R. Pitt
June 18, 2004

Module 6: Temporary Ponds for Construction Site Sediment Control

Introduction

The use of temporary ponds for sediment control is a common practice at many construction sites. In some cases, these ponds are re-built after the construction period and used as permanent ponds for stormwater control. However, in many cases, they are filled in and their area used as part of the land development. Because sediment ponds have relatively short lives, their design criteria and construction methods differ from more permanent stormwater control ponds. The particle trapping mechanisms are the same for both types of ponds, but the influent hydrology and particle size distributions can be substantially different. The following discussion therefore stresses the special features of temporary sediment control ponds for construction sites. Also discussed are filter fences, for two reasons: 1) small drainage areas are usually controlled using filter fences, while large areas require sediment ponds, they are therefore complementary practices with similar objectives, and 2) filter fences remove sediment from the flowing water in much the same way as sediment ponds, by sedimentation (not “filtration”).

Temporary construction site sediment ponds have sediment loads that are very large while the particulates can be very small. Sizeable accumulations of sediment can therefore occur in short periods of time. Due to the lack of protection from scour, dry detention ponds have much smaller removal benefits than wet ponds (having at lest 3 ft. of standing water). If well designed and properly maintained, suspended solids removals of 70 to 90% can be obtained in wet ponds, while dry ponds seldom provide more than 30% suspended solids reductions.

There are a number of basic design guidelines needed to maximize sediment removal and to minimize potential problems in ponds, including:

- Need at least three feet of permanent standing water over most of the pond to protect sediments from scouring. Additional depth is also needed for sediment storage between cleanout operations.
- Ideally, the pond 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.
- Correct pond side slopes are very important to improve safety and to minimize mosquito problems. An underwater shelf near the pond edge needs to be planted with rooted aquatic plants to hinder access to deep water, if the pond will be in place for several years. Short-term temporary ponds commonly used at construction sites will not enable vegetation to become established.
- 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.
- Protect the inlet and outlet areas from scour erosion and cover the inlets and outlets with appropriate safety gratings. Provide an adequate emergency spillway.

Basic pond design guidelines must also be followed to provide the expected level of sediment removal. The following list is a typical example of these 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 and the Corps of Engineers (SCS 1982) must be followed.
- Pond size is dictated mostly by desired particle control and water outflow rate. For construction sites, the pond water surface should be about 1.5% of the watershed area draining to the pond for approximately 90% suspended solids reductions. If the pond area is only about 0.5% of the drainage area, the resulting removal would be about 65%, or less, of suspended solids. The use of chemicals can increase the removal of sediment in ponds. In an early example, Colston (1974) used alum to increase suspended solids and turbidity removals up to about 85 to 97 percent. More recent examples show similar removal benefits when using chemical-assisted sedimentation.

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 methods.

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. Unfortunately, landscaping is not very effective for temporary pond installations, so pond side slopes are most critical. 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).

Gradual slopes near the water edge and a submerged ledge close to shore are usually the best solution to maximize safety. 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. 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 6-1. 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 to keep trash from interfering with the outlet structures operation.

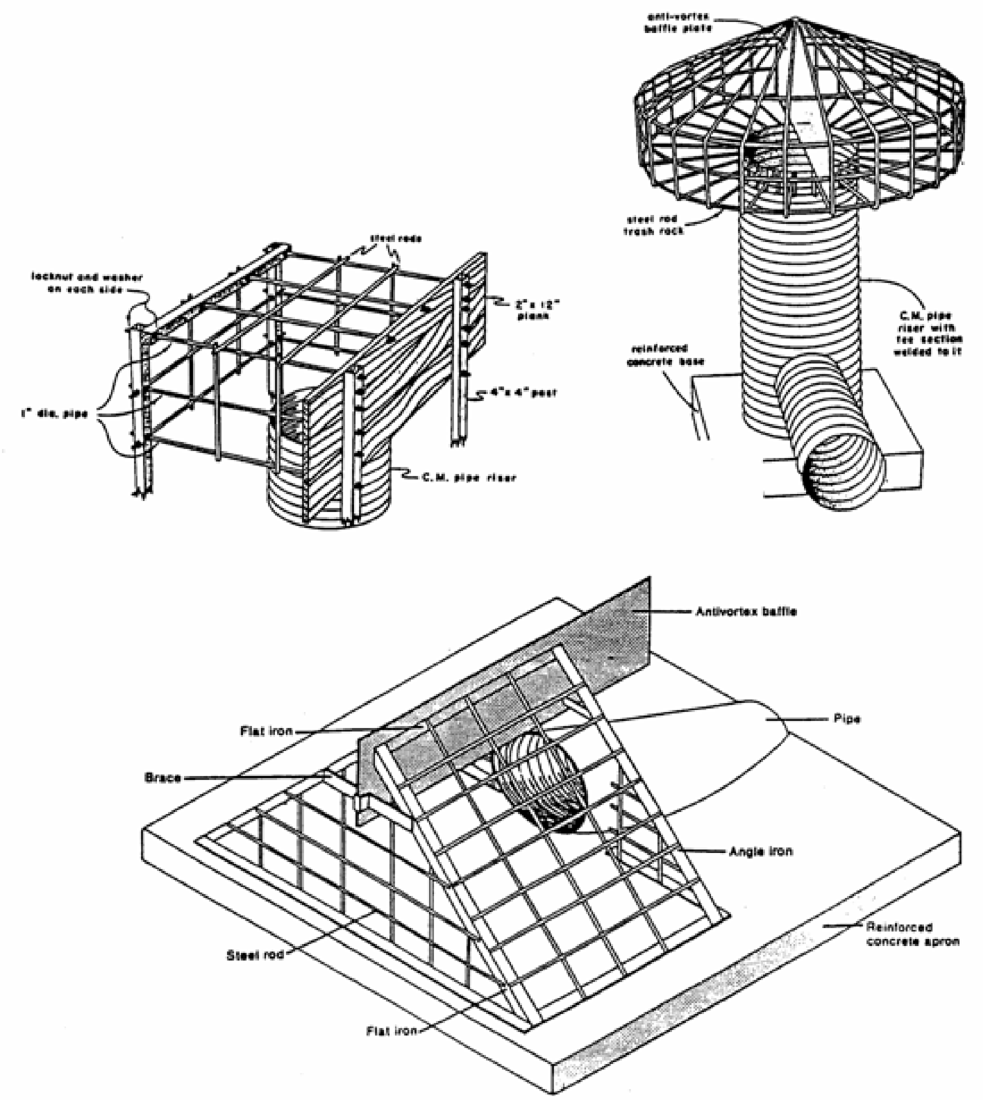


Figure 6-1. Various trash racks and baffles used by the SCS (NRCS). (SCS 1982).

Eccher (1991) lists the following pond attributes to ensure maximum safety:

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 sloped 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.

Maintenance Requirements of Wet Detention Ponds

The most important maintenance for temporary construction site erosion ponds is to conduct periodic inspections and to make sure that the sediment accumulation is not excessive and prematurely filling the pond.

Temporary sediment ponds need to be inspected after each major storm. The inspection should include checking the pond embankments for subsidence and erosion. 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.

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.

During major storms, construction site erosion ponds can literally fill up during a single storm. Most of the sedimentation would occur near the inlet and the resulting sediment accumulation would be very uneven throughout the pond. Normally, sediment removal in a permanent wet pond may be needed about every five to ten years, but may be needed every few months at construction sites. It is therefore necessary to plan for required maintenance during the design and construction of sediment ponds. Ease of access of heavy equipment and the possible paving of a sediment trap near the inlet would ease maintenance problems. 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 top soil 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.

Guidelines to Enhance Pond Performance

The Natural Resources Conservation Service (SCS 1982) has prepared a design manual that addresses specific requirements for such things as anti-seep collars around outlet pipes, embankment widths, types of fill required, foundations, emergency spillways, etc., for a variety of wet detention pond sizes and locations. That manual must be followed for detailed engineering requirements. The Alabama Soil and Water Conservation Committee, Natural Resources Conservation Service, Montgomery, AL, also has prepared the Alabama Handbook for Erosion Control (1993; currently being updated) that describes the construction and maintenance of sediment basins, and many other practices.

Pond Surface Area and Shape

Surface area is one of the most important design considerations for particle removal. 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. Short-circuiting in adequately sized ponds has little detrimental effect on pond performance. However, it can be serious in under-sized ponds. Stagnation is a much more serious problem degrading pond water quality.

Pond Water Depth

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.

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.

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). Sclueler (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 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 create a barrier making unwanted access to deep water difficult for permanent ponds.

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 (such as a floating wier), 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 (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 foe all pond stages).

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 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.

Detention Pond Design Fundamentals
The basic design approaches for wet detention ponds consider either slug flow or completely mixed flow. Martin (1989) reviewed 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 assume 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 (the flushing ratio). 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 moving water in the pond, short-circuiting may reduce the available pond storage volume (and therefore the resident time), with less effective treatment.


Sediment pond at landfill


Permanent pond acting as sediment trap during final construction


Series of small sediment pond at complex construction site (Atlanta, GA)


Temporary pond at highway construction site in area where hauling trucks are washed prior to re-entering roads (WI)

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 6-2.

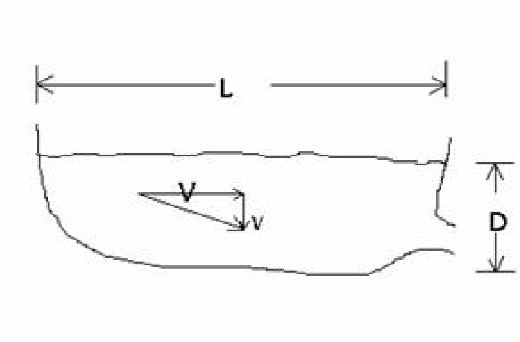


Figure 6-2. Critical Velocity and Pond Dimensions

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:

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. For construction site sediment 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 (as verified during field tests).

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

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

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,

leaving:

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

It is seen that the surface 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 surface overflow rate relationships for variable flow stormwater conditions.

The surface overflow rate (the 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 surface overflow rate (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:

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 settling velocity of v_o
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 (or surface overflow rate) developed previously. 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 6-3 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%.

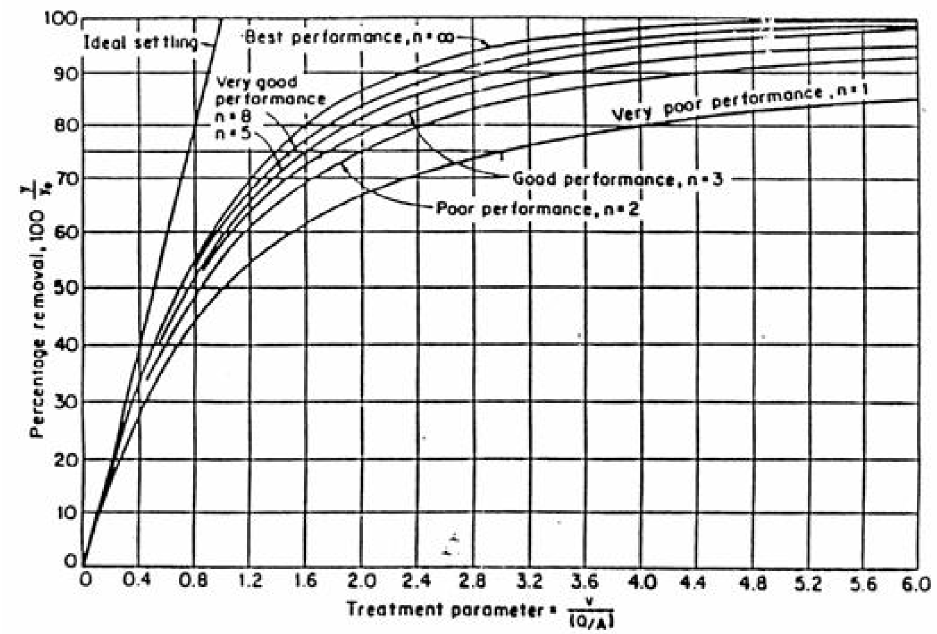


Figure 6-3. Performance curves for settling basins of varying effectiveness (AWWA 1971).

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 the 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 6-4 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.

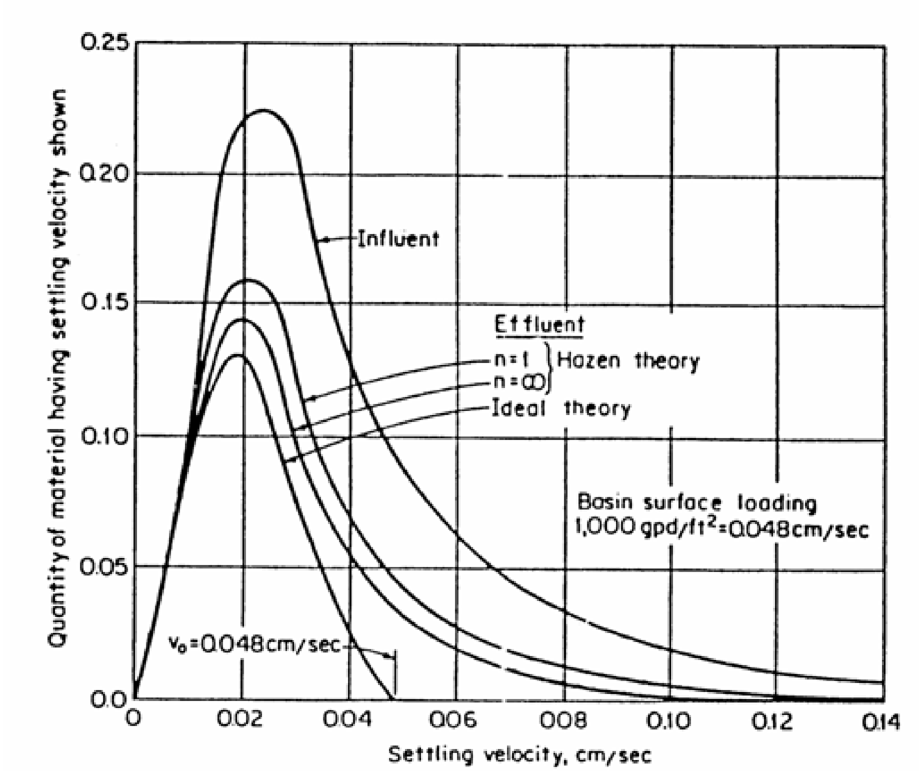


Figure 6-4. Influent and effluent particle settling rate distributions for settling basins of varying effectiveness (AWWA 1971).

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.

Seven events were studied at the Madison, WI, Monroe St, wet detention pond 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 are always a small number of these larger particles. It is generally assumed that short-circuiting is responsible for these large particles. 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.

Residence Time and Extended Detention Ponds

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 runoff based on residence time alone 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.

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. A wet detention pond located immediately upstream of 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 which should be designed for peak flow rate reductions.

The previous discussion on upflow velocity as a design criteria illustrated 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 wet detention ponds during 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.

Runoff Particle Size Distributions

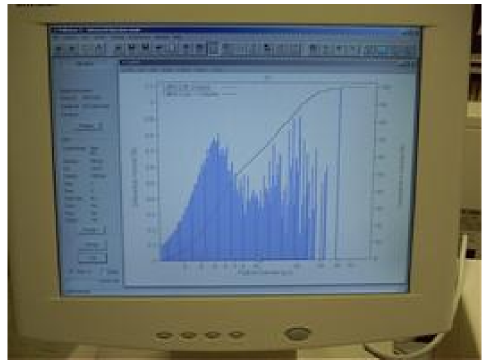
Knowing the settling velocity characteristics associated with stormwater particulates is necessary when designing wet detention ponds. Particle size is directly related to settling velocity (using Stokes law, for example, and using appropriate shape factors, specific gravity and viscosity values) and 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. Probably the earliest description of conventional particle settling tests for stormwater samples was made by Whipple and Hunter (1981).

Cascading sieves (with total solids analyses after each sieve)

Andreseen pipette (miniature settling column)

Coulter Counter Multi-Sizer II2

Coulter Counter Multi-Sizer 3

Multi-Sizer 3 aperture tube and stirrer	 Multi-sizer 3 computer display of particle size distribution
 Pipette for high solids loadings	Field turbidimeter
	Research light microscope with automatic video analyses of particles

Teflon settling column

Different Methods to Characterize Particle Size and Settling Rates

Whipple and Hunter (1981) contradict the assumption sometimes used in modeling detention pond performance that pollutants generally settle out in proportion to their concentrations (first-order rate equations). 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.

Figure 6-5 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 6-5. Particle size distributions for various stormwater sample groups.

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, including inlet bedload contributions. 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.
Figure 6-6 shows the particle size distribution for the inflow events, including bedload, for a series of about 50 runoff events at the Monroe St. detention pond in Madison, WI. 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 material 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 6-6. Inlet particle size distributions observed at the Monroe St. wet detention pond.

Limited data are 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. 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

In addition to high rain energy, many Alabama soils are also highly erosive and result in construction site runoff that is very difficult to control. Based on about 70 construction site erosion samples collected in the Birmingham area (Nelson 1996; Pitt 1998), the characteristics of this runoff include:

- Measured suspended solids concentrations ranged from 100 to more than 25,000 mg/L (overall median about 4,000 mg/L).
- Turbidity ranged from about 300 to >50,000 NTU, with an average of about 4,000 NTU
- Particle sizes: 90% were smaller than about 20 µm (0.02 mm) in diameter and median size was about 5 µm (0.005 mm).
- Measured Birmingham construction site erosion discharges range from about 100 to 300 tons/acre/year

There were obvious relationships between rain conditions and the observed runoff quality during these local Birmingham studies:

Measured conditions:	Low intensity rains (<0.25 in/hr)	Moderate intensity rains (about 0.25 in/hr)	High intensity rains (>1 in/hr)
Suspended solids, mg/L	400	2,000	25,000
Particle size (median), µm	3.5	5	8.5

Nelson 1996 and Pitt 1998

These construction site data would therefore correspond to the "low," or "all NURP" particle size distributions. The particle size distribution of material leaving construction sites is therefore quite small and hard to control. Small particle sizes are much more difficult to remove by most erosion control strategies commonly employed that usually employ sedimentation (sediment ponds and "filter" fences). Settling velocities (or particle sizes) are used with the outflow rate to determine the required surface area for a sediment pond.

These data show that construction site runoff likely has smaller particle size distributions than most stormwater; construction site runoff has median sizes generally in the range of 3 to 8 µm, while stormwater at many locations has larger particles, with median sizes from about 8 to 65 µm.

Particle Settling Velocities

The settling velocities of discrete particles are shown in Figure 6-7, based on Stoke's and Newton's settling relationships. Probably more than 90% of all runoff 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, while construction site runoff particles would be closer 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 III) 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 used. Sieve measurements are limited to sizes greater than about 20 µm, although precision membrane filters can be used for much smaller sizes.

Figure 6-7. Type 1 (discrete) settling of spheres in water at 10° C (Reynolds 1982).

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.

Design Based on NURP Detention Pond Monitoring Results

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 relatively 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 6-8, 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 6-8. Regional differences in detention pond performance (EPA 1983).

Introduction to Storage-Indication Method

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 flows of pollutants may be more appropriate than monitoring simple paired samples of inlet and effluent flows during random events spread over time.

The following discussion on routing includes a fairly simple 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 (www.WinSLAMM.com). 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:

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.

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 6-9 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 6-9. Pond-stage surface area relationship for example problem.

Table 6-1 shows the calculations used to produce the storage-indication figure (Figure 6-10) for the Monroe St. pond. This example reflects some pond modifications that were made to enhance pond performance: 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 10 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 6-1).

Table 6-1. 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] $S + \frac{1}{2} O \Delta t = S + O (\frac{1}{2} \Delta t) = S + 300 (O)$
$\Delta t = 600$ seconds

Figure 6-10. Pond-stage/storage indication curve for example problem.

Design of Wet Detention Ponds for the Control of Construction Site Sediment

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 site engineer 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 (available at www.WinSLAMM.com). These guidelines should therefore be considered as a starting point and modified for specific local conditions. As an example, it may be desirable 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 and other research. For construction sites, these required pond areas are 1.5% of the drainage area for approximately 90% control (arbitrary 5 µm) and 0.5% for 65% control (arbitrary 20 µm). If any undeveloped areas are in the pond drainage, the pond area would have to be increased in area by about 0.6% of those areas. Similarly, if any paved areas were in the drainage, the increase in pond area would need to be 3% of the paved area. Obviously, to be most efficient, any extra drainage areas should be kept to a minimum.

The following table shows how the pond area can be estimated based on drainage area characteristics:

	Land area	Pond size factor	Resulting pond area
Paved area	0.6 acres	3%	0.018 acres
Undeveloped area	3.8 acres	0.6%	0.023 acres
Construction area	27.6 acres	1.5%	0.414 acres
Total:	32.0 acres		0.455 acres

As will be shown in the following example, the total land area needed for the pond will be substantially larger than this value, as this area is the pond surface area during dry weather. The pond freeboard volume (for water quality control), plus the emergency spillway area, will increase the needed area dedicated for the pond.

2) ***The pond freeboard storage*** should be equal to the runoff associated with a 1.25 inch rain from the drainage area 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, the opposite of what should probably occur. 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. About 0.5 inches of runoff would occur at construction sites for sandy soil areas and about 0.6 inches of runoff for clayey soil areas for this rain depth. Again, if other land areas are also in the drainage in addition to the construction area, the pond treatment volume would have to be increased. For any paved areas, the 1.25 inch rain would produce about 1.1 inches of runoff, and for undeveloped areas, the 1.25 inch rain would produce about 0.1 (for sandy soils) to 0.3 (for clayey soils) inches of runoff.

The following table shows how the pond storage volume can be estimated based on drainage area characteristics (assuming clayey soil conditions):

	Land area	Pond WQ volume factor	Resulting pond WQ volume
Paved area	0.6 acres	1.1 inches	0.66 acre-inches
Undeveloped area (clayey)	3.8 acres	0.3 inches	1.14 acre-inches
Construction area (clayey)	27.6 acres	0.6 inches	16.56 acre-inches
Total:	32.0 acres		18.36 acre-inches (1.53 acre-ft)

Figure 6-12 is a schematic showing a cross section of the pond. The area below the invert of the lowest discharge 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 analyses 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 stages of the emergency spillway. In addition, the dead storage area must be provided to minimize scour and to provide sediment storage. At least 3 ft of water must be over the maximum stored sediment depth.

Figure 6-12. Cross-section of pond showing water quality storage portion

3) The selection of the ***outlet devices*** for the wet detention pond (primary water quality device plus emergency spillway). 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 total storage values below each elevation (assuming the noted surface areas for the shallower elevations):

	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 large 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.

Tables 6-2 through 6-5 provide a quick method of selecting appropriate outfall devices for a potential pond location. These tables indicate the minimum amount of pond surface area needed at each stage to provide a five µm critical control level for a variety of conventional outfall devices. Table 6-5 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, for example. 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 6-2. Surface Area Requirements for 5-µm Particle Size Control for Various V-notch Weirs.

Head (ft)	Flow (cfs)	22.5°	Reqd. area (acres)	Flow (cfs)	30°	Reqd. area (acres)	Flow (cfs)	45°	Reqd. area (acres)

		Storage (ac-ft)			Storage (ac-ft)			Storage (ac-ft)	
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 6-3. 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 6-4. 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 6-5. 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

As an example, if a pond required 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 drop tubes from 8” to 24”

Obviously, all stage levels have to be examined and the device selected that provides the desired level of control at the most critical stage (usually at the deepest depth). In most cases, the outlet device that has the largest capacity that meets the discharge requirements should be used. Under-sized discharge devices would likely cause increased flows out the emergency spillway, causing an actual decrease in sediment trapping performance.

These procedures will result in the largest storms that do not enter the emergency spillway to have treatment levels at least equal to the critical particle size specified. As an example, the above calculations focus on the 5 μm particle, at least, being controlled at the highest stage depth of the primary outfall structures in order to provide an approximate worst-case 5 μm control (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 90th 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 90th percentile particle size is typically only 3 μm, or smaller.

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.

Vertical riser with inlet grate (MD photo)	
Temporary outlet made from timber placed at correct elevation and covered with plastic to protect spillway (Auckland Regional Council)	Vertical riser having multiple outlets and wrapped with geotextile fabric

Figure 6-11 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 6-11. SCS TR-55 plot used to size additional freeboard needed for emergency spillway.

The SCS methods can be used to size an emergency spillway. The pond is sized to provide the water quality benefits, and additional storage associated with the emergency spillway stage is taken as Vs in Figure 6-11. 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. 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:

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.
The following example illustrates a compound weir structure, having a drop tube for water quality control, plus a rectangular weir for the emergency spillway. In this example, $q_o = WQ_{out} + \text{emergency spillway}_{out}$

Rain depth for the emergency spillway design (P) = 8 inches
CN = 86; Ia = 0.0366
Therefore, the direct runoff (Q) = 6.2 inches and Ia/P = 0.041
Area (A_m) = 0.021 mi^2 (13.2 acres)
Tc = 20 min (0.3 hr)

The peak unit discharge rate from the tabular hydrograph method is 498 csm/in

The peak discharge is therefore: (498 csm/in)(0.021 mi^2)(6.2 in) = 63.7 ft^3/sec

Also, the volume of runoff for this event is: V_R = [(6.2 in)(13.2 ac)]/12 in/ft = 6.82 ac-ft

The pond surface area is 0.4 acres at the permanent pool depth (the elevation for the water quality outlet invert). Table 6-4 confirms that a 12 inch drop tube structure would work for this pond over a wide range of stage conditions, while providing a desired worst case 5 µm particle control. The outlet flow rate for this drop tube is almost constant for heads of 1 to 6 ft (2.2 ft^3/sec). The maximum desired discharge rate for this pond (for both the water quality outlet plus the emergency spillway) is given as 46.5 ft^3/sec. The ratio of the outlet to the inlet flow rate is therefore:

$q_o/q_i = 46.5/63.7 = 0.73$

The ratio of the storage volume (V_S) to the runoff volume (V_R), for Type II rains (from Figure 6-11) is 0.2, for this ratio of outlet to inlet peak flow rates. The length (L_W in feet) of a rectangular weir, for a given stage (H_W in feet) and desired outflow rate (q_o in ft^3/sec) can be expressed as:

$L_W = q_o/(3.2\ H_W^{1.5})$

The desired q_o for the rectangular weir is 46.5 – 2.2 = 44.3 ft^3/sec. If the maximum stage for the emergency spillway is 1 ft, then length for the emergency spillway is:

LW = 44.3ft^3/sec/[(3.2)(1 ft)$^{1.5}$] = 13.8 ft.

If the water quality outlet had a varying discharge rate for different stages (as is common), then the stage for that outlet must also be known so the actual discharge rate contribution from that outlet to the total discharge rate objective can be used in the calculation. As an example, a 45^o V-notch weir would be a suitable outlet for water quality control for this pond. This weir would provide 5 µm control up to about 1.4 feet of head, for a 0.4 acre pond (assuming the associated storage volume is adequate). At this stage, the discharge rate is about 2.5 ft^3/sec. With another foot of storage and stage for the maximum elevation of the emergency spillway (2.4 ft above the invert of the V-notch), the V-notch weir discharge rate would increase to about 10 ft^3/sec. The residual discharge objective for the emergency rectangular weir would therefore now be: 46.5 – 11 = 35.5 ft^3/sec, and the length for the emergency spillway would be:

LW = 35.5ft^3/sec/[(3.2)(1 ft)$^{1.5}$] = 11.1 ft.

This method is known to be conservative with resulting over-sized emergency spillways. A computer model should therefore be used to verify the performance of the desired pond configuration for a variety of storm conditions.

4) The ponds must also be constructed according to specific design guidelines to insure the expected performance and adequate safety, such as provided by the US Bureau of Reclamation (1987). The guidelines need to specify such things as pond depth, side slopes, and shape.

Example Pond Design for Construction Site Sediment Control

Table 6-6 shows the conditions for an area on a construction site that needs a sediment pond. The drainage area, 53 acres, is mostly active construction site, but some undeveloped land and paved areas also drain to the pond location. The pond therefore needs to be enlarged to accommodate the additional runoff from these areas. The table shows the percentage of the drainage area needed to be used a pond, along with the pond volume to obtain approximately 90% suspended solids reduction.

Temporary construction site pond filled with sediment

Table 6-6. Size of Pond for Construction Area

	Area (acres)	% of area needed for pond surface	Pond surface area (acres)	Water quality volume (inches of runoff)	Pond volume (acre-inches)
Construction area	37	1.5%	0.56	0.6	22.2
Undeveloped area	14	0.5	0.07	0.3	4.2
Paved area	2	3.0	0.06	1.1	2.2
	53		0.69		28.6

The total water quality volume (“live storage”) of the pond is 28.6 acre-inches, or 2.38 acre-ft. The surface of the pond between events (during dry weather) is 0.7 acres, or about 1.3% of this drainage area. The top area of the pond and associated side slopes are calculated based on various assumed pond depths, as shown in a later example.

In this example, a pond depth of 3 ft, and approximate side slopes of 12% and a top area of 0.9 acres are used. An additional 1 ft of storage to accommodate an emergency spillway is also provided, with a maximum top area needed of about 1 acre. The selection of the main discharge device is based on the water surface at the top of this water quality volume. A 12 inch vertical riser pipe, having its opening at the normal pond water surface level, seems to be a good choice, based on Table 6-4 data.

Three feet of standing water is needed above the maximum sediment depth in order to minimize scour. In addition, sacrificial sediment storage must also be provided in the pond. Using RUSLE, the total construction period sediment load to the pond can be estimated. In the following example, it is assumed that the construction period is a half year, and the following conditions apply:

R = 350
LS = 4.95 (based on typical slope lengths of 600 ft at 10% slope)
k = 0.28
C = 0.25 (assuming that ¼ of the construction site area is being actively being worked, and the rest of the area is effectively protected)

The calculated unit area erosion loss for this construction period is therefore about 243 tons per acre per year. Since the construction period is one-half year and the area is 37 acres, the total sediment loss is estimated to be about 4490 tons. For a loam soil, this sediment volume is about 4600 yd^3, assuming the conventional conversion factor of tons x 1.02 = yd^3 for a loam soil. The pond area at the bottom of the 3 ft of standing water is assumed to be about ½ acre, requiring about 2 ft of sediment storage. Therefore, the following lists the pond areas for each depth increment:

Pond depth (ft)	Pond area (acres)
0	0
1	0.35
2	0.50
3	0.57
4	0.63
5	0.70
6	0.77
7	0.73
8	0.90
9	0.97

This design was then entered into WinDETPOND and evaluated. Table 6-7 shows some of the program results for this pond. A series of rains ranging from 0.01 to 4.0 inches was also used. The maximum pond stage is estimated to be about 7.4 ft for the 4 inch rain, more than a half foot below the broad-crested weir emergency spillway. The peak reduction factor (the reduction of the influent peak flow rate at the outfall) is very large for the small events, as expected, and still remains about 0.5 for the largest event. This will help reduce erosive flows to the receiving waters. The "event flushing ratio" indicates the volume of runoff compared to the water volume in the pond before the event. Again, this value is very small for the small events and increases to greater than 1 for rains larger than about 3 inches. The last 2 columns indicate sedimentation performance of the pond. The flow-weighted particle size in the effluent is greater than 4 µm after 3 inches of rain. However, the expected percentage suspended solids control (assuming the "low" particle size distribution) remains greater than 80% for all rains less than about 2 inches. The worst case shown, for the 4 inch rain, drops down to less than 40% control.

Table 6-7. Summarized Results from WinDETPOND to Evaluate Detention Pond at Construction Site

DETPOND for Windows Version 8.4.1

Pond file name: C:\PROGRAM FILES\WINDETPOND\EROSION CONTROL POND EXAMPLE.PND
Pond file description: This is an example of an erosion control pond
Rain file name: C:\Program Files\WinDetpond\BHAMSRCE.RAN
Date of run: 07-18-2002 Time of run: 22:59:47

Detention Pond Water Quality Performance Summary, by Event

Rain Number	Rain Depth (in)	Rain Duration (hrs)	Rain Intensity (in/hr)	Maximum Pond Stage (ft)	Event Inflow Volume (ac-ft)	Peak Reduction Factor (%)	Event Flushing Ratio	Flow-weighted Particle Size(Ideal)	% Part Solids Removed (Ideal)
1	0.01	3.00	0.00	5.00	0.000	1.00	0.000	0.0	100.0
2	0.05	7.00	0.01	5.00	0.002	0.99	0.001	0.0	100.0
3	0.10	8.00	0.01	5.01	0.007	0.99	0.003	0.1	99.8
4	0.25	10.00	0.02	5.07	0.052	0.99	0.022	0.1	99.5
5	0.50	12.00	0.04	5.19	0.137	0.97	0.059	0.3	98.9
6	0.75	14.00	0.05	5.30	0.230	0.94	0.099	0.5	98.2
7	1.00	14.00	0.07	5.42	0.342	0.90	0.147	0.7	96.7
8	1.50	14.00	0.11	5.64	0.610	0.85	0.262	1.2	88.5
9	2.00	14.00	0.14	5.87	0.939	0.78	0.403	1.8	80.2
10	2.50	14.00	0.18	6.26	1.528	0.67	0.656	2.9	68.1
11	3.00	14.00	0.21	6.64	2.266	0.57	0.973	4.0	57.2
12	4.00	14.00	0.29	7.37	4.014	0.50	1.724	6.5	39.1

As noted earlier in Chapters 3 and 4, most of the erosion potential is associated with the numerous moderate (greater than 1 inch) and the few large rains (up to 4 inches) that likely occur during the year. This pond will likely provide 65 to 95+% control for the moderate rains, but will drop off significantly for the largest rains. It is possible to improve the performance of the pond by changing the outlet weir to a smaller capacity device which would provide additional retention for the larger events. Table 6-8 illustrates how this temporary pond would affect the annual particulate solids losses from this construction site. The overall pond performance is expected to be about 75% effective, much less than the initial goal of 90% control. The performance of this pond could be improved if the design was better optimized for the larger, more erosive events. This could be done by choosing a more restrictive outlet device at higher pond stages and also providing more storage, for example.

Table 6-8. Performance of Temporary Sediment Pond at Construction Site (Birmingham rains)

Rain range (inches)	Mid Point Rain (inches)	% of annual R in category	% particulate solids removed for pond	Weighted total annual particulate solids removal (%)
0.01 to 0.05	0.03	0.0	100	0
0.06 to 0.10	0.08	0.1	100	0.1
0.11 to 0.25	0.18	0.7	99.8	0.7
0.26 to 0.50	0.38	3.5	99.5	3.5
0.51 to 0.75	0.63	4.8	98.9	4.7
0.76 to 1.00	0.88	8.2	98.2	8.1
1.01 to 1.50	1.26	16.1	96.7	15.6
1.51 to 2.00	1.76	15.4	88.5	13.6
2.01 to 2.50	2.26	10.9	80.2	8.7
2.51 to 3.00	2.76	7.5	68.1	5.1
3.01 to 4.00	3.5	16.3	57.2	9.3
over 4.01	5.67	16.5	39.1	6.5
4583 events	41.5 years	100.0		75.9 % annual particulate solids removal

Example Detention Pond Shape Calculations

The following discussion presents a calculation example assuming that the wet pond surface is 1.2 acres and the runoff volume for treatment is 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):

$r = (A/\pi)^{1/2} = [1.2\text{acres}(43{,}560\ ft^2 \text{ per acre})/\pi)]^{1/2} = 130$ 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 previously, 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^{o} V-notch weir, or two 22-1/2^{o} V-notch weirs.

Select emergency spillway (mandatory) and additional flood control storage volume (if necessary) using NRCS TR-55 (SCS 1986) procedures.

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, being an anonic polyacrylamides, 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.

<u>*Aluminum Coagulants*</u>
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 0.050 to 0.100 mg/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.

<u>*Field trials using solid floc blocs:*</u>
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 air. A forebay of about $10m^3$ upchannel of the flume was intended to trap bedload and other heavy material which 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 (2400 mg/L vs. 7300 mg/L). During other more moderate events, treated pond effluent concentrations were about 500 mg/L, compared to typical effluent concentrations of about 1000 to 2000 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 initially designed for the field tests. This system would have included a flow measurement weir or flume, an ultrasonic sensor and signal generating unit, and a battery driven dosing pump. The estimated cost was about $6,000, including installation and shelter, but 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. The following photos show an example of this system at a New Zealand construction site, including the main internal components.

Auckland Regional Council rainfall-driven chemical dosing system.

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. The above photo 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 6-9 presents the rainfall catchment area required for different PAC dose rates (at 10.1% Al_2O_3 by weight).

Table 6-9. 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 – 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. The following photo shows the typical multiple decant risers used at Auckland Regional Council sediment pond sites to allow more efficient settling of the floc.

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 time 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 6-10 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 shown in Table 6-10.

Table 6-10. 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

Example Design using Filter Fences for Construction Site Sediment Control

There are three aspects of filter fences that can be evaluated, as demonstrated in the following examples: 1) sediment capture behind the fence, 2) slowing water flowing down a slope, and 3) pressure forces on the fence from the water and resisting forces from the soil on the fence stakes. The first two aspects determine the erosion and sediment control benefits of filter fences, while the third aspect determines how filter fences may fail structurally.

Sediment flowing under hay bale barrier	Well-installed filter fabric fence, with bottom of fabric buried and backfilled to prevent underflow of sediment
	Hay bale barrier along edge of pavement
	Evidence of underflow erosion beneath improperly installed filter fabric fence

Holes in filter fabric fencing	Filter fabric fence installed close to construction area

Multiple rows of filter fabric fences and tree barriers to mark edge of undisturbed zone	
Large sediment load captured by filter fence, maximum load before needed maintenance (J. Voorhees photo)	Same site as prior photo showing sediment load overtopping filter fabric fence due to lack of maintenance (J. Voorhees photo)

Sediment Capture behind Filter Fences

Relatively few field investigations have been conducted to examine the effectiveness of filter fences, and other controls, at construction sites. Important tests have been performed by Barrett, *et al*. (1995), Horner, *et al*. (1990), Schueler and Lugbill (1990), and Smoot, *et al.* (1992). Caltrans is also currently conducting comprehensive tests of construction erosion controls and their results should become available soon.

Perhaps the most comprehensive study of filter fences was conducted by Barrett, *et al*. (1995) at Austin, TX, area highway construction sites, supplemented with controlled laboratory tests. Silt fences at six active highway construction sites were evaluated in terms of suspended solids and turbidity reduction. Two installations used non-woven fabrics, and four installations used woven fabrics. Manual grab sampling was used to obtain representative sediment samples of all size distributions during 10 rains. Uncontrolled discharges due to obvious filter fence failures (mostly undercutting flows or tears in the fabric) were excluded from sampling; only locations where the flows passed through the fabric were sampled. Samples were collected upslope of the pooled water behind the filter fence, in the pool backed up by the filter fence, and downstream of the filter fence. This sampling strategy was used to differentiate sedimentation from filtration effects, and to obtain an overall control efficiency. Because of highly variable concentrations above the pool, most of their data analysis relied on comparisons between the samples collected from the pool and the effluent from the filter fabric, reflecting filtering removal and not sedimentation.

The observed suspended solids removal rates were highly variable, ranging from -61 to 54%, with a median of 0%. Typical effluent suspended solids concentrations after the filter fence were about 500 mg/L. Similar poor results were obtained for turbidity removals (-32 to 49% range, with a median removal of 2%). As indicated by the negative removal rates, the effluent from the filter fences sometimes had greater suspended solids concentrations than were found in the pool. The removal of suspended solids due to sedimentation, however, was estimated to be about 50%, based on partial field observations. At one location where the lower portion of the fabric was clogged, a shallow upstream pool lasted for an extended period and removals of about 65% were measured.

The poor removal efficiency due to filtration was explained by comparing the particle sizes of the suspended solids and the apparent opening sizes of the fabrics (typically from 100 to 1,000 µm). Silt and clay-sized particles comprised the majority of the solids collected (68 to 100%, with a median of 96%) from the pond and below the filter fences. Any large particles present in the flowing waters were thought to have been settled in the pool before the fence. The diameters of the remaining particles passing through the fence were therefore much smaller than the openings in the fabric and were able to pass through unhindered. Earlier work by Schueler and Lugbill (1990) in Maryland substantiated the small particles observed in Texas. During settling column studies on construction site runoff, Schueler and Lugbill found that 90% of the incoming sediment was smaller than 15 µm, with the largest particles observed being only 50 µm. During their sediment pond evaluation tests, however, they did observe sediment deltas forming near the influent location, indicating that sand-sized particles were transported to the sediment ponds and represented a minor portion of the total load. These larger particles were apparently not included in the grab samples as they form part of the bed load.

Barrett, *et al*. (1995) found that filter fence installations are not designed as hydraulic structures, with frequent failures caused by excessive runoff. Runoff around the ends of fences, and even over-topping of the fences was observed several times during their monitoring project. However, other downstream controls were in place to mitigate these failures. Besides failures caused by lack of hydraulic design, they also observed deficiencies in performance that were caused by improper installation and maintenance, including:

- inadequate filter fabric splicing
- fence failure due to sustained over-topping
- unrepaired holes in fabric
- flow beneath fabric due to inadequate trenching of the bottoms of the fabric fences into the ground

Laboratory flume tests were also conducted on filter fabrics, enabling flow rates and suspended solids concentrations to be controlled at specific conditions. Austin silty clay, after passing through a 3 mm sieve, was used to make a test slurry. The median particle size in this mixture was 20 µm, and 30% was finer than 3 µm. The apparent openings in the filter fabrics tested ranged from 600 to 850 µm for 3 woven fabrics and 150 µm for the one non-woven fabric tested. During testing, the woven fabrics had median suspended solids removal rates of 68 to 87% (ranges of 46 to 97%), while the non-woven fabric had a median removal rate of 93% (range of 73 to 99%). The non-woven fabric also had the longest detention times during the tests due to its lower flow rate. In comparison, a rock berm was also tested (having the highest flow rate and therefore shortest detention time) and had a median SS removal efficiency of only 42% (36 to 49% range). The SS reductions in the testing flume was 34% without any controls in place due to sedimentation of the larger test particles while flowing over the rough bed. This high background reduction level therefore significantly reduces these reported flume test measurements with controls. The corrected berm removal rate was only 7%, for example, after taking into consideration the background reductions. Similar reductions would have to be made for the filter fabric test results.

An interesting observation during the flume tests was that while the detention times increased with time since the start of the tests, due to partial clogging of the fabrics, the woven fabrics all had decreased detention times after being exposed to large rains. Apparently, the rains helped wash some of the caked-on mud from the fabrics. This was not observed for the non-woven fabrics where clogging was internal and more permanent. During recent tests on stormwater filtration, several filter types were tested by Clark and Pitt (1999). They found that all of the fabrics examined totally clogged after accumulating a layer of about 3 mm of clay. This clogging layer preferentially forms near the bottom of the fabric, usually indicating the depth of the ponding. This clogging significantly decreases the flow rates through the fabric, allowing extended detention and therefore increased sediment trapping performance.

Barrett, *et al*. (1995) concluded that the poor filtering performance of the filter fences in good condition was due to the small particles in comparison to the large fabric openings. Previously reported high filtration control efficiencies conducted during laboratory experiments were faulty due to the use of unrealistically large test particles. Median particles during field tests at construction sites indicate that almost all of the particles are as silts and clays. The relatively minor sand fractions are easily deposited during sheetflows, or in ponded areas. Sedimentation effectiveness was found to be highly dependent on the detention time in the ponded areas behind the filter fabrics. The detention time is controlled by the geometry of the upstream pond, hydraulic properties of the fabric, and maintenance of the filter fence. Holes in the fabric, under-cutting due to inadequate trenching of the bottom of the fabric, and overtopping or bypassing around the ends of filter fabric fences, all effectively decreased the detention time in the pond behind the fabrics and contributed to very low observed field performance of filter fabrics.

Silt captured on woven filter fabric	Layer of silt captured against bottom edge of newly installed filter fabric fence
Ponded area sediment accumulation and smear of silt on fence	Bulk of sediment captured behind filter fence in ponded area

Heavy sediment load in ponded area	Laboratory test specimens of filter fabrics after flow and sediment capture tests
Sediment in ponded area	Sediment in ponded area
Non-woven filter fabric	Sediment in ponded area

Filter fabric fence on mulched slope(SCS photo)	Woven filter fabric material

Example Calculation of Sediment Capture Behind Filter Fence
It is possible to calculate the expected level of control for a filter fence at a specific site using the upflow velocity concept presented earlier:

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

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. Figure 6-12 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:

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

Figure 6-5 can be used to estimate the approximate suspended solids control corresponding to the critical particle size. For example, if the calculated critical particle size is 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 6-12. Filter fabric conditions and critical particle size controlled.

Filter Fences to Slow Water Flowing Down Critical Slopes

Filter fences intercepting sheetflows may also slow the water flowing down critical slopes. The upslope length of the ponded area will be obviously protected from rain impaction and by flowing water. This length can be estimated for different water depths impounded behind a filter fence. As an example, for a 5% slope and for a 1 ft water depth, the ponding would extend uphill 20 ft. In addition, some of the downslope area beneath of filter fence (if not installed on the toe of the slope, as generally recommended), will also have reduced flow velocities, compared to the same slope without the filter fence. WinDETPOND can be used to calculate the reduction in flow rates for the flows entering the ponded area compared to the discharge water through a filter fence. Generally, non-woven filter fabrics have much lower flow rates compared to woven filter fabrics. The sheetflow calculation information in Chapter 4 can also be used to estimate the flow rates on slopes of different roughness and slopes. As an example, Figure 7-13 (a repeat of Figure 4-12) shows the sheetflow travel times for different slopes having a roughness value of 0.15, corresponding to relatively short grass. A slope of 10% that is 100 ft long would have a travel time of about 5 minutes, or a velocity of about 0.33 ft/sec. There are non-woven fabrics that have flow rates appreciably less than this value, so a filter fabric could result in critical slopes being exposed to reduced periods of high flows. Of course, using multiple filter fences along a slope could help reduce the effective speed of the flowing water, but the accumulative amount of water reaching the lowest fence may be excessive.

Figure 7-13. Sheetflow travel times for different slopes.

Pressure Force on Filter Fences

The pressure equation can be used to calculate the forces acting on filter fences. The following calculation shows the resisting force needed for a 10 ft span of filter fence with 2 ft of standing water:

The momentum equation can be used when the flow rates should be considered:

Basically, the forces acting on a filter fence can be very large and the filter fence stake systems must be selected to withstand this force. In addition, the resisting forces of the soil also act on the fence stake to hold it upright, and also needs to be considered. Wet clayey soils may need long stakes driven deeply in the ground to resist this pressure.

Conclusions

This discussion 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.

Detention ponds are probably the most commonly used runoff quality control devices and have substantial literature documenting their performance and problems. Wet detention ponds have been shown to be very effective, if their surface area are 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. Care must also be taken to minimize safety and environmental hazards associated with ponds.

Physical sedimentation is the main removal process occurring in wet ponds. Temporary sediment ponds at construction sites are most suitable where the area to be controlled is larger than about 10 acres (the typical upper limit for filter fencing). They have been found to be generally the most effective sediment control (after prevention).

References

Akeley, R.P. "Retention ponds for control of urban stormwater quality." In *Proceedings National Conference on Urban Erosion and Sediment Control: Institutions and Technology*, EPA-905/9-80-002, Chicago, January 1980.

Auckland Regional Council. *TP 90: Erosion and Sediment Control*, Auckland, New Zealand. 1999.

Barfield, B. "Analysis of the effects of existing and alternate design criteria on the performance of sediment detention ponds." *1986 Sediment and Stormwater Conference*, sponsored by the Maryland Water Resources Administration, Sediment and Stormwater Division, Salisbury State College, Maryland, July-August 1986.

Barrett, M.E., J.E. Kearney, J.F. Malina, R.J. Charbeneau, and G.H. Ward. *An Evaluation of the Use and Effectiveness of Temporary Sediment Controls*. Center for Research in Water Resources. Technical Report CRWR 261. The University of Texas at Austin. August 1995.

Bedner, R.E. and D.J. Fluke. *Demonstration of Debris Basin Effectiveness in Sediment Control*. Environmental Protection Agency, Industrial Environmental Research Laboratory - Cincinnati, EPA-600/7-80-154. Cincinnati, Ohio, 1980.

Bondurant, J.A., C.E. Brockway, and M.J. Brown. "Some aspects of sedimentation pond design." *Proceedings National Symposium on Urban Hydrology and Sediment Control*, University of Kentucky, Lexington, 1975.

Chambers, G.M. and C.H. Tottle. *Evaluation of Stormwater Impoundments in Winnipeg*, Report SCAT-1, Environment Canada, Ottawa, April 1980.

Clark, S. and R. Pitt. *Stormwater Treatment at Critical Areas, Vol. 3: Evaluation of Filtration Media for Stormwater Treatment*. U.S. Environmental Protection Agency, Water Supply and Water Resources Division, National Risk Management Research Laboratory. EPA/600/R-00/016, Cincinnati, Ohio. 442 pgs. October 1999.

Colston, N.V., Jr. *Characterization and Treatment of Urban Land Runoff*. EPA-670/2-74-096, U.S. Environmental Protection Agency, 1974.

Driscoll, E. D. "Detention and retention controls for urban stormwater." *Engineering Foundation Conference: Urban Runoff Quality - Impact and Quality Enhancement Technology*, Henniker, New Hampshire, edited by B. Urbonas and L. A. Roesner, published by the American Society of Civil Engineers, New York, June 1986.

Eccher. C.J. "Thoughtful design is prime factor in water safety." *Lake Line*. pp. 4-8. May 1991.

EPA (USA Environmental Protection Agency). *Final Report for the Nationwide Urban Runoff Program*. Water Planning Division, PB 84-185552, Washington, D.C., December 1983.

Gietz, R. J. *Urban Runoff Treatment in the Kennedy-Burnett Settling Pond*. For the Rideau River Stormwater Management Study, Pollution Control Division, Works Department, Regional Municipality of Ottawa-Carleton, Ottawa, Ontario, March 1983.

Grizzard, T. J., C. W. Randall, B. L. Weand, and K. L. Ellis. "Effectiveness of extended detention ponds." *Engineering Foundation Conference: Urban Runoff Quality - Impact and Quality Enhancement Technology*, Henniker, New Hampshire, edited by B. Urbonas and L. A. Roesner, published by the American Society of Civil Engineers, New York, June 1986.

Harrington, B. W. *Feasibility and Design of Wet Ponds to Achieve Water Quality Control*. Sediment and Stormwater Division, Maryland Water Resources Administration, July 1986.

Hittman Assoc. *Methods to Control Fine-Grained Sediments Resulting from Construction Activity*. U.S. Environmental Protection Agency, Pb-279 092, Washington, D.C., December 1976.

Horner, R.R., J. Guedry, and M.H. Kortenhof. *Improving the Cost Effectiveness of Highway Construction Site Erosion and Pollution Control*. Washington State Transportation Center, Washington State Dept. of Transportation, Seattle, WA. 1990.

Jones, J. E. and D. E. Jones, "Interfacing considerations in urban detention ponding." *Proceedings of the Conference on Stormwater Detention Facilities, Planning, Design, Operation, and Maintenance*, Henniker, New Hampshire, Edited by W.

DeGroot, published by the American Society of Civil Engineers, New York, August 1982.

Kamedulski, G. E. and R. H. McCuen. “The effect of maintenance on storm water detention basin efficiency.” *Water Resources Bulletin*, Vol. 15, No. 4, pg. 1146, August 1979.

Larcombe, M.F. *Technical Publication on Chemical Removal of Sediment from Earthworks Stormwater*. Auckland Regional Council (New Zealand). 1999.

Linsley, R. K., and J. B. Franzini. *Water Resources Engineering*. McGraw-Hill, New York, 1964.

Marcy, S. J. and J. E. Flack. “Safety considerations in urban storm drainage design.” *Second International Conference on Urban Storm Drainage*, Urbana, Illinois, June 1981.

Martin, E.H. “Mixing and residence times of stormwater runoff in a detention system.” Engineering Foundation Conference: *Design of Urban Runoff Quality Controls*. Potosi, Missouri. July 10 - 15, 1988. Edited by L. A. Roesner, B. Urbonas and M.B. Sommen. pp. 164 - 179. Published by the American Society of Civil Engineers, New York, June 1989.

Mason, J.M.,Jr. “On-site stormwater detention: An overview.” *Public Works*, February 1982.

McCuen, R. H. *A Guide to Hydrologic Analysis Using SCS Methods*. Prentice-Hall, Englewood Cliffs, New Jersey, 1982.

Nelson, John. MSCE published thesis, Department of Civil and Environmental Engineering, UAB. *Characterizing Erosion Processes and Sediment Yields on Construction Sites*. 1996.

Pitt, R. “Runoff controls in Wisconsin’s Priority Watersheds.” *Advanced Topics in Urban Runoff Research* conference. Proceedings edited by B. Urbonas and L.A. Roesner. Engineering Foundation and ASCE, New York. pp. 290-313. 1986.

Pitt, R., and J. McLean. *Toronto Area Watershed Management Strategy Study: Humber River Pilot Watershed Project*. Ontario Ministry of the Environment, Toronto, Ontario, 1986.

Pitt, R. “Sediment Control in Alabama.” *41st Annual Transportation Conference*. Montgomery, AL. February 1998.

Pitt, R. “Detention pond design for water quality improvement.” *National ASCE Hydraulic Conference*. San Francisco, California. July 1993a.

Pitt, R. “The stormwater quality detention pond model (DETPOND).” *26th Annual Water Resources Conference*. University of Minnesota. October 1993b.

Poertner, H.G. *Practices in Detention of Urban Stormwater Runoff.* American Public Works Association, OWRR Contract No. 14-31-0001-3722, Chicago, 1974.

Randall, C. W. “Stormwater detention ponds for water quality control.” *Proceedings of the Conference on Stormwater Detention Facilities, Planning, Design, Operation, and Maintenance*, Henniker, New Hampshire, Edited by W. DeGroot, published by the American Society of Civil Engineers, New York, August 1982.

Schimmenti, F.G. “Stormwater detention basins must control more than runoff.” *American City and County*, December 1980.

Schueler, T. R. *New Guidebook Review Paper No. 2, Wet Pond BMP Design*. Metropolitan Washington Council of Governments, Washington, D.C., 1986.

Schueler, T.R., and J. Lugbill. *Performance of Current Sediment Control Measures at Construction Sites*. Metropolitan Washington Council of Governments, Sediment and Stormwater Administration of the Maryland Dept. of the Environment. 1990.

SCS (now NRCS) (U.S. Soil Conservation Service). *Computer Program for Project Formulation, Hydrology*. Technical Release Number 20 (TR-20). U.S. Dept. of Agriculture, 1982.

Smith, W. G. “Water quality enhancement through stormwater detention.” *Proceedings of the Conference on Stormwater Detention Facilities, Planning, Design, Operation, and Maintenance*, Henniker, New Hampshire, Edited by W. DeGroot, published by the American Society of Civil Engineers, New York, August 1982.

Smoot, J.T., T.D. Moore, J.H. Deatherage, and B.A. Tschantz. *Reducing Nonpoint Source Water Pollution by Preventing Soil Erosion and Controlling Sediment on Construction Sites*. Transportation Center of Tennessee. 1992.

Terstriep, M.L., G.M. Bender, D.C. Noel. *Nationwide Urban Runoff Project, Champaign, Illinois: Evaluation of the Effectiveness of Municipal Street Sweeping in the Control of Urban Storm Runoff Pollution*. Contract No. 1-5-39600, U.S. Environmental Protection Agency, Illinois Environmental Protection Agency, and the State Water Survey Division, University of Illinois, December 1982.

U.S. Bureau of Reclamation. *Design of Small Dams*. U.S Dept. of the Interior, A Water Resources Publication, 3rd edition, 1987.

Vignoles M., and L. Herremans. “Metal pollution of sediments contained in runoff water in the Toulouse city.” (in French). *NOVATECH 95, 2nd International Conference on Innovative Technologies in Urban Storm Drainage*. May 30 - June 1, 1995. Lyon, France. pp. 611-614. Organized by Eurydice 92 and Graie. 1995.

Whipple, W. and J. V. Hunter. “Settleability of urban runoff pollution.” *Journal WPCF*, Vol. 53, No. 12, pg. 1726, 1981.

Wiegand, C., T. Schueler, W. Chittenden, and D. Jellick. “Comparative costs and cost effectiveness of urban best management practices.” Engineering Foundation Conference: *Urban Runoff Quality - Impact and Quality Enhancement Technology*, Henniker, New Hampshire, edited by B. Urbonas and L. A. Roesner, published by the American Society of Civil Engineers, New York, June 1986.