202	308	372	379	298	203	141	106	87	76	65	56	49	44	40	36	32	31	29	26	23	19	0
.50	0	0	0	0	0	0	0	4	21	70	158	258	334	364	270	187	133	102	85	70	60	51	46	41	37	33	31	30	26	23	19	0
.75	0	0	0	0	0	0	0	2	8	30	76	145	219	321	305	241	177	130	102	78	65	55	47	43	38	34	32	30	27	24	19	0
1.0	0	0	0	0	0	0	0	0	0	1	4	15	42	150	267	308	272	209	154	103	79	62	51	45	41	37	33	31	28	25	19	1
1.5	0	0	0	0	0	0	0	0	0	0	0	0	1	10	51	136	226	274	263	195	131	85	62	51	45	41	36	33	29	26	20	6
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	31	86	162	252	239	162	93	64	52	45	41	37	31	28	21	15
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	33	112	202	235	155	92	64	52	45	41	33	29	23	18
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	21	76	182	221	148	90	63	51	45	36	31	24	18

IA/P = 0.50 * * * TC = 0.4 HR * * * IA/P = 0.50

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
0.0	0	0	0	0	0	7	59	168	245	257	213	186	163	128	109	96	88	81	75	67	62	58	54	50	45	43	41	39	35	31	28	0
.10	0	0	0	0	0	0	5	41	125	205	240	222	198	154	123	106	94	86	79	71	64	60	56	51	46	43	42	40	36	32	28	0
.20	0	0	0	0	0	0	3	28	93	168	216	220	205	164	131	110	97	88	81	72	65	60	56	51	46	43	42	40	36	32	28	0
.30	0	0	0	0	0	0	0	2	20	69	135	189	209	192	155	126	107	95	86	77	69	62	57	53	48	44	42	41	37	33	28	0
.40	0	0	0	0	0	0	0	1	14	50	106	161	193	202	163	133	112	98	89	78	70	62	58	53	48	44	42	41	37	33	28	0
.50	0	0	0	0	0	0	0	1	9	37	83	135	174	194	171	140	117	102	91	80	71	63	58	54	49	45	43	41	37	33	28	0
.75	0	0	0	0	0	0	0	0	0	3	15	40	76	147	177	169	146	124	107	90	79	68	60	56	51	47	43	42	38	34	28	0
1.0	0	0	0	0	0	0	0	0	0	0	1	7	21	78	141	173	167	146	125	101	86	73	63	58	53	48	45	42	39	35	28	1
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	5	26	71	121	153	159	139	113	89	72	63	57	53	48	44	40	37	29	7
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	16	45	86	138	150	125	93	74	64	58	53	48	42	39	31	20
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	17	59	112	143	121	91	73	63	57	53	45	40	32	26
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	11	40	101	138	117	90	73	63	57	48	42	34	27

RAINFALL TYPE = II * * * TC = 0.4 HR * * * SHEET 4 OF 10

Exhibit 5-II: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution—continued

HYDROGRAPH TIME (HOURS) — * * * TC = 0.5 HR * * *

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
IA/P = 0.10																																
0.0	17	23	32	57	94	170	308	467	529	507	402	297	226	140	96	74	61	53	47	41	36	32	29	26	23	21	20	19	16	14	12	0
.10	16	22	30	51	80	140	252	395	484	499	434	343	265	162	108	80	65	55	49	42	36	33	29	26	23	21	20	19	16	14	12	0
.20	14	19	25	38	47	69	116	207	332	434	477	449	378	238	149	101	77	62	53	45	39	34	30	27	24	22	20	19	17	14	12	0
.30	13	18	24	35	43	60	97	170	278	382	446	448	401	270	171	114	83	66	56	46	40	34	31	27	24	22	20	19	17	15	12	0
.40	12	15	21	29	33	40	53	83	141	233	332	408	434	361	243	157	107	79	64	51	43	36	32	28	25	22	21	20	17	15	12	0
.50	11	15	20	28	31	37	48	71	118	194	286	367	412	378	271	178	119	86	68	53	44	37	32	29	25	23	21	20	17	15	12	0
.75	9	11	14	19	21	24	27	31	37	49	74	118	182	319	374	328	244	169	117	76	56	43	35	31	28	25	22	21	18	16	12	1
1.0	7	9	12	16	17	19	21	24	27	32	40	55	83	188	309	359	322	245	172	102	68	49	38	32	29	26	23	21	19	16	12	1
1.5	5	7	8	11	12	13	14	15	17	19	21	23	27	43	89	175	269	322	309	225	140	77	49	38	32	29	25	23	20	17	13	5
2.0	3	4	6	7	8	8	9	10	10	11	12	14	15	18	23	35	65	123	202	297	280	181	88	52	39	33	29	26	21	19	14	10
2.5	2	3	4	5	5	6	6	7	7	8	9	9	10	12	15	18	24	36	66	150	244	278	171	87	52	39	33	29	23	20	15	11
3.0	1	1	2	3	3	4	4	4	5	5	6	6	7	8	9	11	13	16	20	37	86	198	263	182	96	56	40	33	26	21	16	11
IA/P = 0.30																																
0.0	0	0	0	1	9	53	157	314	433	439	379	299	237	159	118	95	81	71	65	56	50	46	42	38	34	31	30	28	25	22	19	0
.10	0	0	0	0	1	6	37	117	248	372	416	391	330	218	150	113	92	79	70	60	53	47	43	39	35	32	30	29	26	22	19	0
.20	0	0	0	0	1	4	26	87	194	313	382	388	349	244	167	122	97	82	72	62	54	48	43	39	35	32	30	29	26	22	19	0
.30	0	0	0	0	0	0	3	19	64	151	259	341	372	316	223	156	117	94	80	67	58	50	45	41	36	33	31	29	26	23	19	0
.40	0	0	0	0	0	0	2	13	47	116	211	298	354	328	245	172	127	100	83	69	59	51	45	41	37	33	31	29	26	23	19	0
.50	0	0	0	0	0	0	0	1	9	34	89	170	255	341	303	225	161	120	96	76	64	54	47	42	38	34	31	30	27	24	19	0
.75	0	0	0	0	0	0	0	1	4	14	41	89	152	270	305	268	207	155	118	87	70	57	48	44	39	35	32	30	27	24	19	0
1.0	0	0	0	0	0	0	0	0	0	0	2	7	22	98	212	295	285	237	181	120	88	67	53	46	42	38	34	31	28	25	19	2
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	5	30	95	183	249	265	217	152	96	66	53	46	41	37	34	30	26	20	8
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	18	59	125	221	245	182	105	69	54	47	42	38	32	28	22	16
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	21	84	174	230	172	103	69	54	46	42	34	30	23	18
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	13	56	157	217	163	101	68	53	46	37	31	25	18
IA/P = 0.50																																
0.0	0	0	0	0	0	2	26	89	170	217	229	200	179	144	119	104	93	85	78	70	64	59	55	51	46	43	41	40	36	32	28	0
.10	0	0	0	0	0	0	1	18	65	135	190	216	205	170	137	115	101	91	83	74	67	61	56	52	47	44	42	40	36	32	28	0
.20	0	0	0	0	0	0	1	12	47	106	162	198	203	178	145	121	105	94	85	76	68	61	57	52	48	44	42	40	37	32	28	0
.30	0	0	0	0	0	0	0	1	8	34	82	135	177	194	168	139	117	102	92	80	71	63	58	54	49	45	43	41	37	33	28	0
.40	0	0	0	0	0	0	0	0	6	25	63	111	155	189	174	146	122	106	94	82	73	64	58	54	50	45	43	41	37	33	28	0
.50	0	0	0	0	0	0	0	0	4	18	48	90	133	184	177	152	128	110	97	84	74	65	59	55	50	45	43	41	38	33	28	0
.75	0	0	0	0	0	0	0	0	1	7	22	47	80	142	169	164	144	124	108	91	79	68	61	56	51	47	44	42	38	34	28	0
1.0	0	0	0	0	0	0	0	0	0	0	1	3	11	51	112	155	166	154	134	109	91	76	65	59	54	49	45	43	39	35	28	2
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	2	16	50	97	136	154	145	121	95	75	64	58	54	49	45	41	37	29	10
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	18	47	86	134	146	125	94	75	64	58	53	49	42	39	31	21
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	11	44	95	140	127	97	77	65	58	54	45	41	33	26
3.3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	29	86	135	122	95	76	65	58	49	43	35	27

RAINFALL TYPE = II   * * * TC = 0.5 HR * * *   SHEET 5 OF 10

Exhibit 5-II: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution—continued

TRVL TIME (hr)	HYDROGRAPH TIME (HOURS) 11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
IA/P = 0.10												* * * TC = 0.75 HR * * *																IA/P = 0.10				
0.0	13	18	24	36	46	68	115	194	294	380	424	410	369	252	172	123	93	74	61	49	41	35	31	27	24	22	20	19	17	15	12	0
.10	13	17	23	34	42	59	97	162	250	337	395	405	381	279	191	135	100	79	65	51	42	36	31	28	25	22	21	19	17	15	12	0
.20	11	15	20	28	32	39	52	82	135	211	295	362	391	351	255	178	127	95	75	57	46	38	32	29	26	23	21	20	17	15	12	0
.30	11	14	19	26	30	36	47	70	113	179	256	326	379	360	277	196	140	103	80	60	48	38	33	29	26	23	21	20	18	15	12	0
.40	10	12	16	22	25	28	33	42	61	96	151	221	291	367	336	255	182	131	98	69	54	42	34	30	27	24	22	20	18	16	12	0
.50	9	12	16	21	24	27	31	39	53	82	128	190	258	358	343	274	200	144	106	74	56	43	35	30	27	24	22	20	18	16	12	0
.75	8	10	13	17	18	21	23	26	31	39	55	82	122	230	314	329	281	217	161	104	72	51	38	33	29	26	23	21	19	16	12	1
1.0	6	8	10	13	14	15	17	19	21	23	27	32	42	89	177	272	319	303	249	163	105	66	45	36	31	27	24	22	19	17	13	3
1.5	4	6	7	9	10	10	11	12	14	15	16	18	20	27	46	90	163	241	295	275	204	119	66	45	35	31	27	24	20	18	13	7
2.0	3	4	5	6	7	7	8	9	9	10	11	12	13	16	20	28	48	89	151	245	274	213	115	65	44	35	30	27	22	19	14	10
2.5	1	2	3	4	4	5	5	6	6	7	7	8	8	10	12	14	17	24	37	86	170	260	219	127	71	47	36	31	24	20	16	11
3.0	1	1	2	3	3	3	4	4	4	5	5	5	6	7	8	10	11	14	17	30	64	157	247	205	122	70	46	36	27	22	17	12
IA/P = 0.30												* * * TC = 0.75 HR * * *																IA/P = 0.30				
0.0	0	0	0	0	1	6	30	86	174	266	326	348	328	246	181	138	110	92	79	66	57	49	44	40	36	32	31	29	26	23	19	0
.10	0	0	0	0	0	1	4	22	65	137	223	292	329	303	228	170	131	106	89	73	61	52	46	41	37	33	31	29	26	23	19	0
.20	0	0	0	0	0	0	3	15	48	108	185	256	305	321	245	184	141	112	93	75	63	53	46	42	37	34	31	30	27	23	19	0
.30	0	0	0	0	0	0	2	11	36	84	151	221	277	308	260	199	152	120	98	78	65	54	47	42	38	34	31	30	27	23	19	0
.40	0	0	0	0	0	0	0	1	8	27	65	122	188	286	301	243	187	144	114	87	71	57	48	43	39	35	32	30	27	24	19	1
.50	0	0	0	0	0	0	0	1	6	20	50	98	158	263	292	254	200	155	122	91	74	59	49	44	40	35	32	30	27	24	19	1
.75	0	0	0	0	0	0	0	0	0	2	8	23	51	140	231	269	253	211	167	119	90	68	53	46	42	37	34	31	28	25	19	2
1.0	0	0	0	0	0	0	0	0	0	0	0	1	4	29	96	186	249	261	231	169	120	84	61	50	44	40	36	33	29	26	20	5
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	34	91	163	220	241	197	131	83	61	50	44	40	35	31	27	21	12
2.0	0	0	0	0	0	0	3	0	0	0	0	0	0	0	0	2	11	36	85	174	226	200	127	82	60	49	44	39	32	29	22	17
2.5	o	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	37	105	196	214	135	87	62	51	44	36	31	24	18
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	24	96	205	189	130	85	62	50	39	32	26	18
IA/P = 0.50												* * * TC = 0.75 HR * * *																IA/P = 0.50				
0.0	0	0	0	0	0	0	2	16	45	92	137	166	185	170	146	125	110	98	89	79	70	63	58	53	48	44	42	41	37	33	28	0
.10	0	0	0	0	0	0	0	1	11	34	73	115	149	180	163	141	122	107	96	84	74	65	59	54	50	45	43	41	38	33	28	0
.20	0	0	0	0	0	0	0	1	8	25	57	96	131	173	166	146	126	111	99	86	76	66	59	55	50	46	43	41	38	34	28	0
.30	0	0	0	0	0	0	0	0	1	5	18	44	79	143	170	160	141	122	108	92	81	69	61	56	52	47	44	42	38	34	28	1
.40	0	0	0	0	0	0	0	0	0	4	14	34	64	127	166	162	145	127	111	95	82	70	62	57	52	47	44	42	38	34	28	1
.50	0	0	0	0	0	0	0	0	0	0	2	10	26	82	138	162	157	140	123	103	88	75	64	58	53	49	45	43	39	35	28	2
.75	0	0	0	0	0	0	0	0	0	0	1	4	12	47	98	139	154	148	135	113	96	80	67	60	55	50	46	43	39	36	29	3
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	6	30	73	119	146	151	134	113	91	74	63	58	53	48	45	41	37	29	7
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	30	66	105	143	143	117	90	73	63	57	52	48	42	39	30	18
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	11	30	77	121	137	114	88	72	63	57	52	44	40	32	25
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	19	55	111	132	111	87	71	62	56	47	42	34	27
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	12	51	112	128	108	86	71	62	51	44	36	27

RAINFALL TYPE = II * * * TC = 0.75 HR * * * SHEET 6 OF 10

Exhibit 5-II: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution—continued

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
IA/P = 0.10												* * * TC = 1.0 HR * * *																IA/P = 0.10				
0.0	11	15	20	29	35	47	72	112	168	231	289	329	357	313	239	175	133	103	83	63	50	40	33	29	26	23	21	20	17	15	12	0
.10	10	13	17	24	27	33	42	62	95	144	202	260	306	340	293	222	165	126	98	72	56	43	35	30	27	24	22	20	18	15	12	0
.20	10	13	17	23	26	30	38	54	82	123	176	232	281	332	303	238	179	136	105	76	59	45	35	30	27	24	22	20	18	16	12	1
.30	9	12	16	22	24	28	35	48	70	105	152	205	256	323	310	254	193	146	113	81	61	46	36	31	27	24	22	20	18	16	12	1
.40	8	11	14	19	21	23	27	32	42	61	91	132	181	276	318	294	237	181	138	95	70	51	39	32	28	25	23	21	18	16	12	1
.50	8	10	13	18	20	22	25	30	38	53	78	114	159	253	311	300	251	195	149	102	74	53	40	33	29	25	23	21	18	16	12	1
.75	7	8	11	14	16	17	19	21	25	30	38	53	76	146	228	284	293	256	208	143	99	66	46	36	31	27	24	22	19	17	13	2
1.0	5	7	8	11	12	13	14	16	17	19	22	25	31	57	111	188	256	286	272	208	144	90	56	41	33	29	26	23	20	17	13	4
1.5	4	5	6	8	8	9	10	11	12	13	14	15	17	22	33	59	107	171	231	268	235	157	88	56	41	33	29	25	21	18	14	8
2.0	2	3	4	5	5	6	6	7	7	8	9	9	10	12	15	19	27	44	78	157	231	252	167	96	59	42	34	29	23	20	15	11
2.5	1	2	2	3	4	4	4	5	5	6	6	7	7	8	10	12	15	19	27	58	120	214	241	159	94	59	42	34	26	21	16	11
3.0	0	1	1	2	2	3	3	3	4	4	4	5	5	6	7	8	10	12	14	22	44	113	214	231	152	91	58	42	29	23	17	12
IA/P = 0.30												* * * TC = 1.0 HR * * *																IA/P = 0.30				
0.0	0	0	0	0	1	4	16	42	83	137	195	243	271	292	227	178	143	117	98	79	66	55	47	42	38	34	31	30	27	23	19	0
.10	0	0	0	0	0	0	3	12	32	66	113	168	218	279	260	213	169	136	113	88	72	59	49	43	39	35	32	30	27	24	19	1
.20	0	0	0	0	0	0	2	9	24	52	93	143	193	271	271	225	180	145	119	92	75	60	50	44	39	35	32	30	27	24	19	1
.30	0	0	0	0	0	0	1	6	18	41	75	120	169	246	264	234	191	153	125	96	78	62	51	44	40	36	33	31	27	24	19	1
.40	0	0	0	0	0	0	0	1	4	14	32	61	100	190	251	259	222	181	146	109	86	67	53	46	41	37	33	31	28	25	19	2
,50	0	0	0	0	0	0	0	1	3	10	24	49	83	168	237	254	230	191	155	115	90	69	54	47	42	37	34	31	28	25	19	2
,75	0	0	0	0	0	0	0	0	0	1	4	12	25	76	150	213	239	228	198	149	112	82	61	50	44	39	35	32	29	26	20	4
1.0	0	0	0	0	0	0	0	0	0	0	0	1	2	15	51	113	182	226	234	197	150	104	72	56	47	42	38	34	30	27	20	7
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	18	51	104	162	220	210	158	102	71	56	47	42	37	31	28	22	13
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	20	49	121	187	209	152	100	70	55	47	41	34	29	23	17
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	7	32	87	171	199	146	98	69	54	46	37	31	24	18
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	13	62	158	192	151	103	73	56	41	34	26	18
IA/P = 0.50												* * * TC = 1.0 HR * * *																IA/P = 0.50				
0.0	0	0	0	0	0	0	1	7	21	42	71	101	126	160	154	138	123	110	100	87	77	67	60	55	50	46	43	41	38	34	28	1
.10	0	0	0	0	0	0	0	1	5	15	33	58	87	134	156	149	134	120	108	93	82	71	62	57	52	47	44	42	38	34	28	1
.20	0	0	0	0	0	0	0	1	4	12	26	48	74	123	153	153	137	123	111	95	84	72	63	57	52	47	44	42	38	34	28	1
.30	0	0	0	0	0	0	0	0	3	9	20	38	62	111	143	150	140	127	114	98	86	73	63	58	53	48	45	42	39	35	28	1
.40	0	0	0	0	0	0	0	0	0	2	6	16	31	75	120	145	148	137	123	106	91	77	66	59	54	49	45	43	39	35	29	2
.50	0	0	0	0	0	0	0	0	0	1	5	12	25	64	109	139	146	139	127	108	94	79	67	60	55	50	46	43	39	36	29	3
.75	0	0	0	0	0	0	0	0	0	0	2	5	12	39	78	115	136	140	134	117	101	84	70	62	56	51	47	44	40	36	29	4
1.0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	26	59	96	125	139	133	117	97	78	66	59	54	49	46	41	37	29	8
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	9	26	54	86	123	133	119	95	77	66	59	54	49	43	39	31	17
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	10	25	64	104	129	116	93	76	65	58	53	45	41	33	24
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	10	34	84	125	117	96	78	66	59	49	43	35	27
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	32	89	122	114	94	77	66	53	45	37	27
RAINFALL TYPE = II												* * * TC = 1.0 HR * * *																SHEET 7 OF 10				

Exhibit 5-II: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution —continued

HYDROGRAPH TIME (HOURS)

IA/P = 0.10 * * * TC = 1.25 HR * * * IA/P = 0.10

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
0.0	10	13	18	25	29	38	54	81	118	163	213	256	284	311	266	212	163	129	104	78	61	47	37	31	27	24	22	20	18	16	12	1
.10	10	13	17	23	27	34	47	69	102	143	189	234	267	297	274	226	175	138	111	82	64	48	38	31	27	24	22	20	18	16	12	1
.20	9	11	15	20	22	26	31	42	60	88	124	168	212	280	292	261	212	166	131	95	72	53	40	33	28	25	23	21	18	16	12	1
.30	8	11	14	19	21	24	29	38	53	76	108	148	190	263	288	268	224	177	140	101	76	55	41	34	29	25	23	21	18	16	12	2
.40	8	10	13	18	20	23	27	34	46	66	94	130	170	245	282	273	235	188	149	107	80	58	42	34	29	26	23	21	19	16	12	2
.50	7	9	12	16	17	19	22	25	31	41	58	82	114	190	256	279	262	222	178	127	93	65	46	36	31	27	24	22	19	17	13	2
.75	6	8	10	14	15	17	19	21	25	31	41	56	78	139	207	254	265	245	208	152	110	75	51	39	32	28	25	22	19	17	13	3
1.0	5	6	8	10	11	13	14	15	17	19	22	26	33	60	109	173	230	261	255	208	153	100	64	46	36	30	26	24	20	18	13	5
1.5	3	4	5	7	7	8	9	9	10	11	12	13	15	19	27	45	79	130	186	247	239	180	108	68	48	37	31	27	22	19	14	10
2.0	2	3	4	5	6	6	7	7	8	8	9	10	11	13	16	22	35	59	98	171	236	236	156	95	62	44	35	30	23	20	15	11
2.5	1	2	2	3	4	4	4	5	5	5	6	6	7	8	10	12	14	19	28	58	114	197	226	163	102	65	46	36	26	21	16	11
3.0	0	1	1	2	2	2	2	3	3	3	4	4	4	5	6	7	9	10	13	19	35	88	184	218	169	109	70	49	31	24	18	12

IA/P = 0.30 * * * TC = 1.25 HR * * * IA/P = 0.30

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
0.0	0	0	0	0	0	2	9	25	50	86	130	174	208	253	235	201	164	136	115	92	76	61	51	44	39	35	32	30	27	24	19	1
.10	0	0	0	0	0	0	1	6	19	40	71	110	153	217	247	227	191	157	131	103	84	66	53	46	41	36	33	31	28	24	19	2
.20	0	0	0	0	0	0	1	4	14	31	58	93	133	202	239	231	199	165	138	108	87	68	55	47	41	37	33	31	28	25	19	2
.30	0	0	0	0	0	0	0	1	3	10	24	46	77	152	210	236	222	190	158	122	97	74	58	49	43	38	34	32	28	25	20	3
.40	0	0	0	0	0	0	0	0	2	8	19	37	64	134	196	232	225	198	166	127	101	77	59	50	43	38	35	32	28	25	20	3
.50	0	0	0	0	0	0	0	0	0	2	6	14	30	82	151	206	228	217	189	146	113	85	64	52	45	40	36	33	29	26	20	5
.75	0	0	0	0	0	0	0	0	0	1	2	7	15	49	105	164	205	218	205	166	129	95	69	55	47	41	37	33	29	26	20	6
1.0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	32	77	134	185	214	203	166	120	83	63	52	45	39	35	30	27	21	10
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	11	33	72	121	184	203	171	117	82	62	51	44	39	32	29	22	15
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	21	67	132	194	174	123	86	64	52	45	35	31	24	18
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	13	46	121	187	166	119	84	63	52	39	32	25	18
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	44	129	180	160	116	83	63	44	35	27	18

IA/P = 0.50 * * * TC = 1.25 HR * * * IA/P = 0.50

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
0.0	0	0	0	0	0	0	1	5	13	26	44	68	91	125	142	142	128	117	107	94	83	72	63	57	52	47	44	42	38	34	28	2
.10	0	0	0	0	0	0	0	0	3	10	20	36	57	100	129	140	136	125	114	100	88	76	65	59	54	49	45	43	39	35	29	3
.20	0	0	0	0	0	0	0	0	2	7	16	30	48	90	122	139	139	127	117	102	90	77	66	60	54	49	45	43	39	35	29	3
.30	0	0	0	0	0	0	0	0	0	2	5	12	24	59	98	126	137	134	125	109	96	82	69	61	56	51	46	44	40	36	29	4
.40	0	0	0	0	0	0	0	0	0	1	4	10	19	51	89	119	134	136	127	112	98	83	70	62	56	51	47	44	40	36	29	5
.50	0	0	0	0	0	0	0	0	0	1	3	7	15	43	79	112	131	135	129	114	100	85	71	63	57	52	47	44	40	36	29	6
.75	0	0	0	0	0	0	0	0	0	0	0	1	3	15	39	71	102	123	130	125	112	94	78	67	60	54	49	46	41	37	29	9
1.0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	17	40	71	101	121	129	121	103	84	71	62	56	51	47	42	38	30	13
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	10	26	51	92	119	125	105	86	72	63	57	52	44	40	32	23
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	11	35	72	112	122	103	85	71	63	56	47	42	34	26
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	24	66	111	119	101	83	71	62	51	44	36	27
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	23	71	110	116	99	82	70	55	46	37	27

RAINFALL TYPE = II * * * TC = 1.25 HR * * * SHEET 8 OF 10

Exhibit 5-II: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution—continued

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
IA/P = 0.10 — * * * TC = 1.5 HR * * *																																
0.0	9	11	15	21	25	31	41	58	82	112	147	184	216	255	275	236	198	159	129	98	76	57	43	35	30	25	23	21	18	16	12	1
.10	8	10	13	18	20	23	28	37	51	72	98	131	166	226	265	254	226	187	151	113	86	63	46	37	31	26	23	21	19	16	13	2
.20	8	10	13	17	19	22	26	33	45	63	87	116	149	212	259	259	233	197	160	119	90	66	48	38	32	27	24	22	19	16	13	2
.30	7	9	12	16	18	21	24	30	40	55	76	103	134	197	244	255	238	206	169	125	95	68	49	38	32	27	24	22	19	17	13	2
.40	7	8	11	14	15	17	19	23	28	36	49	67	91	151	208	247	252	230	196	146	109	77	54	41	34	29	25	22	19	17	13	3
.50	6	8	10	13	15	16	18	21	26	33	43	59	80	136	194	238	249	235	204	154	115	81	56	42	34	29	25	23	20	17	13	3
.75	5	7	8	11	12	13	14	16	18	21	25	32	42	76	125	179	222	240	233	193	148	102	67	48	38	32	27	24	20	18	13	5
1.0	4	5	7	8	9	10	11	12	13	14	16	18	22	34	59	101	152	201	236	230	193	135	86	59	44	35	30	26	21	18	14	7
1.5	3	4	5	6	6	7	8	8	9	10	11	12	13	16	22	34	58	95	141	203	226	197	131	84	58	43	35	29	23	20	15	10
2.0	1	2	3	4	4	5	5	6	7	7	7	8	9	10	12	16	22	34	56	110	172	218	187	126	82	57	43	34	25	21	16	11
2.5	1	1	2	2	3	3	3	4	4	4	5	5	6	7	8	9	11	14	18	34	69	141	210	190	133	87	60	44	30	23	17	12
3.0	0	0	1	1	2	2	2	2	3	3	3	3	4	5	5	6	8	9	11	16	27	66	149	204	181	128	85	58	35	25	18	12
IA/P = 0.30 — * * * TC = 1.5 HR * * *																																
0.0	0	0	0	0	0	1	6	15	31	53	80	112	144	193	225	208	186	157	134	108	89	70	56	48	42	37	34	31	28	25	20	2
.10	0	0	0	0	0	0	1	4	12	25	43	68	97	157	198	219	203	178	151	120	98	77	60	50	44	38	35	32	28	25	20	3
.20	0	0	0	0	0	0	0	1	3	9	19	35	57	114	168	201	213	196	171	135	108	84	64	53	46	40	36	33	29	26	20	4
.30	0	0	0	0	0	0	0	1	2	7	15	29	48	100	155	193	210	200	177	140	113	87	66	54	46	41	36	33	29	26	20	5
.40	0	0	0	0	0	0	0	0	2	5	12	23	39	87	141	184	207	202	182	146	117	89	68	55	47	41	36	33	29	26	20	5
,50	0	0	0	0	0	0	0	0	0	1	4	9	18	51	101	153	190	205	197	164	131	99	73	58	49	43	38	34	30	26	20	7
.75	0	0	0	0	0	0	0	0	0	0	2	4	9	30	68	116	160	189	197	179	147	110	80	62	52	45	39	35	30	27	21	8
1.0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	20	49	92	138	175	195	178	137	97	72	57	48	42	37	31	28	21	12
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	21	47	85	145	187	178	133	95	71	57	48	42	34	29	23	16
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	13	45	97	162	180	138	99	74	58	49	38	32	25	18
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	31	89	161	174	133	97	72	58	42	34	26	18
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	29	98	160	169	129	95	71	48	37	28	19
IA/P = 0.50 — * * * TC = 1.5 HR * * *																																
.0	0	0	0	0	0	0	0	3	8	16	27	42	59	92	116	128	130	121	112	100	90	78	67	60	55	50	46	43	39	35	29	4
.10	0	0	0	0	0	0	0	2	6	12	22	35	51	84	110	125	128	123	114	102	91	79	68	61	55	50	46	43	39	35	29	4
.20	0	0	0	0	0	0	0	0	1	4	10	18	29	60	91	114	126	128	120	108	97	83	71	63	57	52	47	44	40	36	29	5
.30	0	0	0	0	0	0	0	0	1	3	8	14	24	52	83	108	123	126	122	110	98	85	72	63	57	52	48	44	40	36	29	6
.40	0	0	0	0	0	0	0	0	0	1	2	6	12	31	60	90	112	124	126	116	104	90	75	66	59	54	49	45	41	37	29	8
.50	0	0	0	0	0	0	0	0	0	0	2	4	9	26	53	83	106	121	125	118	106	91	77	67	60	54	49	46	41	37	29	8
.75	0	0	0	0	0	0	0	0	0	0	1	2	5	16	36	62	88	108	119	122	112	97	81	69	62	56	51	47	42	38	30	11
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	10	26	49	75	98	118	121	108	90	76	66	59	54	49	43	39	31	16
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	11	25	45	80	107	118	106	89	75	65	59	53	45	41	32	23
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	11	32	63	100	115	104	87	74	65	58	48	42	34	26
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	16	48	94	113	105	89	76	66	53	45	36	27
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	15	54	96	111	103	88	75	58	48	38	28

RAINFALL TYPE = II * * * TC = 1.5 HR * * * SHEET 9 OF 10

Exhibit 5-II: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution—continued

HYDROGRAPH TIME (HOURS)

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
IA/P = 0.10													* * * TC = 2.0 HR * * *															IA/P = 0.10				
0.0	7	9	12	16	18	21	27	36	49	64	82	104	127	171	201	226	208	193	171	132	105	79	58	45	36	30	26	23	20	17	13	3
.10	6	8	10	14	15	17	20	25	33	43	57	74	94	139	179	204	218	205	188	150	118	88	63	48	38	32	27	24	20	17	13	4
.20	6	8	10	13	14	16	19	23	29	39	51	66	84	128	169	198	213	207	192	157	123	91	65	49	39	33	28	24	20	17	13	4
.30	6	7	9	12	14	15	18	21	27	35	45	59	76	117	159	191	211	208	196	163	128	95	68	51	40	33	28	25	20	18	13	4
.40	5	6	8	11	12	13	15	17	20	24	31	41	53	87	128	167	197	209	205	180	145	106	75	55	43	35	30	26	21	18	14	5
.50	5	6	8	10	11	13	14	16	18	22	28	37	48	78	118	158	190	208	208	185	151	111	77	57	44	36	30	26	21	18	14	5
.75	4	6	7	9	10	11	12	13	15	18	22	27	35	58	91	129	164	191	202	194	167	125	87	63	48	38	32	27	22	18	14	6
1.0	3	4	6	7	8	8	9	10	11	12	14	16	18	28	46	74	110	147	178	201	193	156	108	76	56	43	35	30	23	19	14	8
1.5	2	3	3	5	5	5	6	6	7	8	8	9	10	12	16	23	36	57	86	137	178	195	160	113	79	58	45	36	26	21	16	11
2.0	1	2	2	3	3	4	4	4	5	5	6	6	7	8	10	12	16	23	35	67	112	169	190	154	110	78	57	44	30	23	17	11
2.5	0	1	1	2	2	2	3	3	3	4	4	4	5	6	7	8	9	12	16	28	52	105	170	185	149	107	76	56	35	26	18	12
3.0	0	0	1	1	1	1	1	2	2	2	2	3	3	3	4	5	6	7	8	12	18	41	99	161	180	152	112	80	45	30	19	12
IA/P = 0.30													* * * TC = 2.0 HR * * *															IA/P = 0.30				
0.0	0	0	0	0	0	1	3	8	15	25	38	54	74	115	148	168	185	170	159	131	110	89	70	57	49	42	38	34	29	26	20	5
.10	0	0	0	0	0	0	0	2	6	12	21	32	47	85	124	153	169	180	168	145	120	96	75	60	51	44	39	35	30	26	20	6
.20	0	0	0	0	0	0	0	2	4	10	17	27	41	75	114	146	165	175	170	149	124	99	76	62	52	45	39	35	30	27	21	6
.30	0	0	0	0	0	0	0	0	1	3	7	14	23	49	86	122	151	170	174	160	136	107	82	66	54	47	41	37	31	27	21	8
.40	0	0	0	0	0	0	0	0	1	2	6	11	19	43	77	113	144	165	173	163	140	111	85	67	55	47	41	37	31	27	21	8
.50	0	0	0	0	0	0	0	0	1	2	4	9	16	37	68	104	136	160	171	165	144	114	87	69	56	48	42	37	31	27	21	9
.75	0	0	0	0	0	0	0	0	0	0	1	2	5	15	34	62	96	127	152	167	160	132	100	77	62	52	45	40	32	28	22	11
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	10	24	48	79	111	150	166	153	118	90	71	58	49	43	34	29	23	14
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	10	24	45	88	130	161	148	115	88	70	57	48	37	31	24	17
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	10	32	68	122	157	143	113	87	68	56	42	34	26	18
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	16	51	114	153	144	116	89	70	49	38	27	19
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	15	59	118	150	140	113	88	57	42	29	19
IA/P = 0.50													* * * TC = 2.0 HR * * *															IA/P = 0.50				
0.0	0	0	0	0	0	0	0	1	4	8	13	20	28	51	73	92	104	111	112	106	97	86	75	66	60	54	49	46	41	37	30	7
.10	0	0	0	0	0	0	0	1	3	6	11	17	24	45	68	87	101	109	112	107	98	88	76	67	60	55	50	46	41	37	30	8
.20	0	0	0	0	0	0	0	1	2	5	9	14	21	40	62	82	98	107	111	108	100	89	77	68	61	55	50	47	41	37	30	8
.30	0	0	0	0	0	0	0	0	0	2	4	7	12	26	46	67	86	100	108	111	104	93	80	70	63	57	52	48	42	38	30	10
.40	0	0	0	0	0	0	0	0	0	1	3	6	10	22	41	62	81	96	106	110	105	94	81	71	63	57	52	48	42	38	30	11
.50	0	0	0	0	0	0	0	0	0	0	1	2	4	13	27	46	67	85	99	110	108	98	85	74	66	59	54	49	43	39	31	13
.75	0	0	0	0	0	0	0	0	0	0	0	1	2	7	18	33	52	71	88	104	108	102	89	77	68	61	55	50	44	39	31	15
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	13	25	43	62	87	103	108	97	84	73	65	59	53	45	41	32	20
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	12	24	48	74	99	106	95	83	72	64	58	48	43	34	25
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	17	37	69	99	104	94	82	72	64	52	45	36	27
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	8	27	65	95	102	95	83	73	58	49	38	28
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	32	68	95	101	93	82	64	52	40	28

RAINFALL TYPE = II * * * TC = 2.0 HR * * * SHEET 10 OF 10

Exhibit 5-III: Tabular hydrograph unit discharges (csm/in) for type III rainfall distribution

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
IA/P = 0.10														* * * TC = 0.1 HR * * *														IA/P = 0.10				
0.0	29	38	57	172	241	425	662	531	345	265	191	130	101	83	68	62	58	54	50	44	41	37	32	27	23	21	19	16	14	13	11	0
.10	26	32	47	98	147	210	353	559	540	410	313	231	164	101	80	67	61	57	53	47	43	39	34	28	24	22	19	17	14	13	11	0
.20	25	31	44	86	127	182	296	471	517	446	357	273	200	117	86	70	63	58	54	48	44	39	34	29	24	22	20	17	14	13	11	0
.30	22	28	37	57	76	110	158	250	398	477	457	390	312	178	111	83	69	62	57	51	45	41	36	31	25	23	20	18	15	13	11	0
.40	21	27	35	53	68	96	137	213	336	430	448	410	345	210	128	90	72	64	58	52	46	41	36	31	26	23	20	18	15	13	11	0
.50	19	24	30	43	49	62	85	120	182	284	382	426	415	305	188	120	86	71	63	55	49	43	38	33	27	24	21	19	15	14	11	0
.75	17	22	27	37	41	49	62	84	120	181	258	327	375	353	264	177	120	88	72	59	52	45	39	34	29	25	22	20	15	14	11	0
1.0	13	17	22	27	30	33	37	43	52	66	91	131	190	315	358	307	220	149	104	72	60	50	43	37	32	27	23	21	16	14	12	0
1.5	9	11	14	18	19	21	23	25	27	29	33	37	44	70	134	229	304	318	269	172	106	68	52	44	38	33	28	24	19	15	12	2
2.0	6	8	10	13	14	15	16	17	19	20	22	24	26	32	45	73	130	207	271	292	216	121	68	51	43	37	32	27	21	16	13	6
2.5	3	4	6	8	9	10	10	11	12	13	14	16	17	20	23	29	38	57	97	189	271	244	136	75	53	44	38	33	24	19	14	9
3.0	1	2	4	5	6	6	7	8	8	9	10	11	12	14	16	19	23	28	38	74	146	256	226	131	74	53	44	37	27	21	14	10
IA/P = 0.30														* * * TC = 0.1 HR * * *														IA/P = 0.30				
0.0	0	0	0	48	106	296	597	496	368	300	221	155	125	106	89	83	79	74	69	62	59	54	47	40	35	32	28	25	22	20	17	0
.10	0	0	0	35	82	225	473	488	408	336	260	190	147	113	94	85	80	75	70	63	59	54	48	40	35	32	29	25	22	20	17	0
.20	0	0	0	7	26	64	171	372	449	422	365	295	225	142	109	92	84	79	74	66	61	56	50	43	36	33	30	26	22	20	17	0
.30	0	0	0	5	19	49	130	291	397	414	381	323	258	161	118	96	86	80	75	68	62	57	50	43	37	33	30	27	22	20	17	0
.40	0	0	0	0	3	14	37	99	227	340	389	384	343	229	152	113	94	85	79	71	65	59	52	46	38	34	31	28	23	21	17	0
.50	0	0	0	0	2	10	28	75	177	286	355	374	354	256	170	123	99	87	80	73	66	60	53	46	39	35	31	28	23	21	18	0
.75	0	0	0	0	0	1	4	13	35	86	161	238	296	325	266	194	141	110	93	80	71	63	56	50	43	37	33	30	24	21	18	0
1.0	0	0	0	0	0	0	0	2	6	19	48	99	165	282	311	264	197	144	112	88	77	67	59	52	45	39	34	31	24	22	18	0
1.5	0	0	0	0	0	0	0	0	0	0	0	1	4	29	99	197	265	277	236	162	113	84	69	60	53	46	39	35	28	23	19	2
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	35	94	172	233	253	196	124	83	68	59	52	45	39	31	25	20	8
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	11	37	88	184	235	201	122	83	67	59	52	45	34	27	21	13
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	38	110	222	202	131	88	69	60	52	39	31	22	15
IA/P = 0.50														* * * TC = 0.1 HR * * *														IA/P = 0.50				
0.0	0	0	0	0	0	107	226	282	258	209	155	130	123	107	97	95	91	87	82	78	74	69	61	52	47	43	39	35	32	29	25	0
.10	0	0	0	0	0	71	174	246	254	224	178	146	130	112	100	96	92	88	83	78	75	70	62	53	48	44	40	35	32	29	25	0
.20	0	0	0	0	0	0	48	132	208	239	229	195	162	127	109	99	95	91	87	80	77	72	65	56	49	45	41	36	32	30	25	0
.30	0	0	0	0	0	0	32	99	172	216	225	205	176	136	113	101	96	92	88	81	77	72	65	57	50	45	41	37	32	30	26	0
.40	0	0	0	0	0	0	0	21	73	139	191	213	208	164	131	111	100	95	91	85	79	74	68	60	51	47	43	38	33	30	26	0
.50	0	0	0	0	0	0	0	14	53	110	164	197	204	174	139	116	103	97	92	86	80	75	68	60	52	47	43	39	33	30	26	0
.75	0	0	0	0	0	0	0	5	22	54	96	137	166	180	159	134	115	103	96	89	83	77	70	63	54	48	44	40	33	31	26	0
1.0	0	0	0	0	0	0	0	0	0	2	10	29	60	132	175	169	146	124	109	97	89	81	74	67	59	52	47	43	34	31	27	0
1.5	0	0	0	0	0	0	0	0	0	0	0	0	2	17	58	112	150	159	148	122	104	91	81	74	67	59	51	46	38	32	28	1
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	20	54	98	133	149	133	108	90	80	73	66	58	51	42	34	29	7
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	12	35	87	131	141	111	92	81	74	66	59	46	38	30	18
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	22	63	120	136	110	91	81	73	66	51	42	31	22

RAINFALL TYPE = III * * * TC = 0.1 HR * * * SHEET 1 OF 10

Exhibit 5-III: Tabular hydrograph unit discharges (csm/in) for type III rainfall distribution—continued

HYDROGRAPH TIME (HOURS)

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
IA/P = 0.10												* * * TC = 0.2 HR * * *																IA/P = 0.10				
0.0	27	34	50	119	176	258	448	565	483	358	270	194	137	93	75	65	60	56	52	46	42	38	33	28	24	21	19	17	14	13	11	0
.10	24	30	42	70	103	151	222	372	501	489	402	314	234	128	91	73	64	59	55	49	44	40	35	30	25	22	20	17	15	13	11	0
.20	23	29	39	64	90	131	192	312	438	472	425	351	273	153	101	78	66	61	56	50	45	40	35	30	25	22	20	18	15	13	11	0
.30	21	26	33	48	59	79	114	166	263	380	441	431	378	238	141	96	75	65	60	53	47	42	37	32	26	23	21	18	15	13	11	0
.40	20	25	32	45	54	71	99	144	224	328	404	422	392	271	165	108	81	68	61	54	48	43	37	32	27	23	21	19	15	14	11	0
.50	18	22	28	38	43	51	64	88	125	191	282	363	402	356	241	151	102	78	66	57	51	44	39	34	28	24	22	19	15	14	11	0
.75	15	18	23	30	33	37	42	50	64	87	125	184	253	359	341	260	177	121	89	67	57	48	42	36	31	26	23	21	16	14	12	0
1.0	12	16	20	25	28	30	33	37	43	53	68	94	135	251	345	330	260	181	125	81	63	52	44	38	33	28	24	21	17	14	12	0
1.5	8	10	13	17	18	19	21	23	25	27	30	33	38	55	99	179	266	308	289	201	124	74	54	45	39	34	29	24	19	15	12	2
2.0	5	7	9	12	13	14	15	16	17	19	20	22	24	29	38	57	99	166	238	284	239	141	75	54	44	38	33	28	21	17	13	6
2.5	2	4	5	7	8	9	10	10	11	12	13	14	16	18	22	26	33	46	75	155	240	264	154	83	56	46	39	34	25	19	14	9
3.0	1	2	3	5	5	6	6	7	8	8	9	10	11	13	15	18	21	25	33	59	118	220	250	147	82	56	45	39	28	22	14	10
IA/P = 0.30												* * * TC = 0.2 HR * * *																IA/P = 0.30				
0.0	0	0	0	17	49	115	297	489	462	379	307	230	167	119	98	86	81	76	72	64	60	55	48	41	36	32	29	26	22	20	17	0
.10	0	0	0	12	37	89	228	402	442	400	338	266	200	131	104	89	82	78	73	65	61	56	49	42	36	33	29	26	22	20	17	0
.20	0	0	0	1	8	27	69	175	326	403	401	359	297	182	126	100	88	81	76	69	63	58	51	44	37	34	30	27	23	21	17	0
.30	0	0	0	1	6	20	52	134	262	356	386	368	321	208	140	107	91	83	78	70	64	58	52	45	38	34	31	27	23	21	17	0
.40	0	0	0	1	4	15	40	103	209	307	360	365	336	234	157	116	95	85	79	72	65	59	53	46	39	34	31	28	23	21	17	0
.50	0	0	0	0	0	3	11	30	78	165	260	327	352	305	213	147	111	93	84	75	68	61	55	48	41	35	32	29	23	21	18	0
.75	0	0	0	0	0	1	5	14	37	83	150	220	276	311	264	196	144	112	94	80	72	64	56	50	43	37	33	30	24	21	18	0
1.0	0	0	0	0	0	0	0	0	2	7	20	48	94	215	299	286	230	171	128	95	81	69	61	54	47	40	35	32	25	22	18	0
1.5	0	0	0	0	0	0	0	0	0	0	0	0	2	13	56	139	224	266	253	186	128	89	71	61	54	48	41	36	29	23	19	3
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	17	58	126	196	245	214	140	89	71	61	54	47	40	32	25	20	9
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	21	57	145	228	213	136	89	70	60	53	46	35	28	21	13
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	23	78	180	215	145	94	72	61	54	41	32	22	15
IA/P = 0.50												* * * TC = 0.2 HR * * *																IA/P = 0.50				
0.0	0	0	0	0	0	10	110	206	245	245	208	162	137	117	102	96	93	90	85	79	76	71	63	54	48	44	40	36	32	29	25	0
.10	0	0	0	0	0	7	76	163	217	235	217	181	152	122	106	98	94	90	86	80	76	71	64	55	49	45	41	36	32	29	25	0
.20	0	0	0	0	0	0	5	52	126	187	219	218	193	145	119	104	97	93	89	83	78	73	66	58	50	46	42	37	32	30	26	0
.30	0	0	0	0	0	0	3	36	96	156	198	211	199	155	125	108	99	94	90	84	79	74	67	59	51	46	42	38	32	30	26	0
.40	0	0	0	0	0	0	0	2	24	72	128	175	199	184	147	121	106	98	93	87	81	76	69	61	53	48	44	39	33	30	26	0
.50	0	0	0	0	0	0	0	1	17	54	103	151	183	194	156	127	109	100	95	88	82	76	70	62	54	48	44	40	33	30	26	0
.75	0	0	0	0	0	0	0	0	6	23	52	90	127	174	168	146	124	109	100	92	85	78	72	65	56	49	45	41	34	31	26	0
1.0	0	0	0	0	0	0	0	0	0	0	2	11	29	91	149	169	157	135	117	100	92	83	76	69	61	53	48	43	35	32	27	0
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	7	33	81	128	155	155	130	109	94	83	75	68	60	53	47	39	33	28	2
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	10	34	72	112	145	139	114	93	82	74	67	60	52	43	35	29	9
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	12	33	83	126	139	112	92	81	74	67	59	47	38	30	18
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	21	60	116	133	110	92	81	73	66	51	42	31	22

RAINFALL TYPE = III * * * TC = 0.2 HR * * * sheet 2 of 10

Exhibit 5-III: Tabular hydrograph unit discharges (csm/in) for type III rainfall distribution—continued

IA/P = 0.10 * * * TC = 0.3 HR * * * IA/P = 0.10

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
0.0	25	31	44	84	124	181	287	441	498	451	358	276	204	118	87	70	63	58	54	48	44	39	34	29	24	22	20	17	14	13	11	0
.10	22	28	37	56	74	108	156	244	375	457	453	389	314	180	113	83	69	62	57	51	46	41	36	31	26	23	20	18	15	13	11	0
.20	21	27	35	53	67	94	136	208	319	411	439	406	345	212	130	91	73	64	59	52	46	42	36	31	26	23	21	18	15	13	11	0
.30	19	24	30	42	49	61	83	118	178	272	365	414	409	305	190	121	87	71	63	55	49	43	38	33	27	24	21	19	15	14	11	0
.40	18	23	29	40	46	56	74	103	153	232	321	383	400	329	217	138	96	75	65	57	50	44	38	33	28	24	21	19	15	14	11	0
.50	16	21	26	34	38	43	52	67	91	132	199	280	349	383	297	196	128	91	73	60	53	46	40	35	30	25	22	20	16	14	11	0
.75	14	18	23	30	33	37	43	52	66	91	131	187	251	347	331	260	182	126	92	68	57	49	42	36	31	26	23	21	16	14	12	0
1.0	11	15	18	23	25	28	30	33	38	44	54	71	98	192	296	334	294	221	154	94	69	55	45	40	34	29	25	22	17	15	12	0
1.5	7	9	12	15	17	18	19	21	23	25	27	30	33	45	75	137	222	287	300	233	149	84	57	47	40	35	30	25	20	16	13	3
2.0	4	6	8	11	12	13	14	15	16	17	19	20	22	26	33	46	76	131	201	278	258	165	85	57	46	40	34	29	22	17	13	7
2.5	2	3	5	7	7	8	9	10	10	11	12	13	14	17	20	24	29	39	60	125	213	260	177	94	60	47	40	35	25	20	14	9
3.0	1	2	3	4	5	5	6	6	7	8	8	9	10	12	14	16	19	23	29	48	95	195	247	167	93	60	47	40	29	22	15	10

IA/P = 0.30 * * * TC = 0.3 HR * * * IA/P = 0.30

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
0.0	0	0	0	6	22	58	146	308	424	422	367	303	234	145	111	92	84	79	74	67	62	57	50	43	36	33	30	26	22	20	17	0
.10	0	0	0	4	16	44	112	243	364	402	379	328	266	166	120	97	86	80	75	68	62	57	51	44	37	33	30	27	22	21	17	0
.20	0	0	0	3	12	33	86	190	306	370	376	344	292	189	132	103	89	82	77	69	63	58	51	44	37	34	30	27	23	21	17	0
.30	0	0	0	0	2	8	25	65	149	254	331	361	350	261	175	126	100	88	31	73	66	60	53	46	39	35	31	28	23	21	18	0
.40	0	0	0	0	1	6	19	50	116	208	290	338	346	282	195	138	107	91	83	74	67	60	54	47	40	35	32	28	23	21	18	0
.50	0	0	0	0	0	1	4	14	38	90	168	250	308	333	256	180	131	103	89	78	71	63	56	49	42	36	33	29	23	21	18	0
.75	0	0	0	0	0	0	2	6	17	43	89	150	213	299	286	229	171	129	104	85	75	65	58	51	44	38	34	30	24	22	18	0
1.0	0	0	0	0	0	0	0	0	1	3	9	24	53	153	253	288	257	200	150	105	85	72	62	55	48	41	36	32	26	22	19	0
1.5	0	0	0	0	0	0	0	0	0	0	0	1	2	15	59	138	217	258	248	187	130	91	71	62	55	48	41	36	29	23	19	3
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	36	90	159	239	227	158	97	74	63	55	48	42	32	26	20	10
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	12	37	113	194	224	151	96	73	62	55	48	36	29	21	14
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	24	78	176	211	145	95	73	62	54	41	32	22	15

IA/P = 0.50 * * * TC = 0.3 HR * * * IA/P = 0.50

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
0.0	0	0	0	0	0	2	33	116	193	221	221	200	165	129	110	99	95	92	87	81	77	72	65	56	49	45	41	37	32	30	25	0
.10	0	0	0	0	0	1	23	85	157	200	214	205	178	138	115	102	96	92	88	82	77	73	66	57	50	46	42	37	32	30	26	0
.20	0	0	0	0	0	1	15	62	125	175	201	203	187	147	121	105	98	94	89	83	78	73	66	58	50	46	42	38	32	30	26	0
.30	0	0	0	0	0	0	0	10	45	99	149	184	197	174	140	117	104	97	93	86	80	75	69	61	52	47	43	39	33	30	26	0
.40	0	0	0	0	0	0	0	7	32	76	125	164	189	179	148	123	107	99	94	87	81	76	69	62	53	48	44	39	33	30	26	0
.50	0	0	0	0	0	0	0	0	5	23	59	103	144	183	169	141	119	105	98	90	84	78	71	64	55	49	45	41	33	31	26	0
.75	0	0	0	0	0	0	0	0	2	9	27	55	89	148	168	156	135	117	105	94	87	80	73	66	58	51	46	42	34	31	27	0
1.0	0	0	0	0	0	0	0	0	0	0	1	4	14	59	119	157	163	145	126	105	95	85	77	71	63	55	49	44	36	32	27	0
1,5	0	0	0	0	0	0	0	0	0	0	0	0	0	3	19	56	103	139	151	138	116	97	85	77	70	62	54	48	40	33	28	3
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	21	52	91	135	143	120	96	84	76	69	61	54	44	35	29	11
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	22	65	112	136	117	96	83	75	68	60	48	39	30	19
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	13	45	102	131	114	95	83	75	67	53	43	31	22

RAINFALL TYPE = III * * * TC = 0.3 HR * * * SHEET 3 OF 10

Exhibit 5-III: Tabular hydrograph unit discharges (csm/in) for type III rainfall distribution—continued

HYDROGRAPH TIME (HOURS)

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
IA/P = 0.10												* * * TC = 0.4 HR * * *																	IA/P = 0.10			
0.0	23	29	39	65	91	132	198	308	422	449	417	345	274	162	108	82	68	61	57	50	45	41	35	30	25	22	20	18	15	13	11	0
.10	20	26	33	48	60	80	114	170	262	368	422	418	370	242	149	102	79	67	60	53	47	42	37	32	27	23	21	18	15	13	11	0
.20	20	25	32	45	55	72	100	147	224	320	388	408	383	272	171	114	85	70	62	54	48	43	37	32	27	23	21	19	15	14	11	0
.30	17	22	28	38	43	51	65	88	127	191	277	351	389	349	244	157	108	81	68	58	51	45	39	34	29	24	22	19	15	14	11	0
.40	17	21	27	36	41	47	59	79	111	165	240	314	378	359	268	178	120	88	72	60	52	45	40	34	29	25	22	20	15	14	11	0
.50	15	19	24	31	34	38	44	54	71	98	142	207	278	364	332	243	163	113	84	65	56	48	41	36	31	26	23	20	16	14	12	0
.75	12	16	20	25	27	30	33	38	44	54	70	97	138	249	333	320	257	185	131	85	65	53	44	39	33	28	24	22	17	14	12	0
1.0	11	13	17	22	23	25	28	30	34	38	46	57	75	145	245	322	311	255	188	114	77	58	47	41	35	30	26	23	18	15	12	1
1.5	6	9	11	14	15	17	18	19	21	23	25	27	30	39	59	105	180	255	292	257	176	98	61	48	41	36	31	26	20	16	13	4
2.0	4	6	8	10	11	12	13	14	15	16	17	19	20	24	30	39	61	103	166	253	272	189	98	61	48	41	35	30	23	18	13	8
2.5	2	3	4	6	7	7	8	9	10	10	11	12	13	16	18	22	26	34	49	100	183	255	198	108	66	50	42	36	26	20	14	9
3.0	1	1	2	4	4	5	5	6	6	7	8	8	9	11	13	15	18	21	26	40	77	169	243	185	106	65	49	41	30	23	15	10
IA/P = 0.30												* * * TC = 0.4 HR * * *																	IA/P = 0.30			
0.0	0	0	0	2	10	30	78	177	306	379	379	347	293	187	133	105	90	82	77	69	63	58	51	44	38	34	30	27	23	21	17	0
.10	0	0	0	2	7	22	59	138	250	336	365	353	313	212	146	112	94	84	78	71	64	59	52	45	38	34	31	27	23	21	17	0
.20	0	0	0	1	5	17	45	107	202	292	341	349	325	235	162	121	98	87	80	72	65	59	53	46	39	34	31	28	23	21	17	0
.30	0	0	0	0	1	4	12	34	83	162	249	310	336	298	215	152	116	96	85	76	68	61	55	48	41	36	32	29	23	21	18	0
.40	0	0	0	0	0	3	9	26	64	130	209	276	324	307	234	168	125	101	88	77	70	62	55	49	41	36	32	29	23	21	18	0
.50	0	0	0	0	0	0	2	7	19	49	103	173	242	313	285	216	157	119	98	82	73	65	57	51	44	37	33	30	24	22	18	0
.75	0	0	0	0	0	0	1	3	9	23	52	97	153	253	285	253	199	151	118	91	78	68	59	53	46	39	35	31	25	22	18	0
1.0	0	0	0	0	0	0	0	0	0	1	4	13	30	104	204	276	276	226	175	120	92	75	64	57	50	43	37	33	26	22	19	1
1.5	0	0	0	0	0	0	0	0	0	0	0	0	1	8	36	98	177	236	250	207	148	99	75	63	56	49	42	37	29	24	19	4
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	22	62	124	210	232	176	107	78	65	57	50	43	33	27	20	11
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	24	85	167	219	167	106	77	64	56	49	37	29	21	14
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	15	58	152	207	159	104	76	64	55	42	33	22	15
IA/P = 0.50												* * * TC = 0.4 HR * * *																	IA/P = 0.50			
0.0	0	0	0	0	0	0	10	54	121	182	204	204	191	146	121	106	98	94	90	83	78	73	66	58	50	46	42	38	32	30	26	0
.10	0	0	0	0	0	0	7	38	94	153	187	198	193	155	128	110	100	95	91	84	79	74	67	59	51	46	42	38	32	30	26	0
.20	0	0	0	0	0	0	5	27	71	126	166	187	191	164	134	114	103	96	92	85	80	75	68	60	52	47	43	38	33	30	26	0
.30	0	0	0	0	0	0	0	3	19	54	102	145	173	185	155	129	111	101	95	88	82	76	70	62	54	48	44	40	33	31	26	0
.40	0	0	0	0	0	0	0	2	13	40	81	124	157	180	161	135	116	104	97	90	83	77	71	63	55	49	44	40	33	31	26	0
.50	0	0	0	0	0	0	0	0	1	9	30	64	104	163	174	154	130	113	102	93	86	79	73	66	57	50	46	42	34	31	27	0
.75	0	0	0	0	0	0	0	0	0	3	13	32	59	120	157	162	145	126	111	98	90	82	75	68	60	52	47	43	35	31	27	0
1.0	0	0	0	0	0	0	0	0	0	0	0	2	6	36	90	138	158	152	135	112	98	88	79	72	65	56	50	45	37	32	27	0
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	1	10	37	80	121	148	143	123	102	87	78	71	64	56	49	41	34	28	4
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	13	36	71	121	140	126	101	86	78	70	63	55	44	36	29	13
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	14	49	97	135	122	100	86	77	70	62	49	40	30	20
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	33	88	130	119	98	85	76	69	54	44	32	22

RAINFALL TYPE = III * * * TC = 0.4 HR * * * SHEET 4 OF 10

Exhibit 5-III: Tabular hydrograph unit discharges (csm/in) for type III rainfall distribution—continued

HYDROGRAPH TIME (HOURS)

* * * TC = 0.5 HR * * *

IA/P = 0.10

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
0.0	21	27	35	54	70	97	144	217	316	397	411	388	330	214	139	99	78	67	60	52	47	42	36	31	26	23	21	18	15	13	11	0
.10	19	24	30	43	50	64	86	125	186	273	355	392	390	296	194	129	94	75	65	56	49	43	38	33	28	24	21	19	15	14	11	0
.20	18	23	29	40	47	58	77	109	161	235	315	367	382	318	218	145	103	80	68	57	50	44	39	33	28	24	22	19	15	14	11	0
.30	16	21	26	34	38	44	53	69	95	139	203	278	337	367	289	199	135	98	77	62	54	46	40	35	30	25	22	20	16	14	11	0
.40	16	20	25	33	36	41	49	62	84	121	176	244	306	358	306	220	151	107	83	64	55	47	41	35	30	25	23	20	16	14	12	0
.50	14	18	22	28	31	35	39	46	57	75	106	152	213	323	346	282	202	140	102	73	59	50	42	37	32	27	23	21	16	14	12	0
.75	12	16	20	25	28	30	34	38	45	56	75	104	145	246	319	308	252	187	135	89	67	53	44	39	33	28	24	22	17	14	12	0
1.0	10	12	16	20	22	23	25	28	31	34	39	47	60	110	197	280	309	279	220	138	90	63	49	42	37	31	26	23	18	15	12	1
1.5	6	8	10	13	14	15	17	18	19	21	23	25	27	34	49	82	143	218	283	271	203	116	68	51	43	37	32	27	21	16	13	4
2.0	3	5	7	9	10	11	12	13	14	15	16	17	19	22	27	34	50	82	135	226	265	211	114	67	50	42	37	31	23	18	13	8
2.5	2	3	4	6	7	7	8	9	10	10	11	12	13	16	18	22	26	34	50	102	182	249	197	111	67	50	42	36	26	20	14	9
3.0	1	1	2	3	4	4	5	5	6	6	7	8	8	10	12	14	16	19	23	34	63	144	238	201	121	72	52	43	31	23	15	10

* * * TC = 0.5 HR * * *

IA/P = 0.30

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
0.0	0	0	0	1	4	15	40	101	198	295	345	345	325	232	161	122	100	88	80	72	65	59	53	46	39	34	31	28	23	21	18	0
.10	0	0	0	1	3	11	30	77	158	249	313	335	329	253	178	132	106	91	82	73	66	60	53	47	40	35	31	28	23	21	18	0
.20	0	0	0	0	2	8	23	59	125	208	278	316	324	271	196	144	112	95	85	75	67	61	54	47	40	35	32	28	23	21	18	0
.30	0	0	0	0	0	2	6	17	45	98	171	242	291	313	249	182	136	108	92	80	71	63	56	49	42	36	33	29	24	21	18	0
.40	0	0	0	0	0	1	4	13	34	77	140	208	264	304	263	198	148	115	97	81	72	64	57	50	43	37	33	30	24	21	18	0
.50	0	0	0	0	0	0	1	3	10	26	60	113	177	276	295	244	185	140	111	88	77	67	59	52	45	39	34	31	24	22	18	0
.75	0	0	0	0	0	0	0	1	4	12	29	60	104	204	271	263	222	174	136	101	83	70	61	54	47	40	35	32	25	22	18	0
1.0	0	0	0	0	0	0	0	0	0	1	2	6	16	67	155	235	263	242	198	138	102	80	66	58	51	44	38	34	27	23	19	1
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	4	22	67	138	205	241	221	167	110	79	66	58	51	44	38	30	24	20	5
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	13	42	93	182	225	191	119	83	67	58	51	44	34	27	21	12
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	15	62	139	213	180	117	82	67	58	51	38	30	22	15
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	10	41	127	203	171	114	81	66	57	43	33	23	16

* * * TC = 0.5 HR * * *

IA/P = 0.50

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
0.0	0	0	0	0	0	0	3	24	68	124	174	190	190	162	133	114	103	97	92	85	80	75	68	60	52	47	43	39	33	30	26	0
.10	0	0	0	0	0	0	2	17	51	100	149	177	186	169	140	119	106	99	93	86	81	75	69	61	52	47	43	39	33	30	26	0
.20	0	0	0	0	0	0	1	12	38	79	126	160	181	173	147	124	109	101	95	88	81	76	69	62	53	48	44	39	33	30	26	0
.30	0	0	0	0	0	0	0	1	8	28	62	105	141	176	165	141	120	107	99	91	84	78	71	64	56	49	45	41	33	31	26	0
.40	0	0	0	0	0	0	0	1	6	20	48	86	123	172	172	146	125	111	101	92	85	79	72	65	56	50	45	41	34	31	26	0
.50	0	0	0	0	0	0	0	0	4	15	37	70	105	157	167	151	130	114	104	94	87	79	73	66	57	50	46	42	34	31	27	0
.75	0	0	0	0	0	0	0	0	0	1	6	17	37	91	139	157	150	134	119	103	93	84	76	69	62	54	48	44	35	32	27	0
1.0	0	0	0	0	0	0	0	0	0	0	1	3	9	40	91	135	153	149	135	113	99	88	79	72	65	57	50	45	37	32	27	1
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	5	24	59	101	132	144	130	107	90	80	73	65	57	51	41	34	29	6
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	25	55	106	138	130	105	89	79	72	65	57	45	37	30	15
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	9	36	81	133	133	104	88	79	71	64	50	41	31	21
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	5	24	74	128	122	103	88	78	70	55	45	32	23

RAINFALL TYPE = III * * * TC = 0.5 HR * * * SHEET 5 OF 10

Exhibit 5-III: Tabular hydrograph unit discharges (csm/in) for type III rainfall distribution—continued

HYDROGRAPH TIME (HOURS)

IA/P = 0.10 * * * TC = 0.75 HR * * *

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
0.0	17	22	28	39	45	56	73	104	151	215	281	328	343	310	228	163	121	94	77	63	53	45	39	34	29	25	22	20	15	14	11	0
.10	17	21	27	37	42	51	66	91	131	187	250	302	336	319	247	179	131	101	82	65	55	46	40	35	29	25	22	20	16	14	11	0
.20	15	19	24	31	35	40	48	60	81	114	163	221	275	328	298	229	167	124	96	73	60	49	42	36	31	26	23	20	16	14	12	0
.30	14	18	23	30	33	38	44	55	72	100	142	194	248	320	305	245	182	135	103	76	62	50	42	37	31	27	23	21	16	14	12	0
.40	13	16	21	26	29	32	36	41	50	65	88	124	171	268	313	288	228	170	127	88	68	54	44	38	33	28	24	21	17	14	12	0
.50	12	16	20	25	28	30	34	39	46	59	78	109	150	244	306	294	242	184	138	94	71	56	45	39	34	28	24	22	17	14	12	0
.75	10	13	16	21	23	25	27	30	33	38	46	58	77	140	221	277	287	248	197	133	92	66	50	42	36	31	26	23	18	15	12	1
1.0	8	10	13	16	18	19	21	23	25	27	30	34	39	61	109	181	249	280	265	198	134	85	58	46	40	34	29	25	20	16	12	2
1.5	5	7	9	12	13	14	15	16	17	19	20	22	24	30	40	63	106	167	225	261	226	147	84	58	46	39	34	29	22	17	13	5
2.0	2	4	5	7	8	9	9	10	11	12	13	14	16	18	22	26	34	50	80	155	226	246	158	91	61	47	40	34	25	20	14	9
2.5	1	2	3	5	5	6	6	7	8	8	9	10	11	13	15	18	21	26	34	62	120	209	234	151	90	60	47	39	29	22	15	10
3.0	0	1	2	3	3	3	4	4	5	5	6	7	7	9	10	12	15	17	21	29	50	113	209	224	144	88	59	46	33	25	16	10

IA/P = 0.30 * * * TC = 0.75 HR * * *

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
0.0	0	0	0	0	1	3	8	24	58	113	182	243	283	287	233	178	139	114	98	83	73	64	56	50	43	37	33	30	24	21	18	0
.10	0	0	0	0	0	2	6	18	45	91	151	212	259	284	245	191	149	120	102	85	75	65	57	50	43	37	33	30	24	21	18	0
.20	0	0	0	0	0	0	1	5	14	35	72	125	183	263	277	230	180	142	116	93	80	68	59	52	45	39	34	31	25	22	18	0
.30	0	0	0	0	0	0	1	3	10	26	57	102	156	245	270	240	192	151	122	96	82	69	60	53	46	39	35	31	25	22	18	0
.40	0	0	0	0	0	0	0	1	2	8	20	45	83	182	252	264	226	181	144	108	89	73	62	55	48	41	36	32	26	22	19	0
.50	0	0	0	0	0	0	0	0	2	6	15	35	67	158	235	259	235	192	153	113	92	75	63	56	49	42	36	33	26	22	19	1
.75	0	0	0	0	0	0	0	0	0	0	1	2	7	35	100	178	232	242	217	163	121	90	71	61	54	47	40	35	28	23	19	2
1.0	0	0	0	0	0	0	0	0	0	0	0	1	4	21	68	140	205	236	229	181	135	97	74	63	55	48	41	36	29	24	19	3
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	13	42	94	155	221	212	158	103	77	64	56	49	42	33	26	20	10
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	15	42	113	184	209	151	101	76	63	55	48	36	29	21	14
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	27	80	168	199	146	100	75	63	55	41	32	22	15
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	10	56	154	191	151	105	78	64	48	37	24	16

IA/P = 0.50 * * * TC = 0.75 HR * * *

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
0.0	0	0	0	0	0	0	0	3	13	37	71	108	140	167	156	136	120	108	101	92	85	78	72	64	56	50	45	41	34	31	26	0
.10	0	0	0	0	0	0	0	2	10	28	57	91	124	163	158	140	124	111	103	94	86	79	72	65	57	50	46	41	34	31	27	0
.20	0	0	0	0	0	0	0	0	1	7	21	45	76	135	159	153	136	120	109	98	90	82	74	67	59	52	47	43	35	31	27	0
.30	0	0	0.	0	0	0	0	0	1	5	15	35	62	121	157	157	140	124	112	100	91	83	75	68	60	52	47	43	35	31	27	0
.40	0	0	0	0	0	0	0	0	1	3	11	27	50	107	146	154	143	128	115	102	93	84	76	69	61	53	48	43	35	32	27	0
.50	0	0	0	0	0	0	0	0	0	0	2	8	21	66	118	148	152	139	125	108	97	87	78	71	63	55	49	45	36	32	27	1
.75	0	0	0	0	0	0	0	0	0	0	1	4	10	38	82	122	142	144	133	116	103	91	80	73	66	58	51	46	37	32	28	1
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	5	24	60	102	132	142	133	116	99	86	77	70	62	55	49	40	33	28	3
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	25	55	91	128	136	119	98	85	77	69	62	54	44	36	29	11
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	9	25	66	109	131	116	97	85	76	69	61	48	39	30	18
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	16	47	100	127	114	96	84	75	68	53	43	31	22
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	10	44	102	124	112	95	83	75	59	47	33	23

RAINFALL TYPE = III * * * TC = 0.75 HR * * * SHEET 6 OF 10

Exhibit 5-III: Tabular hydrograph unit discharges (csm/in) for type III rainfall distribution—continued

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
IA/P = 0.10 * * * TC = 1.0 HR * * * IA/P = 0.10																																
0.0	15	19	24	32	37	44	54	71	98	136	181	227	264	297	270	215	164	128	103	78	64	52	43	36	31	26	23	21	16	14	12	0
.10	13	17	22	28	31	35	41	49	64	87	120	161	205	273	289	254	201	155	122	90	71	56	45	38	33	28	24	21	17	14	12	0
.20	13	16	21	27	29	33	38	46	58	77	105	142	184	257	285	263	214	167	130	95	74	57	46	39	33	28	24	22	17	14	12	0
.30	12	16	20	26	28	31	36	42	53	69	93	126	165	240	279	268	225	178	139	100	77	59	47	39	34	29	25	22	17	15	12	1
.40	11	14	18	23	25	27	30	34	40	48	62	83	112	185	251	276	256	213	168	118	87	65	50	41	35	30	26	23	18	15	12	1
.50	11	13	17	22	24	26	29	32	37	45	56	74	99	167	235	270	261	223	179	126	92	67	51	42	36	31	26	23	18	15	12	1
.75	8	10	13	17	18	19	21	23	25	28	31	36	44	72	122	186	239	258	243	189	136	90	62	48	40	34	29	25	20	16	12	2
1.0	6	9	11	14	15	17	18	20	21	23	25	28	32	46	75	124	185	234	253	226	170	110	71	53	43	37	31	27	21	16	13	4
1.5	4	6	8	10	11	12	13	14	15	16	17	19	21	25	32	46	74	118	170	230	239	179	108	70	52	43	36	31	23	18	13	7
2.0	2	3	4	6	7	7	8	9	10	10	11	12	13	16	18	22	28	38	58	111	179	228	185	116	75	54	44	37	27	21	14	9
2.5	1	1	2	4	4	5	5	6	6	7	8	8	9	11	13	15	18	22	28	46	87	167	219	176	113	73	54	43	31	23	15	10
3.0	0	0	1	2	2	2	3	3	3	4	4	5	5	7	8	10	12	14	16	21	32	68	156	210	179	120	78	56	37	27	16	11
IA/P = 0.30 * * * TC = 1.0 HR * * * IA/P = 0.30																																
0.0	0	0	0	0	0	1	5	13	30	57	95	141	186	243	249	213	174	142	119	97	83	70	60	53	46	39	35	31	25	22	18	0
.10	0	0	0	0	0	1	3	10	23	46	79	120	164	230	245	221	183	150	125	101	85	72	61	53	46	40	35	31	25	22	18	0
.20	0	0	0	0	0	0	1	3	7	18	36	65	102	183	233	241	210	174	144	112	92	76	64	56	48	42	36	33	26	22	19	1
.30	0	0	0	0	0	0	1	2	6	14	29	53	86	163	221	237	217	183	151	117	95	78	65	56	49	42	37	33	26	22	19	1
.40	0	0	0	0	0	0	0	0	1	4	11	23	43	107	180	225	233	207	175	133	105	84	68	59	51	44	38	34	27	23	19	1
.50	0	0	0	0	0	0	0	0	1	3	8	18	34	91	162	214	230	213	183	139	109	86	70	60	52	45	39	34	27	23	19	1
.75	0	0	0	0	0	0	0	0	0	0	1	4	9	33	82	145	196	218	211	174	135	101	77	64	56	48	42	37	29	24	19	3
1.0	0	0	0	0	0	0	0	0	0	0	0	1	2	11	37	85	144	192	214	199	160	116	85	69	59	51	44	38	30	25	20	5
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	23	56	104	174	203	177	123	89	71	60	52	45	35	27	21	11
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	8	24	73	139	194	169	120	87	70	59	51	39	30	22	14
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	15	51	127	186	162	117	86	69	59	44	34	23	16
3,0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	35	117	180	164	122	90	71	51	39	25	16
IA/P = 0.50 * * * TC = 1.0 HR * * * IA/P = 0.50																																
0.0	0	0	0	0	0	0	0	2	6	17	34	57	83	127	151	142	130	118	109	99	91	83	75	68	59	52	47	43	35	31	27	1
.10	0	0	0	0	0	0	0	1	5	13	27	47	71	117	146	144	133	121	112	101	92	84	76	68	60	53	48	43	35	32	27	1
.20	0	0	0	0	0	0	0	1	3	10	21	38	60	106	138	143	135	124	114	102	94	85	76	69	61	54	48	44	35	32	27	1
.30	0	0	0	0	0	0	0	0	0	2	7	16	31	73	114	139	142	132	121	108	98	88	79	71	64	56	50	45	36	32	27	1
.40	0	0	0	0	0	0	0	0	0	2	5	13	25	62	104	133	140	134	124	110	99	89	80	72	64	56	50	45	37	32	27	1
.50	0	0	0	0	0	0	0	0	0	0	1	4	10	35	74	112	134	138	131	117	104	93	82	74	67	59	52	47	38	33	28	2
.75	0	0	0	0	0	0	0	0	0	0	0	2	5	19	43	84	115	131	134	123	110	97	85	77	69	61	54	48	39	33	28	3
1.0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	21	49	84	113	129	132	119	103	89	80	72	64	56	50	41	34	28	5
1.5	0	0	0	0	0	0	0	0	o	0	0	0	0	0	1	7	21	46	75	113	128	120	102	88	79	71	63	56	45	37	29	12
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	14	42	82	119	124	104	90	80	72	64	50	41	31	20
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	29	75	117	121	102	89	79	71	56	45	32	23
3,0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	28	80	118	118	101	88	79	63	49	34	24

RAINFALL TYPE = III * * * TC = 1.0 HR * * * SHEET 7 OF 10

Exhibit 5-III: Tabular hydrograph unit discharges (csm/in) for type III rainfall distribution—continued

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
IA/P = 0.10												* * * TC = 1.25 HR * * *																	IA/P = 0.10			
0.0	13	17	22	28	32	37	44	56	75	100	133	170	206	255	264	236	194	155	125	95	75	59	47	39	33	28	24	22	17	14	12	1
.10	13	16	21	27	30	34	41	51	67	89	118	152	188	243	261	243	204	164	133	100	79	61	48	40	34	29	25	22	17	15	12	1
.20	11	15	18	24	26	29	33	38	47	60	79	105	137	203	248	256	232	193	156	115	88	67	51	42	35	30	26	23	18	15	12	1
.30	11	14	18	23	25	27	31	36	43	54	71	94	122	187	237	254	238	203	165	121	92	69	53	43	36	30	26	23	18	15	12	1
.40	10	12	16	20	22	24	26	29	34	40	49	64	84	140	201	242	250	228	193	142	106	77	57	46	38	32	27	24	19	15	12	1
.50	9	12	15	19	21	23	25	28	32	37	45	58	75	126	186	232	248	233	201	150	111	80	58	47	39	33	28	24	19	15	12	2
.75	8	11	14	17	19	20	22	24	27	31	36	44	56	93	144	196	230	237	220	174	131	91	64	50	41	34	29	25	20	16	12	2
1.0	6	8	11	14	15	16	17	19	20	22	25	28	32	46	74	119	170	213	234	218	174	120	79	58	46	38	32	27	21	17	13	4
1.5	3	5	7	9	9	10	11	12	13	14	16	17	18	22	27	38	58	91	134	197	223	194	128	83	60	47	39	33	24	19	14	7
2.0	2	3	4	6	6	7	8	8	9	10	11	12	13	15	18	22	27	38	57	106	166	214	184	124	82	59	47	39	28	21	14	9
2.5	1	1	2	3	3	4	4	5	5	6	7	7	8	10	11	13	16	19	24	38	68	137	206	186	130	87	62	48	33	24	15	10
3.0	0	0	1	2	2	2	2	3	3	4	4	5	5	6	8	9	11	13	16	21	31	66	145	200	178	126	85	61	39	28	17	11
IA/P = 0.30												* * * TC = 1.25 HR * * *																	IA/P = 0.30			
0.0	0	0	0	0	0	1	2	7	18	35	61	94	130	192	222	218	191	161	136	110	93	77	65	56	48	41	36	32	26	22	19	1
.10	0	0	0	0	0	1	2	5	14	28	50	79	113	177	214	219	197	168	142	115	96	79	66	57	49	42	37	33	26	22	19	1
.20	0	0	0	0	0	0	1	4	10	22	41	66	97	162	205	217	202	175	149	119	99	81	67	58	50	43	37	33	26	23	19	1
.30	0	0	0	0	0	0	0	1	3	8	17	33	55	115	174	208	214	195	168	133	108	87	70	60	52	45	39	34	27	23	19	2
.40	0	0	0	0	0	0	0	1	2	6	14	26	45	100	159	200	212	199	174	138	112	89	72	61	53	45	39	35	28	23	19	2
.50	0	0	0	0	0	0	0	0	1	2	5	11	21	60	116	170	203	209	192	156	124	97	76	64	55	47	41	36	29	24	19	3
.75	0	0	0	0	0	0	0	0	0	1	2	5	11	35	79	131	175	201	201	172	140	107	82	67	58	50	43	37	30	24	20	4
1.0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	23	57	105	153	186	198	172	131	95	75	63	54	47	41	32	25	20	7
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	8	24	54	96	157	189	174	127	94	74	63	54	46	35	28	21	12
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	9	24	69	127	182	167	124	93	74	62	53	40	31	22	15
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	10	35	99	176	167	129	96	76	63	46	35	23	16
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	34	107	171	161	125	94	75	53	40	25	17
IA/P = 0.50												* * * TC = 1.25 HR * * *																	IA/P = 0.50			
0.0	0	0	0	0	0	0	0	1	4	10	21	36	55	95	123	138	131	122	114	104	95	86	78	70	62	55	49	44	36	32	27	1
.10	0	0	0	0	0	0	0	0	1	3	8	16	30	66	102	125	134	128	120	108	99	89	80	73	65	57	51	46	37	32	28	2
.20	0	0	0	0	0	0	0	0	0	2	6	13	24	57	94	120	132	130	122	110	101	91	81	73	66	58	51	46	37	33	28	2
.30	0	0	0	0	0	0	0	0	0	1	4	10	19	49	85	113	131	131	124	112	102	92	82	74	66	58	52	47	38	33	28	2
.40	0	0	0	0	0	0	0	0	0	1	3	8	16	42	76	106	126	130	125	114	104	93	83	75	67	59	52	47	38	33	28	3
.50	0	0	0	0	0	0	0	0	0	0	1	2	6	22	51	84	112	127	129	120	109	97	86	77	70	61	54	49	40	33	28	4
.75	0	0	0	0	0	0	0	0	0	0	0	1	3	12	32	61	91	113	125	123	114	101	89	80	72	64	56	50	41	34	28	5
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	22	47	76	102	122	124	111	96	85	76	69	61	54	43	36	29	9
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	8	22	43	82	111	121	109	95	84	76	68	60	48	39	30	16
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	9	30	63	104	118	107	94	83	75	67	53	43	31	21
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	20	58	104	116	106	93	83	74	59	47	33	23
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	19	64	104	114	104	92	82	65	52	35	24

RAINFALL TYPE = III * * * TC = 1.25 HR * * * SHEET 8 OF 10

Exhibit 5-III: Tabular hydrograph unit discharges (csm/in) for type III rainfall distribution—continued

HYDROGRAPH TIME (HOURS)

TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
IA/P = 0.10														* * * TC = 1.5 HR * * *															IA/P = 0.10			
0.0	12	15	19	25	27	31	37	45	57	75	97	122	151	203	231	238	213	182	150	115	91	70	54	44	37	30	26	23	18	15	12	1
.10	10	13	17	21	23	26	29	34	41	52	67	87	111	165	210	233	227	205	173	131	102	77	58	46	39	32	27	24	19	15	12	1
.20	10	13	16	21	23	25	28	32	38	47	61	78	100	152	199	228	231	210	181	138	107	80	59	47	39	33	28	24	19	15	12	2
.30	9	12	15	20	22	24	27	30	36	44	55	70	90	139	188	221	228	215	188	145	112	83	61	48	40	33	28	24	19	15	12	2
.40	8	11	14	18	19	21	23	25	29	33	40	50	64	103	152	196	223	226	208	166	127	92	66	52	42	35	30	25	20	16	13	2
.50	8	10	13	17	18	20	22	24	27	31	37	46	58	93	140	186	216	224	212	173	133	96	69	53	43	36	30	26	20	16	13	3
.75	6	8	11	14	15	16	18	19	21	23	26	31	36	55	87	130	173	205	217	202	165	119	81	60	48	39	33	28	21	17	13	4
1.0	5	6	8	11	12	13	14	15	16	18	19	21	24	31	46	71	109	151	189	214	200	153	102	72	55	44	37	31	23	18	13	6
1.5	3	4	5	7	8	9	10	11	11	12	14	15	16	19	24	31	45	69	103	159	207	207	147	99	71	54	44	36	26	20	14	8
2.0	1	2	3	5	5	6	6	7	8	9	9	10	11	13	16	19	23	31	45	81	132	189	199	142	97	69	53	43	30	22	15	10
2.5	0	1	1	2	3	3	4	4	4	5	6	6	7	8	10	12	14	17	20	31	53	107	179	194	147	102	73	55	36	26	16	10
3.0	0	0	1	1	1	2	2	2	3	3	3	4	4	5	7	8	10	11	13	18	26	51	116	178	188	142	100	71	43	30	18	11
IA/P = 0.30														* * * TC = 1.5 HR * * *															IA/P = 0.30			
0.0	0	0	0	0	0	1	2	5	11	22	38	59	84	138	180	200	195	176	154	125	105	86	71	60	52	44	39	34	27	23	19	2
.10	0	0	0	0	0	0	0	1	4	9	18	31	50	99	149	184	198	190	170	139	115	93	75	63	55	47	40	35	28	23	19	3
.20	0	0	0	0	0	0	0	1	3	7	14	25	41	86	137	176	195	192	175	144	119	95	76	64	55	47	41	36	28	24	19	3
.30	0	0	0	0	0	0	0	0	1	2	5	11	21	53	100	147	181	194	187	159	130	103	81	68	58	50	43	37	29	24	19	4
.40	0	0	0	0	0	0	0	0	0	2	4	9	17	45	88	136	172	192	189	164	135	106	83	69	59	51	43	38	30	24	20	4
,50	0	0	0	0	0	0	0	0	0	0	1	3	7	23	56	101	145	177	190	177	149	116	89	73	62	53	45	39	31	25	20	6
.75	0	0	0	0	0	0	0	0	0	0	0	1	3	13	35	71	113	151	176	184	162	128	97	77	65	55	48	41	32	26	20	7
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	8	24	53	92	132	174	182	153	114	88	72	61	52	45	34	27	21	10
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	9	24	50	103	152	175	148	112	87	71	60	51	38	30	22	14
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	6	24	62	127	170	151	116	89	73	61	45	34	23	16
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	16	58	131	165	146	113	88	72	52	39	25	16
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	16	67	134	161	142	111	87	60	44	27	17
IA/P = 0.50														* * * TC = 1.5 HR * * *															IA/P = 0.50			
0,0	0	0	0	0	0	0	0	1	2	6	13	22	34	65	94	114	129	122	117	108	100	91	81	74	66	58	52	46	38	33	28	2
.10	0	0	0	0	0	0	0	0	0	2	5	10	18	42	73	99	116	126	121	112	104	94	84	76	68	60	53	48	39	33	28	3
.20	0	0	0	0	0	0	0	0	0	1	4	8	15	36	65	93	112	123	122	113	105	95	85	77	69	61	54	48	39	34	28	4
.30	0	0	0	0	0	0	0	0	0	0	1	3	6	20	44	72	97	114	122	118	109	99	88	79	71	63	56	50	41	34	28	5
.40	0	0	0	0	0	0	0	0	0	0	1	2	5	16	38	65	91	110	121	119	110	100	89	80	72	64	57	51	41	34	29	5
.50	0	0	0	0	0	0	0	0	0	0	0	1	4	13	33	59	85	105	118	120	112	101	90	81	73	65	57	51	41	35	29	6
.75	0	0	0	0	0	0	0	0	0	0	0	1	2	7	20	41	66	89	107	118	115	105	93	83	75	67	60	53	43	35	29	8
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	14	31	53	78	106	117	113	100	89	80	72	64	57	46	37	30	12
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	14	29	60	91	115	111	99	88	79	71	63	50	41	31	18
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	20	45	85	113	109	98	87	78	70	56	45	32	22
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	13	42	87	111	107	97	86	78	62	49	34	24
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	39	82	109	109	98	87	70	56	37	25

RAINFALL TYPE = III   * * * TC = 1.5 HR * * *   SHEET 9 OF 10

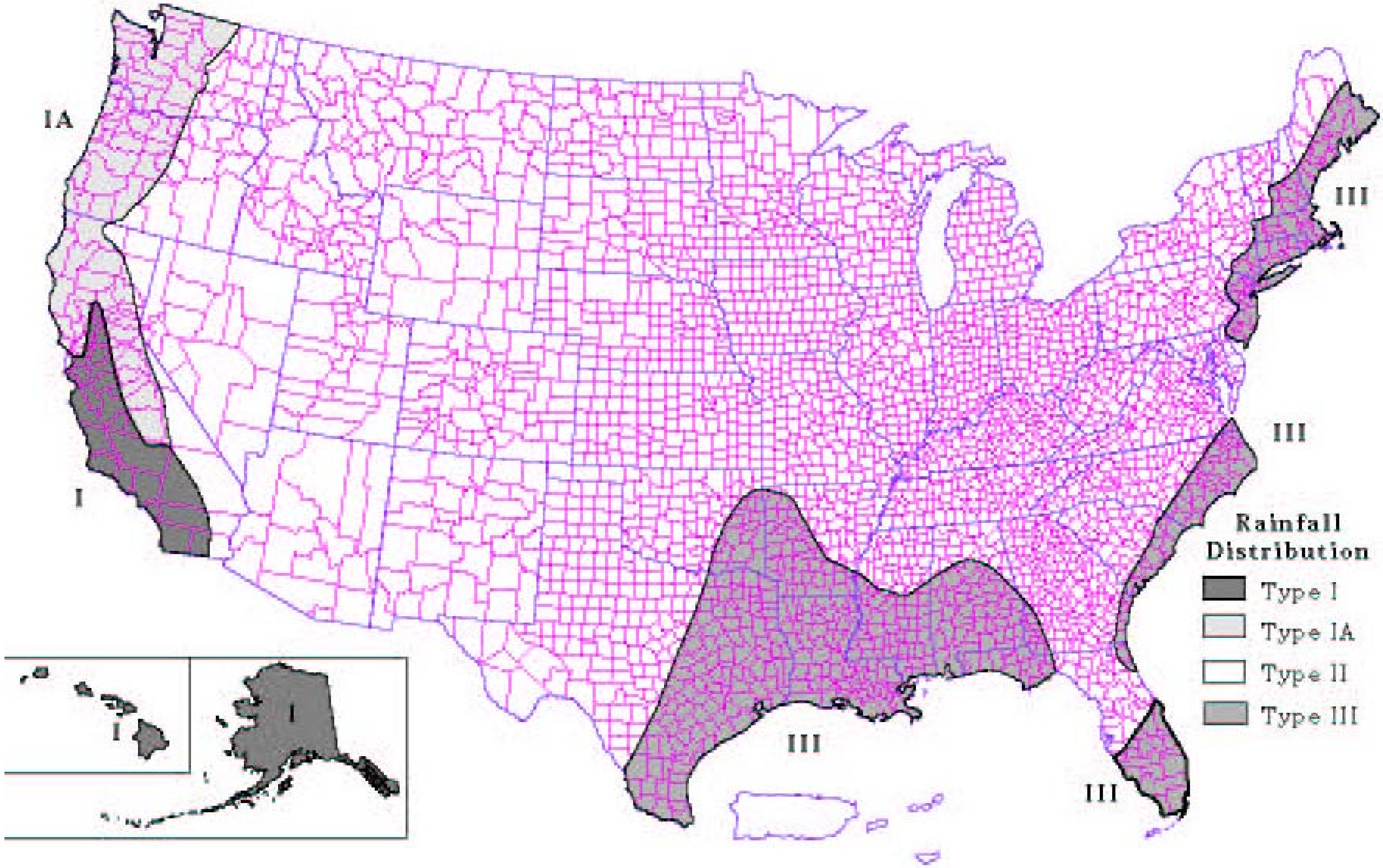
Exhibit 5-III: Tabular hydrograph unit discharges (csm/in) for type III rainfall distribution—continued

HYDROGRAPH TIME (HOURS)

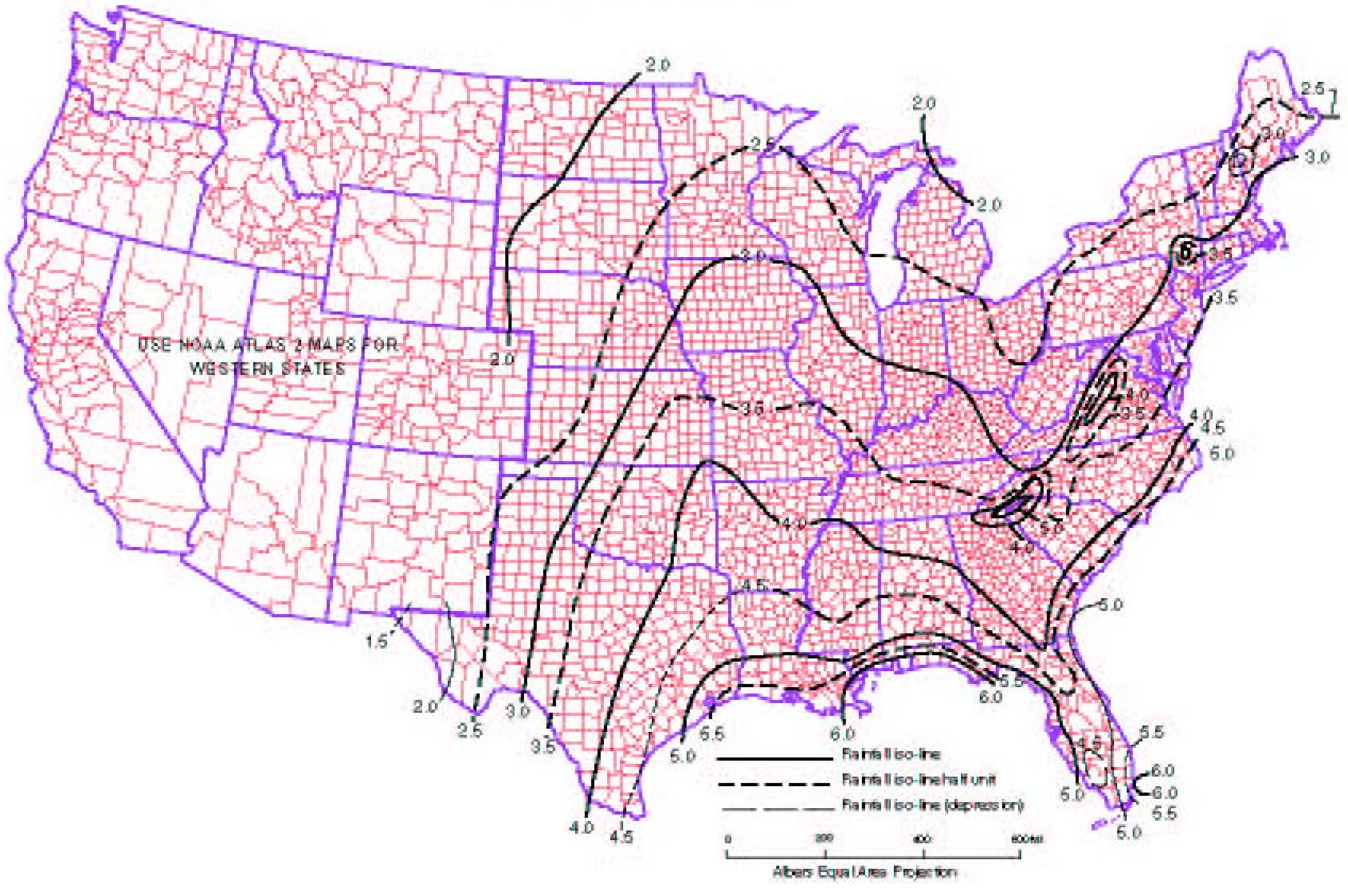
TRVL TIME (hr)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
IA/P = 0.10														* * * TC = 2.0 HR * * *													IA/P = 0.10					
0.0	9	11	15	19	21	23	27	31	39	48	60	75	91	129	164	187	200	191	178	147	119	92	69	55	45	37	31	26	20	16	13	3
.10	8	10	13	17	18	20	22	25	29	36	44	55	68	101	139	170	189	197	188	163	132	101	75	59	47	39	33	28	21	17	13	3
.20	7	10	13	16	18	19	21	24	28	33	40	50	62	93	129	162	185	195	190	168	137	104	77	60	48	40	33	28	21	17	13	4
.30	7	9	12	15	17	18	20	23	26	31	37	46	56	85	120	154	179	193	191	172	142	108	80	62	49	41	34	29	21	17	13	4
.40	6	8	11	14	15	16	18	19	22	25	29	34	42	64	94	129	161	183	192	183	157	120	87	66	53	43	36	30	22	17	13	5
.50	6	8	10	13	14	16	17	19	21	23	27	32	38	58	87	121	153	177	191	185	162	124	90	68	54	44	36	31	22	18	13	5
.75	5	6	8	11	12	13	14	15	16	18	20	23	26	37	55	81	113	144	169	186	179	147	106	78	61	49	40	33	24	19	14	6
1.0	4	5	7	9	10	11	12	13	14	15	17	18	20	27	38	56	82	113	143	176	185	164	121	88	67	53	43	36	26	20	14	7
1.5	2	3	4	5	6	7	7	8	9	10	10	11	12	15	18	23	31	45	65	106	148	180	166	126	92	69	55	44	31	23	15	9
2.0	1	1	2	3	4	4	5	5	6	6	7	8	8	10	12	14	18	23	31	52	87	139	176	160	122	90	68	54	36	26	16	10
2.5	0	0	1	2	2	2	2	3	3	4	4	5	5	6	7	9	11	13	16	22	36	71	132	172	161	126	94	71	45	31	18	11
3.0	0	0	0	1	1	1	1	1	2	2	2	3	3	4	5	6	7	9	10	14	19	35	78	136	168	156	123	92	54	36	20	11
IA/P = 0.30														* * * TC = 2.0 HR * * *													IA/P = 0.30					
0.0	0	0	0	0	0	0	1	2	6	11	18	29	41	75	111	140	159	170	163	145	124	103	84	70	60	51	44	39	30	25	20	4
.10	0	0	0	0	0	0	0	1	2	4	9	15	24	50	84	118	145	160	167	155	134	110	89	74	63	54	46	40	31	25	20	5
.20	0	0	0	0	0	0	0	0	1	3	7	12	20	43	76	110	138	157	165	157	138	113	91	75	64	55	47	41	32	26	20	5
.30	0	0	0	0	0	0	0	0	1	3	5	10	17	38	68	101	131	152	164	159	141	116	93	77	65	56	48	41	32	26	20	6
.40	0	0	0	0	0	0	0	0	0	1	2	4	8	22	45	76	109	137	155	163	151	125	99	81	68	58	50	43	33	26	20	7
.50	0	0	0	0	0	0	0	0	0	1	1	3	7	18	39	69	101	130	151	162	153	128	101	83	69	59	51	44	34	27	20	7
.75	0	0	0	0	0	0	0	0	0	0	0	1	2	6	17	36	63	93	122	151	158	143	114	92	76	64	55	47	36	28	21	9
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	7	18	37	63	93	132	157	153	125	100	82	68	58	50	38	29	21	11
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	12	26	59	100	142	154	128	102	83	70	59	44	34	23	15
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	18	43	93	142	150	125	101	82	69	50	38	24	16
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	12	41	98	141	147	122	99	81	58	43	26	17
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	38	92	135	144	124	102	69	50	30	18
IA/P = 0.50														* * * TC = 2.0 HR * * *													IA/P = 0.50					
0.0	0	0	0	0	0	0	0	0	1	3	6	11	16	33	54	75	91	102	114	114	103	96	87	79	72	64	57	51	41	35	29	6
.10	0	0	0	0	0	0	0	0	1	2	5	9	14	29	49	70	87	99	106	113	104	97	88	80	72	65	58	52	42	35	29	6
.20	0	0	0	0	0	0	0	0	0	1	2	4	7	18	34	54	74	90	101	112	107	99	90	82	75	67	60	53	43	36	29	8
.30	0	0	0	0	0	0	0	0	0	0	1	3	6	15	30	49	69	86	98	111	108	100	91	83	75	68	60	54	43	36	29	8
.40	0	0	0	0	0	0	0	0	0	0	0	1	2	8	19	35	55	73	89	105	110	103	94	86	78	70	62	56	45	37	30	10
.50	0	0	0	0	0	0	0	0	0	0	0	1	2	6	16	31	50	69	85	102	109	104	95	86	78	71	63	56	45	37	30	11
.75	0	0	0	0	0	0	0	0	0	0	0	0	1	4	10	21	37	55	73	93	107	107	97	89	81	73	65	58	47	38	30	13
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	7	15	29	46	72	93	107	103	94	86	78	70	62	50	40	31	16
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	7	15	34	59	89	105	101	93	85	77	69	55	44	32	21
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	10	25	55	90	104	100	92	84	76	61	49	34	23
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	7	24	59	91	103	99	91	83	68	54	36	25
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	28	62	91	101	98	90	75	60	39	26

RAINFALL TYPE = III * * * TC = 2.0 HR * * * SHEET 10 OF 10

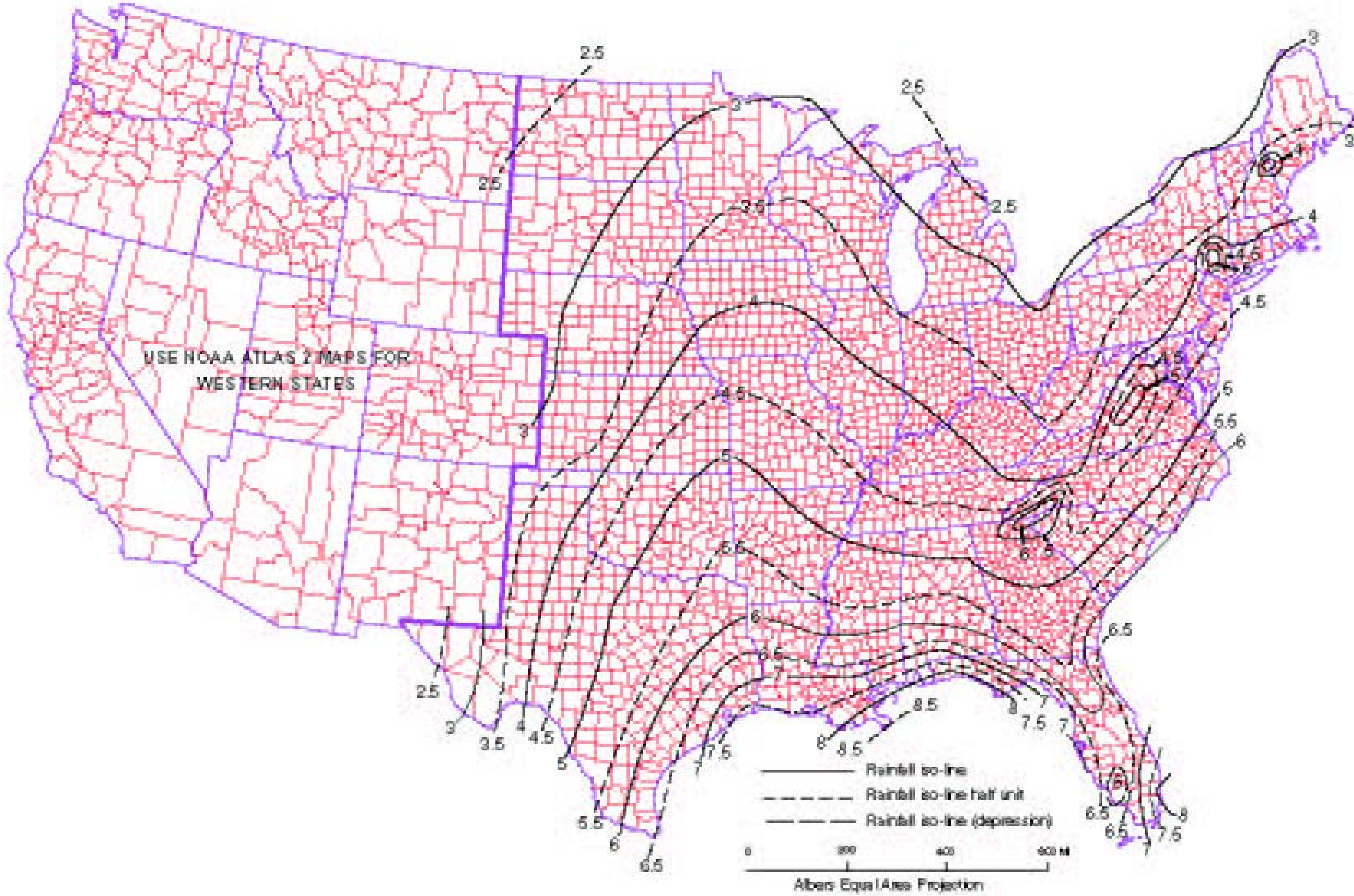
Appendix C-2. Rainfall Distribution for the US (from TR-55, SCS, and TP-40)

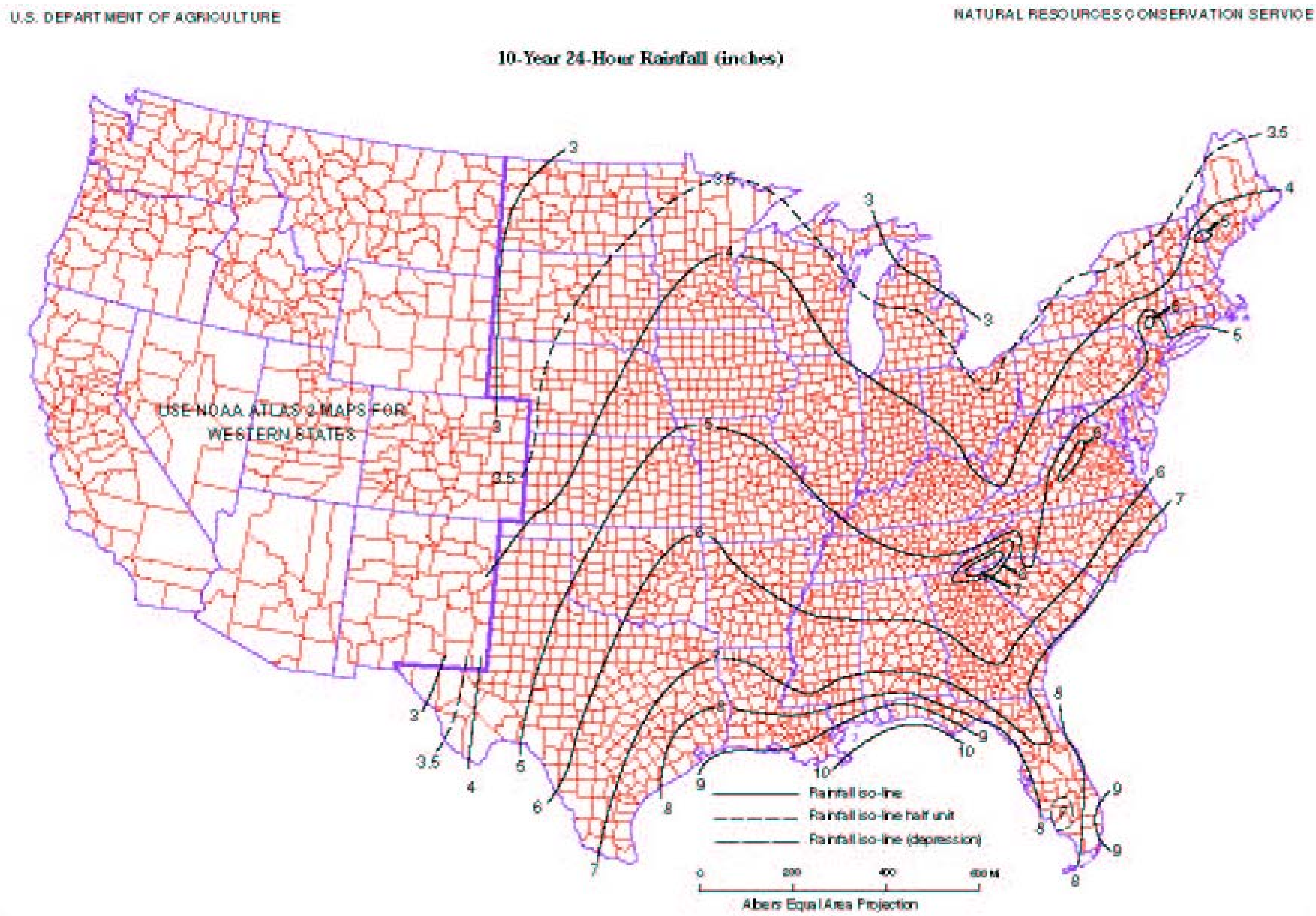

IA
I
III
III
III
III
Rainfall Distribution
Type I
Type IA
Type II
Type III
I
I

2-Year 24-Hour Rainfall (inches)



5- Year 24- Hour Rainfall (inches)

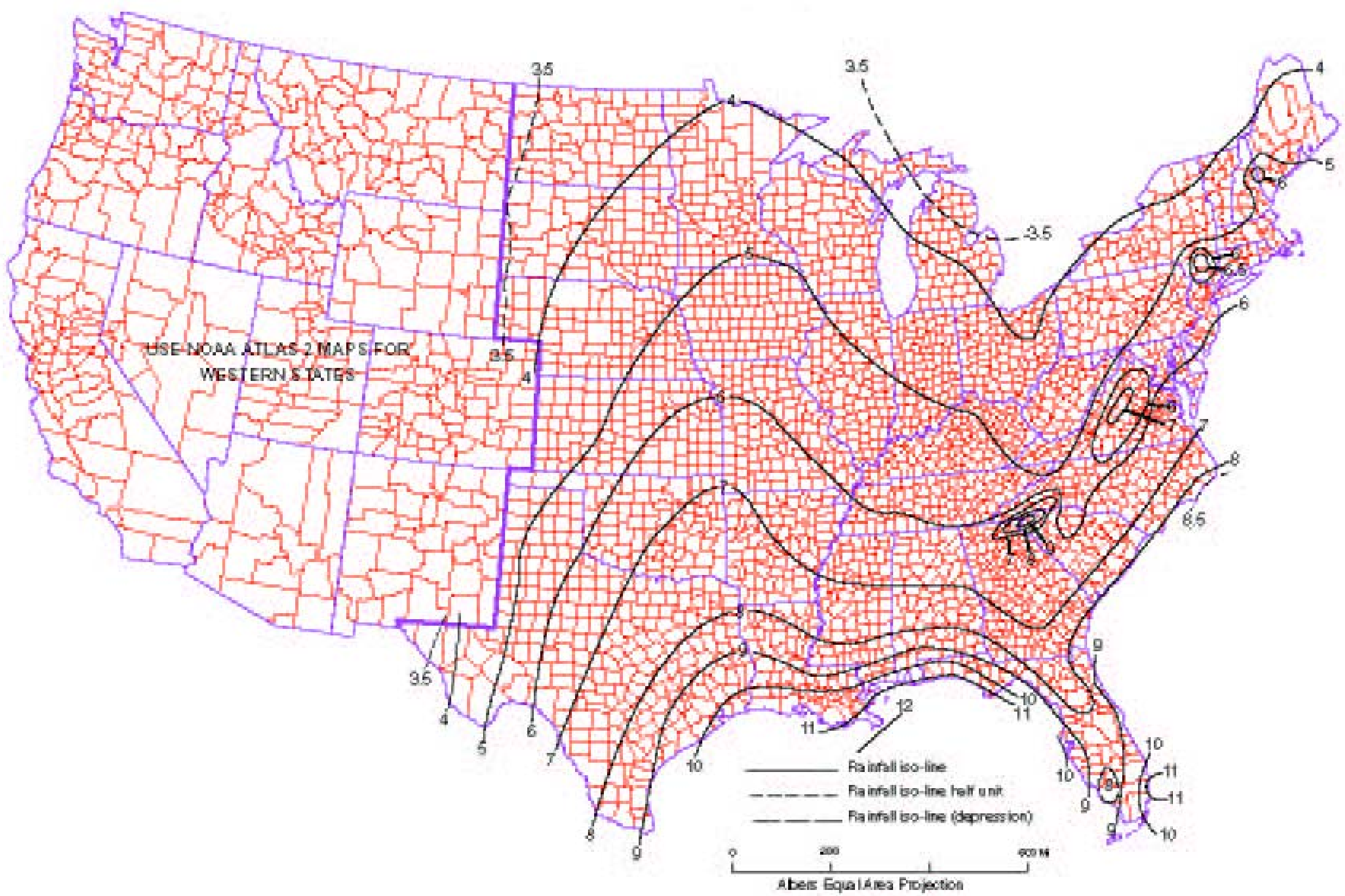



U.S. DEPARTMENT OF AGRICULTURE
NATURAL RESOURCES CONSERVATION SERVICE
10-Year 24-Hour Rainfall (inches)
USE NOAA ATLAS 2 MAPS FOR WESTERN STATES
Rainfall iso-line
Rainfall iso-line half unit
Rainfall iso-line (depression)
Albers Equal Area Projection

U.S. DEPARTMENT OF AGRICULTURE

NATURAL RESOURCES CONSERVATION SERVICE

25-Year 24- Hour Rainfall (inches)



U.S. DEPARTMENT OF AGRICULTURE

NATURAL RESOURCES CONSERVATION SERVICE

50-Year 24-Hour Rainfall (inches)