Regional Stormwater Management Facility System
at the School of Veterinary Medicine, Blacksburg, Virginia

by

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(ABSTRACT)

Continuing development of the Virginia Tech campus is increasing downstream flooding and water quality problems. To address these problems, the University has proposed the construction of a stormwater management facility to control the quantity and quality of stormwater releases to Strouble Creek, a tributary of the New River.

The overall goal of this project is to design a stormwater management facility proposed for the Virginia-Maryland College of Veterinary Medicine at Virginia Tech in Blacksburg, Virginia that will reduce present and anticipated downstream flooding and water quality problems.

Specific objectives of the project are:

- control of flooding in lower areas by reducing the peak discharge while disturbing existing wetlands as little as possible,
- address removal of major NPS pollutants such as total phosphorus (TP), total nitrogen (TN), metals, organic compounds related to petroleum and gasoline, and suspended sediment (SS) from stormwater runoff, and

- design of a dam system that is able to withstand all driving forces and constructed in accordance with governing regulations.

The design requirement to limit wetland disturbance below one acre was maintained. The requirement set by officials of Virginia Tech is based on the Nationwide Permit 26 of the Wetland Regulations. An individual permit process is thus avoided. Considering this demand, however, the freedom of the stormwater management facility design was significantly restricted. Resulting from the previous restrictions mentioned, the facility will include two ponds in series - a lower, dry pond and an upper, wet pond. The stormwater management system is designed to reduce the peak discharge. The dry pond is designed to detain water only for a short period of time, as opposed to the wet pond which is designed to retain water, thereby maintaining a permanent pool of water, and to change the characteristics of runoff.

The wet pond was chosen to be of an Extended Detention wetland type. Aspects such as the availability of suitable area and detention volume governed the decision to make use of this type of stormwater wetland. The constraint on a maximum possible water surface elevation due to the Veterinary School’s road embankment, which crest elevation is at 2023 ft, was considered in the design.

The stormwater management facility was designed to meet water quantity control requirements and to address water quality benefits. Storm water management regulations intending to mitigate the adverse effects of land development to streams and waterways were met. Requirements to limit peak discharges from 2-year and 10-year events to existing discharge levels were achieved.

Several outlet structures for each of the ponds were investigated. The structures proposed are a perforated riser/broad-crested weir for the wet pond and a proportional weir for the dry pond. They were chosen as a result of analyses on hydraulic performance, maximum water surface elevations, drawdown times, peak discharge rates, and pollutant removal capabilities.

The average pollutant removal capability of 75% of TSS, 45% TP, and 25% TN for an extended stormwater wetland, as found in the literature, is expected to be lower for the proposed facility, since the wetland-to-watershed-area ratio is considerably smaller (0.22%) than the required minimum ratio of 1%. However, other suggested desirable parameter for extended detention wetland systems such as required treatment volume, effective flow path length, and dry weather water balance will be maintained.

The structural design of the dams was based on experience and research data. The dams are designed to consist of two zones, shell and core. The core extends as a cutoff trench 4 feet below the ground surface. Additionally, toe drain trenches and anti-seep collars along the pipe where penetrating the dam will be placed to collect and reduce seepage, respectively. Special considerations toward seepage problems were taken into account for both dams by placing a cutoff trench and a toe drain trench.

Dedicated in grateful memory of Dr. James F. Phelan.
My accomplishments are due to immense patience
and generosity.

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1. INTRODUCTION

1.1 Facility Site and Available Data Sources

A stormwater management facility consisting of two ponds in series is proposed for an area northwest of the intersection of Southgate Drive and Duck Pond Drive in Blacksburg, Virginia as shown on the Project Vicinity Map (Drawing No. 1). The area presently contains pastureland, wetland, cropland, and a cross-country running track.

The drainage area for the proposed stormwater management facility consists of the area surrounding Lane Stadium, which includes a mix of land uses consisting of stadium parking lot, lawn areas, and baseball fields plus drainage from the Dairy Center and the Virginia-Maryland College of Veterinary Medicine.

The proposal is based on the Nationwide Permit 26 of the Wetland Regulations which exempts activities that cause the loss or substantial adverse modification of wetlands areas and waters under one acre from the individual permit requirement. In order to avoid an individual permit process which is strongly desired by Virginia Tech authorities, wetland area of slightly less than one acre in extent should be disturbed by the proposed project.

Previous hydrologic investigations have indicated that the lower end of the stadium parking lot, the upstream end of the parking lot adjacent to the Food Processing Facility, Southgate Drive and Duck Pond Drive are subject to flooding. This project does not address these flooding problems.

The use of a retention basin or wet pond is being considered because of its potential for greater removal of constituents like SS, TP, and TN as compared to an extended detention or dry pond system. Two wet ponds in series are proposed, because the area available for retention and detention purposes is limited. Further, VPI&SU intends to support a research program to investigate the performance of a managed wetland relative to a dry pond stormwater facility. Equipment for flow measurement and water quality sampling will be installed at the inflow and outflow sides of the new detention system.

While there are a number of data bases already available on the removal performance of wet ponds, there is an urgent need to expand the rather limited data base that we now have. It has been found by some research projects that wet ponds offer distinct advantages over conventional dry ponds and extended detention ponds. However, the amount of additional pollutant removal that can be achieved by wet ponds is a matter of controversy. This is an extremely important issue, given the substantial increase in construction cost and the additional maintenance requirements associated with permanent pools.

It is believed that the proposed facility will provide effective stormwater management for Virginia Tech. At the same time, the facility offers a rather unique opportunity to create an outdoor laboratory for the training of future engineers and scientists concerned with urban stormwater management and wet pond design.

Data used to determine the direct runoff of the watershed were obtained from the "Stroubles Creek Regional Study and Stormwater Management Plan" prepared by the Roanoke consulting firm Hayes, Seay, Mattern & Mattern (HSMM). Additional design information was obtained from either of the engineering consulting firm Anderson & Associates, Hayes, Seay, Mattern & Mattern, or Froehling & Robertson, subcontractor to Anderson & Associates.

It is important to note that the entire watershed of 446 acres was not subject of hydrologic and hydraulic simulation. Because information on landuse of the School of Veterinary Medicine and detailed areal subdivision could not be obtained from HSMM study, only a 355-acre portion of the watershed was investigated with precipitation-runoff model HEC-1.

1.2 Objectives

The overall goal of this project is to design a stormwater management facility for the Virginia-Maryland College of Veterinary Medicine at Virginia Tech in Blacksburg, Virginia that will reduce present and anticipated downstream flooding and water quality problems.

Specific objectives of the project are:

- control of flooding in lower areas by reducing the peak discharge while disturbing existing wetlands as little as possible,
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- design of a dam system that is able to withstand all driving forces and constructed in accordance with governing regulations.

2. LITERATURE REVIEW

2.1. Wetlands

2.1.1. Terminology of Wetlands

Wetlands occupy an intermediate position between terrestrial and aquatic ecosystems. They often form a part of a large continuum of community types, and therefore it is difficult to set boundaries. Traditionally, wetlands are considered to be transitional ecosystems, often described as successional links in series from open water to land or vice versa (Walter, 1960). More recently, they are being interpreted as stable, nonsuccessional communities in their own right (Van der Alk, 1981). Of the several definitions available, three may be referred to here:

- The International Union for the Conservation of Nature and Natural Resources (IUCN) defines wetlands as "Wetlands are areas of submerged or water saturated land, whether natural or artificial, permanent or temporary, whether the water is static or flowing, fresh, brackish or salt. Water dominated areas to be considered would include marshes, sloughs, bogs, swamps, fens, peatlands, estuaries, bays, sounds, ponds, lagoons, lakes, rivers and reservoirs. Where marine or coastal waters are involved, waters up to the depth of 15 m are included."
- Wastlake (1988) provides a more biologically based definition. "A wetland is an area dominated by specific herbaceous macrophytes, the production of which takes place predominantly in the aerial environment above the water level while the plants are supplied

with amounts of water that would be excessive for most other higher plants bearing aerial shoots."

- A more comprehensive definition prepared by Cowardin (1979) of the U.S. Fish and Wildlife Service states: 'Wetlands are lands transitional between terrestrial and aquatic systems where the water table is usually at or near the surface or the land is covered by shallow water. For purposes of this classification, wetlands must have one or more of the following three attributes:

  (1) at least periodically, the land supports predominantly hydrophytes;

  (2) the substrate is predominantly undrained hydric soil; and

  (3) the substrate is nonsoil and is saturated with water or covered by shallow water at some time during the growing season of the year'.

These three definitions differ greatly. The IBP definition does not include wet areas covered with woody vegetation, shallow water bodies with either hydrophytes or algae as dominant plants, or bogs. These types of ecosystems were not included in the IBP 'Wetlands' synthesis. Although the IUCN definition does not account for soil characteristics, and includes marine coastal water up to 15 m deep, it is close to the Cowardin et al. definition. The latter is the basis for a comprehensive inventory and assessment of American wetlands by the USFWS.

2.1.2. Man-made Wetlands

Man-made wetlands, or those managed for human values, include paddy rice fields and similar crops (marshes), wetland tree plantations (swamps and riparian floodplain forests), fishponds (shallow lakes or standing water in floodplains). These categories can be further distinguished by the management schemes employed.

The use of constructed wetlands for stormwater quality control has attracted a great deal of attention and controversy in recent years. When properly designed, stormwater wetlands have many advantages as an urban best management practice (BMP), including reliable pollutant removal, longevity, adaptability to many development sites and excellent wildlife habitat potential. As with many urban best management practices; however, stormwater wetlands also have some drawbacks, including relatively high land consumption, a need for intensive management after establishment, and the potential for adverse impacts within sensitive watersheds.

Stormwater wetlands can be defined as constructed systems that are explicitly designed to mitigate the impacts of stormwater quality and quantity that occur during the process of urbanization. They do so by temporarily storing stormwater runoff in shallow pools that create growing conditions suitable for emergent and riparian wetland plants. The runoff storage, complex microtopography and emergent plants in the stormwater wetland together form an ideal matrix for the removal of urban pollutants (Schueler, 1992).

2.1.3. Natural versus Stormwater Wetlands

While the design of stormwater wetlands is often informed and based on the ecology of natural wetlands, it is important to keep in mind that stormwater wetlands are distinctly different in hydrology, morphology and ecology from undisturbed wetlands.

Differences between stormwater wetlands and natural non-tidal wetlands within the mid-Atlantic region:

Table 2.1: Differences between Natural and Stormwater Wetlands (Schueler, 1992).

Stormwater Wetlands	Natural Wetlands
Water balance dominated by surface runoff	Water balance often an expression of groundwater
Hydroperiods is “semi-tidal”; inundation and rapid drawdown 10 to 30 times/year	Hydroperiods are more gradual and may change on a seasonal basis
Standing water present year round	Standing water may be seasonal
Wetland boundaries clearly defined	Wetland boundaries vary seasonally due to groundwater shifts
Species diversity established by human intervention or by volunteers; no prior seedbank at site	Species diversity maintained by the seedbank
Simple topographic structure	Complex topographic structure
Relatively few species dominated by emergent types	Diverse number of species, with a mix of tree, shrub, herbaceous and emergent types
Prone to colonization by invasive species such as Typha and Phragmites	Fewer exotic and dominant species, unless site has been disturbed
System requires maintenance	Self-maintaining system
Sediments and water columns enriched with nutrients and trace metals, higher turbidity	Lower quantities of pollutants in the wetland, lower turbidity
High rates of sediment supply	Lower rates of sediment supply
Low to moderate wildlife habitat potential	Moderate to high wildlife habitat value

Natural wetlands that receive urban stormwater input share characteristics of both wetland types. Another key difference is that stormwater wetlands are situated within the urban landscape, and must address the concerns of adjacent residents. Therefore, the designer cannot overlook social factors such as mosquitoes, odors, safety, appearance, passive recreational use, access, and maintenance. A designer is challenged to construct a system that simultaneously mimics the functions of a natural wetland and also fully addresses the needs and concerns the adjacent community.

2.1.4. Wetland Hydrology

At the most general level, the hydrologic cycle is the circulation of water from the earth's surface to the atmosphere and back again. While physical processes predominate, biological processes can also significantly influence pathways involved and rates at which water moves.

The economic importance of water has resulted in development of a tremendous amount of information on factors that control its distribution and movement. Numerous models have been utilized in applying this knowledge, and have contributed to successful resolution of water management problems throughout the world.

In the past, wetland management has normally involved stabilization of water levels at some elevation, either well below or above the ground surface. If fluctuations are permitted, they are controlled within certain limits in terms of water depth and timing.

Water levels in depression at seepage wetlands fluctuate as a function of the overall rise and fall of the surrounding water table. At these sites, the water table does not exhibit large annual fluctuations because water inputs are relatively constant. The constancy can be a function of a climate where water input and output relationships are relatively stable. Water percolating down through the soil will encounter and move along the top of an impermeable stratum and emerge aboveground where this stratum intersects the ground surface. It is in shallow depressions at or somewhere downstream of theses sites that seepage wetlands can develop.

Water level fluctuations also vary as a function of the amount of aquatic or wetland habitat in the general area (Kulczynski, 1949). Fluctuations are minimal where surface water is always present, such as large lakes, man-made impoundments and canals continually receiving water inputs, or sites along a coast.

There are no natural wetlands in the mid-Atlantic region that share this unique “semi-tidal” hydroperiod. The frequent changes in water level of the stormwater hydroperiod imposes severe physiological constraints on the wetland plant community. Consequently, stormwater wetlands tend to have more open standing water than natural wetlands.

2.1.5. Ecology of Stormwater Wetlands

Natural wetlands are more or less “self-maintaining” systems in that they provide ecological services and support a diverse community over time without intervention by man. Stormwater wetlands, on the other hand, would not exist and cannot be maintained without active management. In addition, the semi-tidal hydrologic regime and the simple morphology of stormwater wetlands impose severe constraints on the wetland plant community. These constraints result in the low species diversity typically noted within the stormwater wetland community (Walter, 1960). Conditions within stormwater wetlands often favor exotic and invasive species such as Typha and Phragmites. These aggressive species flourish in the harsh growing environment of a stormwater wetland, and frequently form a mono-specific stand throughout the entire marsh. Ehrenfeld and Schneider (1990) have observed that stormwater influenced wetland communities contained over 65% invasive or exotic species, whereas natural reference wetlands had less than 1% of these undesirable species.

The plant community of stormwater wetlands tends to be dominated by five to ten species of emergent wetland plants. Natural wetland plant communities, by contrast, are much more diverse in respect to both species and forms, and often have 40 to 50 species of trees, shrubs, and herbaceous plants, in addition to emergent wetland plants. The diversity is maintained by the prodigious seedbank within the soil of natural wetlands. Seedbanks may contain up to 100,000 seeds per square meter of nearly fifty species (Leck, 1989). The seeds persist for many years, and will germinate when moisture and temperature are suitable. Stormwater wetlands do not initially possess a seedbank, and therefore lack a “memory” for diversity. Wildlife diversity also appears to be lower in stormwater wetlands than in natural wetlands. Azous (1991) reports that fewer rare reptile, amphibian, bird and mammal species were found in stormwater influenced wetlands of the Pacific Northwest, when compared to natural reference wetlands.

2.1.6. Morphology of Stormwater Wetlands

The morphology of stormwater wetlands is quite different from natural wetlands. To begin with, the boundaries of stormwater wetlands are engineered to be both fixed and constant.

By way of contrast, the boundaries of natural wetlands are often not static, gradually shifting back and forth in response to movement of groundwater and rainfall over time.

Second, the first generation of stormwater wetlands has been unable to fully mimic the complex patterns found in nature. For engineering purposes, stormwater wetlands were typically graded to uniform depths, simple shapes, zero slopes, and possessed little, if any, internal structural complexity. However, current designs include features to imitate the natural microtopography found in natural wetlands.

Third, stormwater wetlands have greater sediment inputs than natural wetlands. High sediment inputs, particularly during the construction phase of development, can overwhelm wetlands, and dramatically reshape their contours. When compared to natural wetlands, the sediment supply to stormwater wetlands is enriched with both nutrients and organic matter. Consequently, stormwater wetlands are eutrophic to hypertrophic, and their nutrient and carbon cycling patterns are often sediment-driven.

2.1.7. Design of Stormwater Wetland Systems

Stormwater wetlands usually fall into one of five basic designs, as described below (Schueler, 1992):

1. Shallow Marsh System (Figures 2.1-2) has a large surface area, and requires a reliable source of baseflow or groundwater supply to maintain the desired water elevations to support emergent wetland plants. Consequently, the shallow marsh system requires substantial space and a sizable contributing watershed area (often in excess of 25 acres) to support the shallow permanent pool.

The majority of shallow marsh system is zero to eighteen inches deep, which creates favorable conditions for the growth of emergent wetland plants. A deeper forebay is located at the major inlet, and a deep micropool is situated near the outlet.

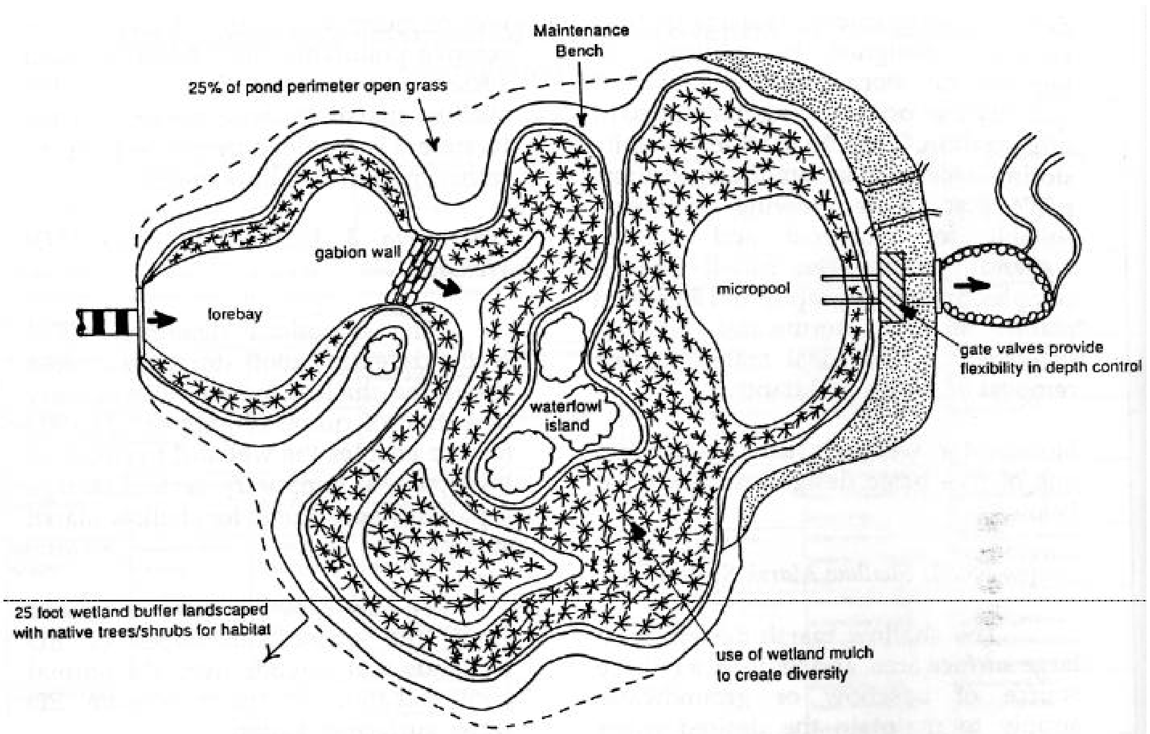

Figure 2.1: Shallow Marsh System (Schueler, 1992)

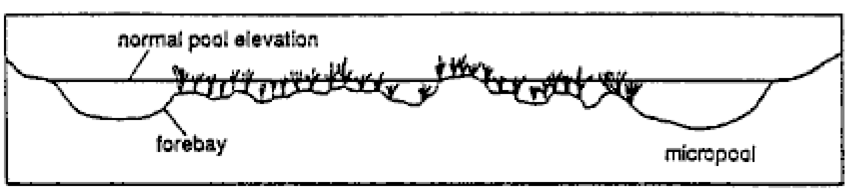

Figure 2.2: Cross-sectional Profile of a Shallow Marsh System (Schueler, 1992)

2. Pond/Wetland System (Figures 2.3-4) utilizes two separate cells for stormwater treatment. The first cell is a wet pond and the second cell is a shallow marsh. The multiple functions of the wet pond are to trap sediments, reduce incoming runoff velocity, and to remove pollutants. The pond/wetland system consumes less space than the shallow marsh, because the bulk of the treatment is provided by the deeper pool rather than the shallow marsh.

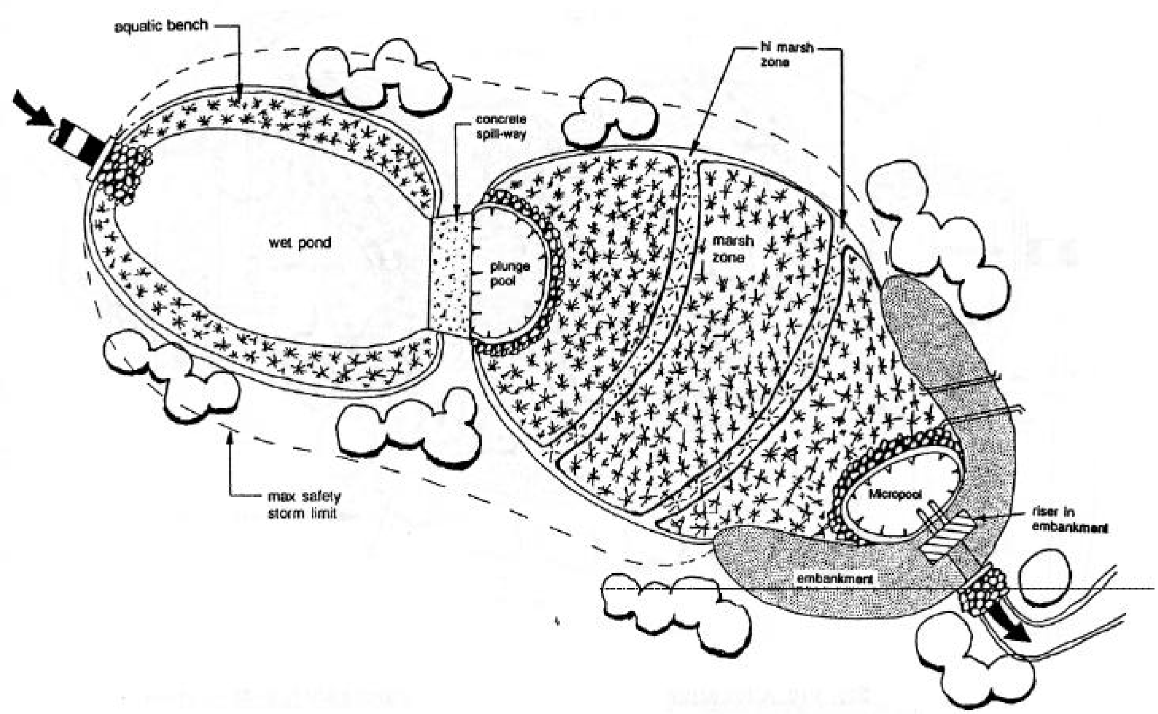

Figure 2.3: Pond/Wetland System (Schueler, 1992)

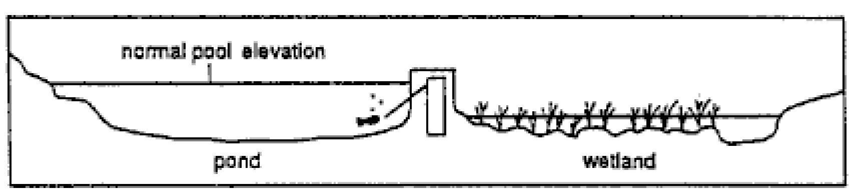

Figure 2.4: Cross-sectional Profile of a Pond/Wetland System (Schueler, 1992)

3. Extended Detention (ED) Wetlands (Figures 4.5-6) create extra runoff storage is created above the shallow marsh by temporary detention of runoff. The ED feature enables the wetland to consume less space, as temporary vertical storage is partially substituted for shallow marsh storage. A new growing zone is created along the gentle side-slopes of ED wetlands that extends from the normal pool elevation to the maximum ED water surface elevation. The water level within an ED wetland can increase by as much as three feet after a storm event, and then returns to normal levels within 24 hours. As much as 50% of the total treatment volume can be provided as ED storage, which helps to protect downstream channels from erosion, and reduce the wetland's space requirement.

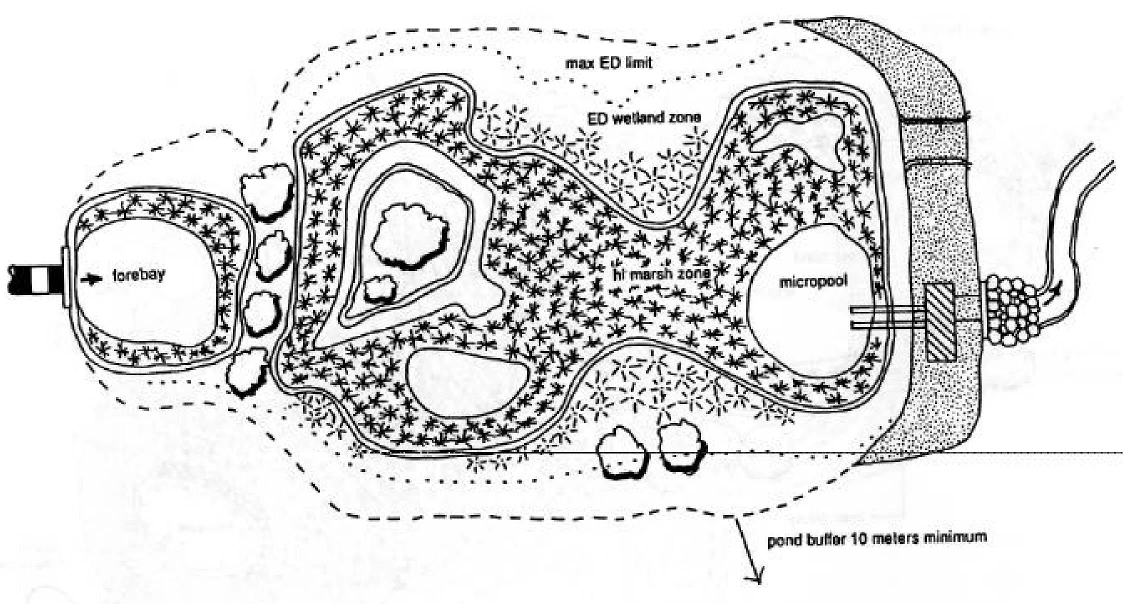

Figure 2.5: Extended Detention Wetland (Schueler, 1992)

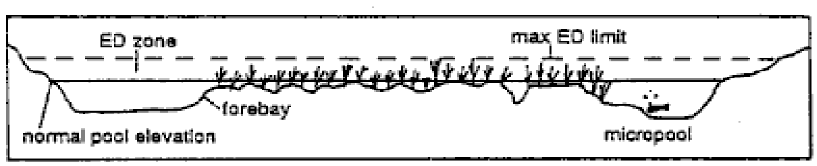

Figure 2.6: Cross-sectional Profile of a Extended Detention Wetland (Schueler, 1992)

4. Pocket Wetlands are usually less than one tenth of an acre in size. Because of their small drainage areas, pocket wetlands usually do not have a reliable source of baseflow, and therefore, exhibit widely fluctuating water levels. In most cases, water levels in the wetland are supported by excavating down to the water table. In drier areas, the pocket wetland is supported only by stormwater runoff, and during extended periods of dry weather will not have standing water. Due to their small size and fluctuating water levels, pocket wetlands often have low plant diversity and poor wildlife habitat value.

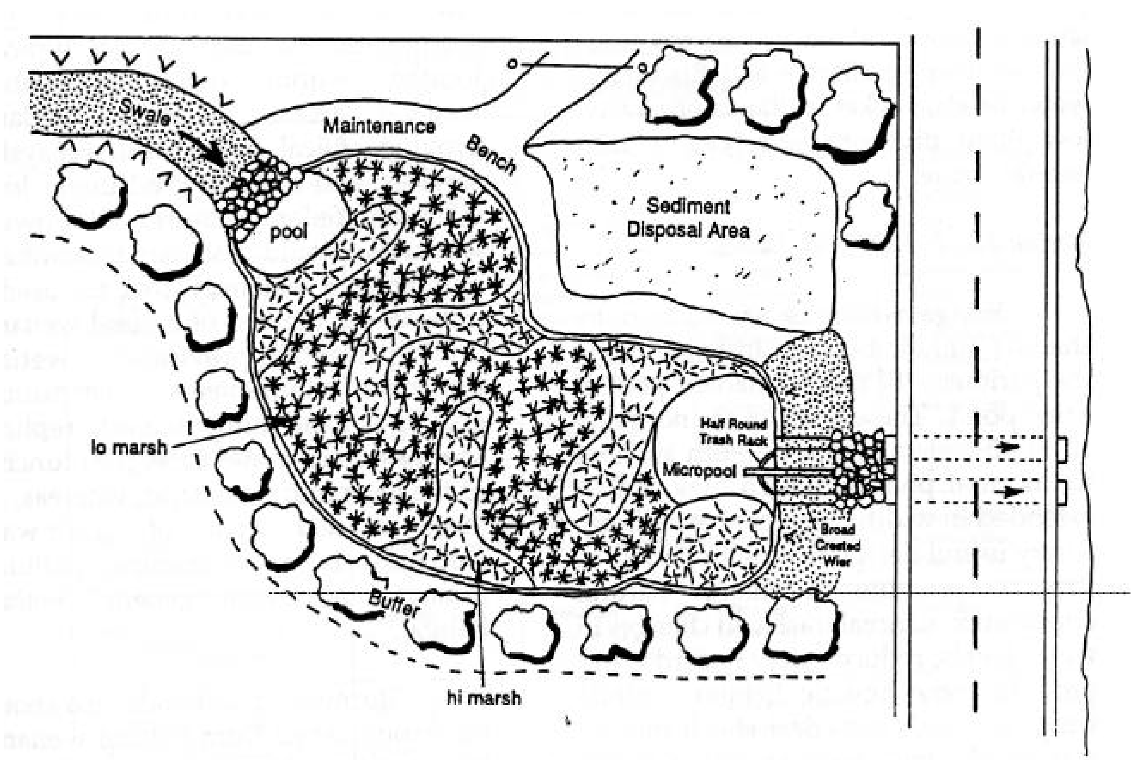

Figure 2.7: Pocket Stormwater Wetland (Schueler, 1992)

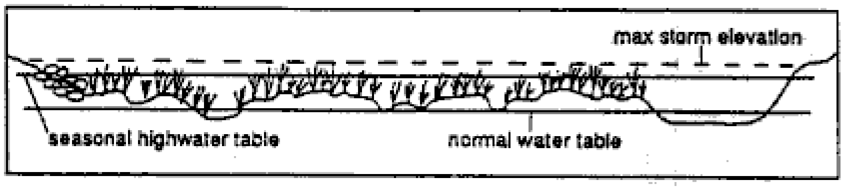

Figure 2.8: Cross-sectional Profile of a Pocket Stormwater Wetland (Schueler, 1992)

The selection of a particular wetland design is usually dependent on three factors:

- contributing watershed area
- available space and
- the desired environmental function for the wetland.

It is important to note that stormwater wetlands are not typically located within delineated natural wetland areas. Natural wetlands provide critical habitat and ecosystem services, and are protected under local, state, and federal statute. Stormwater wetlands should also not be confused with created wetlands that are used to mitigate for the loss of natural wetlands under permitting provisions of wetland protection requirements. The primary goal of wetland mitigation is to replicate species diversity and ecological function of the lost natural wetland; whereas, the more limited goal of stormwater wetlands is to maximize pollutant removal and create ‘generic’ wetland habitat.

Stormwater wetlands also should be distinguished from natural wetlands that receive stormwater runoff as a consequence of upstream development. Although not intended for stormwater treatment, stormwater-influenced wetlands are very common in urban settings. When stormwater runoff becomes a major component of the water balance of a natural wetland, its functional and structural quantities can be severely altered. The end result is that a stormwater-influenced wetland ultimately shares more of the characteristic of a stormwater wetland than a natural wetland.

As a general rule, uncontrolled stormwater should never be introduced into natural wetlands. As a second rule, stormwater wetlands or other stormwater pond systems should never be located within natural wetlands. For a number of reasons, however, it may not always be possible to observe both rules at the same site.

The best process to protect natural wetlands from both uncontrolled stormwater and stormwater controls is to carefully assess the wetland status of the development site prior to design. As a first step, the designer should carefully delineate four items:

- The location and quality of all existing natural wetlands at the site;
- The post-development drainage pattern, noting all catchments;
- Potential locations for stormwater wetlands, based on topography, available space and storage;
- The location of other important environmental features (e.g. forests, steep slopes, ect.).

The next step is to prepare an overlay that combines this information into a single map. The overlay analysis helps to preserve existing wetlands and to identify sites for stormwater wetlands to protect the existing wetlands. It may also define areas where stormwater/wetland conflicts will be unavoidable.

2.1.8. Converting Stormwater-influenced Wetlands to Stormwater Wetlands

Some conflicts with stormwater wetlands are unavoidable. The dilemma for the resource manager in these instances is clear - should one forsake the benefits of the wetland or the benefits of the stream? While no simple guidance can resolve this dilemma, it would seem reasonable that the decision should be made within the context of the larger watershed, i.e., which system provides the greater functional values within the watershed a stormwater-influenced wetland or a stormwater wetland? With this in mind, the following criteria are offered by Schueler (1992) for the extremely limited circumstances under which a natural wetland might be converted:

- The stormwater/wetland conflict cannot be resolved through careful site planning or fingerprinting techniques and on-site stormwater alternatives are not feasible.
- The natural wetland does not possess significant plant diversity or habitat qualities and contains no threatened or endangered species.
- The natural wetland is an incidental wetland that has been created by prior human activity, such as old farm ponds, sediment basins or wetland created by roadway crossings.
- The natural wetland presently is in a degraded state, as typified by a high proportion of invasive species, a monotonic stand of Typha and/or the dominance of open water.
- The natural wetland comprises less than 1% of the contributing watershed area, will receive direct stormwater runoff from a contributing watershed that will have an ultimate imperviousness in excess of 15%, and cannot be protected by reliable upstream runoff controls.

These wetlands will be heavily influenced by stormwater, and are likely to dramatically change character and function after development. Small, isolated (less than one acre) wetlands that are not located in close proximity to the stream system. These wetlands are not expected to provide much of a role in sustaining ecosystem functions within the watershed, after development.

When an existing wetland could be enhanced by adding to its water supply (using the stormwater).

- Introduction of stormwater runoff into an existing wetland after it has been effectively treated by a pond system (i.e., using the wetland to polish urban runoff). This use should be primarily restricted to existing wetlands that have standing water over much of the year.

In many cases, stormwater wetland conflicts can be avoided by “fingerprinting” the stormwater wetland around the natural wetland. Fingerprinting is a technique where the designer adjusts the location of the stormwater wetland and the entry of upstream baseflows and stormflows so as to minimize the impact on a natural wetland or forest.

A common practice before the advent of non-tidal wetland protection regulations was to place an embankment across the stream valley to impound the required storage volume for a stormwater pond or wetland. The end result was the transformation of a natural wetland into a stormwater wetland, with the attendant loss of diversity and functional values. The transformation occurs regardless of whether the natural wetland is replaced by a permanent pool or by a temporary extended detention, although the severity of the transformation is reduced if extended detention is used.

The preferred course of action is to locate the stormwater control in an upstream or offstream location. This is easier said than done, as some quantity of baseflow is usually required to maintain water elevations within a stormwater wetland. In some cases, it is possible to split the needed baseflow away from the stream into an off-line stormwater wetland situated well away from the stream. Another alternative is to rely on smaller upland pocket wetlands. This stormwater wetland design can be applied to serve very small drainage areas that lie outside of the perennial stream network. The widespread use of pocket wetlands; however, does have some disadvantages, particularly with regard to reliability in pollutant removal, maintenance, diversity and habitat quality.

Another fingerprinting technique involves pond sequencing, i.e., employing a series of smaller pools and wetland areas in the stream valley above and below the existing wetland. In this scenario, the total stormwater storage is provided in multiple cells, so that runoff is pre-treated before it enters the existing wetland. While this scenario will still result in significant stormwater influence to the existing wetland, it does reduce overall degradation.

2.2. Wetland Regulations

2.2.1. Criteria for Wetland Determination

For most landowners, the starting point in wetland regulation is the question of wetland jurisdiction: whether any of their land is subject to wetlands regulation and if so, the extent of area. In most instances, landowners will hope to minimize wetlands jurisdiction to avoid the necessity of going through the permit process and the chance of not being allowed to use their land as intended. In many instances; however, landowners will only need to establish the area of wetlands jurisdiction and proceed with the project accordingly. Also, the inquiring entity in some instances will be a potential buyer whose major concern is likely to be evaluating the land's potential use and consequent value, not minimizing wetland jurisdiction.

The only definitive way to determine if an area is subject to wetlands jurisdiction, and consequently permitting requirements, is to seek a determination from the Corps of Engineers (Corps) district office. The Corps regulations provide that the district offices are to perform this function. In sharp contrast with the permit process, no procedures are set forth in the regulations for the jurisdictional determination; in fact, no requirement exists for input from other federal agencies, state agencies, the landowner, or the public. Sometimes, in controversial situations, the Corps either unilaterally or in agreement with the landowner seeks input from others, particularly the Fish and Wildlife Service (FWS) and the Environmental Protection Agency (EPA).

The jurisdictional determination is final with the district engineer. There is no provision for administrative appeal to the division engineer or Corps headquarters in Washington or right to an adjudicatory hearing as there is for denial of an NPDES permit.

Wetland jurisdiction is the most complicated and consistently litigated regulatory issue. The Clean Water Act limits federal jurisdiction to "navigable waters", which it then defines as "waters of the United States". The Corps regulations define "waters of the United States" in seven categories, of which one include wetlands. Another of the categories is adjacent wetlands. Wetlands are defined by the Corps regulations as: "Those areas that are inundated or saturated by surface or groundwater at a frequency and duration sufficient or support, and that under normal circumstances to support, a prevalence of vegetation typically adopted for life in saturated soil conditions.

The Corps supplements its regulations with Regulatory Guidance Letters (1988) that provide guidance to the districts on specific issues that arise including jurisdictional issues.

2.2.2. Nationwide Permits - Individual Permits

The most significant general permits are called nationwide permits (NWPs). These general permits are issued by the Corps headquarters in Washington, D.C. and apply throughout the country. Some general permits have been issued on a regional or state basis, reflecting the decentralized administration of the wetland program by the Corps. These general permits are not listed in the Corps Section 404 regulations as are the nationwide permits. Information about them can be obtained from Corps division and district offices.

There are 26 nationwide permits. Except for four of them, it is not necessary that the landowner even inform the Corps of the activity. Rather, so long as the landowner meets the condition and management practices applicable to all NWPs, he can simply proceed with the activity. Among the more important of the fourteen conditions are that the discharges not occur in areas of concentrated shellfish production, not jeopardize a threatened or endangered species, or not adversely affect historic properties either on or determined to be eligible for listing on the National Register of Historic Places. There are eight management practices which require to the maximum extent practicable, among other things, that the discharges be minimized, and avoided in spawning areas during spawning season and in breeding areas for migratory waterfowl, and that heavy equipment used for work in wetlands be placed on mats (Law of Wetland Regulation, 1989).

The Corps regulations vest the division engineer with discretionary authority to override a nationwide permit in a specific instance and require an individual permit. This discretionary authority, which the regulation requires be based on concerns for the aquatic environment, has been employed only rarely. Its use is impeded by the fact that the regulations do not generally require that the district engineer be notified of the activity in advance.

A related question is whether more than one nationwide permit can be used for a single project. As a general matter, the Corps allows this practice, stating as an example in a Regulatory Guidance Letter that "a project involves a fill resulting in up to one acre of impacts above the headwater as well as a minor road crossing fill, is authorized under a combination of NWPs 26 and 14." The Regulatory Guidance Letter notes, however, that Nationwide Permit 26 cannot be used twice on the same project to increase the allowable acreage of a single such permit.

Another important aspect of general permits is the question of their applicability if an activity qualifying for a general permit is a feature of a larger activity requiring an individual permit. A Regulatory Guidance Letter (1988) states that "portions qualifying for a NWP should be able to function or meet their purpose without the portion requiring an individual permit.

2.2.3. Nationwide Permit 26

The Corps issued regulations on July 19, 1977, before the enactment of the 1977 Clean Water Act Amendments, that allowed discharges above headwater into non-tidal rivers, streams, and adjacent wetlands involving less than ten acres. Thus, under the new regulation, a landowner could fill a hundred acres of wetlands or more without seeking Corps' approval if those wetlands were isolated or located above headwater.

In December of 1982, 16 environmental organizations filed suit against the Corps challenging its amended regulations and the nationwide permit for above headwater and isolated waters in particular. Among the plaintiffs were the National Wildlife Federation, the National Audubon Society, the National Recourses Defense Council, and the Sierra Club. On February 10, 1984, the court approved a settlement of that case requiring the Corps to promulgate new regulations. The settlement required that the Corps modify Nationwide Permit 26 to require individual permits for areas ten acres or larger and notification for areas larger than one acre but less than 10 acres.

On March 29, 1984, in compliance with the settlement agreement, the Corps solicited public comment on Nationwide Permit 26 as established in the court settlement. The Corps issued an Environmental Assessment on Nationwide Permit 26 on September 10, 1984, and concluded in it that an environmental impact statement was not required. Then, the Corps promulgated new regulations on October 5, 1984, implementing the settlement agreement on Nationwide Permit 26 and establishing the current Nationwide Permit 26. It represented a major victory for the environmentalists since it reduced the jurisdictional area of isolated wetlands excluded from unlimited to ten acres, and for between one and ten acres required a notification process that could lead to an individual permit. Only areas less than one acre were totally excluded from federal jurisdictional coverage (Even here they must meet conditions, 33 CFR 330.5(b) and management practices, 33 CFR 330.6, applicable to all nationwide permits, and are subject to the infrequently used discretionary authority to require an individual permit, 33 CFR 330.8).

Nationwide Permit 26 is the most important general permit and has been the most controversial. This importance is due to the large amounts of wetland losses that can occur under it. Nationwide Permit 26 applies to isolated wetlands and waterbodies and to those located above headwaters. Headwaters are defined as the “point on a non-tidal stream above which the average annual flow is less than five cubic feet per second”. Many Corps districts have maps showing the location of headwaters on rivers and streams, thus eliminating the need for making the calculation.

The notification procedure that applies to areas between one and ten acres requires the landowner to notify the Corps, the Corps to notify various federal and state agencies for comment, and the division engineer to make a decision whether to require an individual permit (Regulatory Guidance Letter, 1988).

There has been substantial dissatisfaction with the notification procedure. Some Corps districts and Fish and Wildlife Service offices, believing that 20 days is not sufficient time in which to make decision, have required or recommended, in the case of the Fish and Wildlife Service, individual permits in each instance. Other Corps districts have basically accepted all Nationwide Permit 26 notifications without requiring an individual permit. Still other Corps districts have allowed them unless there is an objection from the Fish and Wildlife Service. In these instances where the Corps district automatically accepts the Wish and Wildlife Service’s automatic objection to activities they have not had time to review, it is important for the landowner to confer with the Fish and Wildlife Service in advance to attempt to satisfy it on project design or offsetting mitigation. If this advance work is not done, the landowner will have to go through the more onerous and time-consuming individual permit process. On the other hand, if the Fish and

Wildlife Service has been satisfied in advance, the notification is more likely not to raise objection, allowing the nationwide permit to be obtained in 20 days.

2.2.4. Other Nationwide Permits

Two other important nationwide permits allow the construction of outfall or water discharge structures where the discharge has been permitted by EPA, and discharge of material for backfill or bedding for utility lines. Another nationwide permit allows for repair or replacement of a currently serviceable structure while another allows minor road crossing fills for crossing a non-tidal waterbody provided that the crossing is culverted or bridged. A minor road crossing is defined as involving the discharge of less than 200 cubic yard of fill material below the plane of ordinary high water. Discharge and dredging that do not exceed ten cubic yards of material is also allowed by two separate nationwide permits. Another nationwide permit allows docks and other structures constructed in artificial canals within principally residential developments where the connection of the canal to navigable waters has been previously approved by the Corps.

The Corps NEPA regulations provide some basis for arguing that other laws apply to projects that are eligible for a nationwide permit. They state when discussing categorical exclusions from the environmental assessment (EA) or EIS requirement that “compliance with the Endangered Species Act, the Fish and Wildlife Coordination Act, the National Historic Preservation Act, the Clean Water Act, and other like acts, is always mandatory, even for actions not requiring an EA or EIS. Similarly, it can be argued that those laws are applicable to activities qualifying for general permits. Even if they are applicable, determining exactly how they are applicable to activities that are not reviewed by or subject to notification requirements of the Corps and the extent of their applicability poses difficult questions.

The Corps often establishes exemptions to jurisdiction while simultaneously expanding it. Thus, when the Corps broadly defined the interstate commerce requirement in the preamble to its November 13, 1986, regulations, it also set forth exemptions to overage. Although it characterized these exemptions as already existing ones, it certainly had not been clearly stated or understood previously. The most significant exemptions noted in the preamble to the regulations are drainage or irrigation ditches excavated in dry land, artificially irrigated areas which would revert to upland if irrigation ceases, and water-filled depressions created in dry land during construction activities that are filled before construction ends. The other two exemptions are:

- for artificial lakes or ponds created by excavating or diking dry land to collect and retain water and used exclusively for stock watering, irrigation, settling basin, or rice growing, and
- artificial swimming pools or ornamental pools created by excavating or diking dry lands for aesthetic purposes (Law of Wetland Regulation, 1989).

2.3. Stormwater Management Regulations

The stormwater management regulations are applicable to:

- every locality that establishes a local stormwater management program, and
- every state agency that, after January 1, 1991, undertakes any land clearing, soil movement, or construction activities involving soil movement or land development.

2.3.1. Requirements on Stormwater Management

Administrative procedures on maintenance and inspections are an integral aspect of a stormwater management plan. The Stormwater Management Regulation cover such procedures. They include the following points:

- Responsibility for the operation and maintenance of stormwater management facilities, unless assumed by a governmental agency, shall remain with the property owner and shall pass to any successor or owner. If portions of the land are to be sold, legally binding arrangements shall be made to pass the basic responsibility to successors in little. These arrangements shall designate for each project the property owner, governmental agency, or other legally established entity to be permanently responsible for maintenance.
- A schedule of maintenance inspections shall be incorporated into the local ordinance. Ordinances shall also provide that in case where maintenance or repair is neglected, or the stormwater management facility becomes a danger to public health or safety, the locality has the authority to perform the work and to recover the costs from the owner.
- At a minimum, stormwater management facilities shall be inspected on a semi-annual basis and after any storm which causes the capacity of the facility to be exceeded.
- During construction of the stormwater management facility, localities shall make inspections on a regular basis.

2.4. Water Quality Aspects

2.4.1. Removal Pathways Within Stormwater Wetlands

The basic intent of a stormwater wetland is to create a shallow matrix of sediment, plants, water and detritus that collectively removes multiple pollutants through a series of complementary physical, chemical and biological pathways.

Some of the primary removal pathways operating within stormwater wetlands are described below (Schueler, 1992):

- Sedimentation: As with other conventional pond systems, sedimentation (or gravitational settling) is perhaps the dominant removal pathway for particulate pollutants operating within a stormwater wetland. The morphology and vegetation within stormwater wetlands create ideal settling characteristics, particularly in comparison to other pond systems. The sheetflow conditions across the wetland, slower runoff velocities and hydraulic resistance afforded by the wetland vegetation, are all very effective at promoting settling. In addition, the root network of emergent plants helps to stabilize sediments, thereby reducing the potential for their resuspension. Much of the sedimentation in wetlands (and ponds) will usually occur in the immediate vicinity of the inlet.
- Adsorption to Sediment/Emergent Plants/and Detritus: The second primary removal pathway is the adsorption of pollutants to the surfaces of suspended sediments, bottom sediments, wetland vegetation and organic detritus. Adsorption is a key removal pathway for phosphorus, trace metals, and some hydrocarbons. The importance of adsorption as a

removal pathway increases as the contact time with this complex of surfaces increases. Longer contact time in stormwater wetlands can be achieved by creating a surface area to volume ratio. A number of techniques can be used to increase contact time:

- create a complex and variable microtopography within the stormwater wetland,
- establish dense stands of emergent wetland vegetation,
- design for sheet flow or shallow flow conditions,
- incorporate organic soils into the bottom of the wetland, and
- allow wetland detritus to accumulate within the basin.

- Physical Filtration by Plants: These dense network of emergent wetland plants acts as a filter for incoming stormwater as it passes through a stormwater wetland. While physical filtration is a relatively ineffective pathway for the removal of nutrients and trace metals, it is very effective at removing trash, debris, and other floatables found in urban runoff. In addition, the filtering action of the wetland plants reinforces the effectiveness of other removal pathways, such as sedimentation, adsorption, and microbial removal.
- Microbial Activity: The complex of surfaces within the stormwater wetland provide favorable conditions for active microbial growth. Billions of microscopic bacteria consume carbon and nitrogen compounds within both water column and the organic sediments of the wetland. Microbial processes are effective in removing nitrogen (via the nitrification/denitrification process) and organic matter (via aerobic decomposition). Microbial activity consumes oxygen, and often can completely deplete oxygen within the top layer marsh sediments. The combination of partially decomposed organic matter and

low oxygen common to wetlands helps to immobilize many trace metals into less mobile sulfide, oxide or hydroxide compounds that are less likely to be released to the biota.

- Uptake by Wetland Plants: Uptake of pollutants by emergent wetland plants is an indirect removal pathway for nutrients and metals. In nearly all cases, emergent wetland plants take up these substances through their roots, rather than from their leaves. Therefore, the influence of wetland plant uptake is primarily on pollutants that have been previously deposited in the sediments. Exceptions include some species of submerged or floating aquatic vegetation. Although the uptake does not remove nutrients and metals from the water column, it does create new exchange sites within the sediment for future adsorption of pollutants. The nutrients and metals that are translocated to the leaves and stems are not permanently retained, but can be released to the water column when the above-ground part dieback in the Fall.
- Uptake by Algae: Uptake by planktonic or benthic algae is an important removal pathway for soluble pollutants such as phosphate and ammonia within stormwater wetlands. The large volume of standing water within stormwater wetlands is an optimum environment for the growth of algae. Free-floating algae take up the nutrients and convert them into biomass which, in turn, settle out to the wetland sediment. The contribution of algae uptake to the nutrient removal capacity of wetlands is frequently overlooked. Galli (1992) surveyed over twenty wetlands and ponds in Maryland, and found that the wetlands as a group has a higher trophic index, which is a measure of system productivity. Algal mats are often observed on the sediment surface of shallow wetlands, and these mats are thought to be effective in removing nutrients as well.

- Augumented Retention or Detention: Not all the removal pathways in a stormwater wetland are related to the wetland/plant/sediment matrix. In many designs, up to half of the total treatment volume is devoted to a permanent pool and/or temporary extended detention. These treatment volumes can augment sedimentation and algal uptake, and can increase the total contact time within the wetland for adsorption, filtration, and microbial activity. These removal pathways take on added importance during the non-growing season, when marsh removal pathways are not as effective.

2.4.2. Pollutant Removal Capability and Rates of Stormwater Wetlands

The pollutant removal performance of nearly 25 stormwater wetland systems has been reported to date. Although the stormwater wetland systems monitored have differed greatly in their design and treatment volume, most have shown moderate to excellent pollutant removal capability under a range of environmental conditions. From a review of these studies, it is evident that:

- The most reliable overall performance was achieved by pond-wetland systems. In these systems, the permanent pool pretreats runoff before entering a shallow marsh. The pool also reduces runoff velocities and provides considerable pollutant removal. Of particular note has been the demonstrated capability of these systems to provide consistently higher levels of phosphorus and nitrogen removal than other stormwater wetland designs (Figures 2.9 and 2.10).

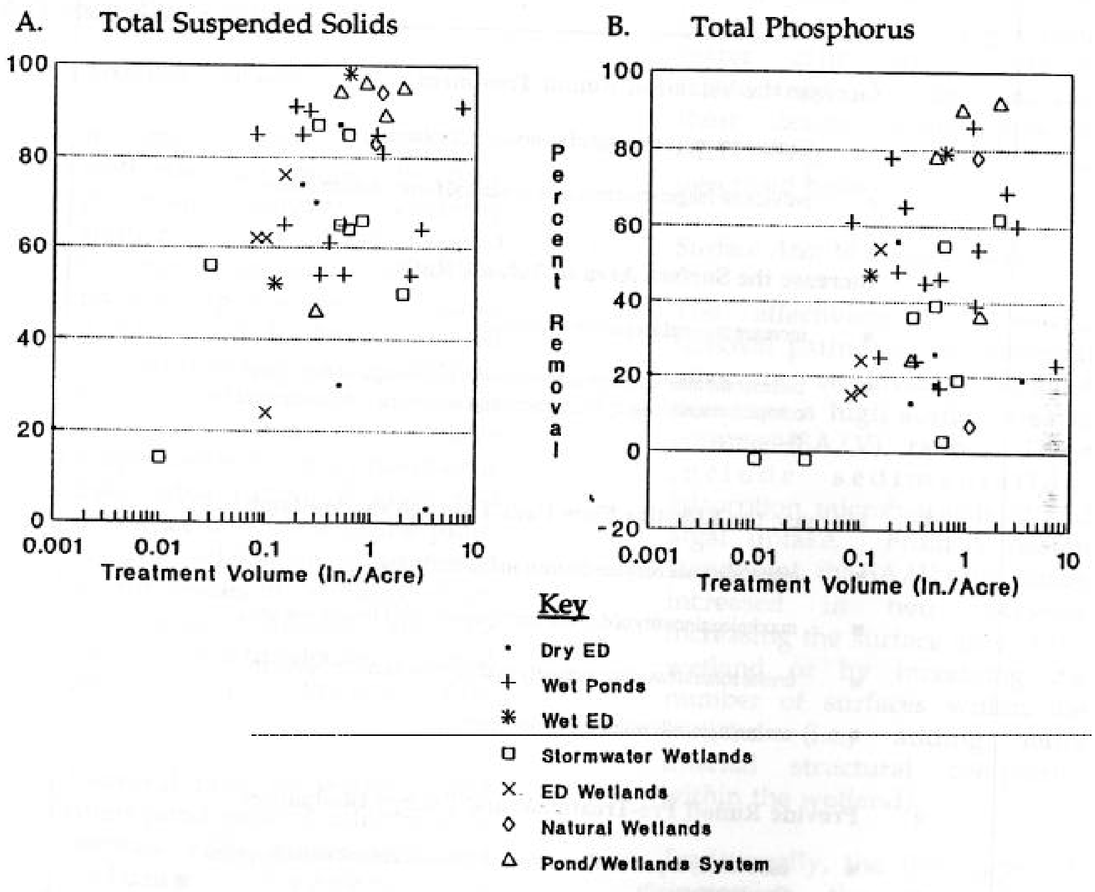

Figure 2.9: Pollutant Removal Performance of Stormwater Ponds and Wetlands: by Treatment Volume (Schueler, 1992)

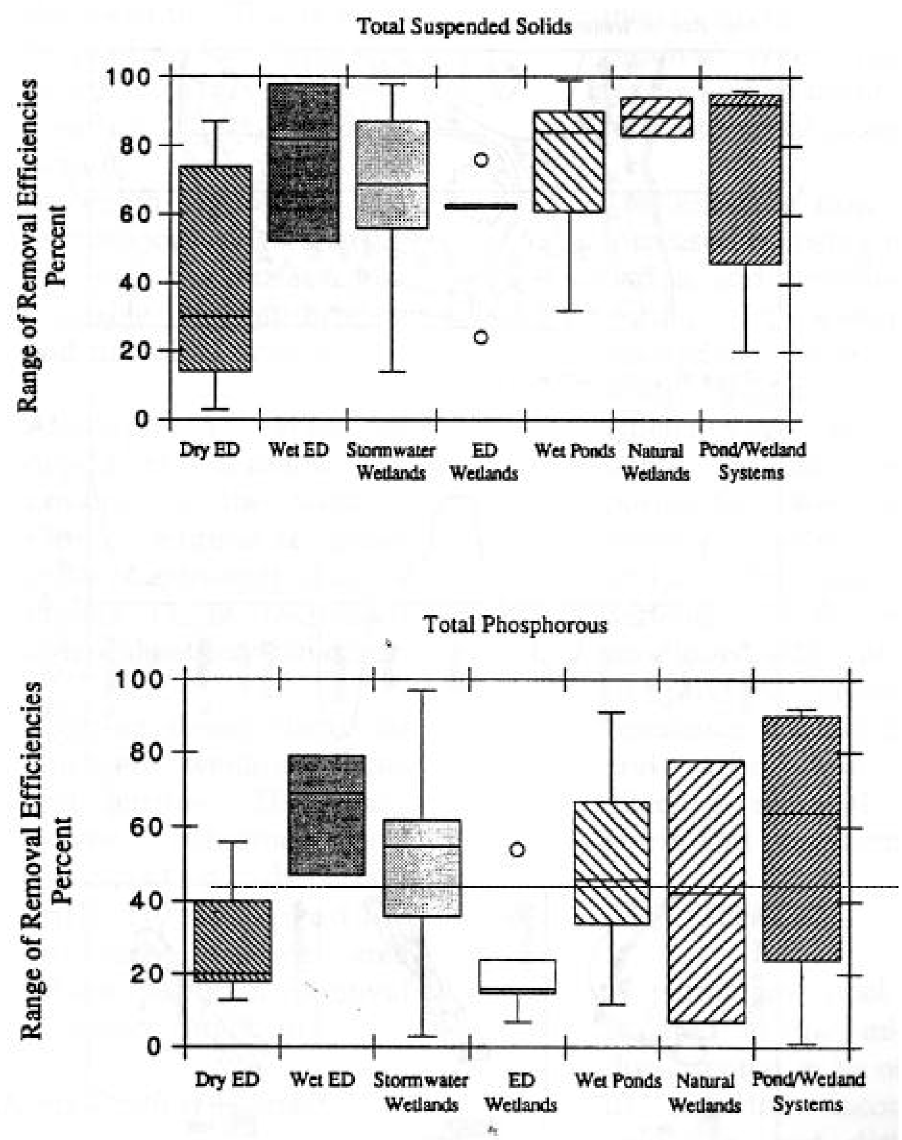

Figure 2.10: Pollutant Removal Performance of Stormwater Ponds and Wetlands: by Pond Type (Schueler, 1992)

- The performance of pocket wetlands has never been monitored. For a number of reasons, it is quite likely that pocket wetlands will not perform as reliable as other stormwater wetland designs.

First, pocket wetlands are excavated to the groundwater, which means that much of the treatment volume capacity is used up by groundwater rather than stormwater. Second, pocket wetlands often lack forebays, are prone to resuspension, and often lack the dense vegetative cover of larger wetlands (Galli, 1992).

- Performance also declines if stormwater wetlands are covered with ice or receive snowmelt runoff. Obert et al. (1989) reports that ice and snowmelt conditions experienced in Minnesota reduced average pollutant removal by 25 to 50%. Nevertheless, good removal rates over the entire year were still observed at the site.

2.5. The HEC-1 - Hydrologic and Hydraulic Calculations

2.5.1. Program Capabilities

The HEC-1 computer program was developed for use in analyzing the hydrologic processes of flood events in river basins varying in size and complexity from small urban catchment to a large multi-basin river system. It is used as a basic tool to determine runoff from synthetic as well as historical events.

Fundamentally, the HEC-1 program has the capability to simulate the precipitation-runoff process and compute flood hydrographs at desired locations in a basin. The physical characteristics of the river basin are represented by an interconnected system of geographic and hydrologic components. The basin boundaries are delineated, and the land area is divided into subbasins on the basis of study objectives and hydrologic characteristics. The model is structured so that lumped parameter can be used to simulate the precipitation-runoff process. Channel routing reaches provide the connecting link between subbasins. Thus, the basic hydrologic components of the model include land-surface runoff from each subbasin, channel and reservoir routing, and the combining of hydrographs at confluences (Hoggan, 1989).

2.5.2. Theoretical Assumptions and Limitations

A river basin is represented as an interconnected group of subareas. The assumption is made that the hydrologic processes can be represented by model parameters which reflect average conditions within a subarea. If such averages are inappropriate for a subarea then it would be necessary to consider smaller subareas within which the average parameters do apply. Model parameters represent temporal as well as spatial averages. Thus, the time interval to be used should be small enough such that averages over the computation interval are applicable.

There are several important limitations of the model. Simulations are limited to a single storm due to the fact that provision is not made for soil moisture recovery during periods of no precipitation. The model results are in terms of discharge and not stage, although stages can be printed out by the program based on a user specified rating curve. A hydraulic computer program (HEC-2 e.g.) is generally used in conjunction with HEC-1 to obtain stages. Streamflow routings are performed by hydrologic routing methods and do not reflect the full St. Vernant equations which are required for very flat river slopes. Reservoir routings are based on the Modified Puls techniques which are not appropriate where reservoir gates are operated to reduce flooding at downstream locations.

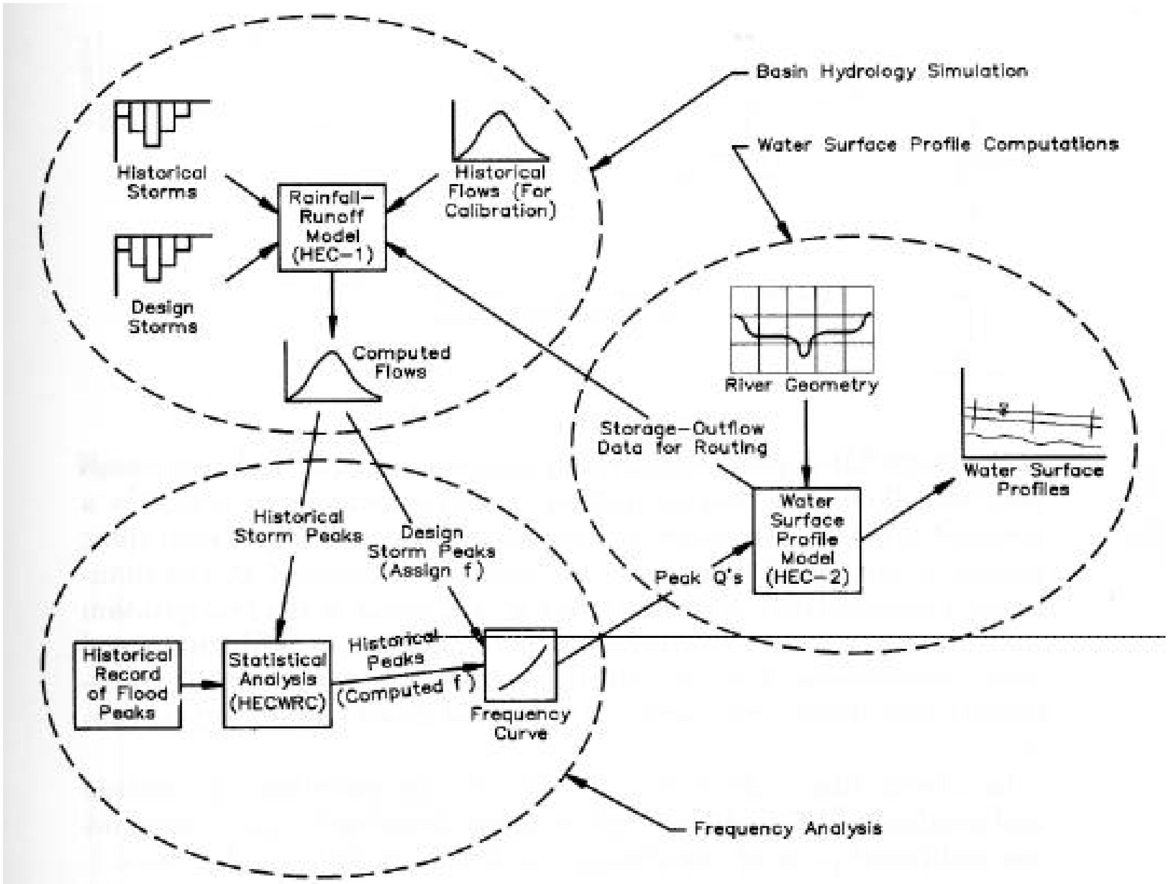

Figure 2.11: Schematic of a Floodplain Hydrologic and Hydraulic Study (Hoggan, 1989)

2.5.3. Model Components / Stream Network Model Development

The stream network simulation model capability is the foundation of the HEC-1 program. All other program computation options build on this option's capability to calculate flood hydrographs at desired locations in a river basin.

A river basin is subdivided into an interconnected system of stream network components using topographic maps and other geographic information. A basin schematic diagram of these components is developed by the following steps:

- The study area watershed boundary is delineated first. In a natural or open area this can be done from a topographic map. However, supplementary information, such as municipal drainage maps, may be necessary to obtain an accurate depiction of an urban basin's extent.
- Segmentation of the basin into a number of subbasins determines the number and types of stream network components to be used in the model. Two factors impact on the basin segmentation: the study purpose and the meteorological variability throughout the basin. First, the study purpose defines the areas of interest in the basin, and hence, the points where subbasin boundaries should occur. Second, the variability of meteorological processes and basin characteristics impacts on the number and location of subbasins. Each subbasin is intended to represent an area of the watershed which, on the average, has the same hydraulic/hydrologic properties. Further, the assumption of uniform precipitation and infiltration over a subbasin becomes less accurate as the subbasin becomes larger. Consequently, if the subbasins are chosen appropriately, the average parameters used in the components will more accurately model the subbasins.
- Each subbasin is represented by a combination of model components. Subbasin runoff, river routing, reservoir, diversion and pump components are available to the user.

- The subbasin and their components are linked together to represent the connectivity of the river basin. HEC-1 has available a number of methods for combining or linking together outflow from different components. This step finalizes the basin schematic.

The subbasin land surface runoff component, such as subbasins 10, 20, 20, ect. In Figure 2.12(a) or equivalently as element 10 in Figure 2.12(b) is used to represent the movement of water over the land surface and in stream channels.

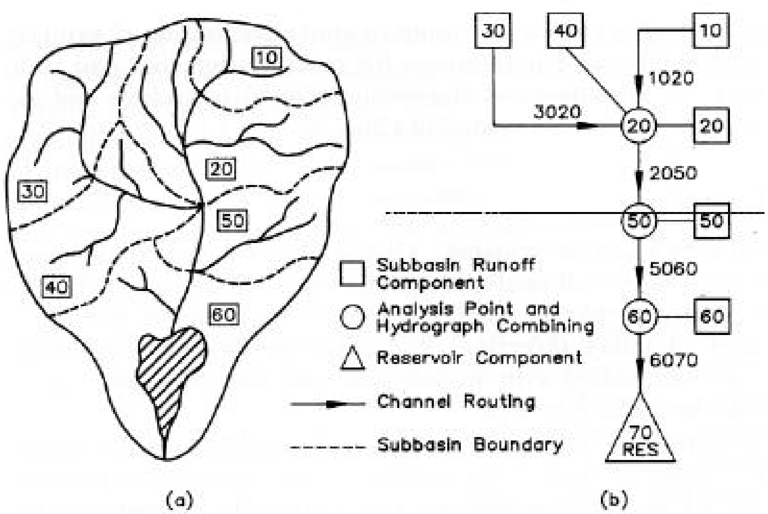

Figure 2.12: River Basin Simulation of the HEC-1 Model (Hoggan, 1989)

(a) Map Subdivided Basin

(b) Network Diagram of Model Components

The input to this component is a precipitation hyetograph. Precipitation excess is computed by subtracting infiltration and detention losses based on a soil water infiltration rate function. Note that the rainfall and infiltration are assumed to be uniform over the subbasin. The resulting rainfall excesses are then routed by the unit hydrograph or Kinematic Wave technique to the outlet of the subbasin producing a runoff hydrograph. The unit hydrograph technique produces a runoff hydrograph at the most downstream point of the subbasin. If that location for the runoff computation is not appropriate, it may be necessary to further subdivide the subbasin or use the Kinematic Wave method to distribute the local inflow.

The Kinematic Wave rainfall excess-to-runoff transformation allows for the uniform distribution of the land surface runoff along the length of the main channel (e.g. subbasin 60, Figure 2.12, runoff could be laterally distributed between point 50 and 60 instead of being lumped at point 60). This uniform distribution of local inflow (subbasin runoff) is particularly important in areas where many lateral channels contribute flow along the length of the main channel.

2.5.4. River Routing Components of HEC-1

A river routing component, element 1020, Figure 2.12, is used to represent flood wave movement in a river channel. The input to the component is an upstream hydrograph resulting from individual or combined contributions of subbasin runoff, river routings or diversions. If the Kinematic Wave method is used, the local subbasin distributed runoff (e.g. subbasin 60 as described above) is also input to the main channel and combined with the upstream hydrograph as it is routed to the end of the reach. The hydrograph is routed to a downstream point based on the characteristics of the channel. There are a number of techniques available to route the runoff hydrograph.

Consider the use of subbasin runoff components 10 and 20 and river routing reach 1020 in Figure 2.12(b) and the corresponding subbasins 10 and 20 in Figure 2.12(a) The runoff from component 10 is calculated and routed to control point 20 via routing reach 1020. The runoff hydrograph at analysis point 20 can be calculated by methods employing either the unit hydrograph or Kinematic Wave techniques. In the case that the unit hydrograph technique is employed, runoff from component 10 is calculated and routed to control point 20 via routing reach 1020. Runoff from subbasin 20 is calculated and combined with the outflow hydrograph from reach 1020 at analysis point 20. Alternatively, runoff from subbasins 10 and 20 can be combined before routing in the case that the lateral inflows from subarea 20 are concentrated near the upstream end of the reach 1020. In the case that the Kinematic Wave technique is employed, the runoff from subbasin 20 is modeled as a uniformly distributed lateral inflow to reach 1020. The runoff from subbasin 10 is routed in combination with this lateral inflow via reach 1020 to analysis point 20.

A suitable combination of subbasin runoff component and river routing components can be used to represent the intricacies of any rainfall-runoff and stream routing problem. The connectivity of the stream network components is implied by the order in which the data components are arranged. Simulation must always begin at the uppermost subbasin in a branch of the stream network. The simulation (succeeding data components) proceeds downstream until a confluence is reached. Before simulating below the confluence, all flows above that confluence must be computed and routed to that confluence. The flows are combined at the confluence and the combined flows are routed downstream. In Figure 2.12, all flows tributary to control point 20 must be combined before routing through reach 2050.

Distributed outflow from a subbasin may be obtained by utilizing combinations of three conceptual elements: overland flow planes, collector channels, and a main channel. The Kinematic

Wave routing technique can be used to route rainfall excess over the overland flow planes. Either the Kinematic Wave or Muskingum technique can be used to route lateral inflows through a collector channel, and upstream and lateral inflow through the main channel. However, both methods cannot be inter-mixed.

The stream network system, or flow chart of the HEC-1 model, used to obtain outflow hydrographs of the watershed is presented in Figure A1.

2.5.5. Kinematic Wave Routing

In the Kinematic Wave interpretation of the equations of motion, it is assumed that the bed slope and water surface slope are equal and acceleration effects are negligible. The momentum equation then simplifies to:

$$S_f = S_0 \tag{2.1}$$

where S_f is the friction slope and S_0 is the channel slope.

Thus, flow at any point in the channel can be computed from Manning's formula, Eq. 4.2.

Since the momentum equation has been reduced to a simple functional relation between area and discharge ($Q = \alpha A^m$), the movement of a flood wave is described solely by the continuity equation

$$\frac{A}{t} + \frac{Q}{x} = q \tag{2.2}$$

The channel routing computation can be utilized independently of the other elements of the subbasin runoff. The cross-sectional geometry is limited to circular, triangular, square, rectangular, and trapezoidal shapes. Theoretically, a flood wave routed by the Kinematic Wave technique through these channel sections is translated, but does not attenuate (although a degree of attenuation is introduced by the finite difference solution). Consequently, the Kinematic Wave routing technique is most appropriate in channels where flood wave attenuation is not significant, as is typically the case in urban areas.

2.5.6. Muskingum Routing

The Muskingum method computes outflow from a reach using the following equations:

QOUT(2) = (CA - CB) * QIN(1) + (1 - CA) * QOUT(1) + CB * QIN(2) (2.3)

$$CA = \frac{2 * \Delta t}{2 * AMSKK * (1 - X) + \Delta t} \tag{2.4}$$

$$CB = \frac{\Delta t - 2 * AMSKK * X}{2 * AMSKK * (1 - X) + \Delta t} \tag{2.5}$$

where QIN is the outflow to the routing reach [cfs],

QOUT is the outflow from the routing reach [cfs],

AMSKK is the travel time through the reach [hours], and

X is the Muskingum weighting factor ($0 \le X \le 0.5$)

The routing procedure may be repeated for several subreaches (designated in the HEC-1 model as NSTPS) so the total travel time through the reach is AMSKK. To insure the method's computational stability and the accuracy of computed hydrographs, the routing reach should be chosen so that:

$$\frac{1}{2(1 - X)} \le \frac{AMSKK}{NSTPS * \Delta t} \le \frac{1}{2X} \tag{2.6}$$

2.5.7. SCS Dimensionless Unit Hydrograph

The parameter for the synthetic unit hydrograph can be determined from gage data by employing the parameter optimization option. Otherwise, these parameters can be determined from regional studies or from guidelines given in references for each synthetic technique. There are three synthetic unit hydrograph methods available in the model.

Input data for the Soil Conservation Service, SCS (1985), dimensionless unit hydrograph method consists of a single parameter, TLAG, which is equal to the lag [hrs] between the center of mass of rainfall excess and the peak of the unit hydrograph. Peak flow and time to peak are computed as:

$$TPEAK = 0.5 * \ t + TLAG \qquad (2.7)$$

$$QPK = 484 * AREA/TPEAK$$

(2.8)

where TPEAK is the time to peak of unit hydrograph [hours],

t is the duration of excess [hours] or computational interval,

QPK is the peak flow of unit hydrograph [cfs/inch], and

AREA is the subbasin area]mi^2].

The unit hydrograph is interpolated for the specified computation interval and computed peak flow from the dimensionless unit hydrograph.

The selection of program computation interval, which is also the duration of the unit hydrograph, is based on the relationship t = 0.2 * TPEAK (HEC-1 User's Manual, 1990). There is some latitude allowed in this relationship; however, the duration of the unit graph should not exceed t = 0.2 * TPEAK. These relations are based on an empirical relationship, TLAG = 0.6 * Tc, and 1.7 * TPEAK = t + Tc, where Tc is the time of concentration of the watershed. Using these relationships, along with equation

$$1 - AI = 1.414\ (1 - T)^{1.5} \qquad 0.5 < T < 1 \tag{2.9}$$

where AI is the cumulative area as a fraction of the total subbasin area and

T is the fraction of time of concentration

It is found that the duration should not be greater than t ≤ 0.29 * TLAG.

2.5.8. Rainfall-Runoff Simulation

The HEC-1 model components are used to simulate the rainfall-runoff process as it occurs in an actual river basin. The model components are based on simple mathematical relationships which are intended to represent individual meteorological, hydrologic and hydraulic processes which comprise the precipitation-runoff process. These processes are separated into precipitation, interception/infiltration, transformation of precipitation excess to subbasin outflow, addition of baseflow and flood hydrograph routing.

A precipitation hyetograph is used as the input for all runoff calculations. The specified precipitation is assumed to be basin average (i.e., uniformly distributed over the subbasin). Any of the options used to specify precipitation produce a hyetograph. The hyetograph represents average precipitation (either rainfall or snowfall) depth over a computation interval.

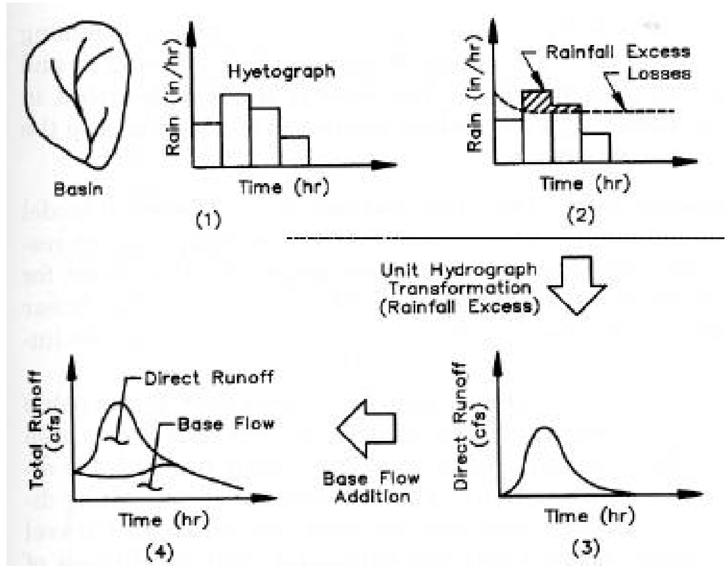

Figure 2.13: Unit Hydrograph Method (Hoggan, 1989)

(1) Determine Rainfall Hyetograph

(2) Subtract Losses to Obtain Rainfall Excess

(3) Transform Rainfall Excess into Direct Runoff

(4) Add Base Flow to Obtain Total Runoff

Land surface interception, depression storage and infiltration are referred to in the HEC-1 model as precipitation losses. Interception and depression storage are intended to represent the surface storage of water by trees or grass, local depressions in the ground surface, in cracks and crevices in parking lots or roofs, or in an surface area where water is not free to move as overland flow. Infiltration represents the movement of water into the soil.

Two important factors should be noted about the precipitation loss computation in the model. First, precipitation which does not contribute to the runoff process is considered to be lost from the system. Second, the equations used to compute the losses do not provide for soil moisture or surface storage recovery. (The Holtan loss rate option is an exception in that soil moisture recovery occurs by percolation out of the soil moisture storage.) This fact dictates that the HEC-1 program is a single-event-oriented model.

The precipitation loss computations can be used with either the unit hydrograph or Kinematic Wave model components. In the case of the unit hydrograph component, the precipitation loss is considered to be a subbasin average (uniformly distributed over the entire subbasin). On the other hand, separate precipitation losses can be specified for each overland flow plane in the Kinematic Wave component. The losses are assumed to be uniformly distributed over each overland flow plane.

In some instances, there are negligible precipitation losses for a portion of a subbasin. This would be true for an area containing a lake, reservoir or impervious area. In this case, precipitation losses will not be computed for a specified percentage of the area labeled as impervious.

There are five methods that can be used to calculate the precipitation loss. Using any one of the methods, an average precipitation loss is determined for a computation interval and subtracted from the rainfall/snowmelt hyetograph. The resulting precipitation excess is used to compute an outflow hydrograph for a subbasin. A percent imperviousness factor can be used with any of the loss rate methods to guarantee 100% runoff from that portion of the basin (HEC-1 User’s Manual, 1990).

2.5.9. SCS Curve Number Computational Method

The Soil Conservation Service, U.S. Department of Agriculture, has instituted a soil classification system for the use in soil survey maps across the country. Based on experimentation and experience, the agency has been able to relate the drainage characteristics of soil groups to a curve number, CN (SCS, 1985). The SCS provides information on relating soil group types to the curve number as a function of soil cover, land use type and antecedent moisture conditions.

Precipitation loss is calculated based on supplied values of CN and IA (where IA is an initial surface moisture storage capacity in units of depth). CN and IA are related to a total runoff depth for a storm by the following relationships:

$$ACEXS = \frac{(ACRAN - IA)^2}{ACRAN - IA + S} \tag{2.10}$$

$$S = \frac{1000 - 10 * CN}{CN} \tag{2.11}$$

where ACEXS is the accumulated excess [inches],

ACRAN is the accumulated rainfall depth [inches], and

S is the currently available soil moisture storage deficit [inches].

In the case that the user does not wish to specify IA, a default value is computed as:

$$IA = 0.2 * S. \tag{2.12}$$

Substituting this approximation in Equation (2.11) gives

$$Q = \frac{(P - 0.2P)^2}{P + 0.8P} \tag{2.13}$$

This relation is based on empirical evidence established by the Soil Conservation Service. Since the SCS method gives total excess for a storm, the incremental excess (the difference between rainfall and precipitation loss) for a time period is computed as the difference between the accumulated excess at the end of the current period and the accumulated excess at the end of the previous period.

2.6. Reservoir Routing and Outlet Structure Characteristics

2.6.1. VT/PSUHM Program and Reservoir Routing

The first version of the PSUHM was issued in October 1984 in a Pennsylvania State University continuing education short course on computational methods in stormwater management. Since then, the original software has been modified substantially through annual revisions. The present hydrologic and hydraulic capabilities in VT/PSUHM include design storm and hyetograph calculations, curve number weighting, rational and modified rational methods, SCS curvilinear unit hydrograph, SCS TR-55 tabular hydrograph, Muskingum channel routing, modified Puls reservoir routing, hydrograph combining, and plotting and sizing of outlets in a multi-stage detention structure (MSRM).

VT/PSUHM provides a reservoir routing option applying the modified Puls routing method (Chow, 1959). This routing method is a variation of the storage routing method described by Henderson (1966). It is applicable to both channel and reservoir routing. The modified Puls method routes a hydrograph through a reservoir using the storage indication method also described in Viessman et al. (1989). This method is based on mass conservation principles and the hydraulics of the reservoir outlet structure. The user is prompted to supply an elevation-storage curve, which was created by the MSRM module, an elevation-outflow curve, based on the basin geometry and obtained from AutoCAD drawings, and an inflow hydrograph.

The modified Puls method uses a finite-difference form of the continuity equation combined with a storage indication curve (2S/Δt + O vs. O). The finite-difference equation for two points in time is:

$$(I_n + I_{n+1}) + \left(\frac{2S_n}{\Delta t} - O_n\right) = \left(\frac{2S_{n+1}}{\Delta t} + O_{n+1}\right) \tag{2.14}$$

in which the unknowns are S_{n+1} and O_{n+1} on the right-hand side. I is known for all n, and S_n and O_n are known for the initial time step. Therefore, the right-hand side of Equation (2.14) can be calculated. Values of S_{n+1} and O_{n+1} are then used as input on the left-hand side and the calculation is repeated for the second time interval and so on. The storage indication curve is a plot of 2S/Δt + O vs. O. Thus, once the right-hand side of Equation (2.14) has been determined, one can read values of O directly from the curve.

2.6.2. Perforated Riser Outlet Structure

The rate of discharge from a vertical riser with a single column of perforations of diameter D_O [inches] spaced a distance h [ft] apart vertically can be computed from the following relationship assuming an elevation datum at the centerline of the lowest perforation

$$Q(z) = \sum_{i=1}^{n} Q_i \tag{2.15}$$

where n is z/h

z is the elevation of the water level above the lowest perforation,

i is the perforation number with “1” being the lowest perforation, and

Q_i is the discharge through perforation i [cfs] and can be represented as

$$Q_i = 0.0267 * D_O^2 * H_i^{0.5} \tag{2.16}$$

H_i, the head [ft] on perforation i, can be represented as a function of the elevation z and the spacing between the perforations as $H_i = z - (i-1)h$. If the coefficient 0.0267 D_O^2 in Equation (2.16) is set to a_o, the discharge from a vertical riser with one column of perforations can be represented as

$$Q(z) = a_o \sum_{i=1}^{n} \left(z - (i-1)h\right)^{0.5} \tag{2.17}$$

Evaluations of Equation (2.17) done by Jarrett (1996) yield the rating curves shown in Figures 2.14 and 2.15 for 0.5- and 1.0-ft vertical spacing, respectively. Each rating curve is of the form

$$Q = az^m \tag{2.18}$$

where $a = f(h, D_O)$ for one column of perforations (a_t and a_b),

m = 0.5 for the zone between the elevation of the lowest perforation and the elevation of the second perforation, and

m = 1.38 was determined by regression of Equation (2.18) results for the zone between the second perforation and the top of water storage.

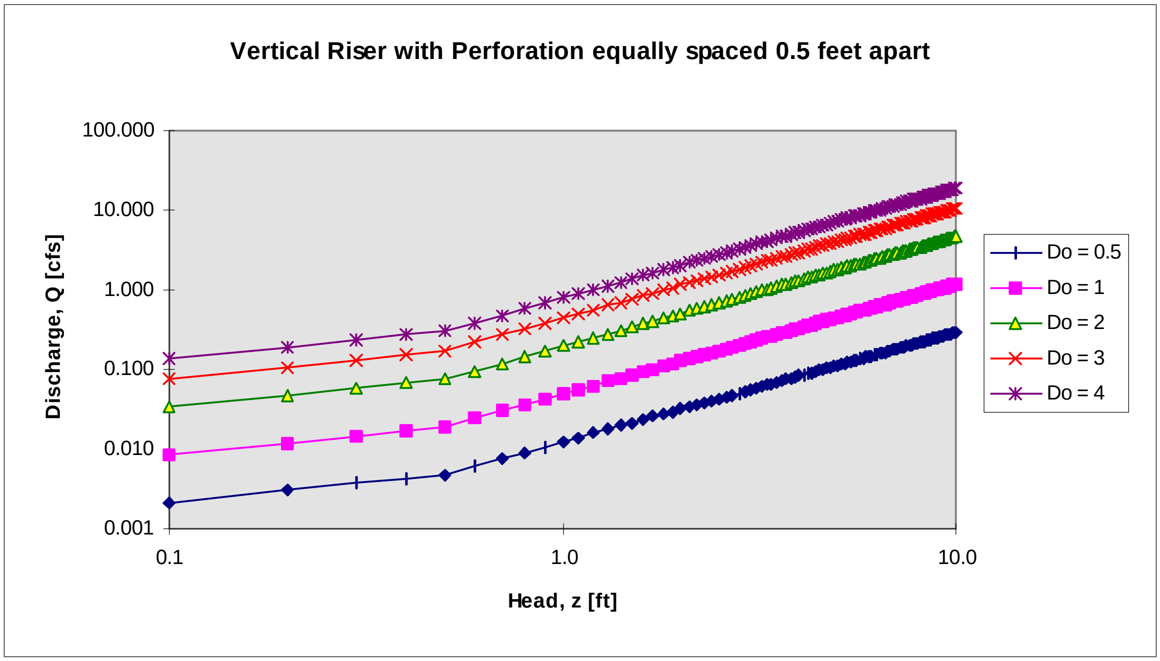

Figure 2.14: Vertical Riser with Perforation Equally Spaced 0.5 Ft Apart

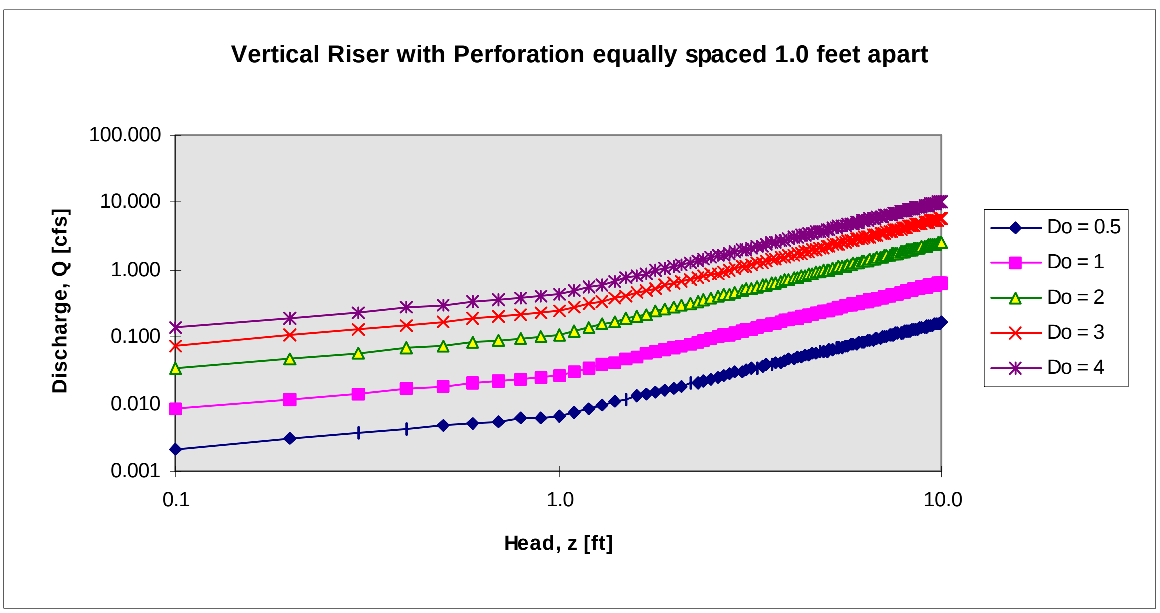

Figure 2.15: Vertical Riser with Perforation Equally Spaced 1.0 Ft Apart

Table 2.2: Relation of a_t and a_b with Respect to D_O and h

h [ft]	Do [in]	a_b	a_t
0.5	0.5	0.0068	0.0123
0.5	1.0	0.0267	0.0492
0.5	2.0	0.1070	0.1960
0.5	3.0	0.2400	0.4420
0.5	4.0	0.4270	0.7890
1.0	0.5	0.0068	0.0068
1.0	1.0	0.0267	0.0267
1.0	2.0	0.1070	0.1070
1.0	3.0	0.2400	0.2400
1.0	4.0	0.4270	0.4270

With recognition that the discharge relationship for the top and bottom zones of the perforated risers are equal at the elevation of the number 2 perforation it is possible to refine equation (2.18) into a more easily used design equation. At the number 2 perforation, the discharge from the top zone discharge equation, $Q_t = a_t z^{1.38}$ is equal to the discharge from the bottom zone discharge equation, $Q_b = a_b z^{0.5}$. By equating these two relationships, a relationship between a_t and a_b can be developed as $a_b = a_t z^{0.88}$.

The discharge for multiple columns of perforations of the riser, is determined by

$$a_{tot} = n * a_1 \qquad (2.19)$$

where n is the number of columns of perforation and

a_1 is the riser design factor for a perforated riser with only one column of perforations.

2.6.3. Skimmer Outlet Structure

A skimmer, as developed by Jarrett (1996), is shown schematically in Figure 2.16. The skimmer floats on the water surface, where the “C” enclosure is supposed to provide the necessary buoyancy to suspend the riser. The “C” enclosure also keeps floating trash from the horizontal perforated water entry pipe centered within the “C” closure, located just under the water surface. The base of the riser is fixed, via a 4-inch diameter flexible hose to a 4-inch diameter pipe which receives the water discharged from the basin. The flexible hose permits articulation of the riser.

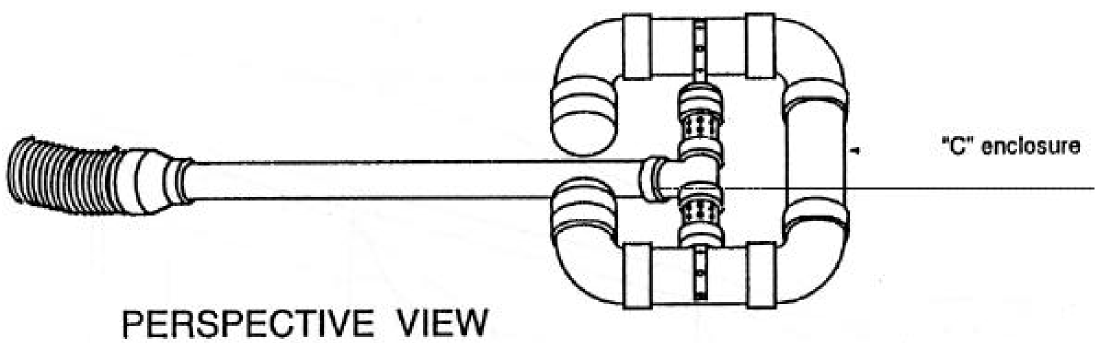

Figure 2.16: Perspective View of Skimmer (Jarrett, 1996)

The flow of water through the skimmer is controlled by an orifice located at either the top or base of the riser. The flow of water through the orifice is controlled by:

$$Q = 0.0267 * D_O^{\ 2} * H^{0.5} \tag{2.20}$$

where Q is the discharge [cfs],

D_O is the orifice diameter [in], and

H is the depth of water [ft] driving water through the orifice.

The driving head is the vertical elevation difference between the water surface and the center of the orifice (H = d + b).

In order to develop an expression to predict the skimmer outflow rate from Equation (2.20), it is necessary to determine how the driving Head, H, varies as the length of the skimmer riser L, from the flexible hose to the articulating pin connecting the riser to the floating “C” closure, and the distance from the articulating pin to the orifice in the riser. The unknown H is the sum of d + b. Considering angle relations one will finally obtain the expression H = d + l/L (D - d). By substituting into Equation (2.20), Q can be expressed as

$$Q = 0.0267 * D_O^{\ 2} * \left(d + \frac{l}{L}(D - d) \right)^{0.5} \tag{2.21}$$

All skimmer dimensions must be in the same units, preferably ft.

In the case where the orifice is at the top of the riser, the discharge is nearly constant for most of the dewatering period and declines only when nearly all of the impounded water has been removed. In the case where the orifice is at the base of the riser, the discharge rate increases rapidly to a peak, then declines as dewatering proceeds. Note that the dewatering time for the two configurations is quite different, even though the orifice is the same diameter in both cases. In tests run by Jarrett (1996), dewatering required nearly twice as long when the orifice was at the base rather than at the top of the riser. By changing the length dimensions of the skimmer, as well as the orifice diameter, any dewatering time can be made to occur.

2.6.4. Proportional Weir

The proportional weir (or “Sutro” weir, though the Sutro shape is only one of many similar shapes) is distinguished from other control devices by having a linear head-discharge relationship achieved by allowing the discharge area to vary non-linearly with head. Although more complex to design and construct, a proportional weir may significantly reduce the required storage volume for a given site.

Much of the initial work was done on proportional weirs in the late 1800’s to early 1900’s. After the invention of the Sutro weir, the interest in the proportional weir vanished. Then in the 1970’s and 1980’s, new designs and research increased the interst in proportional weir design again. Much of recent work has been done in India. This work strays from the conventional Sutro weir to incorporate new shapes in both the base weir and the complementary weirs using some unique combination. Some designs include inverted V-notch weirs, logarithmic complementary weirs with rectangular, triangular, parabolic trapezoidal and reverse parabolic base weirs, and weirs with circular bottoms or circular tops. Each weir has its own set of equations for both flow and shape design considerations; however, each still has the property of a linear head-discharge curve.

Most detention facilities that serve multiple frequency storms are ponds stacked with their runoff volumes one atop another in the same pond, with the invert of an outlet occurring at the top of the storage for the next lowest frequency storm analyzed. Thus, with multiple outlets, there is a transition period from, say, a 10-year to a 25-year storm. At the intermediate storm frequencies, the non-proportional weir releases peak flows that are higher than historic discharges. With a single proportional weir, all storms between design frequencies are maintained at their historic flow rates.

As a good example of a well-functioning proportional weir the construction of such a weir for the new Meridian Office Park just south of Denver should be mentioned. The detention facility was designed for a 1300-ac site south of Denver, developed by Hartford Insurance.

The projected impervious area for developed on-site lots was 55%, meaning increased peak storm flow rates would result. The basin considered for the proportional weir application was 1400 ac, including contributing off-site area of 890 ac. This currently undeveloped land was assumed to release stormwater at historic rates in the future even if development occured there. The on-site area was 510 ac located in the lower end of the basin. The county required that the developed storm flow rate be restricted to the maximum historic flow only during the 100-year storm. Engineers decided to design a structure that would serve a wide range of storm frequencies up to a maximum of the 100-year storm. The outlet structure chosen was a large proportional weir.

The maximum storage volume and a restricted reservoir depth of less than 10 ft was used, because a pond depth of 10 ft or greater in this case would have classified the pond as a dam under State of Colorado Engineer’s criteria and would have required time-consuming project submittals and approvals.

Values of height of the rectangular base and the width of the weir were approximately 1 ft and 40 ft, respectively.

The proportional weir has one unique characteristic that distinguishes it from other flow measuring or control devices. It has a linear head-discharge relationship.

Formulas for design of proportional weirs are as follows (Corcoran, 1995):

$$Q = C \cdot (2 \cdot g)^{0.5} \cdot a^{1/2} \cdot b \cdot \left(h \cdot a/3\right) \tag{2.22}$$

where C = 0.62 ± 0.02

g = 32.2 ft/s^2

$$Q = 4.97 \cdot a^{1/2} \cdot b \cdot \left(h \cdot a/3\right) \tag{2.23}$$

$$x/b = 1 - 2/ \cdot \left(ARCTAN \sqrt{y/a} \right) \tag{2.24}$$

Where Q is the quantity of flow [cfs],

a is the heigth of the rectangular base [ft],

b is the width of the weir [ft], and

the diminesions of h, x, and y are [ft].

2.7. Embankment Construction and Design

The basic principle of design should be the production of a satisfactory functional structure at a minimum total cost. For minimum cost, the dam must be designed for maximum utilization of the most economical materials available, including material which must be excavated for its foundations and for appurtenant structures.

An earthfill dam must be safe and stable during all phases of construction and operation of the reservoir. To accomplish that, the following criteria must be met (Bureau of Reclamation, 1965):

- The embankment must be safe against overtopping during occurrence of the inflow design flood by the provision of sufficient spillway and outlet works capacity.
- The slopes of the embankment must be stable during construction and under all conditions of reservoir operation, including rapid drawdown of storage dams.
- The embankment must be designed so as not to impose excessive stresses upon the foundation.
- Seepage flow through the embankment, foundation, and abutments must be controlled so that no internal erosion takes place and so there is no sloughing in the area where the seepage emerges. The amount of water lost through seepage must be controlled so that it does not interfere with planned project functions.
- The embankment must be safe against overtopping by wave action.

The upstream slope must be protected against erosion by wave action and the crest and downstream slope must be protected against erosion due to wind and rain.

For embankment heights less than 50 feet, compaction of cohesive soils at optimum water content and approximately Proctor maximum dry density insures sufficient air, even in most compressible soils, to preclude the development of pore water pressure of appreciable magnitude.

Placing of the material dry of optimum may result in :

- low density for the same compactive effort
- increased permeability of the embankment
- excessive softening and settlement on saturation by the reservoir, resulting in possible cracking of the fill.

On the other hand, the water content should not be appreciably greater than optimum for Proctor maximum dry density because difficulties have been experienced with unstable fills when very wet soils were used, even in dams of low height. The foregoing considerations result in the recommended practice of compacting cohesive soils in the cores of small dams close to the optimum water content at Proctor maximum dry density.

3. REGULATORY FRAMEWORK

3.1. Safe Dams Act

National responsibility for the protection and coordination of dam safety lies with the Federal Emergency Management Agency (FEMA). In addition, state agencies have some responsibility for administration of the provisions of the Federal Dam Safety Act. Under the federal regulations, a dam is an artificial barrier that does or may impound water, that is 20 feet or greater in height, or that has a storage volume of 30 acre-feet or more. Detailed engineering requirements are given in the regulations for new dams and these regulations should be consulted for all engineering requirements. A number of exemptions are allowed from the Safe Dams Act and the cognizant state office should be contacted to resolve questions. (Debo and Reese, 1995)

3.2. Wetland Regulations

There are 26 nationwide permits pertaining to wetlands. Landowners may modify wetlands under 22 National Wetland Permits (NWP), but four of the permits require approved individual permits before wetland modification. As the landowner meets the condition and management practices applicable to the NWPs, the landowner can proceed with wetland disturbance. Among the more important of the fourteen conditions are that the discharges not occur in areas of concentrated shellfish production, not jeopardize a threatened or endangered species, or not adversely affect historic properties either on or determined to be eligible for listing on the National Register of Historic Places. There are eight management practices, which require to the maximum extent practicable, among other things, that discharges be minimized, and avoided in spawning areas during spawning season and in breeding areas for migratory waterfowl, and that heavy equipment used for work in wetlands be placed on mats (Law of Wetland Regulation, 1989).

The Corps of Engineers promulgated new regulations on October 5, 1984, for isolated wetlands and waterbodies and for those located above headwaters. Headwaters are defined as the “point on a non-tidal stream above which the average annual flow is less than five cubic feet per second”. Nationwide Permit 26 exempts from the individual permit requirement activities that cause the loss or substantial adverse modification of wetlands areas and waters under one acre and requires notification to the Corps if the affected wetlands area is one acre or greater and less than ten. The Corps has 20 days after notification to decide whether to require an individual permit. Only areas less than one acre were totally excluded from federal jurisdictional coverage (Regulatory Guidance Letter, 1988).

3.3. Permit on Wetland Conversion

The actual authority to permit the conversion of a natural wetland to a stormwater wetland solely resides with the state and/or federal wetland permitting agency, and will usually be made on a case by case basis. The applicant will need to secure a 401 water quality certification and a 404 wetland permit to proceed, and will often be required to provide extensive supporting data to make the change.

The regulatory status of stormwater wetlands and other stormwater ponds is still being defined as of this writing. In particular, the circumstances whereby stormwater wetland systems can be located in degraded urban wetlands, or even in "waters of the United States" are actively being reviewed by regulatory authorities at both the regional and national level (Debo and Reese, 1995).

3.4. Stormwater Management Regulations

The Virginia Department of Conservation and Recreation (1990) defines a stormwater management facility as a device that controls stormwater runoff and changes the characteristics of that runoff including, but not limited to, the quantity and quality, the period of release or the velocity of the flow. A stormwater retention basin or retention basin is defined as a stormwater management facility, which similar to a detention basin, temporarily impounds runoff and discharges its outflow through a hydraulic outlet structure to a downstream conveyance. It may also include a permanent impoundment and is normally wet, even during non-rainfall periods. Storm runoff inflows are temporarily stored above this permanent pool level.

3.4.1. Requirements on Stormwater Management

The Stormwater Management Regulations (1990) provide several requirements or technical criteria. The most important ones for the present project are:

- A stormwater management plan for a land development project shall be developed so that from the site, the post-development peak runoff rate from a 2-year storm and a 10-year storm, considered individually, shall not exceed their respective pre-development rates.
- The design storms shall be defined as either a 24-hour storm using the rainfall distribution recommended by the U.S. Soil Conservation Service when using U.S. Soil Conservation methods or as the storm of critical duration that produces the greatest required storage volume at the site when using a design method such as the Rational Method.
- Localities shall require impounding structures that are not covered by the Virginia Dam Safety Regulations to be checked for structural integrity and floodplain impacts for the 100-year storm event.
- Pre-development and post-development runoff rates shall be verified by calculations that are consistent with good engineering practices and are acceptable to the locality.
- Outflows from a stormwater management facility shall be discharged to an adequate channel, or velocity dissipaters shall be placed at the outfall of all detention and retention basins and along the length of any outfall channel as necessary to provide a non-erosive velocity of flow from the basin to a channel.

In addition to the technical criteria, the Department of Conservation and Recreation has set the following water quality requirements.

- For a detention basin, the water quality volume shall be detained and released over 30 hours. The detention time is a brim-drawdown time and therefore, shall begin at the time of peak storage of the water quality volume in the detention basin. If this requirement would result in an outlet opening smaller than three inches in diameter or the equivalent cross sectional area, the period of detention shall be waived so that three inches will be the minimum outlet opening used.

- For a retention basin, the volume of the permanent pool must be at least three times greater than the water quality volume.

3.5. Minimum Standards of Erosion and Sediment Control Regulations

The purpose of the regulations is to form the basis for the administration, implementation, and enforcement of the Act, the Erosion and Sediment Control Law. The intent of these regulations is to establish a framework for compliance with the Act while at the same time providing flexibility for innovative solutions to erosion and sediment control concerns (Virginia Department of Conservation and Recreation. 1996).

The regulations include minimum standards, of which a few are listed below.

- Permanent or temporary soil stabilization shall be applied to denuded areas within 7 days after final grade is reached on any portion of the site. Temporary soil stabilization shall be applied within 7 days to denuded areas that may not be at final grade but will remain dormant for longer than 30 days. Permanent stabilization shall be applied to areas that are to be left dormant for more than one year.
- During constructions, soil stockpiles and borrow areas shall be stabilized or protected with sediment trapping measures. The applicant is responsible for the temporary protection and permanent stabilization of all soil stockpiles on site as well as borrow areas and soil intentionally transported from the project site.

- A permanent vegetative cover shall be established on denuded areas not otherwise permanently stabilized. Permanent vegetation shall not be considered established until a ground cover is achieved that, is uniform, mature enough to survive and will inhibit erosion.
- Stabilization measures shall be applied to earthen structures such as dams, dikes, and diversions immediately after installation.
- Cut and fill slopes shall be designed and constructed in a manner that will minimize erosion. Slopes that are found to be eroding excessively within one year of permanent stabilization shall be provided with additional slope stabilizing measures until the problem is corrected.
- Whenever water seeps from a slope face, adequate drainage or other protection shall be provided.
- Before newly constructed stormwater conveyance channels or pipes are made operational, adequate outlet protection and any required temporary or permanent channel lining shall be installed in both the conveyance channel and receiving channel.
- When work in a live watercourse is performed, precautions shall be taken to minimize encroachment, control sediment transport and stabilize the work area to the greatest extent possible during construction. Nonerodible material shall be used for the construction of causeways and cofferdams. Earthen fill may be used for these structures if armored by nonerodible cover materials.
- The bed and banks of a watercourse shall be stabilized immediately after work in the watercourse is completed.
- Where construction vehicle access routes intersect paved or public roads, provision shall be made to minimize the transport of sediment by vehicular tracking onto paved surface. Where

sediment is transported onto paved or public road surface, the road surface shall be cleaned thoroughly at the end of each day.

- All temporary erosion and sediment control measures shall be removed within 30 days of final site stabilization or after temporary measures are no longer needed, unless otherwise authorized by the local program authority.
- Properties and waterways downstream from the development sites shall be protected from sediment deposition, erosion and damages due to increases in volume, velocity, and peak flow rate of stormwater runoff for the stated frequency storm of 24-hour duration.

4. HYDROLOGY AND MODELLING CHARACTERISTICS

4.1. Data and Model Information

The data used to determine the direct runoff from the 355-acre portion of the watershed was obtained from the “Stroubles Creek Regional Study and Stormwater Management Plan” (1995) of Hayes, Seay, Mattern & Mattern (HSMM). This plan prepared a storm drainage structure inventory, determined existing and future flow conditions, and made recommendations to improve drainage on the Virginia Tech campus.

Hayes, Seay, Mattern & Mattern divided the 3300-acre watershed, which included most of Virginia Tech and the Town of Blacksburg, into two components - a detailed study area and a non-detailed study area. Unfortunately, the proposed project area was not included in the detailed study area. Thus, limited but sufficient data were available to model the hydrologic characteristics of the watershed. HSMM quantified the severity and frequency of existing flooding and projected future flooding conditions. Upon the request of Virginia Tech, HSMM reran their hydrologic/hydraulic model XP-SWMM and generated outflow hydrographs for the proposed stormwater management facility. However, information on landuse of the School of Veterinary Medicine and detailed areal subdivision could not be concluded from their study. For this reason, nodes of their generated network system representing only a 355-acre portion of the watershed were used in investigations with a different precipitation-runoff model.

The HSMM study area can be roughly divided into three portions. The upper third includes single-family and multi-family residences. This area has substantial development. The middle third of their study area generally lies close to the Lane Stadium. This area contains a great deal of built-up or “urban” portion of the Virginia Tech campus. The majority of future development, within this portion of the study area, will be extending the urban character of the area by constructing the new softball field and the track-and-field building. The lower third of the study area, between Duck Pond Drive and U.S. Route 460 Bypass, is largely undeveloped, except for the Veterinary Medicine School.

Since the XP-SWMM program is proprietary, the program could not be used in the present analysis. Consequently, the public domain of the hydrologic/hydraulic model HEC-1, which was developed for use in analyzing the hydrologic processes of flood events in river basins varying in size and complexity from a small urban catchment to a large multi-basin river system, was used to compute the outflow hydrographs for different design storm events after proceeding channel and reservoir routing procedures.

A drainage structure inventory (Drawings No. 3 and 4) prepared by HSMM was used to construct the stream network system (Figure A-1) required by HEC-1. Basin boundaries were delineated from Drawings 3 and 4 on the basis of study objectives and hydrologic characteristics (Figure A-1 and Table A-3). The assumption was made that the hydrologic processes are represented by model parameters that reflect average conditions within a subarea.

4.2. Watershed Characteristics

The entire watershed was subdivided by HSMM into numerous subbasins. For all computational purposes, the identification numbers defined by HSMM were maintained. Each subbasin has a unique identification number that is associated with its storm drainage structure. The subbasin delineation was performed using Virginia Tech's and the Town of Blacksburg's 1 in = 100 ft areal mapping with 2 ft contour intervals and were verified by visual inspection of the watershed by engineers of HSMM. Areal photography for the mapping was performed in 1992. The subbasin delineation was digitized and placed as a layer in the mapping digital files. The subbasin areas were calculated using ArcCAD.

Subbasins were delineated for each drainage structure within the detailed study areas, resulting in subbasin areas that typically did not exceed 1 to 2 acres along the storm sewer pipe system and 20 to 40 acres in the open-channel portion of the system. The delineation of subbasins for individual drainage structures allowed watershed characteristics to be assigned and hydrographs to be determined for each drainage structure and other significant locations within the area investigated.

4.3. Runoff Curve Numbers and Loss Rates

Existing land use conditions were determined from studying the 1 in = 100 ft areal mapping, supplemented with visual observations. Land uses were characterized as low, medium, and high density residential, commercial, industrial or agricultural/open/vacant. Future land use was projected based on build out within the Town of Blacksburg and future projects identified by Virginia Tech.

Soil types were determined using the Soil Survey of Montgomery County (SCS, 1985). The soil types were cross-referenced to the appropriate hydrologic soil classifications (A, B, C, or D) using SCS Technical Release No. 55 (SCS, 1986). Land use and hydrologic soil classification were used in calculating runoff curve numbers. Runoff curve numbers for each subbasin were determined. Coverage for each land use and hydrologic soil group was generated and overlaid to provide percentages of each combination of land use and hydrologic soil group within each subbasin. The percentages were then applied to the matrix shown in Table 4.1, and the subbasin composite runoff curve numbers were generated. The runoff curve numbers used in the HEC-1 model to calculate discharges from each subbasin for present and future conditions are presented in Table A-1.

Table 4.1: Runoff Curve Number

Land Use	Runoff Curve Numbers for each Hydrologic Soil Classification			
	A	B	C	D
Agriculture/Open/Vacant	39	61	74	80
Commercial	89	92	94	95
Farm	59	74	82	86
Gravel	76	85	89	91
High Density Residential	77	85	90	92
Industrial	81	88	91	93
Low Density Residential/Open	51	68	79	84
Medium Density Residential	57	72	81	86
Open/Gravel	69	80	86	89
Univ 0% Impervious	45	67	74	80
Univ 15% Impervious	60	71	78	83
Univ 20% Impervious	62	72	79	84
Univ 30% Impervious	67	75	81	85
Univ 40% Impervious	72	78	83	87
Univ 50% Impervious	76	81	86	89
Univ 60% Impervious	76	84	88	90
Univ 75% Impervious	81	88	91	93
Univ 80% Impervious	86	91	93	94
Univ 88% Impervious	91	93	94	95
Univ 90% Impervious	92	94	95	96
Univ 92% Impervious	93	94	96	97
Univ 95% Impervious	94	95	97	98
Univ 100% Impervious	98	98	98	98
Water	99	99	99	99
Woods	25	55	70	77

As the table indicates, the future development of the Virginia Tech campus results in decreasing pervious area, which is reflected by the increase of the watershed-wide curve number by approximately one.

Land surface interception, depression storage and infiltration representing the movement of water into the soil are referred to in the HEC-1 model as precipitation losses. The SCS curve number option was used to calculate the loss rates. Note that the equations used to compute the losses do not provide for soil moisture or surface storage recovery (HEC-1 User's Manual, 1990).

4.4. Travel Time / Time of Concentration Computations

By definition, the time of concentration is the time required for water to flow from the most hydraulically remote point of the basin to the location being analyzed. Thus, the time of concentration is the maximum time for water to travel through a watershed, which is not always the maximum distance from the outlet to any point in the watershed. (Debo et al, 1995)

Distributed outflow from a subbasin was obtained by utilizing combinations of three conceptual elements: overland flow planes, collector channels, and a main channel. The overland flow planes and the collector channels were considered in the travel time computations, whereas the main channel was subject to channel routing procedures.

A large number of methods are available for estimating travel times. The computational method chosen for the project of the detention facilities is the SCS Segmental Method (1986). According to the computational procedure, the watershed was subdivided into three different segments to compute the time of concentration (T_C). The three segments are:

- Overland flow: Time of concentration was determined based on the equation

$$T_C = \frac{0.007 * (n * L)^{0.8}}{P_2^{\ 0.5} * S^{0.4}} \tag{4.1}$$

where n = Manning's n for sheet flow (n = 0.24),

P_2 = 2 year, 24-hr rainfall [in] (P_2 = 3.0 inches),

L = overland flow length [ft], and

S = overland slope [ft/ft]

This equation was developed by SCS (1986) for the Revised TR 55. It should be noted that the maximum overland flow length of 200 ft was not exceeded. Concentrated flow: Refers to swale and gully flow which forms just above a defined channel. The SCS (1986) Average Velocity Chart was chosen to calculate the time of concentration of the segment (Figure A-3).

- Channel flow: The travel time of the channel flow segment is defined channel as

$$v = \frac{1.486}{n} R^{2/3} S^{1/2} \tag{4.2}$$

where v is the average flow velocity [ft/s],

S is the channel bed slope [ft/ft],

R is hydraulic radius [ft], and

n is Manning's resistant factor

To apply Manning's equations, either depth or discharge must be assumed to calculate the velocity. This requires assumptions concerning controlling conditions. For reason of simplification, a trapezoidal channel with 2:1 side slopes, 10 ft bottom width (w), a discharge

(Q) of 10 cfs, and a Manning's factor (n) of 0.06 were assumed to calculate the travel time of the channel flow segment.

The time of concentration computations are listed at Table A-5. The data required to perform the calculations were obtained from AutoCAD databases of the study area that include slope, flow length, and land cover data for each subbasin and channel reach.

HSMM used a different, but similar approach to calculate travel time (HSMM, 1995). Their results are presented in Table A-5.

Both procedures used to compute travel time are related to the stream network determined from Drawings No. 3 and 4. Within the stream network, direct runoff was routed to the outlet of the drainage basin (D14), which is located at the intersection of Southgate Drive and Duck Pond Drive. In using the Kinematic Wave Method to model channel flow, a modified time of concentration was required, since the method is not applicable for short stream reaches. Thus, several reaches of the stream system had to be combined and consequently, the stream network system was rearranged (Figure A-4). Detailed travel time calculations are listed at Table A-6.

The SCS dimensionless unit hydrograph option of the HEC-1 model was used to generate appropriate storm hydrographs for subbasins. This option requires a data input of SCS lag times in hours. The lag time calculations for each of the subbasins are included at Table A-5.

It is recognized that the travel time varies with the magnitude of flood events. However, in the interest of time, the channel travel time was assumed to be the same for all events. The predicted areal discharges should be viewed with this assumption in mind.

4.5. Rainfall

The HEC-1 model evaluated conditions for several synthetic design storm events. The return period, duration, total rainfall volume, and temporal distribution of the total rainfall volume were parameters considered in the development of the design storms. Three 24-hour design storms were required: the 2-year, 10-year, and 100-year return periods. The design rainfall depths for 24-hour storms with various return frequencies were obtained from National Weather Service (1983). The storm duration was always 24 hours. The SCS Type II 24-hour distribution was used to distribute the rainfall and develop the synthetic design storms. Total 24-hour rainfall depths for each design storm are listed in Table 4.2.

Table 4.2: Design storm rainfall depths

Design Storm Rainfall Depths	
Return Frequency [years]	Total Rainfall [inch]
2	3.0
10	4.9
100	7.0

The SCS Type II scaled rainfall distribution is the appropriate distribution for the area of the southwestern part of Virginia. Synthetic hyetographs were generated using the VT/PSUHM program and were later imported to the HEC-1 model. Rainfall depth and distribution are parameters used in the HEC-1 model to calculate hydrographs.

4.6. River Routing Procedures

Both the Kinematic Wave and the Muskingum method were used to route inflow through the main channel, but were not inter-mixed. The Kinematic Wave and the Muskingum routing procedures were applied independently from each other on the watershed.

Kinematic Wave channel routing computations can be utilized independently of other elements of the subbasin runoff. The cross-sectional geometry is limited to circular, triangular, square, rectangular, and trapezoidal shapes. Theoretically, a flood wave routed by the Kinematic Wave technique through these channel sections is translated, but does not attenuate. Consequently, the Kinematic Wave routing technique is most appropriate in channels where flood wave attenuation is not significant, as is typically the case in urban areas (Hoggan, 1989).

The initial stream network (Figure A-1), as used for the Muskingum Method, was not appropriate for the Kinematic Wave routing procedure, so that it was necessary to further combine subbasins, as to be seen at Figure A-5.

To insure the Muskingum method's computational stability and the accuracy of computed hydrographs, routing reaches (NSTPS) had to be chosen appropriately (HEC-1 User's Manual, 1990). Detailed information about the NSTPS computations are shown at Table A-4.

4.7. HEC-1 Results for Design Storms

The peak discharges of the 10- and 100-year flood events computed using HEC-1 differed greatly from those computed by HSMM. After reviewing literature of the HEC-1 modelling performance, it had to be expected. The XP-SWMM program used by HSMM is capable of splitting the flow according to network geometry if pipe and conduit capacities are reached. As water overflows from manholes, the excess water proceeds as open channel flow on the surface along the path of least resistance. The HEC-1 model does have an option of flow splitting; however, it has to be manually defined. For project simulations conducted , there was no flow splitting assumed, because this procedure is very time-consuming due to the trial and error nature of required computations. An additional source for the discrepancy in the hydrographs is that XP-SWMM can account for the hydraulics of full pipe flow conditions while HEC-1 simulates unlimited open channel flow conditions. Consequently, HEC-1 cannot account for water storage within the system. The exaggerated peak discharges of the HEC-1 computations are the result of these differences. A sample input file for the HEC-1 model is provided in Figure A-5.

The hydrographs of Figures 4.1-4.6 were obtained using both the HEC-1 program and the XP-SWMM hydrographs of HSMM. The point of comparison was node D14, which is situated at the intersection of Duck Pond Drive and Southgate Drive (refer to Drawing No. 4 for location) and represents a 355-acre portion of the watershed.

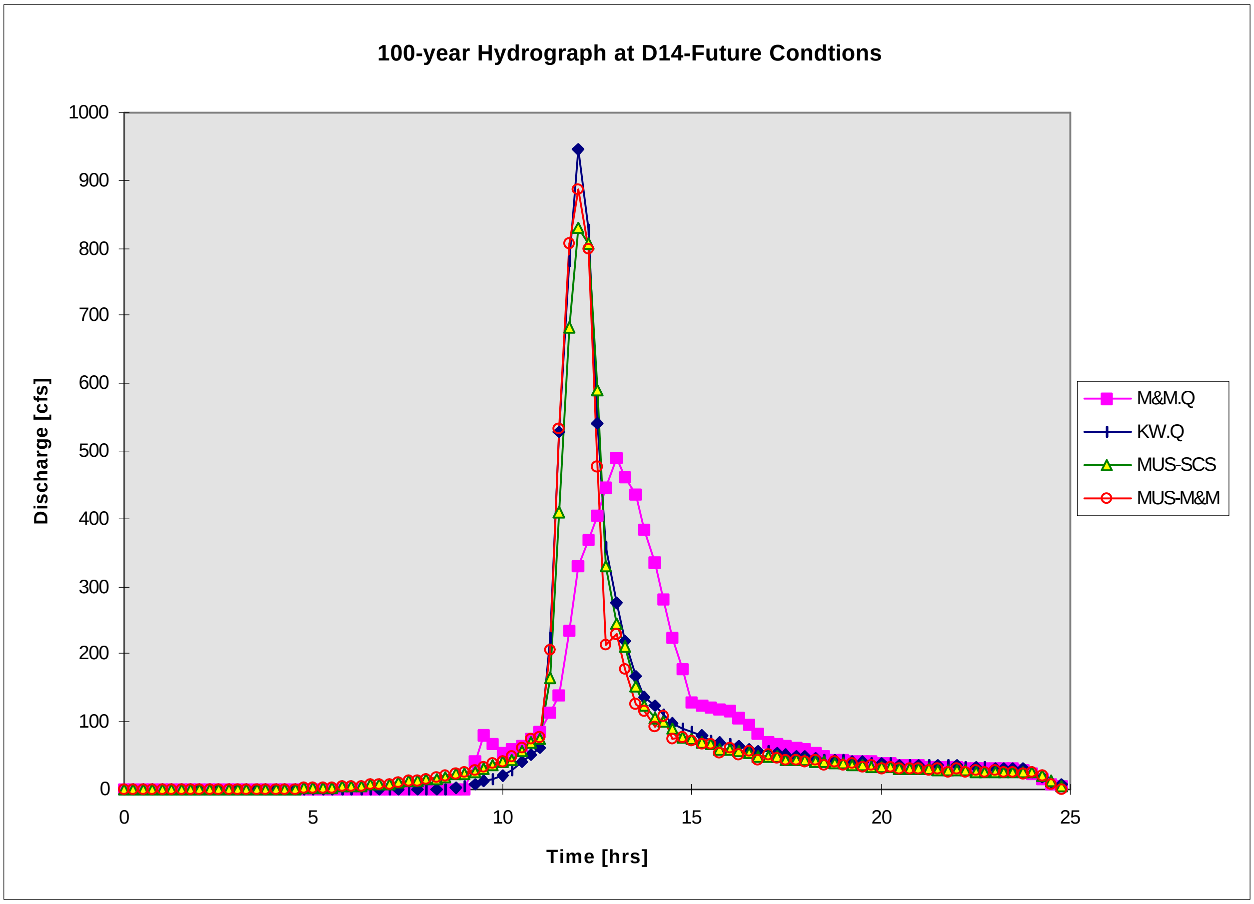

Figure 4.1: 100-year Hydrograph at D14 for Future Conditions (M&M Q = XP-SWMM hydrograph obtained by Hayes, Seay, Mattern & Mattern; KW. Q = HEC-1 hydrograph applying Kinematic Wave routing procedure and SCS travel time computation method; MUS-SCS = HEC-1 hydrograph applying Muskingum routing procedure and SCS travel time computation method; MUS-M&M = HEC-1 hydrograph applying Muskingum routing procedure and HSMM travel time computation method)

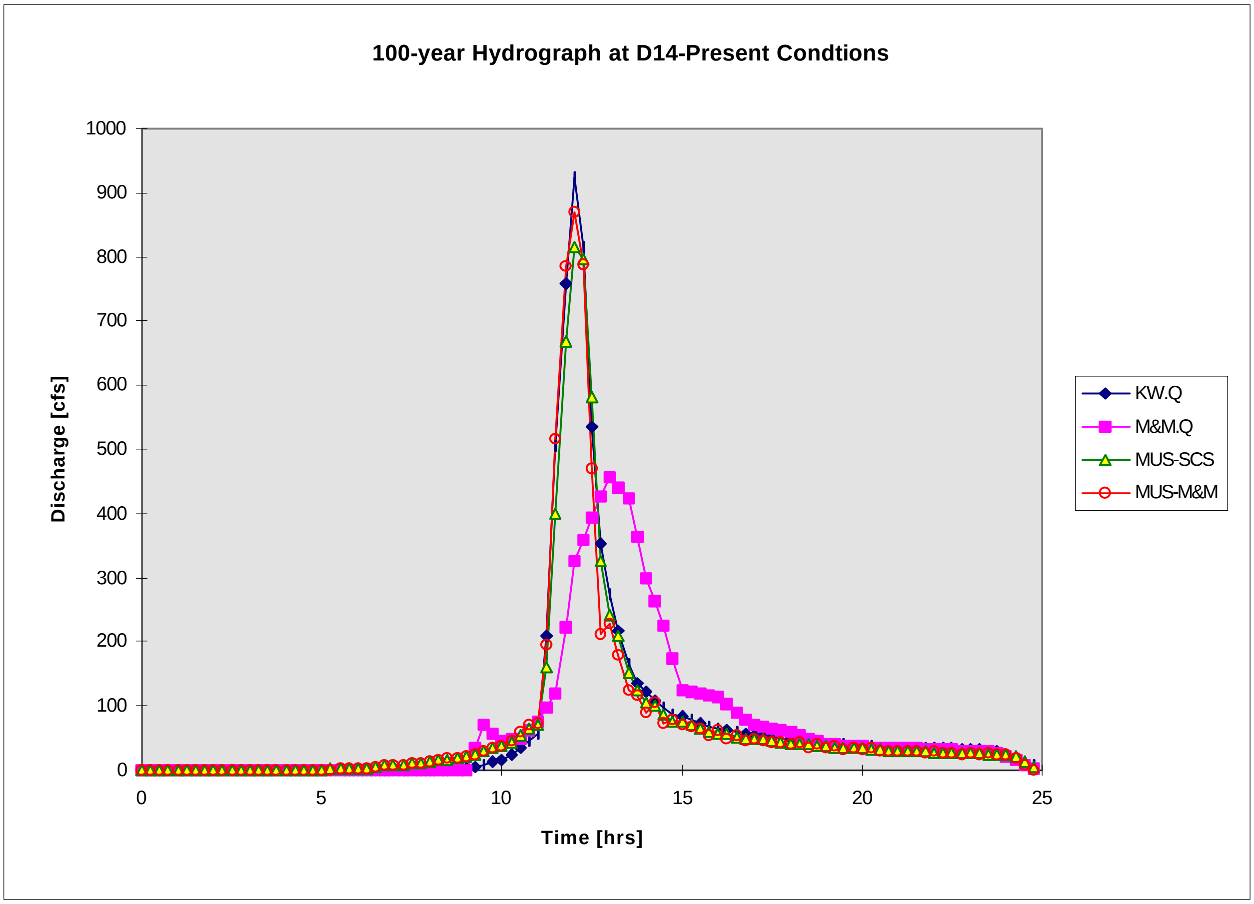

Figure 4.2: 100-year Hydrograph at D14 for Present Conditions (M&M Q = XP-SWMM hydrograph obtained by Hayes, Seay, Mattern & Mattern; KW. Q = HEC-1 hydrograph applying Kinematic Wave routing procedure and SCS travel time computation method; MUS-SCS = HEC-1 hydrograph applying Muskingum routing procedure and SCS travel time computation method;

MUS-M&M = HEC-1 hydrograph applying Muskingum routing procedure and HSMM travel time computation method)

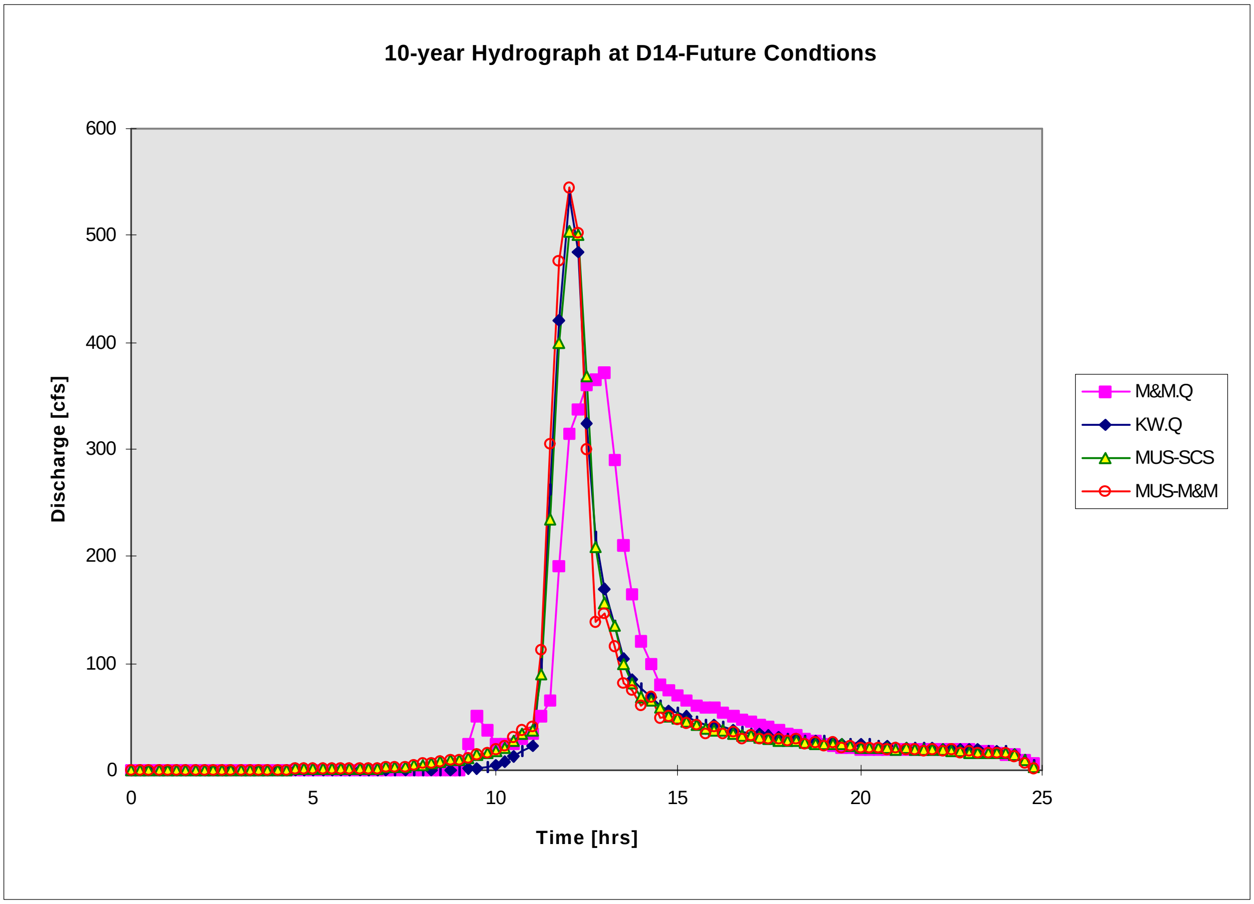

Figure 4.3: 10-year Hydrograph at D14 for Future Conditions (M&M Q = XP-SWMM hydrograph obtained by Hayes, Seay, Mattern & Mattern; KW. Q = HEC-1 hydrograph applying Kinematic Wave routing procedure and SCS travel time computation method; MUS-SCS = HEC-1 hydrograph applying Muskingum routing procedure and SCS travel time computation method; MUS-M&M = HEC-1 hydrograph applying Muskingum routing procedure and HSMM travel time computation method)

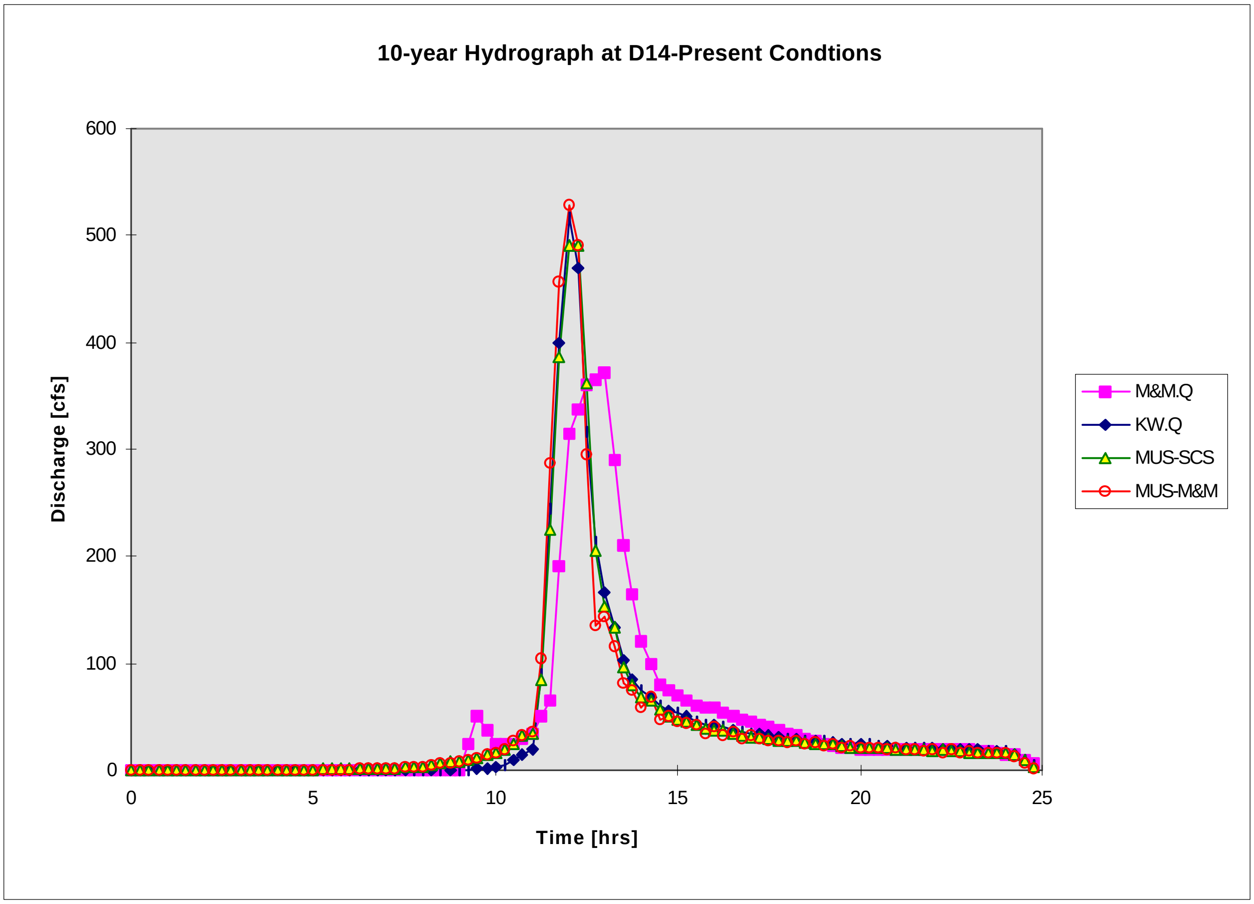

Figure 4.4: 10-year Hydrograph at D14 for Present Conditions (M&M Q = XP-SWMM hydrograph obtained by Hayes, Seay, Mattern & Mattern; KW. Q = HEC-1 hydrograph applying Kinematic Wave routing procedure and SCS travel time computation method; MUS-SCS = HEC-1 hydrograph applying Muskingum routing procedure and SCS travel time computation method;
MUS-M&M = HEC-1 hydrograph applying Muskingum routing procedure and HSMM travel time computation method)

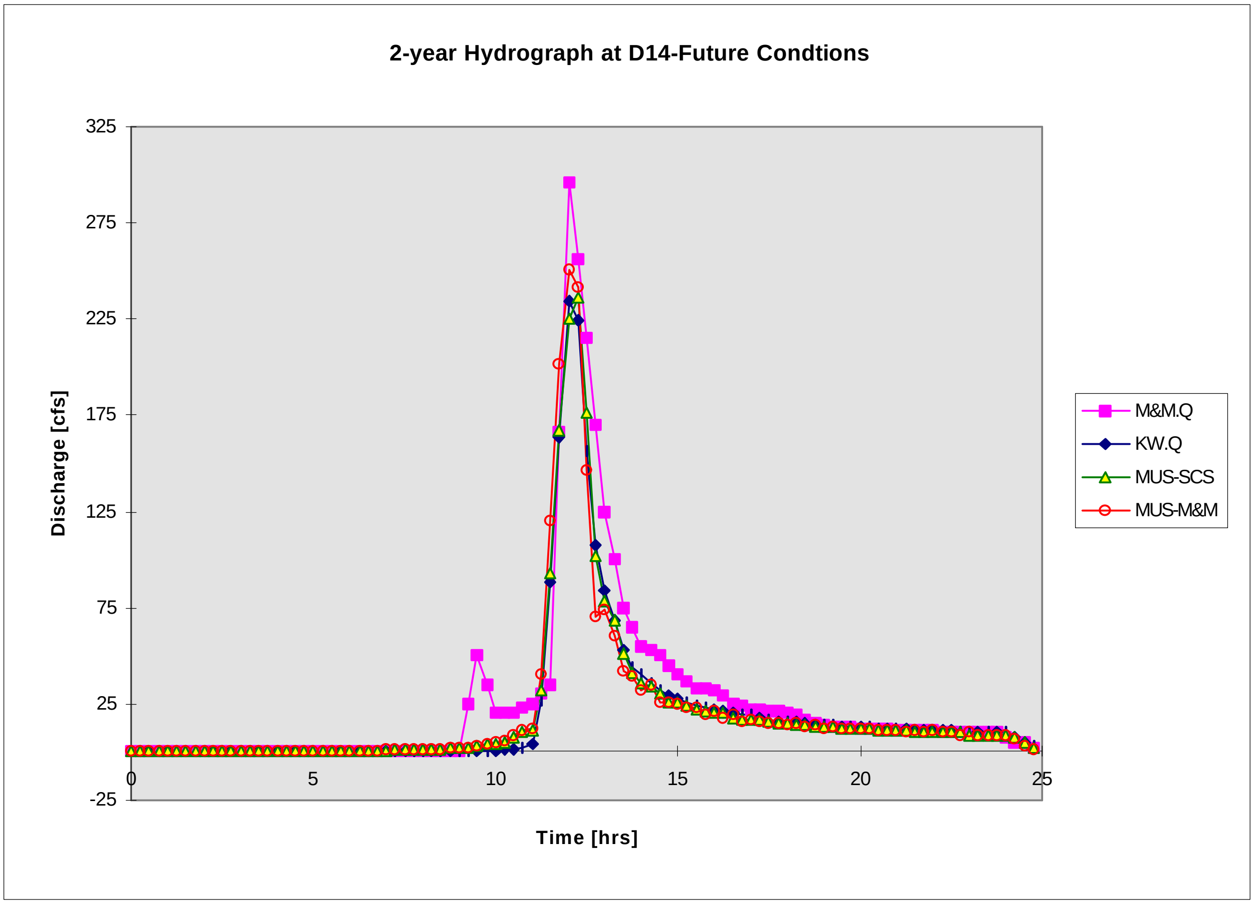

Figure 4.5: 2-year Hydrograph at D14 for Future Conditions (M&M Q = XP-SWMM hydrograph obtained by Hayes, Seay, Mattern & Mattern; KW. Q = HEC-1 hydrograph applying Kinematic Wave routing procedure and SCS travel time computation method; MUS-SCS = HEC-1 hydrograph applying Muskingum routing procedure and SCS travel time computation method; MUS-M&M = HEC-1 hydrograph applying Muskingum routing procedure and HSMM travel time computation method)

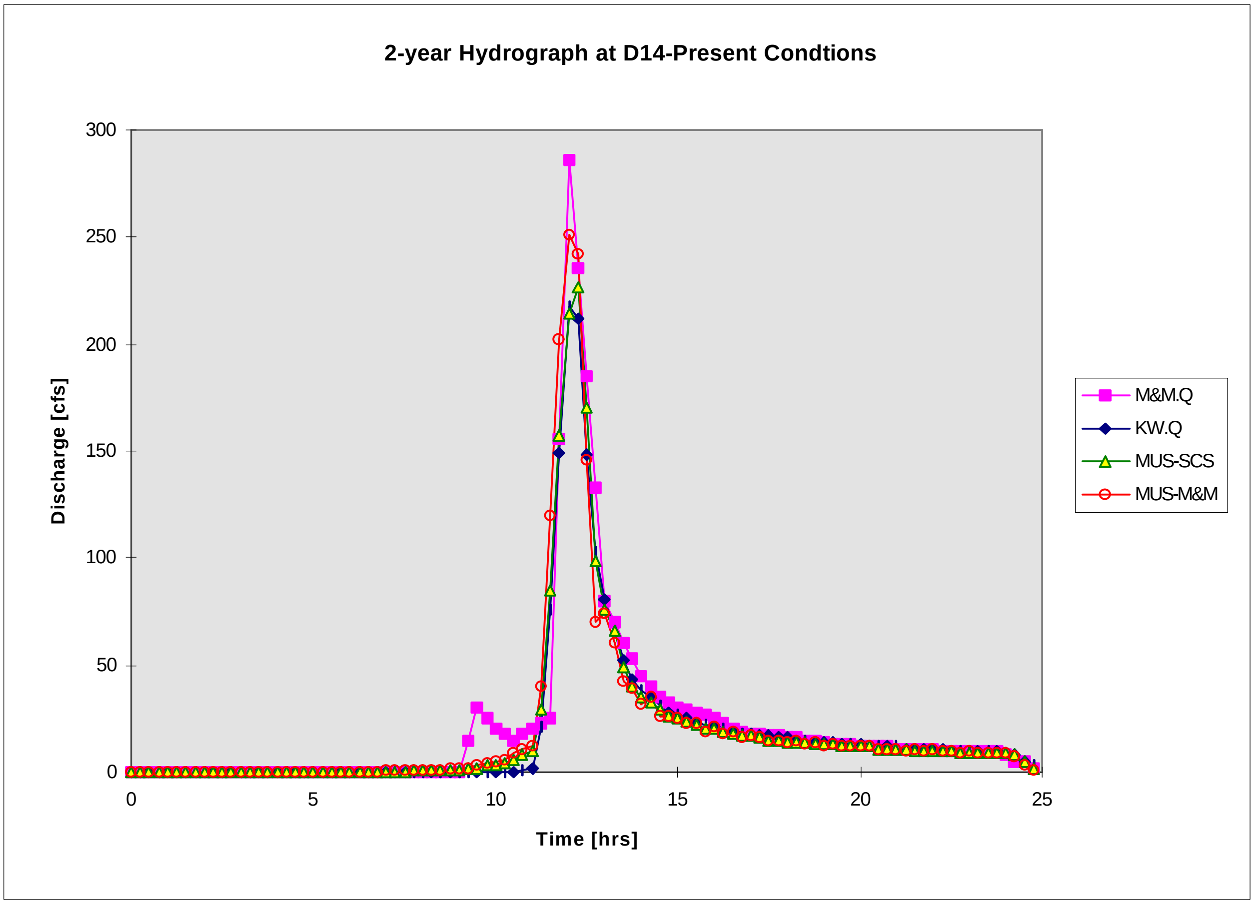

Figure 4.6: 2-year Hydrograph at D14 for Present Conditions (M&M Q = XP-SWMM hydrograph obtained by Hayes, Seay, Mattern & Mattern; KW. Q = HEC-1 hydrograph applying Kinematic Wave routing procedure and SCS travel time computation method; MUS-SCS = HEC-1 hydrograph applying Muskingum routing procedure and SCS travel time computation method; MUS-M&M = HEC-1 hydrograph applying Muskingum routing procedure and HSMM travel time computation method)

Comparison of the hydrographs clearly reveals a mismatch of the results computed for 10-year and 100-year storm events simulated by XP-SWMM and HEC-1, respectively. In Figure 4.4, the peak discharge for a 100-year flood event simulating future landuse conditions as computed by XP-SWMM model is 489 cfs or approximately 400 cfs below the amount calculated by HEC-1 model. Similarly, differences of 70% in peak discharge of 10-year flood events were also encountered (Table 4.3 and Figures 4.5-4.10).

Table 4.3: Peak Discharges for Various Flood Events Computed by XP-SWMM and HEC-1 (KW = Kinematic Wave routing procedure applying SCS travel time computation method; MUS-SCS = Muskingum routing procedure applying SCS travel time computation method; MUS-M&M = HEC-1 Muskingum routing procedure applying HSMM travel time computation method)

Conditions	Flood Event	XP-SWMM Model [cfs]	HEC-1 Model [cfs]		
			KW-SCS	MUS-SCS	MUS-M&M
Present	100-year	489	945	830	886
	10-year	378	536	503	544
	2-year	296	234	236	251
Future	100-year	457	924	816	870
	10-year	371	517	490	529
	2-year	286	218	227	242

The Montgomery County Flood Insurance Study (FIS) done in 1979 used a model similar to HEC-1. The XP-SWMM model predictions are superior to those of HEC-1 because of XP-SWMM’s ability to simulate pipe flow conditions, which HEC-1 approximates using open channel flow.

As Figures 4.1-4.6 indicate, the peak discharge rates predicted by HEC-1 and XP-SWMM differ by approximately 100% for the 100-year flood and 80% for the 10-year flood. The peak lag times of the 100-year and the 10-year flood hydrographs differ by 60 and 45 minutes, respectively. The lags in peak runoff computed by the XP-SWMM program are due to its ability to account for back-ups in the stormwater system. While the peak discharge rates and times to peak varied the total runoff volumes generated by the two models were essentially the same (Table A-7).

The manually estimated peak discharge of 598 cfs for the 100-year storm, future conditions, for the 355-acre portion of the drainage area corresponds approximately with the XP-SWMM result of 489 cfs. The peak discharge estimation for the 100-year flood event supporting the XP-SWMM results can be seen at Table A-2. It is emphasized that all reservoir calculations dealt with in the next sections are based on hydrographs computed by the XP-SWMM model and represent the discharge of the entire watershed of 446 acres.

5. HYDRAULICS - RESERVOIR ROUTING AND OUTLET STRUCTURES

The Virginia Tech / Penn State Urban Hydrology Model (VT/PSUHM) was used to design the outlet structures of the two ponds in series and to perform the reservoir routing through both. The design inflow hydrograph (100-year, future conditions) for the wet pond facility is shown in Figure 5.1. The hydrographs used in the wet pond facility design for the 2-year, 10-year, and 100-year storm events were those obtained from the XP-SWMM program. The HEC-1 hydrographs were not used because of the problems previously discussed. Since no detailed areal and landuse information on the subbasin of the Veterinary Medicine School were available, the impact of this region was concluded from response in changes of the hydrographs between points D14 (see Drawing No. 4) and SB4, which is approximately 400 ft further downstream of the D14 location. At SB4, the entire runoff of the Veterinary Medicine School area (89 ac) and the runoff of the 355-acre portion of the watershed are combined. Unfortunately, no data tables of hydrographs at SB4 were available, thus graphical outputs of the XP-SWMM had to be digitized.

Because the VT/PSUHM program requires equal time steps, the digitized hydrographs of the XP-SWMM model were converted to a uniform time basis of 5 minutes. Linear interpolation was used to estimate intermediate points. Exceptions were made to maintain the peak discharges.

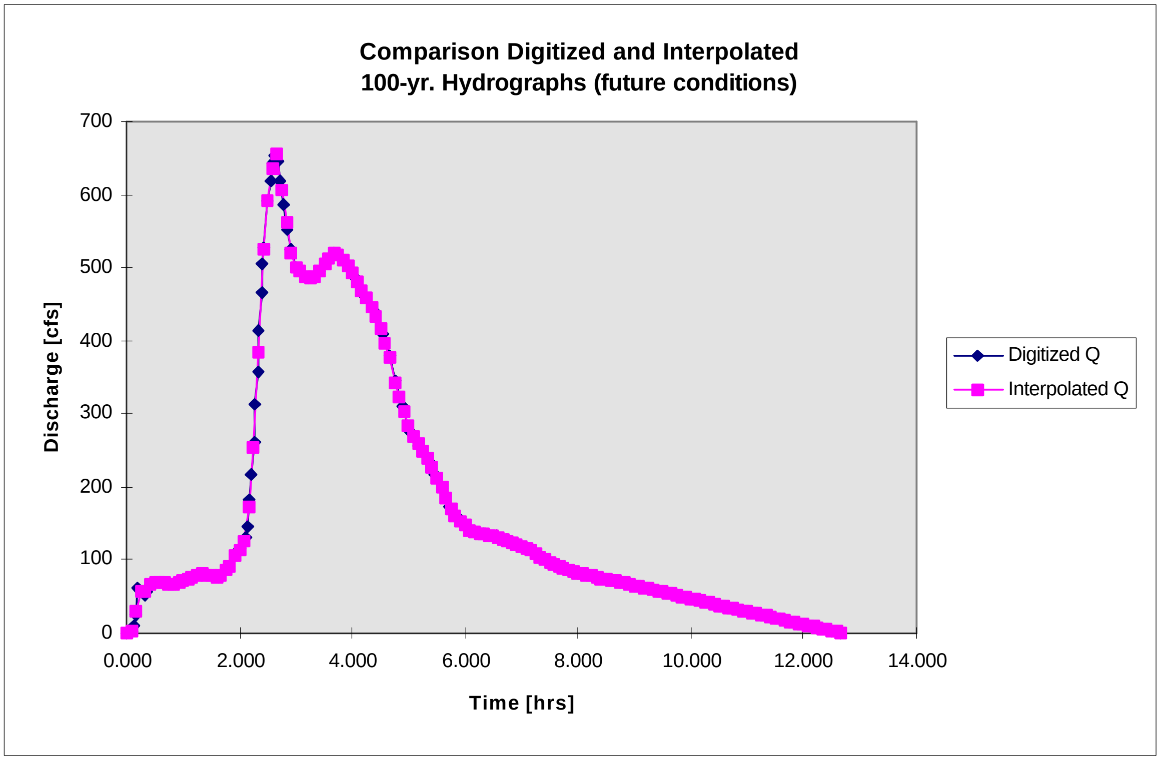

Figure 5.1: Comparison of Digitized and Interpolated Design Hydrograph Obtained from the XP-SWMM Program (100-year, Future Conditions)

An excellent match of data points was achieved for the 100-year design flood, as Figure 5.1 constitutes. Similar matches were achieved for the 10-year and 2-year flood events. It is assumed that no inaccuracies were caused by the transformation to equal-time-step hydrographs.

5.1. The Urban Hydrology Model - VT/PSUHM

The VT/PSUHM is a menu-driven interactive model written in the BASIC programming language for IBM-compatible personal computers. The model contains 17 programs that perform various hydrologic and hydraulic tasks. The task are organized into five types of operations: utilities, rainfall, runoff, routing, and surface profiles. Data can be saved as a file that includes hyetographs, hydrographs, subarea characteristics, elevation-storage charts, elevation-outflow charts, and detention outlet geometry. These data, which are of ASCII code, can be accessed by other modules later through the data file management system. The present version of VT/PSUHM includes design storm and hyetograph calculations, curve number weighting, rational and modified rational methods, SCS curvilinear unit hydrograph, SCS TR-55 tabular hydrograph, Muskingum channel routing, modified Puls reservoir routing, hydrograph combining, and plotting and sizing of outlets in a multi-stage detention structure (MSRM).

The MSRM module of the VT/PSUHM program routes a hydrograph through a multiple outlet detention facility. Written by Chamberlain (1986), the model contains a subroutine that computes the hydraulic performance curve for an outlet structure with up to 10 openings or stages. This capability is useful when trying to reduce runoff peaks for several return periods through the use of one structure. The subroutine can model rectangular, v-notch and proportional weirs, riser pipe inlets, emergency spillways, circular and rectangular orifices, open and grated drop inlets, discharge pipes, outfall culverts, and outfall channels. MSRM adjusts outlet capacity for riser box submergence and for inlet-outlet control in the outfall culvert.

The multi-stage outlet design and routing model in the VT/PSUHM is especially useful in simulating multiple outlets required for the design of the wet and dry detention facilities to control baseflow and the 100-year flood event. The outlet routine is capable of developing a complex rating curve representing up to 10 different outlets or stages. The program outputs take the form of elevation-storage-discharge tables that can be passed to the reservoir routing module for full analysis of detention facilities.

5.2. Reservoir Routing

VT/PSUHM provides a reservoir routing option applying the modified Puls routing method (Chow, 1959). This routing method is a variation of the storage routing method described by Henderson (1966). It is applicable to both channel and reservoir routing. The modified Puls method routes a hydrograph through a reservoir using the storage indication method also described in Viessman et al. (1989). This method is based on mass conservation principles and the hydraulics of the reservoir outlet structure. The user is prompted to supply an elevation-storage curve, which was created by the MSRM module, an elevation-outflow curve, based on the basin geometry and obtained from AutoCAD drawings, and an inflow hydrograph.

5.3. Baseflow Determination

To size the basin outlet required to maintain a permanent pool elevation for the wet pond, the watershed baseflow is usually calculated based on historical data. Since no data were available, except a flow depth measurement taken by engineers of the consulting firm Anderson & Associates on April 25, 1996, the baseflow computed is assumed to be a gross estimation. According to individuals familiar with the stream, this day was not necessarily a dry weather day and baseflow was probably overestimated. However, it is safer to assume a slightly higher baseflow with respect to remaining storage capacity of the pond than to underestimate the steady inflow.

Based on the flow depth measurement, the baseflow of the entire watershed (446 ac) was determined to be 1.95 cfs using Manning's equation (Eq. 4.2) with an assumed bed slope of 1.16%, a bottom width of 6.0 ft, and a measured flow depth of 1.375 in.

5.4. Wet Detention Facility

As a result of 0.5 in of rainfall over disturbed areas, whereas disturbance is caused by clearing, grading, and excavating, the dry storage portion of the wet detention facility is required to contain excess surface runoff volume of 4.8 ac-ft. This required value is based on calculations done by the stormwater management engineer, Dr. George B. Williamson, of the Department of Conservation and Recreation. Dr. Williamson revealed that the Department of Conservation and Recreation requires an equal ratio of wet to dry storage volume. The dry and wet storage volume each of 4.8 ac-ft was used in the design, although reasonable doubts on an effective water treatment are acknowledged. Thus, according to the Virginia Stromwater Management Regulations (1996), the wet storage volume should be three times the size of dry storage.

Due to site constraints such as area, volume, and maximum water surface elevation, the engineering possibilities were largely restricted. Resulting from numerous options investigated, the following parameters of the wet pond were obtained:

- bottom elevation at 2014.5 due to excavation,
- permanent pool elevation at 2018.1 due to 4.8 ac-ft wet storage (Table 5.1),

- emergency spillway elevation at 2020.2 due to 4.8 ac-ft dry storage (Table 5.2),
- maximum possible dam height at elevation 2023.0 due to road embankment impounding the wet pond and to prevent the School of Veterinary Medicine from flooding.

As determined by the VT/PSUHM program, the emergency spillway is 100 ft wide and has side slopes of 4:1 (H:V). The emergency discharge coefficient was determined to be 3.0, which is a typical value for grassed broad crests (Bureau of Reclamation, 1965). Principal spillways were designed to carry the baseflow maintaining the desired permanent pool table at a certain elevation.

A storage-elevation curve was obtained from AutoCAD Drawings No. 1 and 2. Tables 5.1 and 5.2 show derivations of the storage volumes with respect to pond elevations.

Table 5.1: Total Storage of Wet Pond

Elevation [ft]	Area [ft^2]	Average Area [ft^2]	Increment [ft]	Incremental Volume [ft^3]	Cumulative Volume [ft^3]	Cumulative Volume [ac-ft]
2014.5	0				0	0.00
		23078	0.5	11539		
2015.0	46156				11539	0.26
		52160	1.0	52160		
2016.0	58163				63699	1.46
		62917	1.0	62917		
2017.0	67671				126616	2.91
		74328	1.0	74328		
2018.0	80984				200943	4.61
		86259	1.0	86259		*
2019.0	91534				287202	6.59
		98769	1.0	98769		
2020.0	106003				385971	8.86
		111502	2.0	223003		
2022.0	117000				608974	13.98

*4.8 ac-ft (204.088 ft^3) occurs at 2118.10

Table 5.2: Dry Storage of Wet Pond

Elevation [ft]	Area [ft^2]	Average Area [ft^2]	Increment [ft]	Incremental Volume [ft^3]	Cumulative Volume [ft^3]	Cumulative Volume [ac-ft]
2018.089	0				0	0.00
		40998	0.0	41		
2018.090	81996				41	0.00
		81996	0.0	820		
2018.100	81996				861	0.02
		86765	0.9	78089		
2019.0	91534				78949	1.81
		98769	1.0	98769		
2020.0	106003				177718	4.08
		111502	2.0	223003		**
2022.0	117000				400721	9.20

** 4.8 ac-ft (204.088 ft^3) occurs at 2020.20

5.4.1. Perforated Riser Outlet Structure

A standard principal spillway configuration for urban and construction site basins is the perforated riser. Jarrett (1993) developed a procedure for the design and analysis of a linear stage-storage curve for the zone between the bottom perforated inlet and the design water surface of the principal spillway.

Warner and Schwab (1989) compared a perforated riser, a drop inlet, and two types of siphons for trapping efficiency using the Sediment, Erosion, Discharge by Computer Aided Design Model (SEDCAD). The perforated riser and siphon tubes consistently outperformed the drop inlet by reducing the peak discharge, stage , and sediment concentration. The sediment retention efficiency for a perforated riser was found to be 60% for a laboratory scale sedimentation basin (Engle and Jarrett, 1991).

A perforated riser was selected as the outlet structure for the wet pond facility because of its performance with respect to peak discharges, retention times, drawdown times, maximum pool elevations, and trapping efficiency.

A perforated riser with two columns, a perforation spacing of 0.5 ft, and a perforation diameter of 4 in was designed using the procedure of Jarrett (1996) to discharge the baseflow. The discharge from the perforated riser as a function of pool elevation is shown in Figure 5.2.

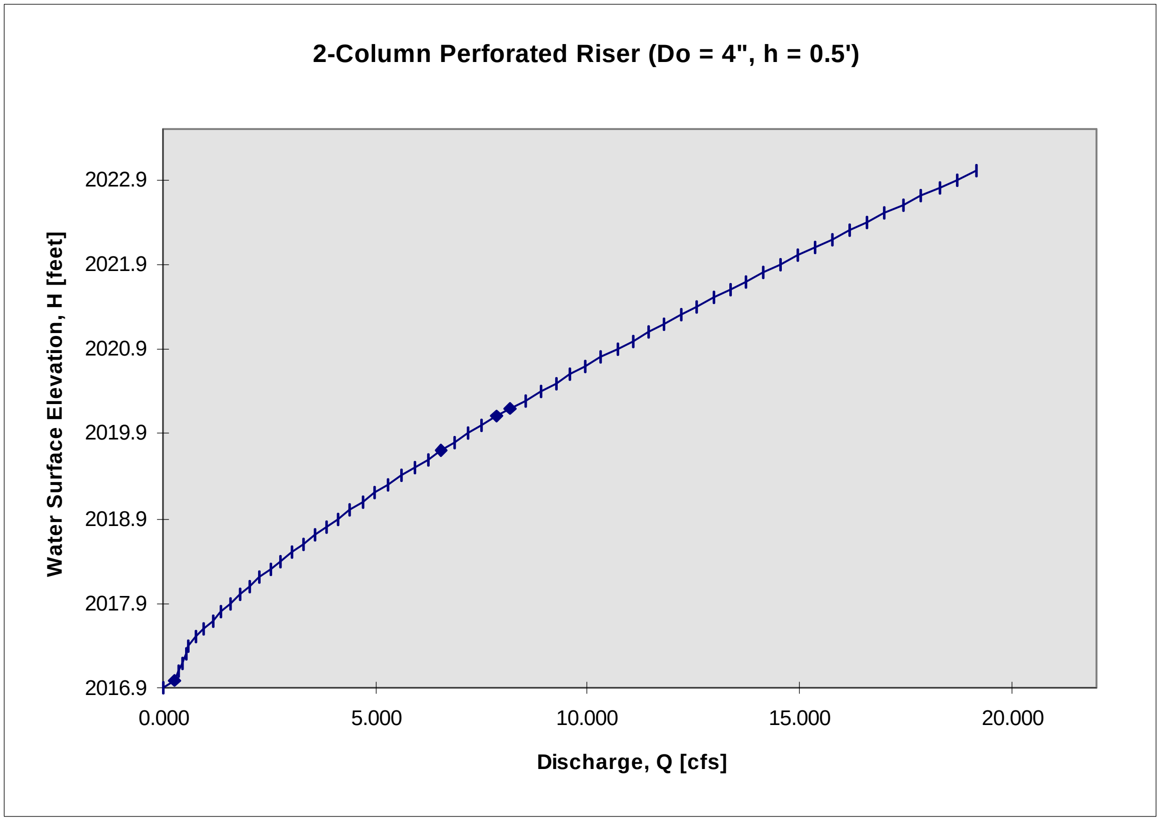

Figure 5.2: Discharge-Elevation Curve of the Designed Perforated Riser

(2 columns, D_O = 4", h = 0.5')

The diameter of the collector pipe passing through the dam was determined by means of rearranging the Manning formula (Eq. 4.2).

$$D = \left(\frac{2.56 \cdot n \cdot Q}{S^{0.5}} \right)^{0.375} \tag{5.7}$$

Routing a 100-year design storm through the reservoir and including the discharge of the emergency spillway, a maximum pool elevation of 2021.82 ft was obtained. Corresponding with the maximum pool elevation, a peak discharge of 14.2 cfs for the perforated riser was computed. The resultant diameter of the outfall conduit for a slope of 1.0% (0.5 ft / 50 ft) and a Manning's n value of 0.024 is 2.5 ft (30 in).

The outfall conduit will have the following features:

- Outfall culvert elevation: 2016.5 ft
- Outfall culvert diameter: 2.5 ft
- Outfall culvert length: 50 ft
- Outfall culvert slope: 0.01 ft/ft
- Outfall culvert Manning's n: 0.024

The results of hydraulic performance (H-Q) and the drawdown time computations are listed in detail at Figures 5.5-5.7 and at Table 6.4, respectively. The discussion of its performance is done at the end of Section 5.5.

5.4.2. Culvert Outlet Structure

The structure consists of an inclined PVC-pipe that will discharge water from a the wet pond. A manhole divides the pipe in two sections - an inclining (from elevation 2016.0 to 2016.5) 8-in inlet pipe and a 2-ft concrete outfall culvert. Since the VT/PSUHM program cannot handle negative pipe slopes, the inlet elevation was assumed to be at 2016.5 ft. Accordingly, the pipe slope was considered to be of zero inclination. The inclining pipe was selected because water should be withdrawn from a medium water stratum to avoid trash and sediment transport from the water surface and the bottom of the wet pond, respectively.

The discharge pipe inlet structure is determined for the following conditions:

- Discharge pipe inlet elevations: 2016.0 ft
- Discharge pipe diameter: 8 in
- Discharge pipe length: 16 ft
- Discharge pipe slope: - 0.031 ft/ft
- Discharge pipe's Manning n: 0.011

An outfall culvert will have the following features:

- Outfall culvert invert elevation: 2016.5 ft
- Outfall culvert diameter: 1.0 ft
- Outfall culvert length: 34 ft
- Outfall culvert slope: 0.0118 ft/ft
- Outfall culvert Manning's n: 0.013

5.4.3. Grate (Drop) Outlet Structure

Flow into drop inlets is calculated using both weir and orifice equations based on the assumption that the inlets experience weir flow at low heads, transitioning to orifice flow as the inlet becomes submerged. To account for this transition, flow is calculated using both equations, with the smaller of the two values used as the actual drop inlet outflow.

The input data required for drop inlet consists of the crest elevation, effective inlet perimeter, effective inlet flow area, and discharge coefficient. Grate drop inlets can be of irregular geometry, as long as the effective perimeters and flow areas are known.

The drop grate inlet structure chosen for design purposes has the characteristics:

- Crest elevation: 2017.8 ft
- Effective inlet perimeter: 4 ft
- Effective inlet flow area: 1 ft
- Discharge coefficient: 3.1

An outfall culvert will have the following features:

- Outfall culvert elevation: 2016.5 ft
- Outfall culvert diameter: 2.0 ft
- Outfall culvert length: 50 ft
- Outfall culvert slope: 0.01 ft/ft
- Outfall culvert Manning’s n: 0.013

5.4.4. Skimmer Outlet Structure

Another outlet structure considered was a skimmer. A skimmer, as described in Jarrett (1996), is a floating outlet device. The base of the riser is fixed, via a flexible hose to a pipe which receives the water discharged from the basin. The flexible hose permits articulation of the riser.

The dimensions of the skimmer, as investigated by Jarrett (1996), are different from those required to pass the baseflow of 1.95 cfs. For the proposed wet pond, the required orifice diameter, D_O, would be of 13 in in contrast to 1.2 in used in tests by Jarrett (1996). The discharge characteristic of skimmers with different D_O, orifice diameter, values can be seen in Figure 5.3.

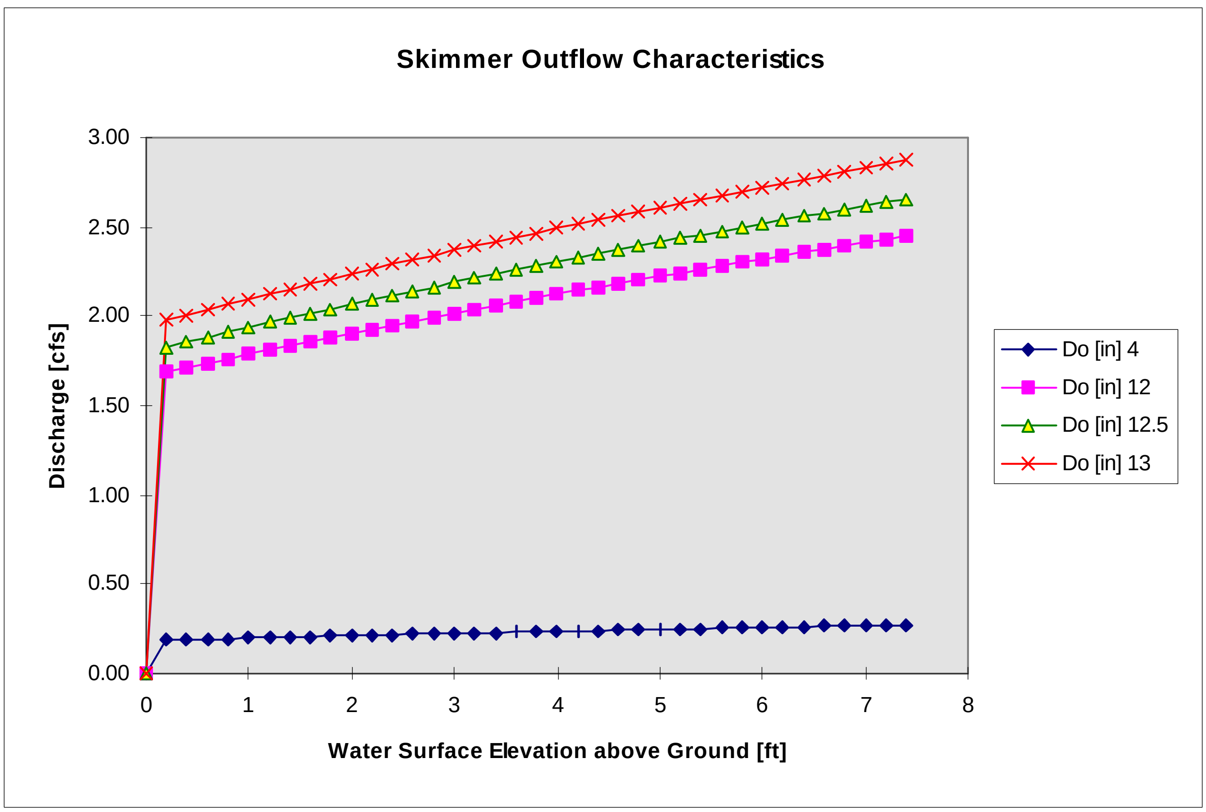

Figure 5.3: Head-Discharge Relationship of Skimmer Outlet Structures

Despite the fact that a skimmer will cause more of the inflow water to be retained for a longer period of time, thus providing greater opportunity for suspended sediment to settle from the water, it was not selected for an outlet of the wet pond because of its experimental nature. According to considerations of Dr. Williamson, a skimmer could be damaged by floating trash carried into the pond by a flood. Reservoir routing with a skimmer outlet structure was not performed.

5.4.5. Wet Pond Analysis

Data assessment reveals that the structures reduce the peak discharge, but do not delay the peak time. As shown in Figures 5.4-5.6, all of the outlet structures investigated impact the hydraulic performance by these means. Only the 2-year flood peak discharge shows a time lag of 5 minutes. However, all examined hydrographs concur in shape and magnitude for the storm events considered.

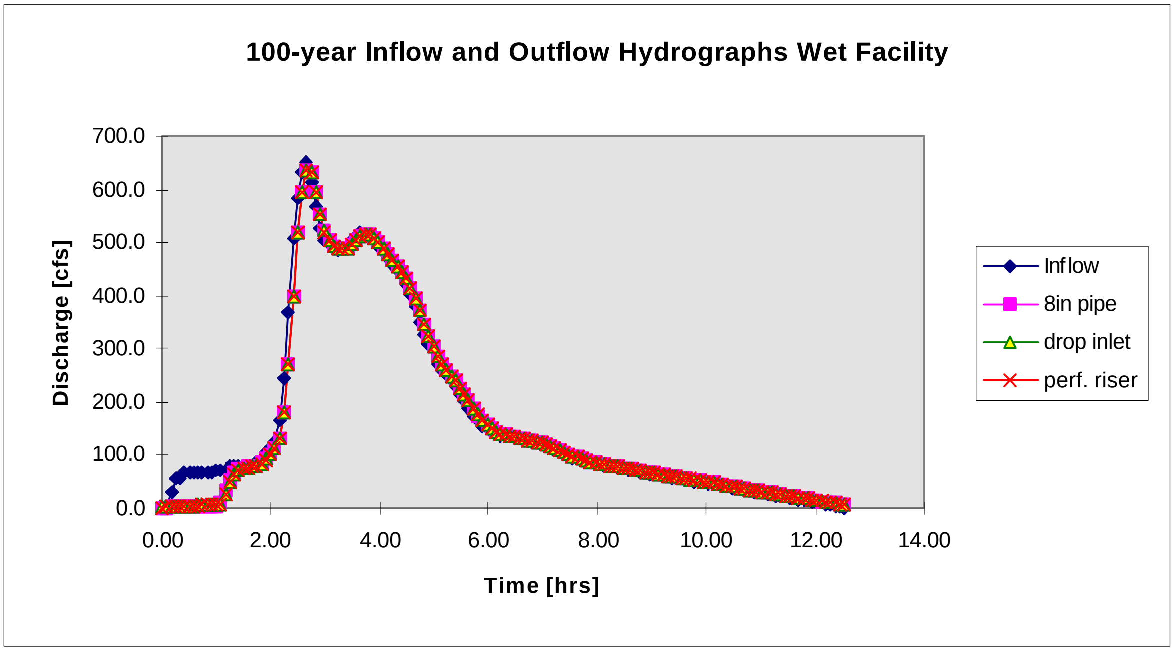

Figure 5.4: 100-year Inflow and Outflow Hydrographs Wet Facility

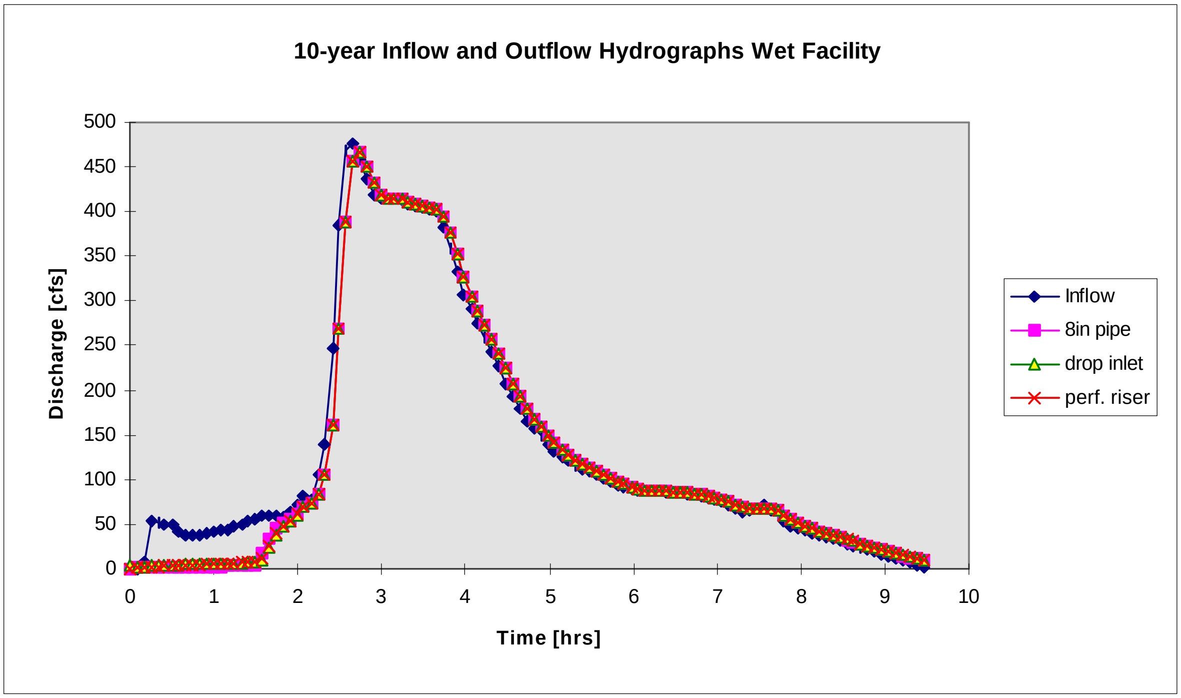

Figure 5.5: 10-year Inflow and Outflow Hydrographs Wet Facility

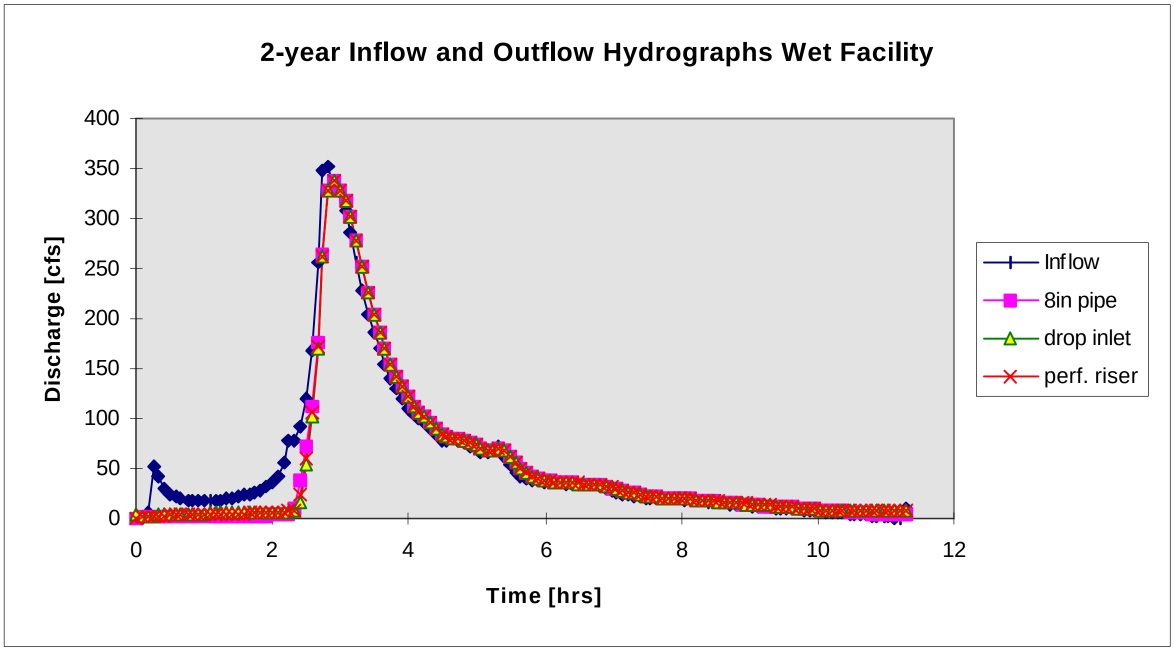

Figure 5.6: 2-year Inflow and Outflow Hydrographs Wet Facility

Table 5.3: Inflow and Peak Discharges of Different Wet Pond Outlet Structures

		Peak Discharges		
	Wet pond	100-year Storm [cfs]	10-year Storm [cfs]	2-year Storm [cfs]
Inflow		652.26	476.61	352.76
Outflow	8-in pipe	635.15	465.66	337.53
	grate inlet	635.07	465.62	337.35
	perf. riser	635.13	465.64	337.45

Table 5.4: Maximum Water Surface Elevation due to Different Wet Pond Outlet Structures

	Maximum Water Level		
	100-year Flood [cfs]	10-year Flood [cfs]	2-year Flood [cfs]
8-in pipe	2021.63	2021.53	2021.27
grate inlet	2021.62	2021.52	2021.26
perf. riser	2021.61	2021.52	2021.26

Figures 5.4-5.6 summarize the results of the hydraulic calculations. As shown, there was a negligible decrease in peak discharge of approximately 17 cfs for a 100-year event and 15 cfs for a 2-year event. Referring to Table 5.4, the maximum water surface elevations did not vary greatly by storm return period. Drawdown time computations are presented in a later section of the paper. The outflow hydrographs of the wet pond are used as inflow hydrographs of the dry pond.

5.5. Dry Detention Facility

Based on requirements of the Department of Conversation and Recreation, a dry pond is to be placed downstream of the wet pond. It must be able to detain a 2-year flood, thus a storage capacity of 13.8 ac-ft is required. After excavation, the required storage capacity will be achieved with a pool elevation at 2012.0 (Table 5.5).

Table 5.5: Total Storage of Dry Pond (Quantity) Facility

Elevation [ft]	Area [ft^2]	Average Area [ft^2]	Increment [ft]	Incremental Volume. [ft^3]	Cumulative Volume [ft^3]	Cumulative Volume [ac-ft]
2004.9	0				0	0.00
		2649	0.1	265		
2005.0	5298				265	0.01
		21084	1.0	21084		
2006.0	36870				21349	0.49
		46227	2.0	92454		
2008.0	55584				113803	2.61
		90215	2.0	180429		
2010.0	124845				294232	6.75
		149724	2.0	299447		
2012.0	174602				593679	13.63
		221133	2.0	442265		
2014.0	267663				1035944	23.78

The proportional weir was selected as the outlet structure of the dry pond. For structural details like dam zoning and stability investigations, refer to Section 7.

5.5.1. Proportional Weir

Stormwater Detention ponds are a popular way of minimizing flow peaks downstream of a new development. Flooding potential may be further reduced by adopting the more sophisticated outlet of a proportional weir.

In handling stormwater flows of 2, 10, and 100-year frequencies, the detention pond and proportional weir outlet work extremely well. Not only will the detention pond and outlet handle the design storms, but it will do so in a manner superior to conventional outlets for storms with frequencies of 2 and 100 years.

Figure 5.7 indicates significant parameters of a proportional weir.

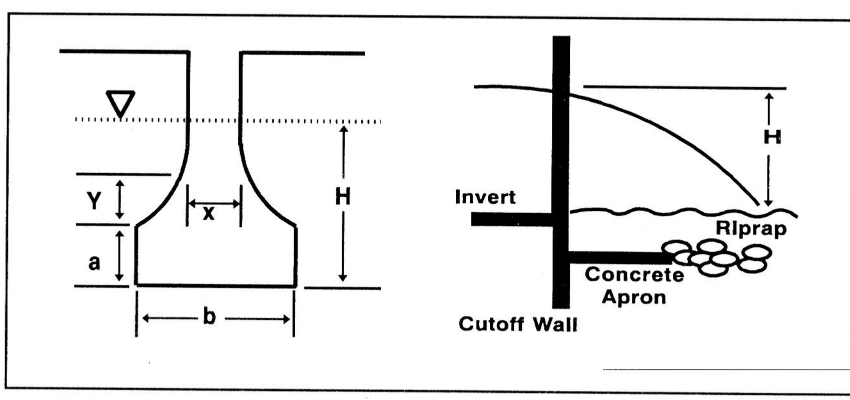

Figure 5.7: Definition of Proportional Weir Parameters (Corcoran, 1995)

In propotional weir design, dimension “a”, the heigth of the rectangular base, the design discharge, “b”, the width of the weir, is calculated until reasonable dimensions are determined. Using these values and Equation 2.24, the coordinates of the weir were found (Figure B-1).

The dimensions of the propotional weir of the designed for the dry pond are as following:

Table 5.6: Shape Parameter of the Proportional Weir

Heigth of the rectangular base	Width of the weir
2.25 ft	11.00 ft

The linear relationship between pool elevation and discharge was obtained using Equation 2.23 and is presented in Figure 5.8. Detailed computations are also listed at Table B-1.

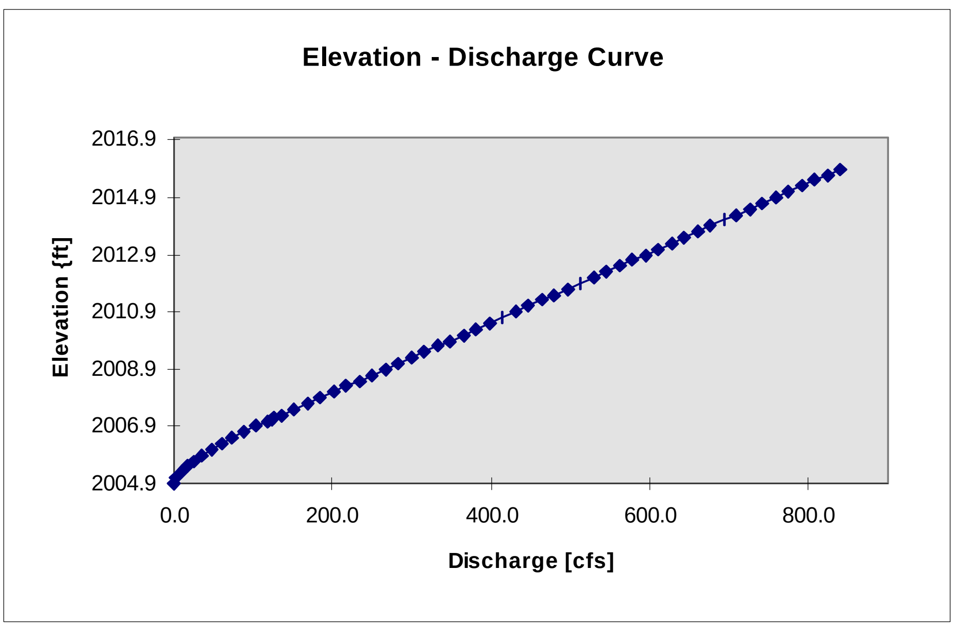

Figure 5.8: Head-Discharge Curve of Proportional Weir

With known storage versus depth (Table 5.5) and depth versus discharge rate relationships (Figure 5.8), a storage versus discharge relationship was derived. The modified Puls procedure was applied to route each storm (2, 10, and 100-year) through the dry pond. The inflows to the dry pond were obtained from discharges of wet pond. The inflow and outflow hydrographs for the dry pond are shown in Figures 5.9-5.11.

5.5.2. Culvert Outlet Structure

A projected 2-ft pipe, in combination with a broad-crested emergency spillway of 20 ft length, where the weir crest elevation is at 2010.0, was designed and its hydraulic performance evaluated.

The 2-ft outfall culvert will have the following parameters:

- Outfall culvert invert elevation: 2004.9 ft
- Outfall culvert diameter: 2.0 ft
- Outfall culvert length: 100 ft
- Outfall culvert slope: 0.006 ft/ft
- Outfall culvert Manning's n: 0.018

Figures 5.9-5.11 show outflow hydrographs and provide comparisons to the inflow hydrograph.

5.5.3. Dry Pond Analysis

In Figures 5.9-5.11, reduction and delay in peak discharge can be seen. The inflow hydrograph of the dry pond is not unique; however, for reasons of comparison and since the outflow hydrographs of the wet pond are similar, only one inflow hydrograph is presented.

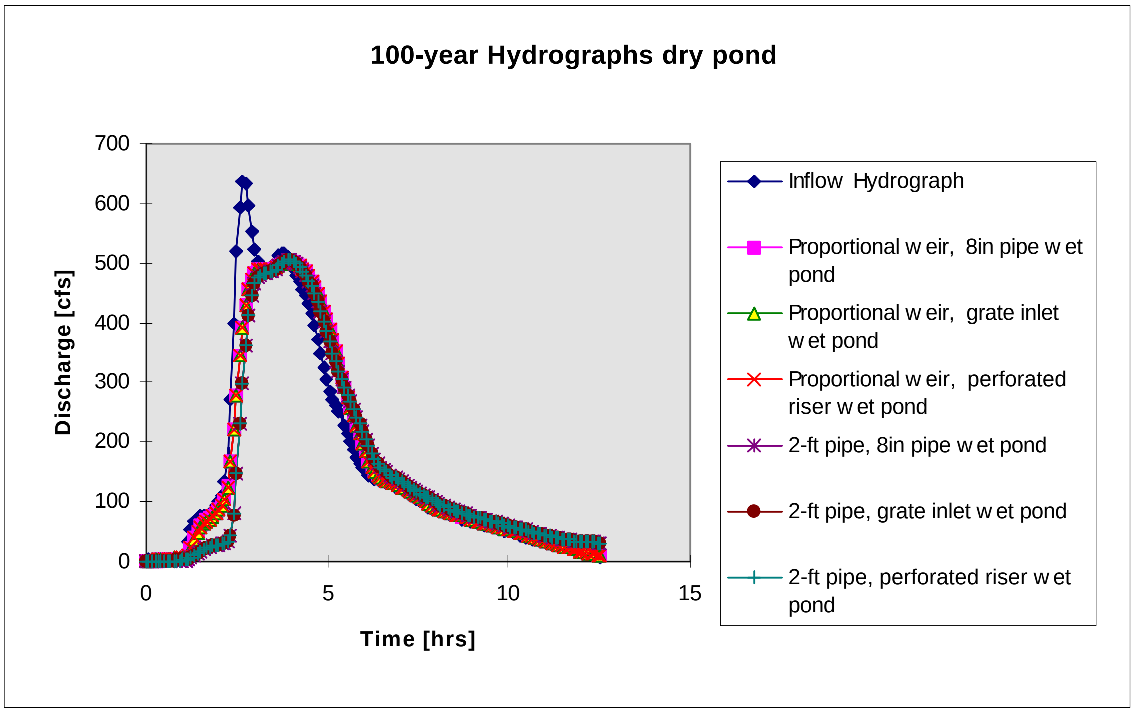

Figure 5.9: 100-year Inflow and Outflow Hydrographs Dry Facility

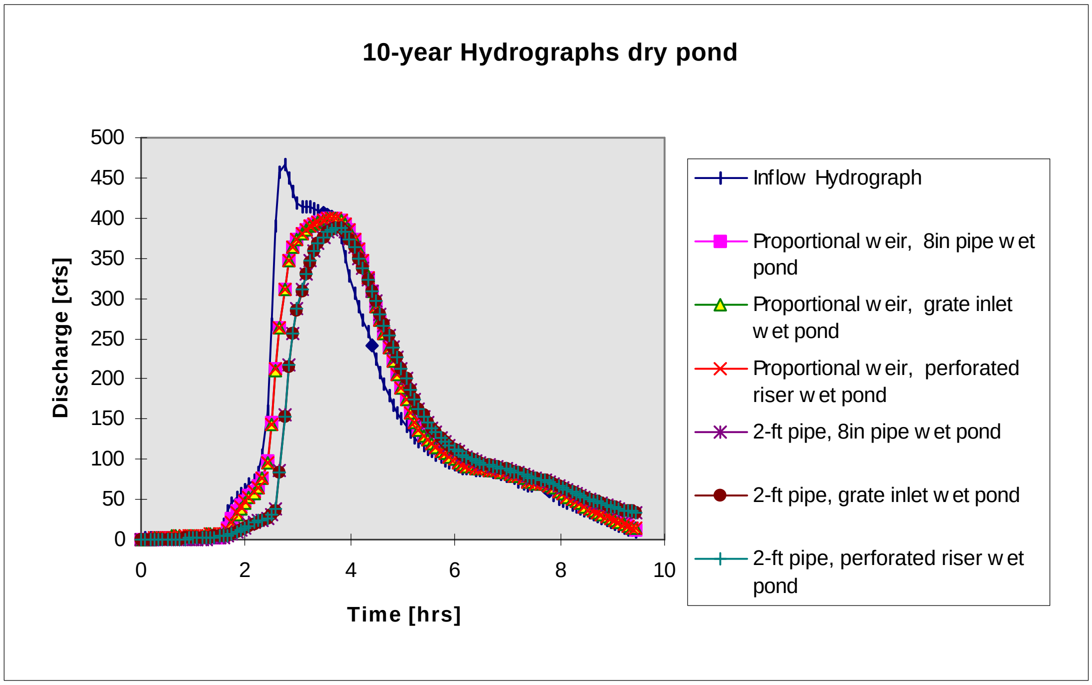

Figure 5.10: 10-year Inflow and Outflow Hydrographs Dry Facility

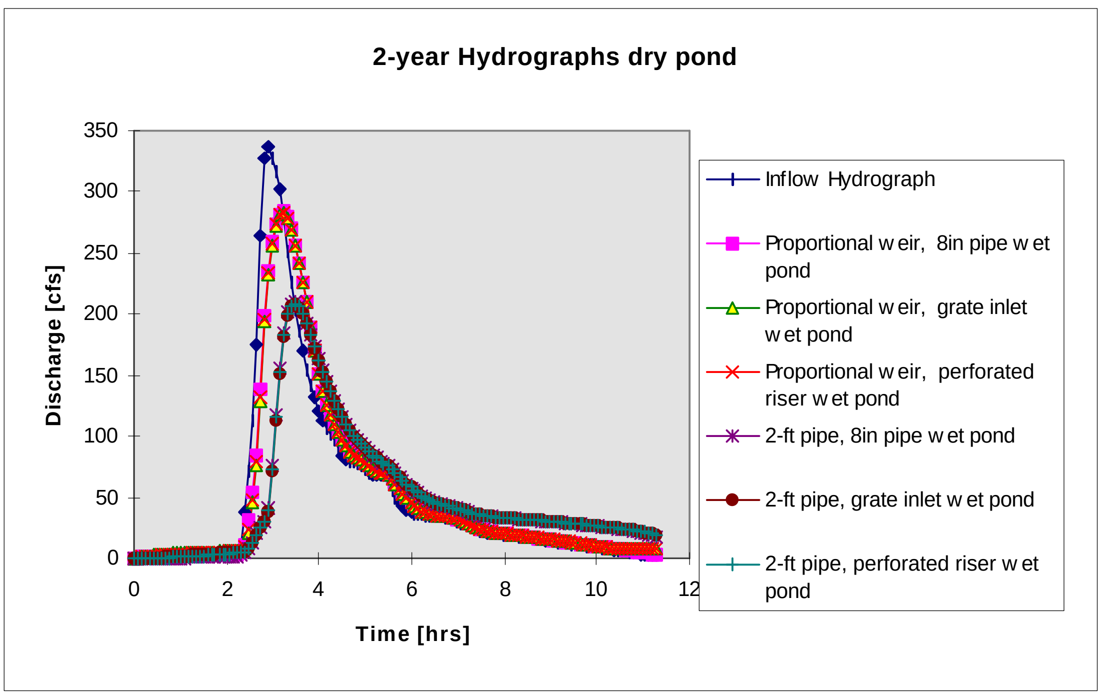

Figure 5.11: 2-year Inflow and Outflow Hydrographs Dry Facility

Table 5.7: Inflow and Peak Discharges of Different Dry Pond Outlet Structures

			Peak Discharges		
	Wet pond	Dry pond	100-year Flood [cfs]	10-year Flood [cfs]	2-year Flood [cfs]
Inflow (present conditions)			652.26	476.61	329.28
Inflow (future conditions)			636.07	487.67	352.76
	8-in pipe	prop.weir	502.56	395.05	284.16
	grate inlet	prop.weir	502.55	395.02	283.39
	perf. riser	prop.weir	502.55	395.04	283.65
	8-in pipe	2' pipe/sp.way	504.57	388.44	209.80
	grate inlet	2' pipe/sp.way	504.56	388.31	208.39
	perf. riser	2' pipe/sp.way	504.57	388.38	209.00

Table 5.8: Maximum Water Surface Elevation due to Different Dry Pond Outlet Structures

		Maximum WSE		
Wet pond	Dry pond	100-year Flood [ft]	10-year Flood [ft]	2-year Flood [ft]
8-in pipe	prop.weir	2011.77	2010.52	2009.11
grate inlet	prop.weir	2011.77	2010.51	2009.10
perf. riser	prop.weir	2011.77	2010.51	2009.11
8-in pipe	2' pipe/sp.way	2013.87	2013.19	2012.00
grate inlet	2' pipe/sp.way	2013.87	2013.19	2011.99
perf. riser	2' pipe/sp.way	2013.87	2013.19	2012.00

The requirements of the stormwater regulations were met by the design and the peak discharges of the 2-year and 10-year events were reduced below the present levels, as indicated in Table 5.7.

Generally, differences in the outlet structures of the wet pond have no significant impact on the peak discharge from the dry pond. But there is a significant discrepancy between the 2-ft pipe including emergency spillway and the proportional weir in hydraulic performance. Remarkably, the proportional weir has a maximum water surface elevation of 2 ft to 3 ft less than the 2-ft pipe for the 2-year and 100-year storm events, respectively (Table 5.8). In accordance with hydraulic characteristics of a proportional weir, the peak discharge of the proportional weir is higher for a 2-year flood event than the peak discharge for the 2-ft pipe. Because of a linear head-discharge ratio, the proportional weir releases less water than the 2-ft pipe in a 100-year flood event. It releases less water than the pipe (Table 5.7).

In general, the dry pond delays the peak discharge. Depending on type of outlet structure and the flood event, a lag of some 80 minutes was determined for a 100-year flood with both types of outlet structures. With a 2-ft pipe outlet structure, the peak outflow for a 2-year flood occurs approximately 35 minutes later than the peak inflow. In contrast, a peaking difference of approximately 20 minutes for the same flood event was determined with a proportional weir. Table 5.8 indicates a shift in the discharge peak times. Maximum water surface elevations (Table 5.8) can be correlated to storage volume used by referring to Table 5.5.

6. WATER QUALITY BENEFITS OF STORMWATER WETLANDS

6.1. Pollutant Removal Capability and Rates of Stormwater Wetlands

The basic intent of a stormwater wetland is to create a shallow matrix of sediment, plants, water and detritus that collectively removes multiple pollutants through a series of complementary physical, chemical and biological pathways. The use of stormwater wetlands to remove pollutants from urban runoff has attracted great interest in recent years, and their performance under widely different environmental and runoff conditions has been documented in numerous research studies. However, most of the reported performance monitoring research has been focused on smaller, experimental wetlands of non-standard design. Thus, only limited data is available to assess the performance of the standard stormwater wetland systems described by Schueler (1992). The projected performance of such systems, however, can be inferred from performance data obtained from the experimental wetlands and other pond systems.

The pollutant removal performance of nearly 25 stormwater wetland systems has been reported to date. Although the stormwater wetland systems monitored have differed greatly in their design and treatment volume, most have shown moderate to excellent pollutant removal capability under a range of environmental conditions. From a review of these studies, it is evident that:

- The removal rates for stormwater wetlands are similar to conventional pond systems, such as dry extended detention and wet ponds. In many cases, suspended solid removal rates were higher than conventional ponds, which reflects the better settling conditions in ponds that are often found in wetland systems. Conversely, phosphorus removal rates in wetland systems were more variable and in some cases slightly lower than wet ponds. The exact reason for the reduced phosphorus removal is not clear yet, but may reflect complex phosphorus cycling patterns often associated with wetlands. In fact, some wetlands actually exported ammonia nitrogen and phosphate at certain times of the year.

- Extended detention (ED) ponds with wetlands typically perform better than ED ponds without wetlands. Four performance studies have been reported, and these suggest a moderate to high capability to remove particulate pollutants, and a low to moderate capability to remove soluble pollutants. However, the four ED wetland systems investigated had inadequate treatment volumes (0.8 to 0.15 inches of runoff per contributing acre), which may have limited their ability to provide reliable levels of soluble pollutant removal.
- Several studies indicate that the performance of stormwater wetlands declines slightly during the non-growing season, and more strongly when the wetland plants dieback in the Fall (releasing a portion of the nutrients stored in above-ground biomass). The lower performance can be attributed to the lack of wetland structure and lower temperatures that reduce microbial and algae activity.
- Some evidence exists that stormwater wetland performance increases as the wetland ages, at least in the first several years. This is attributed to the greater density of plants and organic materials that accumulate in the wetland over time. Initially, many stormwater wetlands do not possess much organic matter because they are excavated to depleted mineral subsoils.

- In general, it is apparent that stormwater wetlands outperform natural wetlands of similar size (Strecker et al., 1992). This undoubtedly reflects the longer detention and/or retention of runoff that is designed into constructed systems.

Based on the analysis of the performance studies (Schueler, 1992) and the various removal pathways in wetlands, it is expected that stormwater wetlands in the Mid-Atlantic area can reliably achieve the long-term removal rates outlined in Table 6.1, if they are designed as specified in Table 6.2.

Table 6.1: Projected Long Term Pollutant Removal Rates for Stormwater Wetlands in the Mid-Atlantic Region (Schueler, 1992)

Pollutant	Removal Rate
Total Suspended Solids	75%
Total Phosphorus	45%
Total Nitrogen	25%
Organic Carbon	15%
Lead	75%
Zinc	50%
Bacteria	2 log reduction

Table 6.2: Sizing Criteria for Stormwater Wetlands (Schueler, 1992)

Sizing Criteria	Shallow Marsh	Pond/Wetland	ED Wetland	Pocket Wetland
Runoff Treatment Volume (V_t)	Capture 90% of the annual runoff volume from site, V_t = (1.25 inches) (Runoff Coefficient)(Site Area), Minimum V_t of 0.25 watershed-inches			
Wetland to Watershed Area Ratio	0.02	0.01	0.01	0.01
Allocation of Surface Area (%)	20 - deep 40 - lo marsh 40 - hi marsh	45 - deep 25 - lo marsh 30 - hi marsh	20 - deep 35 - lo marsh 45 - hi marsh	10 - deep 40 - lo marsh 50 - hi marsh
Allocation of Treatment Volume (%)	40 - pool 60 - marsh 0 - ED	70 - pool 30 - marsh 0 - ED	20 - pool 30 - marsh 50 - ED	20 - pool 80 - marsh 0 - ED
Flow Path a.) Length to width ratio b.) dry weather path	 1:1 2:1	 1:1 2:1	 1:1 2:1	 N.A. 2:1
Water Balance	Confirm inflow rate > 0.002 cfs/acre, compute water balance during dry weather			Confirm dry weather water table elevation in field
Extended Detention	Not Employed	Not Employed	EDv = 50% of V_t, 12 to 24 hrs ED range ≤ 3 ft	Not Employed

- Minimum Treatment Volume: The pollutant removal capability for any stormwater wetland is limited by how much stormwater bypasses the wetland without treatment. The 90% storm event (quarterly return interval) is recommended by Schueler (1992) as a target capture efficiency because it is realistically achievable. A higher design criteria, such as the 99% storm event (2-year return interval), would require treatment of the runoff from nearly 2.9 inches of runoff. The treatment volume (V_t) can be derived once the maximum rainfall volume has been selected (1.25 inches), using the relationship between site imperviousness and the storm runoff coefficient (R_V):

$$R_V = 0.05 + 0.009(I) \tag{6.1}$$

$$R_V = 0.23$$

where I is percent site imperviousness, which was assumed to be 20%.

Thus, the required treatment volume for the site equals (Schueler, 1992):

$$V_t = ((1.25)(R_V)(A) / 12)(43{,}560) \tag{6.2}$$

$$V_t = ((1.25)(0.23)(446) / 12)(43{,}560)$$

$$\underline{V_t = 465{,}457\ ft^3 > 209{,}088\ ft^3\ (available)} = (4.8\ ac\text{-}ft)(43{,}560\ ft^2/ac)$$

The minimum requirement of 0.25 watershed-inches applied to the watershed and the proposed facility can be calculated to:

$$V_{t,\ min} = ((0.25)(446) / 12)(43{,}560) \tag{6.3}$$

$$\underline{V_{t,\ min} = 404{,}745\ ft^3}$$

It is obvious that the required treatment volume, V_t, is not obtained. However, if the extended detention volume of 4.8 ac-ft is added, the minimum requirement will be maintained.

$$V_{t,\ avail.} = (2)(4.8\ ac\text{-}ft)(43{,}560\ ft^2/ac)$$

$$\underline{V_{t,\ avail.} = 418{,}176\ ft^3 > 404{,}745\ ft^3 = V_{t,\ min}}$$

- Surface Area Requirement: Generally, the pollutant removal capability of a stormwater wetland becomes more consistent when the wetland-to-watershed area ratio exceeds 2% (Nichols, 1983, Strecker et al, 1992). The reliability of pollutant removal also tends to increase as the wetland to watershed-area-ratio (WWAR) increases (Figure 6.1) although the relationship is not always consistent.

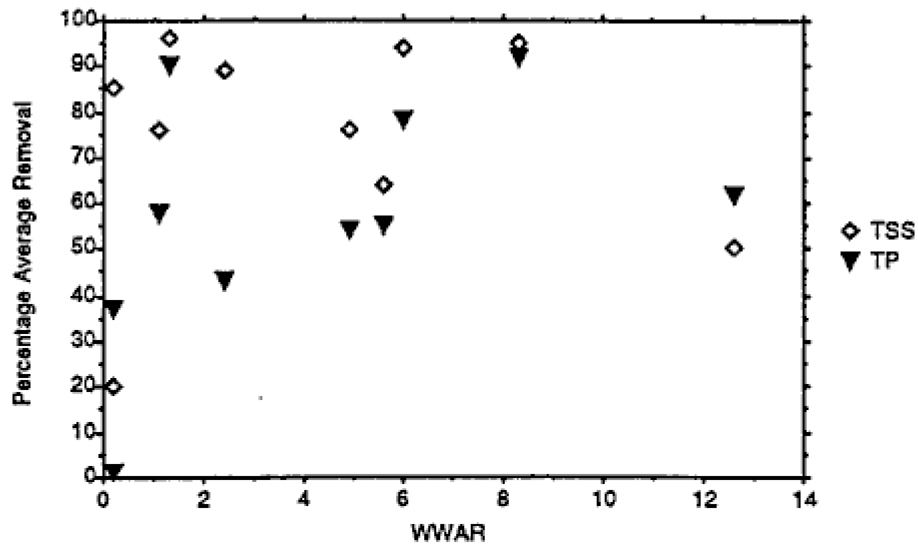

Figure 6.1: Pollutant Removal as a Function of the WWAR (Strecker et al, 1992)

Thus, Schueler (1992) recommends that a minimum WWAR of 2% be used to size the surface area of shallow marsh wetlands. To his mind, a smaller ratio of 1% can be justified in the case of Pond wetlands and ED wetlands, as the pool and extended detention treatment storage components partially substitute for shallow wetland areas. However, this criteria is not feasible at the site of the School of Veterinary Medicine. It had to be waived (WWAR = 0.99 / 446 * 100 = 0.22%) and the pollutant removal capability is expected to be lower.

- Depth / Area and Treatment / Volume Allocation: The depth / area allocation within a wetland is important for a number of reasons. Foremost, it is used to achieve the maximum surface-area-to-volume ratio possible within the wetland, by creating a series of depth zones that will produce "microtopography". Four basic depth zones are possible - deepwater, lo marsh, hi marsh and semi-wet areas.

The deepwater areas can be further broken down to the three components - forebay, micropools and deepwater channels. The deepwater zones are from 1.5 to 6 feet deep, and support little emergent vegetation, but may also support submerged or floating aquatic vegetation.

The lo marsh zone ranges from 6 to 18 inches below the normal pool, and the hi marsh zone ranges from 6 inches below the pool up to the normal pool. Generally, the hi marsh zone will support greater density and diversity of emergent wetland plants than the lo marsh zone. Therefore, it will possess a higher surface-area-to-volume ratio. The semi-wet zone is greatest for the extended detention wetlands and ranges from zero to 2 feet above the normal pool.

General targets for allocating the total treatment volume (V_t) are shown in Table 6.2. The table also illustrates the relative dominance of permanent pool, shallow marsh, and extended detention storage. More detailed, treatment volume and area allocations for the extended detention wetland design are described in Table 6.3. Both allocation criteria represent targets rather than standards, and are used to apportion the basic treatment methods to maximize removal efficiency.

Table 6.3: Guidelines for the Allocation of Depth Zones and Treatment Volume in an Extended Detention Wetland Systems

Target Allocation	Percent of	
	Surface Area	Treatment Volume
Forebay	5	10
Micropool	5	10
Deepwater	0	-
Lo Marsh	40	20
Hi Marsh	40	10
Semi-wet	10	50

Estimating the surface area and treatment volume from Drawing No. 1, the design will meet the guidelines. However, the forebay treatment volume will slightly be less than the target of 10%.

- Flow Path Length: This criteria is intended to create the longest possible flow path through the wetland system, and thereby increase contact time over the surface area of the marsh. The effective flow path can be maximized in two ways:

  - by increasing the length-to-width ratio of the entire wetland design (general rule: length-to-width ratio should be greater to or equal to 1, to prevent short circuiting) and
  - by increasing the dry-weather-flow path within the wetland to attain maximum sinuosity (placing of wedges of hi marsh perpendicular to the flow path, as shown in Figure 6.2).

The design criteria for stormwater wetlands, as proposed by Schueler (1992), is to achieve a dry weather flow path of 2:1 or greater, using both techniques. The extended detention wetland proposed is designed to have a ratio of 5:1 with hi marshes to partially compensate the insufficient WWAR.

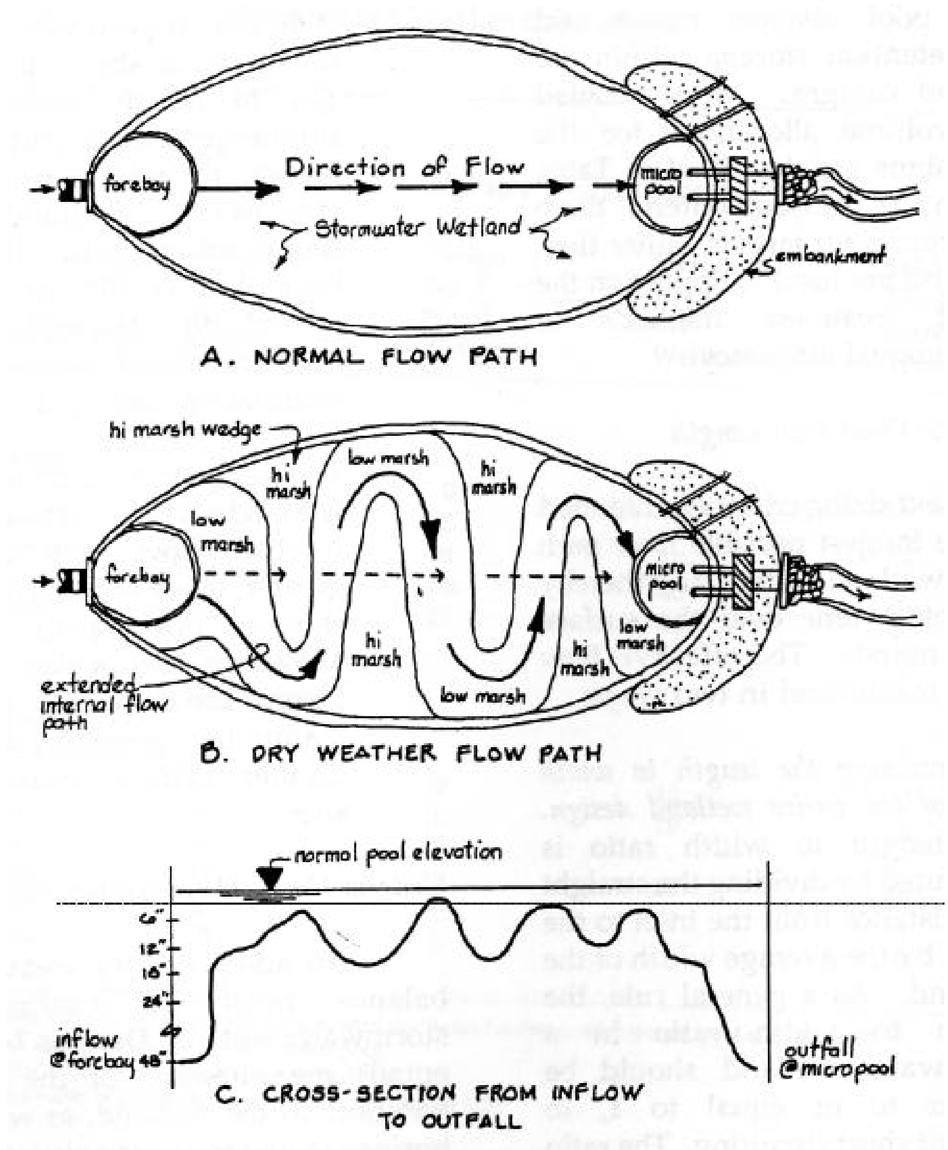

Figure 6.2: Use of Hi Marsh Wedges to Extend the Internal Flow Path Through a ED Wetland

- Dry Weather Water Balance: Schueler (1992) emphasizes in his guidelines that an adequate dry weather water balance must be confirmed for the design of a ED wetland. This entails measurement of the incoming flow to the wetland, as well as borings to determine the elevation of the water table and soil permeability rates. Infiltration losses depend on the underlying soil type, and can be significant in case of karst and fractured bedrock. A general rule of thumb is that a baseflow of 0.002 cfs per acre is needed to maintain water elevations in a stormwater wetland during dry weather, assuming that infiltration losses are minor. Thus, the entire drainage area of 446-acre should supply a steady inflow of 0.89 cfs (0.002 * 446) to the wetland. Compared to the baseflow measurement of 1.95 cfs, a permanent pool elevation can be maintained in case of no karst and fractured rock beneath.

- Extended Detention Volume: The design followed unique sizing rules for ED wetlands defined by Schueler (1992):

  - The volume of extended detention is not larger than 50% of the total volume (V_t). The sizing criteria was considered by proposing a design with equally wet and dry storage of 4.8 acre-feet.

  - The target ED detention time for this volume should be 12 to 24 hours. As shown for the outlet structure of a perforated riser at Table 6.4 , the detention time will be 12.82 hours for the 100-year storm event.

Table 6.4: Drawdown Times of the 100-year Flood for Outlet Structures Investigated

Outlet Structure		Elevation Above MSL	Pool Elevation		Drawdown Time for the 100-year Flood	
Wet pond	Dry pond	[ft]	Permanent [ft]	Bottom [ft]	[hrs]	[days]
8-in pipe		2021.63	2018.1		20.56	0.86
grate inlet		2021.62	2018.1		11.27	0.47
perf. riser		2021.61	2018.1		12.82	0.53
	prop.weir	2012.30		2004.9	1.59	0.07
	2-ft pipe	2013.87		2004.9	9.98	0.42

- The ED orifice must be well protected from clogging. Schueler (1992) recommends a reverse-slope submerged orifice or a perforated half-round hood over a broad crested weir. The options considered for the project are: perforated riser, 8-inch reverse slope submerged orifice, and a grate inlet including a broad-crested weir as an emergency spillway.
- The maximum ED water surface elevation will not be greater than three feet above the normal pool elevation (2020.2 ft - 2018.1 ft = 2.1 ft), because large and frequent changes in water level are not conductive to the growth of dense or diverse stands of emergent wetland plants.

7. CONSTRUCTIONAL ASPECTS OF DAMS AND OUTLET STRUCTURES

7.1. Regional Geology

Available geologic references (Froehling & Robertson, 1996) report that the site lies within the Valley and Ridge geologic provinces in southwestern Virginia, and is underlain by Cambrian-aged rocks of the Elbrock Formation. Quaternary-aged, fine-grained alluvium overlies the Elbrock over the majority of the site. Alluvium has been deposited in part from the tributary to nearby Stroubles Creek that passes through the site. Alluvial soils are reported to consist of rounded cobbles, boulders, sand, and gravel. The Elbrock consists of interbedded dolomite and limestone with lesser shale and siltstone. Often, these rocks weather to form a highly variable bedrock surface consisting of troughs and pinnacles that may greatly fluctuate in elevation within short lateral distances.

Referring to Keller (1957), continued subsurface dissolution of the carbonate bedrock may lead to development of a highly irregular bedrock surface consisting of pinnacles (high points) and troughs (low areas). Dissolution below the surface of the rock results in voids ranging in size from isolated vugs less than two inches in diameter up to extensive caverns many feet across and several thousand feet in extent. Over time, the soils overlying areas of dissolved rock may subside, in a continual process of subsurface chemical erosion of rock and infilling of underlying cavities by overburden soils. Karst landforms characterized by sinkholes and other solutional features are common in areas underlain by rocks of the Elbrock formation. The development of the ground surface depression may result from gradual subsidence or may occur rapidly.

At the proposed detention facility site, there are numerous variations on sinkhole development. Regardless of the mode of development, it is important to note (Keller, 1957) that man-made changes in the soil stress and water regime can greatly accelerate sinkhole development. Natural geologic processes that might otherwise occur over thousands of years can occur within several years or even months. Construction activities such as site grading, building construction, and water impoundment have reportedly caused sinkholes to develop rapidly or to collapse suddenly.

7.2. Site Exploration and Subsurface Conditions

The consulting firm Anderson & Associates conducted a subsurface exploration program and a geotechnical analysis was conducted by Froehling & Robertson, Inc. in June 1996. Froehling & Robertson, Inc. drilled 29 borings, B-1 through B-29, to auger refusal or eight ft maximum. The exploration program developed by Anderson & Associates was performed on 25 - 27 June 1996 at the locations shown on the Drawing No. 1 and 2. The elevation of the ground surface at each boring location was estimated by interpolation of elevation contours and elevation points on the Probe Hole Sketch (Drawings No. 1 and 2) by Anderson & Associates.

The test borings were performed in accordance with generally accepted practice using an ATV-mounted CME-55 rotary drill rig. A continuous split-spoon sampling was performed from 0.5 ft to either 8 ft maximum or the auger refusal depth. Hollow-stem augers were advanced to pre-selected depth, the center plug was removed, and disturbed soil samples were recovered with a standard split spoon sampler (1.375 in ID, 2.0 in OD) in general accordance with ASTM D 1586, the Standard Penetration Test (Terzaghi et al, 1996).

The site, which is located near the Veterinary Medicine School, is characterized by a bended tributary of the Stroubles Creek. Over the entire area, numerous rock outcrops can be noted and are generally more heavily concentrated at the southeastern end of the proposed wet facility. However, no sinkholes or other karst features could be observed. Heavy vegetative growth covers the project site, and soft ground surface condition is prevalent. During the time of the drilling, it was reported that water was flowing in the tributary.

The recent subsurface exploration of Froehling & Robertson generally encountered alluvial soils over disturbed residual soils. Some of the borings did not penetrate the alluvial soils while others continued into decomposed rock. The alluvial soils encountered are typical of the high variability present in an alluvial deposit. Soil type, thickness, and consistency in an alluvial deposit tend to change rapidly over short lateral and vertical distances. Residual soils and decomposed rock encountered are typical of those that weather from the carbonate rocks supposedly found in the Blacksburg area.

In general, the alluvial soils consisted of fine-grained silts and clays that were classified by visual-manual methods in the laboratory of Froehling & Robertson (1996) as OL, ML, CL, and CH, and coarse-grained alluvium classified as SC and SM.

Residual soils were encountered by Froehling & Robertson (1996) as fine-grained clays and silts visually-manually classified as CL, CH, ML, MH and coarse-grained residual sands as SM and SW.

Auger refusal, which was interpreted as the presence of rock, occurred in 20 of the 29 borings prior to reaching the intended termination depth of 8 ft. Auger refusal occurred at depths ranging from 1 to 7 ft, as partially shown on the selected boring logs of Appendix D. But the depth to auger refusal may vary intermediate of the boring locations and may exist at shallow depths in unexplored areas of the site.

According to the boring logs, measurable subsurface water was encountered in 3 of the 29 borings in an uncased borehole immediately upon completion. Fluctuations in subsurface water levels and soil moisture are very likely with changes in precipitation, runoff, and season.

As can be seen at the cross-sections of Figures C-1 and C-2, residual and alluvial materials are of high variability. The boring logs B-1 through B-5 (Table C-1 and Figure C-3) and B-13 through B-15 (Table C-1 and Figure C-2) represent the cross-section at the sites of the dams with temporary and permanent impoundment, respectively. (refer also to Drawings No. 1 and 2)

7.3. River Diversion During Construction

The design for the dams, which are to be constructed across the stream channel, considers diversion of the streamflow around and through the structure. The diversion works were incorporated into the overall construction program. For earthfill dams, where considerable areas of foundation and structure excavation are exposed and overtopping of the embankment while under construction may result in serious damage, the importance of eliminating the risk of flooding generally is great.

For the small dams which will be constructed in a single season at the same time, only the floods that may occur during that season need to be considered. The Bureau of Reclamation (1965) states that it should be sufficiently conservative to provide for the largest flood likely to occur in a 5-year period.

A partial diversion for the wet pond will be necessary. The existing streambed will be able to handle a 5-year flood event, if the left bank of the channel, where the outlet structure is to be located, is stabilized with riprap and slightly raised. Thus, the riser pipe outlet structure can be placed and will later discharge the water when the dam gap is filled.

During the construction of the proportional weir, a temporary channel involving a gap through the right side of the earthfill dam will be utilized to divert streamflow. The estimated discharge for a 5-year flood will be approximately 420 cfs. Assuming a 1%-slope, a channel roughness of 0.02 (wood), the diversion area calculated using the Manning's equation (Equation 4.2) is 8 ft wide and 4.5 ft deep.

7.4. Embankment Construction and Design

7.4.1. Embankment Construction and Stability

The stability of an embankment is determined by its ability to resist shearing stresses, since failure occurs by sliding along a shear surface. Shearing stresses result from externally applied loads, such as reservoir and earthquake, and from internal body forces caused by weight of the soil and the embankment slopes. The external and internal forces also produce comprehensive stresses normal to any potential sliding surface. These comprehensive stresses both contribute to the shearing strength of the soil and to the development of destabilizing pore water pressures. The magnitude of pore water pressures from comprehensive forces depends on the compressibility and of the compacted soil and of the amount of air it contains.

The basic principle of design should be the production of a satisfactory functional structure at a minimum total cost. For minimum cost, the dam must be designed for maximum utilization of the most economical materials available, including material which must be excavated for its foundations, capacity increase, and for appurtenant structures. The yardage from these sources constitutes an appreciable portion of the total embankment quantity. Consequently, it strongly influences the design of the dams. Although these materials are less desirable than soil from borrow areas, economy requires that they be employed to the maximum practicable extent. The zoned type of dam allows for maximum utilization of material that must be excavated for the foundation of the dam, spillway, and outlet works. The feasibility of using materials from structural excavation is influenced by the sequence of construction operations. Adequate placing areas must be identified to use materials from the reservoir and cutoff trench excavations in the embankments without having to stockpile and later rehandle large quantities of earth and rock.

The upper 6-inch surficial soil layer will be wasted and not used for embankment fill. Drawings No. 1 and 2 point out the locations where the surficial material will be moved. The piles will be situated above the highest pool elevation and silt fences will be placed to prevent erosion into the pond prior to fill stabilization.

The slopes of an earthfill dam depend on the type of dam and on the nature of the material for construction. The Bureau of Reclamation (1965) published guidelines to design small dams on saturated fine-grained soils with a safety factor of 1.5 assuming that no drainage occurred in the foundation during construction. According to their recommendations, the slope for 10-ft dams on saturated soils of either medium consistency (SPT of approximately 4 to 10 blows per ft), stiff consistency (approximately 11 to 20 blows per ft), or hard consistency (greater than 20 blows per ft) should be of a ratio of at least 3:1 (H:V). Based on this recommendation, the upstream slopes of the dams will be 4:1 and the downstream slopes 3:1. A flatter upstream slope is used to eliminate expensive slope protection because of wave actions.

To control seepage through and under the proposed dams, a cutoff trench will be constructed along the centerline of the dams, because alluvial soils extend fairly deep beneath the structures. The cutoff trench should be a minimum of 4 ft wide and penetrate this soil layer into undisturbed residual soils. Based on a review of boring logs B-1 through B-5 and B-13 through B-15, a trench four to six ft deep will be required to minimize potential seepage. The trench should be constructed the full length of the dam. Since there are no data of the depth of alluvium layers, because the borings were terminated at a maximum depth of 8 ft below the ground surface, it is assumed that the layer will extend further. Therefore, the design will be based on a pervious foundation that is not completely penetrated by a cutoff trench. The cutoff trench backfill is designed to extend upward as a core through the center of the dam to an elevation at least as high as the maximum detained water elevation.

The core minimum width of 8 ft was selected for both practical and theoretical reasons. It was taken as the minimum, because it will permit economical placement and compaction of impervious embankment material by construction equipment, which includes trucks, dozers, and sheepfoot rollers. The criterion given by the Bureau of Reclamation (1965), that the thickness of the core at any elevation can not be less than the height of embankment at that elevation, was adopted so that the average hydraulic gradient through the core will be less than unity. Steeper gradients will result in high seepage forces and the necessity for construction of high-quality filter zones, which, for small dams of under 10-ft elevations, are neither economical nor practicable from a construction control standpoint. Furthermore, if the core were thinner, it could potentially be ruptured by differential settlement of the foundation. With the minimum size core centrally located, the stability of the zoned embankment is not greatly affected by the nature of the soil comprising the core.

The cutoff trench and core backfill material may be furnished either from areas to be cut during dam excavation or from an off-site source. Because of the high variability of soils encountered in the subsurface exploration, it is not certain that on-site borrow will be satisfactory with respect to quantity and uniformity. Materials from the site are likely to contain gravel, cobbels, boulders, and broken rock without significant deposits of clean, uniform clays and silts.

The Bureau of Reclamation (1965) recommends a minimum freeboard, which is the vertical distance between the crest of the embankment and the maximum reservoir water surface, of 3 ft for reservoirs having a fetch of less than one mile. The crest elevation of the dry pond will be at elevation 2014.90 ft. Thus with the highest water surface being at 2011.77 ft, the minimum freeboard will be maintained. A survey of the Corps of Engineers substanciated the premise that dumped riprap is by far the preferable type of upstream slope protection. So, riprap will be placed on the upstream side of the dam. As opposed to that, the wet pond will not maintain a freeboard of 3 ft. Because of insufficient freeboard of the pond, the upstream side of the dam will be protected with grass and downstream side, that is exposed to possible flood damage due to high probabilty of overtopping, by placing of riprap. Since the emergency spillway extends almost entirely over the length of the dam, there is little chance of damage to the dam structure itself.

A minimum crest width should be specified to provide a safe percolation gradient through the embankment at the level of the full reservoir. Because of practical difficulties in determining this factor, the crest width is, as a rule, determined empirically and largely by precedent. The following formula is suggested by the Bureau of Reclamation for the determination of crest width for small earthfill dams:

$$w = \frac{z}{5} + 10 \tag{7.1}$$

where w is the width of crest [ft] and

z is the height of dam [ft] above the lowest point in the streambed (z = 10 ft).

For ease of construction with power equipment, the same agency recommends that the minimum width should not be less than 12 ft. Based on a minimum core width of 8ft, the crest width of the dry pond was chosen to 25 ft. Similarly, the crest of the wet pond almost exclusively consist of the emergency spillway, which is 100 ft long and approximately also 25 ft wide.

7.4.2. Embankment Protection

There is a wide variety of possible bank protection methods. Riprap will be used for downstream and grass for upstream embankment protection. Maynord (1987) and Reese (1984) proposed a procedure for riprap placement including the following assumptions and limitations:

- minimum riprap thickness equal to D_{100}
- the value of d_{85}/d_{15} less than 4.6

- maximum velocity less than 18 ft/s.

Riprap needs to be placed downstream of the dams. From the continuity equation, the flow velocity on the emergency spillway of the wet pond was calculated to be approximately 3.8 ft/s, considering a weir width of 100 ft, a 100-year storm event discharge over the spillway of approximately 607 cfs, and a resulting flow depth of 1.7 ft. The crest of the spillway will be lined with grass, e.g. Kentucky bluegrass, that can withstand permissible velocities of 4 ft/s according to USDA (1954). The downstream face of the wet pond will experience higher flow velocity. Estimations by the Manning's formula (Eq. 4.2) and the continuity equation gives an approximate flow velocity (Manning's formula -> flow depth of 0.47 ft, continuity equation -> v = 12.5 ft/s). According to a figure of Neill (1984), for an allowable mean velocity of 12.5 ft/s, the D_{50} should be of at least 200 mm (8 inches). The equations were solved by a trial and error procedure, since n, the channel roughness for riprap lined channels, is dependant on D_{50}. The n value, D_{50} in ft, is expressed as:

$$n = 0.0395 \cdot \left(D_{50}\right)^{1/6} \qquad (7.2)$$

The riprap layer thickness should be a minimum of D_{100}, and the D_{85}/D_{15} value should be less than 4.6. Further, stone should be angular in shape. The riprap should be extended at least 20 ft downstream the toe of the dam to prevent scour.

The mean flow velocity of the proportional weir, the outlet structure of the dry pond, will be approximately 8.5 ft/s for a 100-year flood event solved by the continuity equation. From Neill (1984), a riprap of D_{50} = 100 mm (4 inches) could serve as a safe bed material to prevent the channel from scouring. However, since the mean flow velocity was very roughly estimated, the same riprap as used for the downstream slope of the wet pond should be placed. The stabilization of the channel is necessary to guarantee channel stability at least 50 ft downstream of the proportional weir.

7.4.3. Requirements and Specifications on Compaction

For construction of small dams, the placement of the embankment material at the Proctor optimum water content and maximum laboritory density is critical. The water content should not be appreciably greater than optimum for Proctor maximum dry density because difficulties have been experienced with unstable fills when very wet soils were used, even in dams of low height. The foregoing considerations result in the recommended practice of compacting cohesive soils in the cores of small dams close to the optimum water content at Proctor maximum dry density.

Other important variables affecting construction of earthfill embankments are distribution of soils, placement, roller characteristics, number of roller passes, thickness of layers, maximum size and quantity of gravel size in the material, condition of the surface layers after rolling, and effectiveness of power tamping in places inaccessible or undisireable for roller operations, such as anti-seep collars.

Adequate inspection and laboratory testing are essential to maintain the control of earthfill construction. Inspections need to cover the determination of the thickness of the compacted layer. A layer that is spread too thick will not give the desired density for given compaction conditions. Usually, 8 to 9 inches for a 6-inch compacted lift of earthfill is a proper spread thickness.

The removal of oversized rock from earthfill embankment material is most efficiently done prior to delivery on the emankment. However, smaller amounts of oversized rock can be removed by hand picking or rock rakes. Common fill for the embankment dams and pond side slopes should be free of organics and rocks greater than 3 inches in diameter, and should be compacted to at least 93% of the soils maximum dry density, as determined by the standard Proctor method (ASTM D 698).

The inspector in charge at the construction site has to make sure that the specified number of roller passes differing by soil type was made on each lift. An oversight in maintaining the proper number of passes may lead to considerable drop in the degree of compaction.

The cutoff trench and the core should be backfilled with high-plasticity soils (CH or MH) compacted in thin lifts (6 inches) to at least 93% of the soils's maximum dry density as determined by the Standard Proctor method (ASTM D 698). The plasticity index (ASTM D 4318) of the backfill material should be greater than 20 but less than 50 (SM, SC, ML). Depending on availability at local construction firms, a sheepfoot roller type is to be used for compaction of cohesive materials. However, the number of roller passes to obtain a 93% maximum dry density of the soil may vary with the weight and length of the compactors.

Mechanical tamping, used around structures, along abutments and in areas inaccessible to the rolling equipment, should be carefully watched by the inspector and checked by frequent density tests. The procedure to be followed for mechanical tamping will depend greatly on the type of the tamper used.

7.5. Seepage and Piping

Foundations of dams consisting of recent alluvial deposits are composed of relatively pervious sands overlying impervious geological formations. Basic problems are found in pervious foundations; one pertains to the amount of underseepage, and the other is concerned with the forces exerted by the seepage. Loss of water through underseepage may be of economic concern for a storage dam but of little consequence for a detention facility.

Upon a precise determination of the coefficient of permeability of the foundation, a rough approximation of the amount of underseepage may be made by use of the Darcy formula:

$$Q = k \cdot i \cdot A \qquad (7.3)$$

where Q is discharge [cfs],

k is the coefficient of permeability of the foundation [ft/s],

i is the hydraulic gradient or

i = h/L = difference in head [ft] / length of path [ft], and

A is gross area of foundation through which flow takes place [ft^2]

Table 7.1 is a summary of values obtained on more than 1500 soil tests performed in the Engineering Laboritries of the Bureau of Reclamation in Denver, Colorado, arranged according to the main soil classification groups. The data for this table were obtained from reports for which laboratory soil classifications were available.

Table 7.1: Permeability, Unified Soil Classification System

Soil group	Permeability, k		
	[ft/yr]	[ft/sec]	[cm/sec]
GW	27000.0	8.56E-04	2.61E-02
GP	64000.0	2.03E-03	6.19E-02
GM	0.3	9.51E-09	2.90E-07
GC	0.3	9.51E-09	2.90E-07
SW	-	-	-
SP	15.0	4.76E-07	1.45E-05
SM	7.5	2.38E-07	7.25E-06
SM-SC	0.8	2.54E-08	7.73E-07
SC	0.3	9.51E-09	2.90E-07
ML	0.6	1.87E-08	5.70E-07
ML-CL	0.1	4.12E-09	1.26E-07
CL	0.1	2.54E-09	7.73E-08
OL	-	-	-
MH	0.2	5.07E-09	1.55E-07
CH	0.1	1.59E-09	4.83E-08
OH	-	-	-

Further, a very general estimation of permeability of soils can be obtained from a table given by Duncan (1996).

Table 7.2: Average Permeability Values after Duncan (1996)

Soil group	average k	
	[cm/sec]	[ft/sec]
coar.sand	1.00E-01	3.28E-03
fine sand	1.00E-02	3.28E-04
silty sand	1.00E-04	3.28E-06
silt	1.00E-06	3.28E-08
clay	1.00E-07	3.28E-09

The seepage quantity for the dams will be calculated based on a permeability value of 1.00E-07 ft/sec, which was determined as an average for ML and SL soils (Tables 7.1 and 7.2). The seepage quantities were computed by means of Equation (7.3):

- Wet pond facility:

Assumptions: - Seepage occurs beneath the cutoff trench and through the embankment only.

- Max. difference in head: 2021.84 - 2016.00 = 5.84 ft
- Length of path: 12.6 + 11/0.7 + 4 + 22.8 = 55.1 ft
- Seepage area (per ft depth): 140 ft * 1 ft = 140 ft^2/ft

Q = 1.0E-07 ft/sec * 5.84 ft / 55.1 ft * 140 ft^2/ft

Q = 1.5E-06 ft^3/sec per ft depth beneath cutoff trench

<u>Q = 1.3E-01 ft^3/day per ft depth beneath cutoff trench</u>

Seepage through dam under baseflow conditions:

- Max. difference in head: 2018.10- 2016.00 = 2.10 ft

Q = 1.0E-07 ft/sec * 2.10 ft / 55.1 ft * 140 ft^2/ft

Q = 5.3E-07 ft^3/sec per ft depth beneath cutoff trench

<u>Q = 4.6E-02 ft^3/day per ft depth beneath cutoff trench</u>

- Dry pond facility:

Assumptions: - Seepage occurs beneath the cutoff trench and through the embankment only.

- Max. difference in head: 2011.77 - 2004.90 = 6.87 ft
- Length of path: 31.8 + 8/0.7 + 17 + 40 = 100.2 ft

- Seepage area (per ft depth): 320 ft * 1 ft = 320 ft^2/ft

Q = 1.0E-07 ft/sec * 6.87 ft / 100.2 ft * 320 ft^2/ft

Q = 2.2E-06 ft3/sec per ft depth beneath cutoff trench

<u>Q = 1.9E-01 ft^3/day per ft depth beneath cutoff trench</u>

The accuracy of the amount of underseepage as determined by Darcy's formula depends on the homogeneity of the foundation and the accuracy with which the factor of permeability is determined. The results obtained indicate only the order of magnitude of seepage and are likely to be conservative. Generally speaking, the seepage quantity is small considering that the depth below the cutoff trench does not extend infinitely and the foundation consists of stratified soil layers.

The flow of water through a pervious foundation produces seepage forces as a result of the friction between the percolating water and the walls of the pores of the soil through which it flows. When the cross-sectional area through which flow takes place is restricted, as under the dams, the velocity of the seepage for a given flow is increased. The increase in velocity is accompanied by an increase in friction loss, and the seepage force is correspondingly increased. As the water percolates upward at the downstream toe of the dams, the seepage force tends to lift the soil trying to float out the soil. The erosion would progress backwards along the flow line until a "pipe" would be formed to the reservoir, allowing rapid escape of reservoir storage and subsequent failure of the dam. Experience has shown that this action can be slow and accumulative and the resulting failure can be a sudden upheaval of the foundation at the downstream toe of the dam. This type of piping failure has been referred as "failure by heave".

Relatively impervious foundations or pervious foundations with adequate cutoff trenches are not susceptible to piping because impervious soil offers so much resistance to the flow of water that the reservoir head is largely dissipated in overcoming friction before the downstream toe of the dam is reached. The proposed dam design including cutoff trenches accounts for these circumstances.

Another type of piping failure is due to internal erosion that starts in springs near the downstream toe and proceeds upstream along bases of the dams, the walls of conducts, and bedding planes in the foundations. This type of failure is also termed as “failure by subsurface erosion”. Experience has shown that many failures due to piping are of the subsurface erosion type as a result of seepage following minor geological weakness. For reasons of applicability to the determination of the safety against blowout piping only, which is virtually independent of the grain size and gradation of the foundation, and because of the lack of detailed foundation exploration, the construction of flow nets is not required for the design of foundations for small dams.

However, to guarantee a controlled seepage collection it is best to place a seepage trench at the toe of the dams.

7.6. Structural Details of the Dams

7.6.1. Cutoff Trench

The Darcy formula for seepage indicates that the amount of seepage is directly propotional to the cross-sectional area of the foundation. The action of a partial cutoff trench is similar to that of an obstruction in a pipe - the flow is reduced because of the loss of the head due to the obstruction, but the reduction in flow is not directly proportional to the reduction in the area of the pipe. Experiments by Craeger (1945) on homogeneous isotropic pervious foundations have demonstrated that a cutoff extending 50 percent of the distance to the impervious stratum will reduce the seepage by only 25 percent; an 80 percent cutoff penetration is required to reduce the seepage by 50 percent.

A partial cutoff trench is effective in stratified foundations by intercepting the more pervious layers in the foundation and by substancially increasing the vertical path the seepage must take. The partial cutoff trenches of 4-ft depth and width for the detention facilities increase the seepage path and finally intercept a clayey, impervious layer. Considering that there are heads of approximately 6 and 7 ft only, the partial cutoff trenches are assumed to be effective.

7.6.2. Toe Drain Trench

Toe drain trenches are commonly installed along the downstream toes of dams. The purpose of these drains is to collect the seepage emerging and discharging from lower soil strata of the foundation. The seepage is led to an outfall pipe which discharges into the downstream channel, below the dam. Toe drains are also used on impervious foundations to insure that any seepage that may come through the foundation or the embankment is collected and that the goundwater is kept below the surface sufficiently to avoid the creation of unsightly boggy areas below the dam. The plastic drainpipes are placed in trenches at 4-ft depth insuring effective interseption of the seepage. The bottom width of the trench is 2 ft and a minimum pipe size, recommended by the Bureau of Reclamation (1965), of 6 inches with 1/2-inch openings is used (Detail in Drawings No. 1 and 2). The drainpipe will be surrounded by a filter to prevent clogging of the drains by inwash of fine material or piping of foundation material into the drainage system.

There are some rational approaches to the design of filters, most generally credited to Terzaghi (1996) and as a result of experimentations to the Corps of Engineers and the Bureau of Reclamations. The latter authorities recommend the following limits to satisfy filter stability criteria and to provide ample increase in permeability between base and filter. The criteria are:

- $\frac{D_{15,\,filter}}{D_{15,base\ \ material}} = 5\ to\ 40$,

  provided that the filter does not contain more than 5 percent of material finer than 0.074 mm (No. 200 sieve).

- $\frac{D_{15,\,filter}}{D_{85,base\ material}} \leq 5$

- $\frac{D_{85,\,filter}}{D_{\max,drainpipe}} \geq 2$

- The grain-size curve of the filter should be roughly parallel to that of the base material.

Since there are no grain-size curves of the foundation material available, it is conservative to provide a two-layer filter consisting of sand (SW) and gravel (GW). The D_{85} of the gravel should be no smaller than 1 inch and the grain-size curves have to meet the filter criteria mentioned above. The soil types SW and GW will meet the requirements assuming that a ML type soil being adjacently, as to be seen at the Table C-1.

7.7. Constructional Considerations of Outlet Structures

7.7.1. Anti-Seep Collars

All earth dams with a central core have some seepage through them. If seepage occurs without dislodging and removing soil particles, no damage will result. However, if soil paticles are washed away in the seepage, severe problems may develop. When piping occurs the flow passages and leakage increases until a possible failure mode is reached. Piping can not just develop in earth dams where the filter between fine soil and coarse material in the embankment is not properly designed, it can also occur where the fill material joins the abutment material or where it joins a solid structure such as an outlet conduit. If the fill material is not carefully placed and hand tamped next to the surfaces of discontinuity, the less consolidated material may be the starting point for piping. One way of reducing the possibility of piping around buried conduits is to construct collars around them.

The collars are thought to block any flow passages along the exterior of the conduit, and the distance the seepage water must travel is increased by the collar. In contrast to the theoretical considerations, proper compaction is difficult to achieve around these collars causing partial ineffectiveness in practice. Particular care must be taken in compacting the material adjecent to the collar.

The perforated riser pipe through the detention, wet pond, facility dam requires a certain number of seepage collars. SCS (1986) recommends anti-seep collars based on the saturated length and the pipe diameter. Referring to the figure of Appendix C9, the anti-seep collar size required was determined to be 5 ft by 5 ft for 2 collars and a saturated length of 50ft.

7.7.2. Constructional Considerations of the Proportional Weir

The proportional weir outlet structure will be constructed of reinforced concrete, with each side supported by a counterfort extending perpendicular to the weir in the upstream direction. The weir edge is to be constructed by bending two steel channel sections to the dimensions outlining the weir using equation (5.10). Channels are also used as the bottom form for the cast-in-place concrete. A sharpened flat piece of steel is then welded to the upstream channel edge to give the final weir edge. The reinforced concrete walls will be 1 ft thick.

An energy dissipater consisting of 4-12 inch riprap and depressed 1 ft should be placed immediately downstream. The weir discharges into the natural, open channel, allowing the weir to function unsupressed.

8. COST ESTIMATION

Cost estimations were done by applying cost data (Waiver, 1993). The book aims primarily at commercial and industrial projects costing up to $500,000 and up, or multi-family housing projects (Waiver, 1993). Costs are primarily for new construction or major renovation of buildings rather than repairs or minor alternations. Although Waiver (1993) does not specifically address dam construction sites, all activities required for a stormwater management construction are provided.

The cost estimation includes overhead and profit. It uses the Masterformat system of classification and numbering as developed by the Construction Specifications Institute (CSI) and Constructions Canada (CSC).

While overall construction costs may vary relative to general economic conditions, price fluctuations within the industry are dependent upon many factors. The regional cost component was acknowledged and considered by applying a city code index of 0.873 given in Waiver (1993) for the closest location (Roanoke, Virginia). Costs in Waiver's book were based on 1993 data so an inflation factor of 0.962 was introduced, which was obtained from a later edition.

Anderson & Associates (1996) based their cost estimation on their own prices and services. Therefore, the cost estimations may differ significantly. A direct comparison with their cost analysis was difficult, because they did not breakdown their costs in detail (Table D-1). However, overall costs can be compared. The overall construction costs of Anderson & Associated were $319,000 compared to $271,500 by this analysis(Table D-2). The major costs were engendered by grading and excavating processes. They accounted for approximately 75% of the total costs.

As is typical for an estimation, the calculated total costs represent a rough approximation only. Many things cannot be foreseen. There may be additional costs if an unexpectedly large amount of rock must be excavated, long periods of rainfall occur, numerous fissures that have to be sealed or covered with a liner are discovered, or growth of invasive and undesired plants occurs in the wet pond. The overall costs can be considerably lowered if a dump place or export location for excavated material can be found in the immediate vicinity of the construction site. Hauling costs for the graded material represent approximately 50% of the grading costs. Hence, money can be saved by allocating a suitable and nearby site to deposit excess material.

It should be noted that neither Anderson & Associates estimates nor those presented herein estimations include maintenance costs, which will occur, because the wet pond facility requires some maintenance.

9. SUMMARY AND CONCLUSIONS

Based on the requirement that the maximum wetland disturbance is to be kept below one acre, it was decided that the stormwater management facility will include two ponds in series - a lower dry pond and an upper wet pond.

The stormwater management facility for a 446-acre watershed on the Virginia Tech campus was designed to provide water quantity control and benefit water quality. The design meets all storm water management regulations. Peak discharges for 2-year and 10-year events are limited to existing discharge rates. The requirement for the detention of the first 0.5 inch of stormwater runoff from developed sites for water quality purposes is also met.

Hydrologic modelling was used to confirm hydrograph computations made by an engineering consulting firm Hayes, Seas, Mattern & Mattern. The HEC-1 program was used to model rainfall events of 24-hour duration and 2-year, 10-year, and 100-year return periods. Simulations showed that the HEC-1 software did not adequately simulate conduit flow and open channel flow at the same time. Differences in travel time calculations between the HEC-1 calculations and those obtained by HSMM were accounted for in this study. A routing procedure different from that used by HSMM, produced a time of concentration approximating the value obtained by HSMM given the subarea size and curve numbers used by HSMM in their computations. Also investigated was the response of the HEC-1 model with both Muskingum and Kinematic Wave routing and two different methods for calculating the time of concentration. However, regardless of the routing and travel time computational procedures, no match with the resultant hydrograph of HSMM for the 10-year and 100-year storms was obtained. The difference in peak flow and times to peak was the result of the different hydrologic models used. The XP-SWMM program used by HSMM is capable of handling pipe flow and it has the ability to account for backwater effects in the stormwater sewer system. The XP-SWMM model is thus able to more realistically simulate the system than the HEC-1 model, which does not consider pipe flow. In contrast to the differences in peak discharge and time-to-peak calculations, the HEC-1 and the XP-SWMM programs produced approximately the same runoff volume from a 355-acre portion of the drainage area.

The design hydrograph, a 100-year storm event based on future land use conditions from the runoff calculations of HSMM was used in the design of the detention ponds. The design storm had a peak discharge of 652.6 cfs to the wet pond facility.

Four different outlet structures were considered for the wet pond and two for the dry pond. A constraint on the design of the outlet structure was that a discharge form a point near the water surface should be avoided since floating trash could either damage or clog the discharge structure. For these reasons, the drop inlet and the skimmer outlets were not selected. A perforated riser was selected for the wet pond outlet, because it has better pollutant removal capabilities than an inclining projected pipe. The emergency spillway was designed as a broad-crested weir. It will extends over the entire length of the dam, because freeboard for the road embankment adjacent to the School of Veterinary Medicine needs to be provided.

It is recommended that the baseflow should be redetermined on a basis of measurements over a period representing statistical reliability. The baseflow calculated was based on a single measurement of the flow depth only. It is very unlikely that precisely the discharge at that day corresponded to the baseflow. The designer emphasizes that the perforated riser was sized on the basis of the ‘pseudo’ baseflow.

A proportional weir was found to be the most suitable outlet structure for the dry pond facility. This outlet consists of a single structure, in contrast to the other structural solution, which involves a combination of a culvert and an emergency spillway. The proportional weir reduced the peak discharge by approximately 130 cfs and the peak was delayed by 80 minutes.

With respect to pollutant removal requirements and local characteristics of the site, an extended detention wetland based on research done by Schueler (1992) is proposed. Referring to Schueler’s findings, stormwater wetlands in the Mid-Atlantic area may reliably achieve minimum long-term removal rates 75% of TSS, 45% TP, and 25% TN if designed according to recommended criteria. Schueler’s criteria were met with respect to required minimum extended treatment volume, effective flow path length, and dry weather water balance; however, not on wetland to watershed area ratio. As a result of the very small, 0.22% wetland to watershed area ratio, high rate of the pollutant removal will not be achieved by the facility proposed. Since little research has been done on wetland to watershed area ratio much smaller than 1%, the design would offer excellent opportunities to gain valuable data on performance and pollutant removal rates of small-sized wetlands.

Geotechnical analysis undertaken by Froehling & Robertson revealed that the stormwater management facility site is underlain by alluvium, residual soils, and rock. The geological report of Froehling & Robertson indicates that there are chances that the rock and soil formations at the bottom of the dams may contain cracks and other high permeable strata. To determine the possibility of seepage under the dam, a cutoff trench is proposed at the center of both dams to penetrate the stratified foundation and increase the path of seepage. A drainage trench at the toe of the downstream side of the dams is also recommended to collect seepage and to discharge it to the receiving stream. Anti-seep collars are recommended for the riser pipe to prevent seepage along the pipe.

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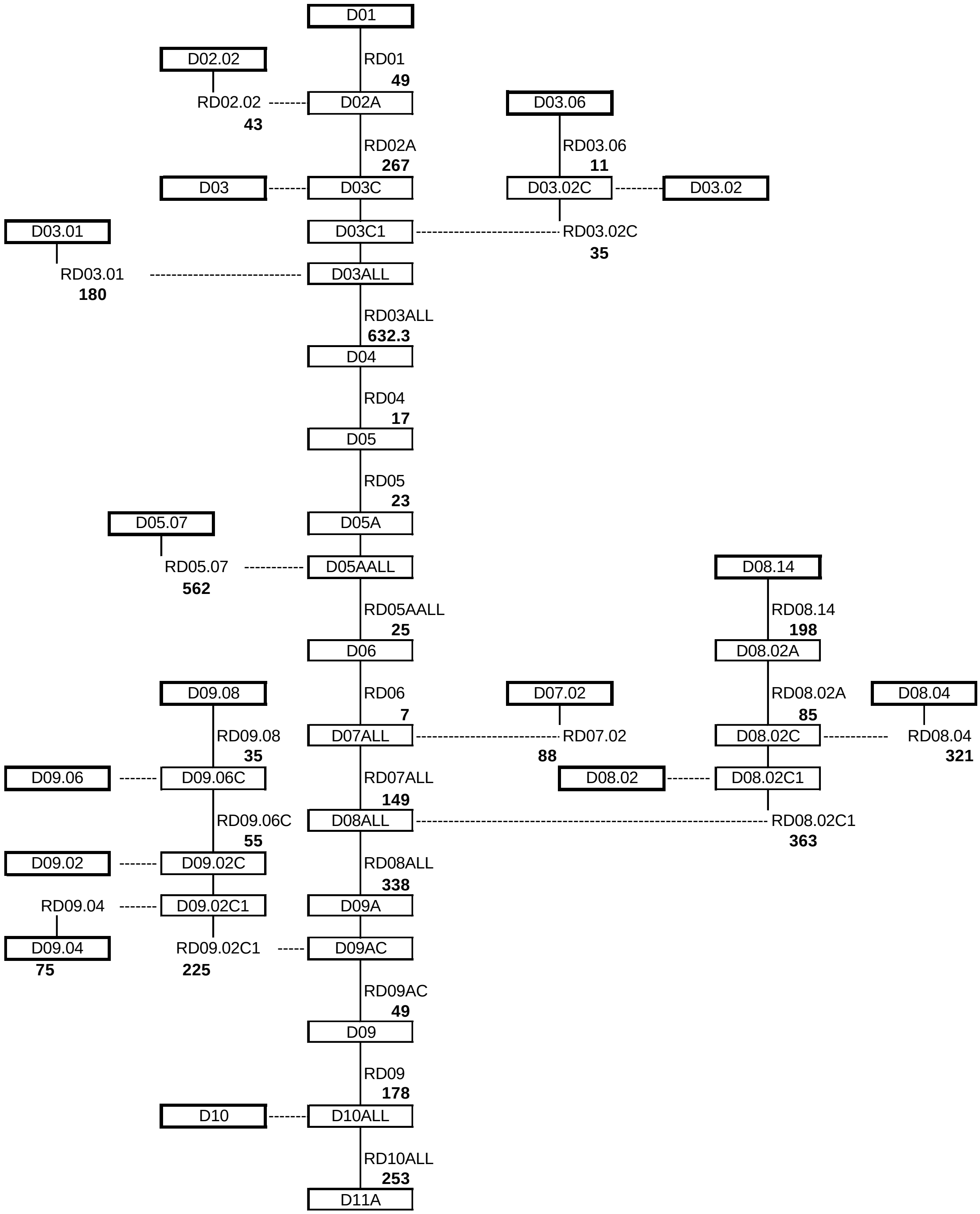
APPENDICES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D03.06
RD02A
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RD03.06
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D03C
D03.02C
D03.02
D03.01
D03C1
RD03.02C
35
RD03.01
180
D03ALL
RD03ALL
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D04
RD04
17
D05
RD05
23
D05.07
D05A
RD05.07
562
D05AALL
D08.14
RD05AALL
25
RD08.14
198
D06
D08.02A
D09.08
RD06
7
D07.02
RD08.02A
85
D08.04
RD09.08
35
D07ALL
RD07.02
88
D08.02C
RD08.04
321
D09.06
D09.06C
RD07ALL
149
D08.02
D08.02C1
RD09.06C
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D09.04
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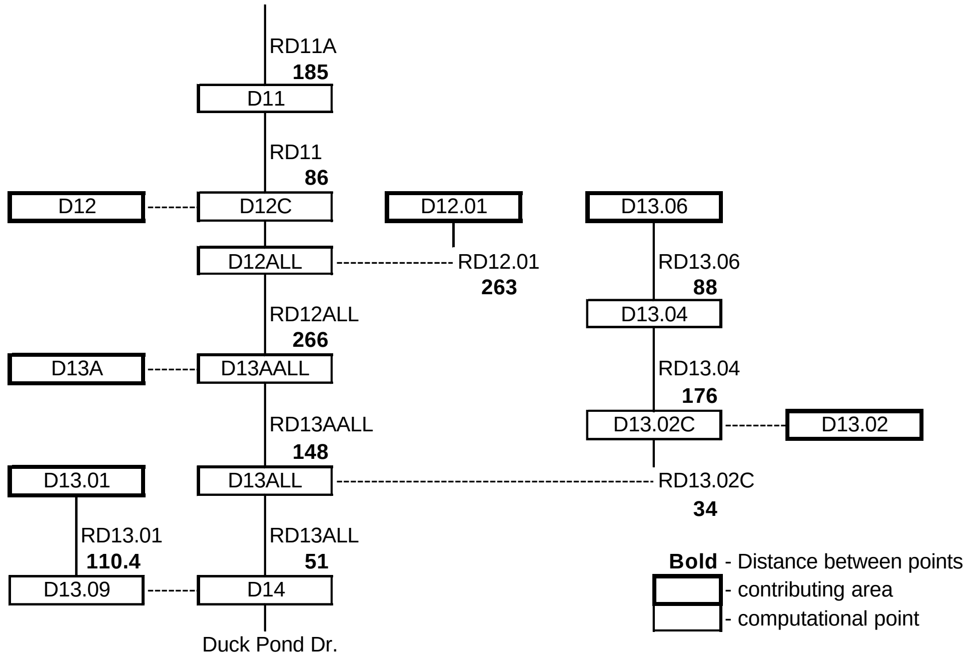
D10
D10ALL
RD10ALL
253
D11A



(No further areal information -> comparison with computation done by Hayes, Seay, Mattern & Mattern)

Figure A-1: Hec-1 Flowchart Used to Model the Prpoposed Stormwater Management Facility [Muskingum Routing Method]

Table A-1: Areal and CN Data Obtained from Hayes, Seay, Mattern & Mattern

Name SB	Area		Curve Number		Area weighted CN	
	[acres]	[miles]	Present	Future	Present	Future
D01	108.95	0.170	80.87	81.34	13.77	13.85
D02.02	1.79	0.003	89.50	89.50	0.25	0.25
D03	0.85	0.001	93.43	94.34	0.12	0.13
D03.01	5.44	0.009	85.56	85.56	0.73	0.73
D03.02	0.48	0.001	93.49	93.50	0.07	0.07
D03.06	3.29	0.005	93.18	93.18	0.48	0.48
D05.07	86.31	0.135	84.41	85.33	11.39	11.51
D07.02	7.38	0.012	89.03	92.71	1.03	1.07
D08.14	4.82	0.008	84.78	84.78	0.64	0.64
D08.02	63.42	0.099	72.05	72.31	7.14	7.17
D08.04	4.48	0.007	76.27	77.37	0.53	0.54
D09.02	11.95	0.019	86.58	93.64	1.62	1.75
D09.04	1.45	0.002	74.12	74.11	0.17	0.17
D09.06	1.43	0.002	89.33	90.10	0.20	0.20
D09.08	6.85	0.011	80.05	82.26	0.86	0.88
D10	6.57	0.010	83.65	91.65	0.86	0.94
D12	1.03	0.002	80.00	80.54	0.13	0.13
D12.01	8.08	0.013	82.05	84.20	1.04	1.06
D13.01	6.32	0.010	79.13	79.13	0.78	0.78
D13.02	1.00	0.002	68.52	68.52	0.11	0.11
D13.06	18.52	0.029	73.16	73.89	2.12	2.14
D13A	4.94	0.008	78.10	79.94	0.60	0.62
	355.35	0.555			80.35	81.40

Table A-2: Peak Discharge Estimation for the 100-year Design Flood

Equivalent Diameter: 24 ft^2 = 4 ft * 6ft = PI/4*D^2
-> D = SQRT(24*4/PI)
Capacity: - Circular (eqivalent diameter)
= 1.49/n*R^(2/3)*S^(1/2)

<u>Estimation of areal discharge:</u>

- Slope estimation:

10%	564.4	ft	Average Slope	2.76
85%	4797.5	ft		

- Drainage area:

355.35 acres = 1438055 m^2

- CN:

CN = 80	-> DR	= 4.69"	
R = 7"	Figure A-2 (SCS, 1986)		CN = 80.35
CN = 85	-> DR	= 5.25"	-> DR = 4.73"
R = 7"	Figure A-2 (SCS, 1986)		-> DR = 120.14 mm

- P-Q:

(Fig. 13-6)	-> S = 1%	-> 0.1	m^3/s per mm DR
(Fig. 13-7)	-> S = 4%	-> 0.17	m^3/s per mm DR
S = 2.76%		-> PQ = 0.141	m^3/s per mm DR

P - Q = 120.14 * 0.141
P - Q = 16.94 m^3/s
P - Q = 598.23 cfs

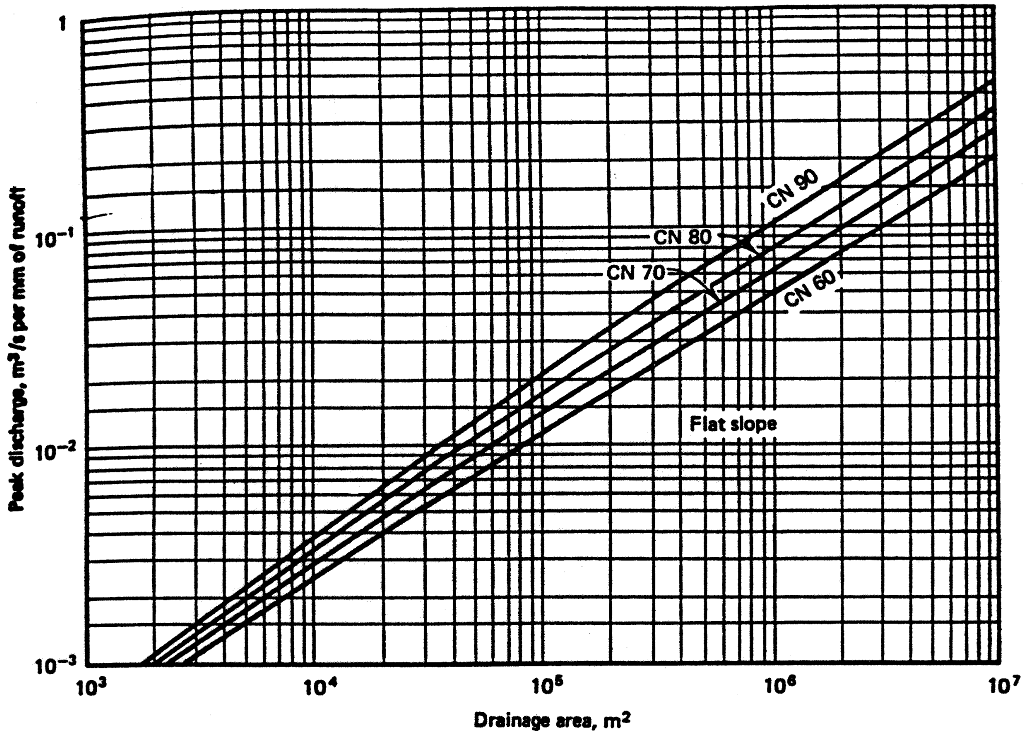


FIGURE 13-6
Peak rate of discharge for small watersheds—flat slope. 1%

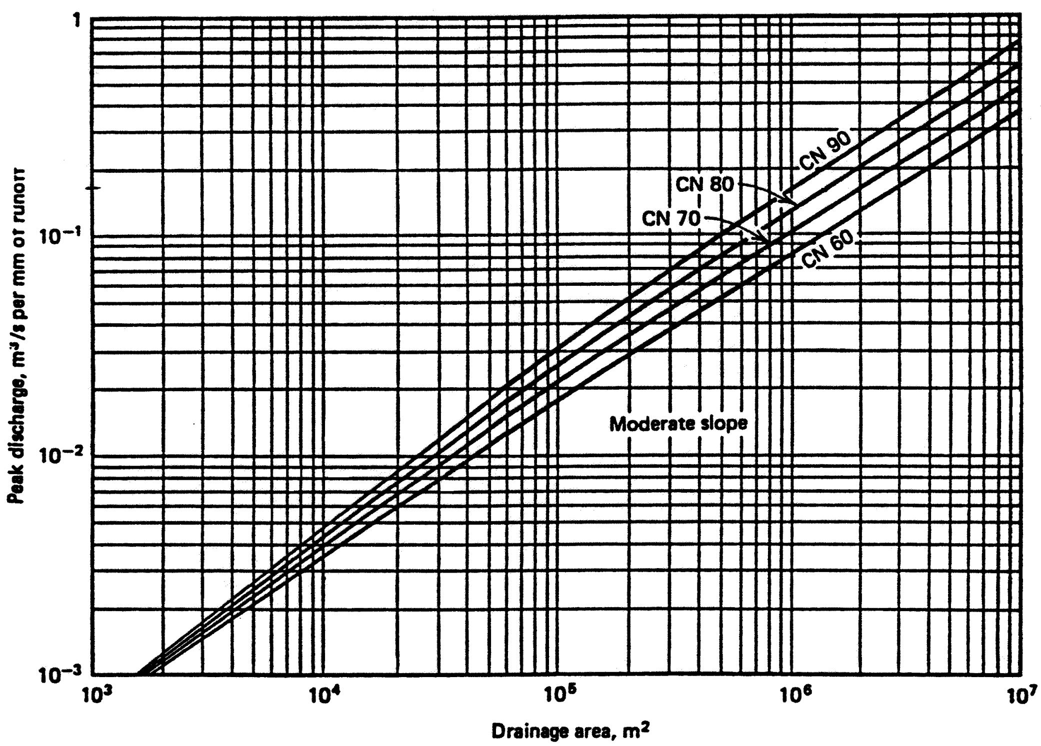


FIGURE 13-7
Peak rate of discharge for small watersheds—moderate slope. 4%

Figure A-2: Peak Rate Discharge Curves for 1% and 4% Slopes (SCS, 1986)

Table A-3: Stream Network and Connectivity Used for the HEC-1 Model

										Single Distances	Acc. Dist.		Slope [%]
										300	300.0	10%	3.31
Routing	Route		Length	Slope	n	Pipe Dia.	Shape	Equiv. Dia	Capacity	1270	1570.0		
ID	From	To	[ft]	[%]		[ft]		[ft]	[cfs]	1350	2920.0		
RD01	D01	D02A	49.0	4.0	0.013	4	CIRC	4	288.06	49.0	2969.0		
RD02.02	D02.02	D02A	43.3	2.5	0.013	1.5	CIRC	1.5	16.65		2969.0		
RD02A	D02A	D03C	267.7	0.7	0.013	4	CIRC	4	120.50	267.7	3236.7		
RD03.06	D03.06	D03.02C	10.7	2.1	0.013	2	CIRC	2	32.87		3236.7		
RD03.02C	D03.02C	D03C1	34.9	2.5	0.013	2	CIRC	2	35.87		3236.7		
RD03.01	D03.01	D03ALL	179.7	1.5	0.013	1.5	CIRC	1.5	12.90		3236.7		
RD03ALL	D03ALL	D04	632.3	1.2	0.013	4	CIRC	4	157.78	632.3	3869.0		
RD04	D04	D05	16.8	1.3	0.045	6	TRAP	5.53	112.41	16.8	3885.8		
RD05	D05	D05A	22.5	1.3	0.045	6	TRAP	5.53	112.41	22.5	3908.3		
RD05.07	D05.07	D05AALL	561.7	0.3	0.013	2	CIRC	2	12.42		3908.3		
RD05AALL	D05AALL	D06	25.3	1.1	0.045	6	TRAP	5.53	103.41	25.3	3933.6		
RD06	D06	D07ALL	7.2	3.4	0.045	6	TRAP	5.53	181.80	7.2	3940.8		
RD07.02	D07.02	D07ALL	87.6	0.1	0.013	2	CIRC	2	7.17		3940.8		
RD07ALL	D07ALL	D08ALL	148.9	0.3	0.045	6	TRAP	5.53	54.00	148.9	4089.7		
RD08.14	D08.14	D08.02A	197.8	3.3	0.013	1.5	CIRC	1.5	19.13		4089.7		
RD08.02A	D08.02A	D08.02C	85.2	3.1	0.013	1.5	CIRC	1.5	18.54		4089.7		
RD08.04	D08.04	D08.02C	321.5	2.8	0.013	1.5	CIRC	1.5	17.62		4089.7		
RD08.02C1	D08.02C1	D08ALL	363.4	0.1	0.013	2	CIRC	2	7.17		4089.7		
RD08ALL	D08ALL	D09A	337.8	0.8	0.045	6	TRAP	5.53	88.19	337.8	4427.5		
RD09.08	D09.08	D09.06C	34.6	1.4	0.013	1	CIRC	1	4.23		4427.5		
RD09.06C	D09.06C	D09.02C	54.8	0.2	0.013	1	CIRC	1	1.60		4427.5		
RD09.04	D09.04	D09.02C1	75.1	3.1	0.013	1	CIRC	1	6.29		4427.5		
RD09.02C1	D09.02C1	D09AC	224.7	0.2	0.013	1.25	CIRC	1.25	2.90		4427.5		
RD09AC	D09AC	D09	48.5	0.6	0.045	6	TRAP	5.53	76.37	48.5	4476.0		
RD09	D09	D10ALL	178.2	2.2	0.045	6	TRAP	5.53	146.24	178.2	4654.1	85%	2.2
RD10ALL	D10ALL	D11A	253.4	1.1	0.045	6	TRAP	5.53	103.41	253.4	4907.5		
RD11A	D11A	D11	185.0	1.0	0.045	6	TRAP	5.53	98.59	185.0	5092.4		
RD11	D11	D12C	86.0	1.3	0.045	6	TRAP	5.53	112.41	86.0	5178.4		
RD12.01	D12.01	D12ALL	262.6	1.1	0.013	1.25	CIRC	1.25	6.79		5178.4		
RD12ALL	D12ALL	D13AALL	266.4	1.2	0.045	6	TRAP	5.53	108.00	266.4	5444.8		
RD13AALL	D13AALL	D13ALL	148.0	0.5	0.045	6	TRAP	5.53	69.72	148.0	5592.8		
RD13.06	D13.06	D13.04	87.7	1.2	0.013	1.5	CIRC	1.5	11.54		5592.8		
RD13.04	D13.04	D13.02C	176.3	1.2	0.018	1.25	CIRC	1.25	5.12		5592.8		
RD13.02C	D13.02C	D13ALL	33.5	7.0	0.013	1.5	CIRC	1.5	27.87		5592.8		
RD13ALL	D13ALL	D14	51.3	3.3	0.045	6	TRAP	5.53	179.11	51.3	5644.1		
RD13.01	D13.01	D14	110.4	2.1	0.024	3	CIRC	3	52.50				
								Sum of Length [ft]:		5644.1			

Table A-4: Muskingum Input Data for Routing Procedure

Routing ID	Route From	Route To	Length [ft]	Slope [%]	n	Pipe Dia. Equi. Dia [ft]	Shape	Velocity [ft/sec]	K (AMSKK) [sec]	K (AMSKK) [hrs]	K (AMSKK) [min]	NSTPS
RD01	D01	D02A	49.0	4.0	0.013	4.0	CIRC	22.92	2	0.0006	0.04	421
RD02.02	D02.02	D02A	43.3	2.5	0.013	1.5	CIRC	9.42	5	0.0013	0.08	196
RD02A	D02A	D03C	267.7	0.7	0.013	4.0	CIRC	9.59	28	0.0078	0.47	32
RD03.06	D03.06	D03.02C	10.7	2.1	0.013	2.0	CIRC	10.46	1	0.0003	0.02	880
RD03.02C	D03.02C	D03C1	34.9	2.5	0.013	2.0	CIRC	11.42	3	0.0008	0.05	294
RD03.01	D03.01	D03ALL	179.7	1.5	0.013	1.5	CIRC	7.30	25	0.0068	0.41	37
RD03ALL	D03ALL	D04	632.3	1.2	0.013	4.0	CIRC	12.56	50	0.0140	0.84	18
RD04	D04	D05	16.8	1.3	0.045	5.5	TRAP*	4.68	4	0.0010	0.06	252
RD05	D05	D05A	22.5	1.3	0.045	5.5	TRAP*	4.68	5	0.0013	0.08	187
RD05.07	D05.07	D05AALL	561.7	0.3	0.013	2.0	CIRC	3.95	142	0.0395	2.37	6
RD05AALL	D05AALL	D06	25.3	1.1	0.045	5.5	TRAP*	4.31	6	0.0016	0.10	153
RD06	D06	D07ALL	7.2	3.4	0.045	5.5	TRAP*	7.57	1	0.0003	0.02	951
RD07.02	D07.02	D07ALL	87.6	0.1	0.013	2.0	CIRC	2.28	38	0.0107	0.64	23
RD07ALL	D07ALL	D08ALL	148.9	0.3	0.045	5.5	TRAP*	2.25	66	0.0184	1.10	14
RD08.14	D08.14	D08.02A	197.8	3.3	0.013	1.5	CIRC	10.83	18	0.0051	0.30	49
RD08.02A	D08.02A	D08.02C	85.2	3.1	0.013	1.5	CIRC	10.49	8	0.0023	0.14	111
RD08.04	D08.04	D08.02C	321.5	2.8	0.013	1.5	CIRC	9.97	32	0.0090	0.54	28
RD08.02C1	D08.02C1	D08ALL	363.4	0.1	0.013	2.0	CIRC	2.28	159	0.0442	2.65	6
RD08ALL	D08ALL	D09A	337.8	0.8	0.045	5.5	TRAP*	3.67	92	0.0255	1.53	10
RD09.08	D09.08	D09.06C	34.6	1.4	0.013	1.0	CIRC	5.38	6	0.0018	0.11	140
RD09.06C	D09.06C	D09.02C	54.8	0.2	0.013	1.0	CIRC	2.03	27	0.0075	0.45	33
RD09.04	D09.04	D09.02C1	75.1	3.1	0.013	1.0	CIRC	8.01	9	0.0026	0.16	96
RD09.02C1	D09.02C1	D09AC	224.7	0.2	0.013	1.3	CIRC	2.36	95	0.0264	1.59	9
RD09AC	D09AC	D09	48.5	0.6	0.045	5.5	TRAP*	3.18	15	0.0042	0.25	59
RD09	D09	D10ALL	178.2	2.2	0.045	5.5	TRAP*	6.09	29	0.0081	0.49	31
RD10ALL	D10ALL	D11A	253.4	1.1	0.045	5.5	TRAP*	4.31	59	0.0163	0.98	15
RD11A	D11A	D11	185.0	1.0	0.045	5.5	TRAP*	4.11	45	0.0125	0.75	20
RD11	D11	D12C	86.0	1.3	0.045	5.5	TRAP*	4.68	18	0.0051	0.31	49
RD12.01	D12.01	D12ALL	262.6	1.1	0.013	1.3	CIRC	5.54	47	0.0132	0.79	19
RD12ALL	D12ALL	D13AALL	266.4	1.2	0.045	5.5	TRAP*	4.50	59	0.0164	0.99	15
RD13AALL	D13AALL	D13ALL	148.0	0.5	0.045	5.5	TRAP*	2.90	51	0.0142	0.85	18
RD13.06	D13.06	D13.04	87.7	1.2	0.013	1.5	CIRC	6.53	13	0.0037	0.22	67
RD13.04	D13.04	D13.02C	176.3	1.2	0.018	1.3	CIRC	4.18	42	0.0117	0.70	21
RD13.02C	D13.02C	D13ALL	33.5	7.0	0.013	1.5	CIRC	15.77	2	0.0006	0.04	423
RD13ALL	D13ALL	D14	51.3	3.3	0.045	5.5	TRAP*	7.46	7	0.0019	0.11	131
RD13.01	D13.01	D14	110.4	2.1	0.024	3.0	CIRC	7.43	15	0.0041	0.25	61

* Conrete Box of 4' x 6'

Area Box (4x6) = 24 sq. ft
A = Pi/4*D^2
-> D = SQRT(4*A/Pi)
= SQRT(4*24/Pi)
D (ft) = 5.53 (equivalent radius)

v = Bankful velocity
v = 1.49/n*(D/4)^(2/3)*S^0.5
v = v(wave), since there are closed conduits only
K = L/v

Computational Stability Criteria:

1/(2*(1-X)) < K[min]/(NMIN*NSTPS) < 1/(2*X)
1/(2*(1-0.5)) < K[min]/(15*NSTPS) < 1/(2*0.5)
1 < K[min]/(15*NSTPS) < 1
NSTPS = 15/K[min]

X = 0.5
NMIN = 15 (No. of minutes in computational interval)
NSTPS = Integer steps for the Muskingum routing Method

Table A-5: HSMM and SCS Travel Time Computations [Muskingum Method]

Name SB	Type	Character	Hayes, Seay, Matter & Mattern						Type	SCS Method					
			Length [ft]	Slope [%]	Velocity [ft/sec]	Time [min]	Time [hrs]	Tlag = 0.6*Tc		Length [ft]	Slope [%]	Velocity [ft/sec]	Time [min]	Time [hrs]	T lag = 0.6 * Tc
D01	Overland *	Forest	300	3.31	0.46	10.86	0.18		Overland ***	200	3.31		20.98	0.35	
	Swale *	Grassy	1270	3.31	2.73	7.75	0.13		Swale ****	1370	3.31	2.95	7.74	0.13	
	Channel **	Channel	1350	3.65	9	2.50	0.04		Channel *****	1350	3.65	9	2.50	0.04	
						21.11	0.35	0.21					31.22	0.52	0.312
D02.02	Overland	Forest	300	8.67	0.74	6.76	0.11		Overland	200	8.67		14.27	0.24	
	Swale	Grassy	100	8.67	4.41	0.38	0.01		Swale	200	8.67	4.8	0.69	0.01	
	Channel	Channel	200	2	6.7	0.50	0.01		Channel	200	2	6.7	0.50	0.01	
						7.63	0.13	0.08					15.46	0.26	0.155
D03	Swale	Grassy	330	2.95	2.58	2.13	0.04		Swale	330	2.95	2.7	2.04	0.03	
						2.13	0.04	0.02					2.04	0.03	0.020
D03.01	Swale	Grassy	700	4.99	3.35	3.48	0.06		Swale	700	4.99	3.6	3.24	0.05	
						3.48	0.06	0.03					3.24	0.05	0.032
D03.02	Swale	Paved	600	2.29	3.03	3.30	0.06		Swale	600	2.29	3.05	3.28	0.05	
						3.30	0.06	0.03					3.28	0.05	0.033
D03.06	Overland	Forest	300	8.22	0.72	6.94	0.12		Overland	200	8.22		14.58	0.24	
	Swale	Grassy	130	8.22	4.3	0.50	0.01		Swale	230	8.22	4.6	0.83	0.01	
						7.45	0.12	0.07					15.41	0.26	0.154
D05.07	Overland	Forest	300	4.19	0.52	9.62	0.16		Overland	200	4.19		19.09	0.32	
	Swale	Grassy	1780	4.19	3.07	9.66	0.16		Swale	1880	4.19	3.25	9.64	0.16	
	Channel	Channel	1630	1.08	4.8	5.66	0.09		Channel	1630	1.08	4.8	5.66	0.09	
						24.94	0.42	0.25					34.39	0.57	0.344
D07.02	Overland	Forest	300	7.04	0.67	7.46	0.12		Overland	200	7.04		15.51	0.26	
	Swale	Grassy	670	7.04	3.98	2.81	0.05		Swale	770	7.04	4.3	2.98	0.05	
						10.27	0.17	0.10					18.50	0.31	0.185
D08.14	Overland	Forest	300	5.44	0.59	8.47	0.14		Overland	200	5.44		17.20	0.29	
	Swale	Grassy	1060	5.44	3.5	5.05	0.08		Swale	1160	5.44	3.75	5.16	0.09	
						13.52	0.23	0.14					22.35	0.37	0.224
D08.02	Overland	Forest	300	3.09	0.44	11.36	0.19		Overland	200	3.09		21.56	0.36	
	Swale	Grassy	1510	3.09	2.64	9.53	0.16		Swale	1610	3.09	2.8	9.58	0.16	
	Channel	Channel	760	1.81	6.1	2.08	0.03		Channel	760	1.81	6.1	2.08	0.03	
						22.97	0.38	0.23					33.22	0.55	0.332
D08.04	Overland	Forest	300	3.3	0.46	10.87	0.18		Overland	200	3.3		21.00	0.35	
	Swale	Grassy	590	3.3	2.72	3.62	0.06		Swale	690	3.3	2.9	3.97	0.07	
						14.48	0.24	0.14					24.97	0.42	0.250
D09.02	Swale	Grassy	1100	2.92	2.56	7.16	0.12		Swale	1100	2.92	2.75	6.67	0.11	
						7.16	0.12	0.07					6.67	0.11	0.067
D09.04	Overland	Forest	300	8.23	0.72	6.94	0.12		Overland	200	8.23		14.57	0.24	
	Swale	Grassy	170	8.23	4.3	0.66	0.01		Swale	270	8.23	4.6	0.98	0.02	
						7.60	0.13	0.08					15.55	0.26	0.156
D09.06	Swale	Grassy	950	2.88	2.54	6.23	0.10		Swale	950	2.88	2.75	5.76	0.10	
						6.23	0.10	0.06					5.76	0.10	0.058

Name SB	Type	Character	Done by Hayes, Seay, Matter & Mattern					
			Length [ft]	Slope [%]	Velocity [ft/sec]	Time [min]	Time [hrs]	Tlag = 0.6*Tc
D09.08	Overland	Forest	300	3.41	0.47	10.64	0.18	
	Swale	Grassy	950	3.41	2.77	5.72	0.10	
	Swale	Paved	280	1.9	2.76	1.69	0.03	
						18.05	0.30	0.18
D10	Overland	Forest	300	1.07	0.26	19.23	0.32	
	Swale	Grassy	450	1.07	1.55	4.84	0.08	
						24.07	0.40	0.24
D12	Swale	Grassy	210	0.95	1.46	2.40	0.04	
	Channel	Channel	110	1.45	5.5	0.33	0.01	
						2.73	0.05	0.03
D12.01	Overland	Forest	300	7.7	0.7	7.14	0.12	
	Swale	Grassy	375	7.7	4.16	1.50	0.03	
	Channel	Channel	150	0.44	2.7	0.93	0.02	
						9.57	0.16	0.10
D13.01	Overland	Forest	300	14.17	0.95	5.26	0.09	
	Swale	Grassy	20	14.17	5.64	0.06	0.00	
	Channel	Channel	600	1.67	6.1	1.64	0.03	
						6.96	0.12	0.07
D13.02	Swale	Grassy	400	1.33	1.73	3.85	0.06	
						3.85	0.06	0.04
D13.06	Overland	Forest	300	1.32	0.29	17.24	0.29	
	Swale	Grassy	550	1.32	1.72	5.33	0.09	
	Swale	Grassy	190	18.25	6.4	0.49	0.01	
	Channel	Channel	570	0.94	4.8	1.98	0.03	
						25.04	0.42	0.25
D13A	Swale	Grassy	1100	1.31	1.72	10.66	0.18	
						10.66	0.18	0.11

Type	By means of SCS Method					
	Length [ft]	Slope [%]	Velocity [ft/sec]	Time [min]	Time [hrs]	T lag = 0.6 * Tc
Overland	200	3.41		20.73	0.35	
Swale	1050	3.41	3	5.83	0.10	
Swale	280	1.9	2.8	1.67	0.03	
				28.23	0.47	0.282
Overland	200	1.07		32.96	0.55	
Swale	550	1.07	1.7	5.39	0.09	
				38.35	0.64	0.383
Swale	210	0.95	1.55	2.26	0.04	
Channel	110	1.45	5.5	0.33	0.01	
				2.59	0.04	0.026
Overland	200	7.7		14.97	0.25	
Swale	475	7.7	4.5	1.76	0.03	
Channel	150	0.44	2.7	0.93	0.02	
				17.65	0.29	0.177
Overland	200	14.17		11.73	0.20	
Swale	120	14.17	6	0.33	0.01	
Channel	600	1.67	6.1	1.64	0.03	
				13.70	0.23	0.137
Swale	400	1.33	1.82	3.66	0.06	
				3.66	0.06	0.037
Overland	200	1.32		30.30	0.51	
Swale	650	1.32	1.81	5.99	0.10	
Swale	190	18.25	6.9	0.46	0.01	
Channel	570	0.94	4.8	1.98	0.03	
				38.72	0.65	0.387
Swale	1100	1.31	1.8	10.19	0.17	
				10.19	0.17	0.102

* Overland flow times were determined based on the Upland Method of the National Eng. Handbook.
longest overland flow length used: 300 ft

** Channel Flow assumptions: trapezoidal channel with 2:1 side slopes, 10 ft bottom width, a flow heigth of 4 ft, and a manning's factor of 0.06

*** Overland flow times were determined based on the SCS Segmental Method.
longest overland flow length used: 200 ft
Tc = 0.007*(n*L)^0.8/(P2^0.5*S^0.4)   where: n = 0.24
P2(24hrs) = 3.0 in

**** Average velocities for estimating travel time for shallow concentrated flow (paved/unpaved)
Chart at Fig. A-3 (SCS, 1986)

***** Channel flow assumptions: trapezoidal channel with 2:1 side slopes (m:1), 10 ft bottom width (w), a flow heigth (h) of 4 ft, and a manning's factor (n) of 0.06
v = 1.49/n*((w*h+m*h^2)/(w+2*(1+m^2)^0.5)^(2/3)*S^0.5

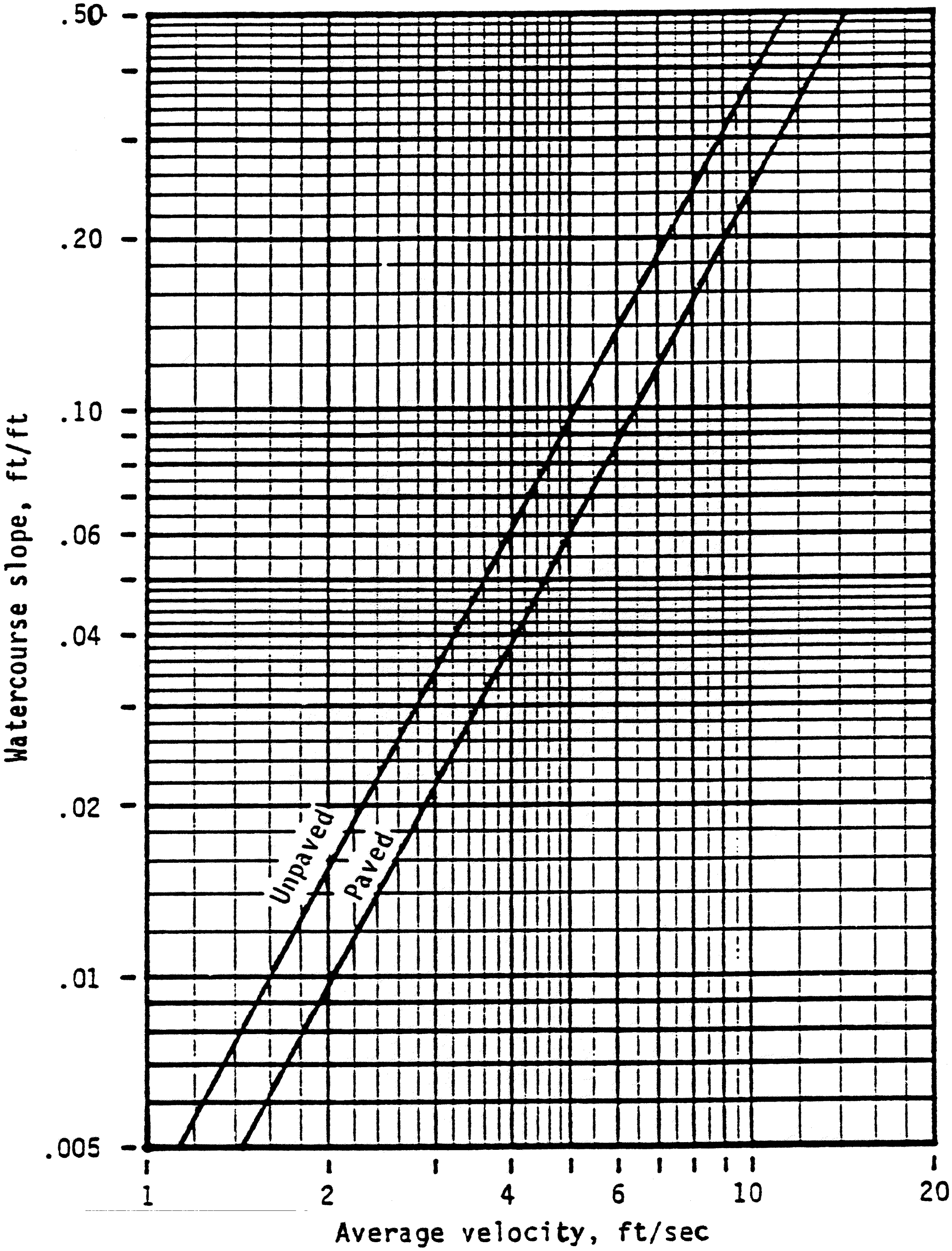
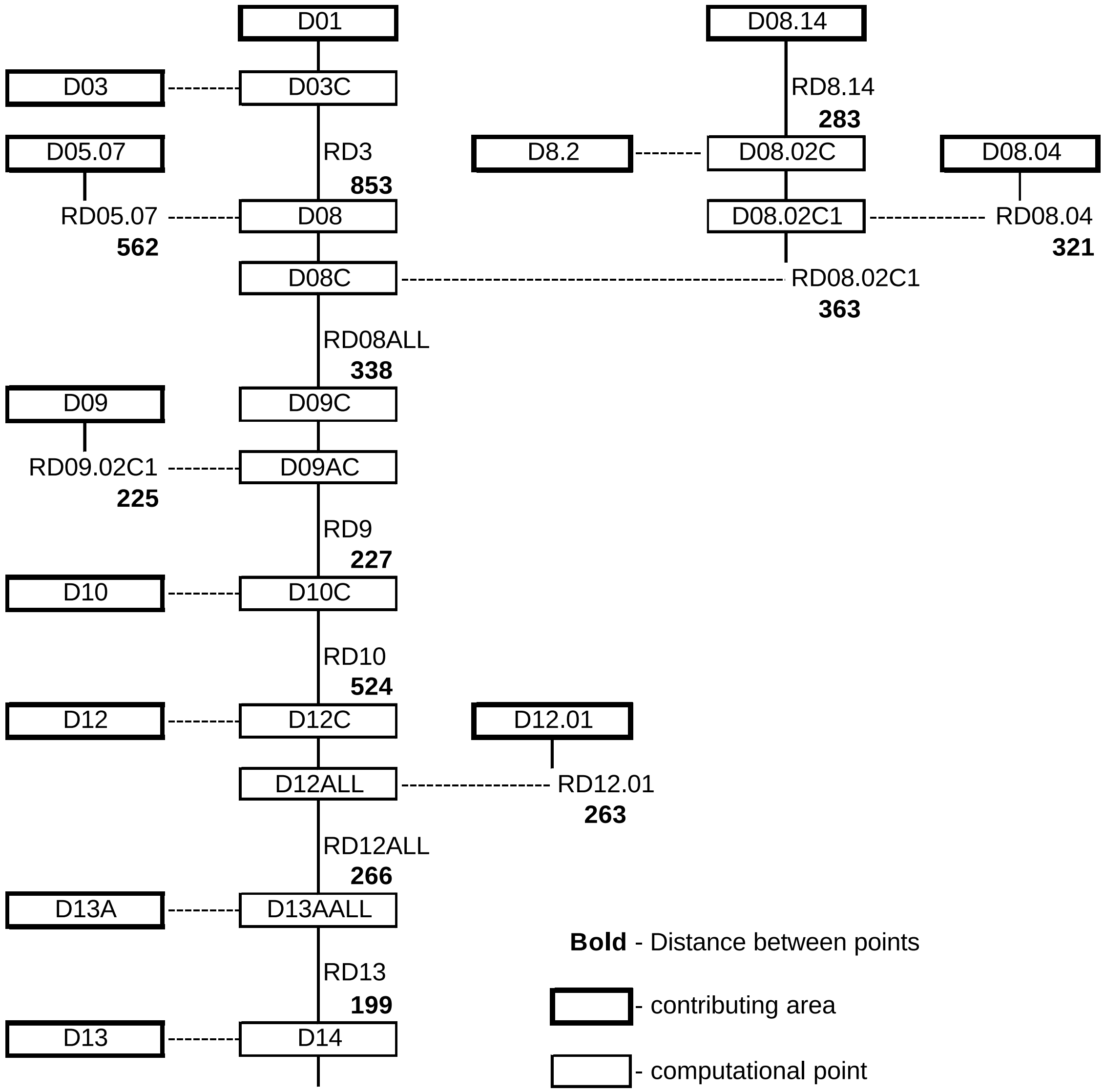


Figure A-3: Average Flow Velocities for Estimating Travel Time for Shallow Concentrated Flow (SCS, 1986)



(No further areal information -> comparison with computation done by Hayes, Seay, Mattern & Mattern)

Figure A-4: Hec-1 Flowchart Used to Model the Proposed Stormwater Management Facility [Kinematic Wave Method]

Table A-6: SCS Travel Time Computations [Kinematic Wave]

New ID	Name	Area		Curve Number		Type	Character	Length	Slope	Velocity	Time		T lag =
	Basin	[acres]	[miles]	Present	Future			[ft]	[%]	[ft/sec]	[min]	[hrs]	0.6 * Tc
	D01	109.0	0.170	80.87	81.34	Overland *	Forest	200	3.3		20.98	0.35	
	D02.02	1.8	0.003	89.50	89.50	Swale **	Grassy	1370	3.3	3.0	7.74	0.13	
		110.7	0.173	81.01	81.47	Channel ***	Channel	1350	3.7	9.0	2.50	0.04	
						Tr ****	Pipe	49	4.0	22.9	0.04	0.00	
D1											31.25	0.52	0.31
Channel Routing													
	RD02A					(4' , n = 0.013)	Pipe	268	0.7				
	D03	0.9	0.001	93.43	94.34	Overland	Forest	200	8.2		14.58	0.24	
	D03.02	0.5	0.001	93.49	93.50	Swale	Grassy	230	8.2	4.6	0.83	0.01	
	D03.06	3.3	0.005	93.18	93.18	Tr	Pipe	11	2.1	10.5	0.02	0.00	
	D03.01	5.4	0.009	85.56	85.56	Tr	Pipe	35	2.5	11.4	0.05	0.00	
D3		10.1	0.016	89.10	89.17						15.48	0.26	0.15
Channel Routing													
	RD03ALL					(4' , n = 0.013)	Pipe	632	1.2				
	RD04					(6'x4', n = 0.045)	Box	17	1.3				
	RD05					(6'x4', n = 0.045)	Box	23	1.3				
RD3						(4' , n = 0.013)	Pipe	672	1.2				
	D05.07	86.3	0.135	84.41	85.33	Overland	Forest	200	4.2		19.09	0.32	
						Swale	Grassy	1880	4.2	3.3	9.64	0.16	
						Channel	Channel	1630	1.1	4.8	5.66	0.09	
D05.07											34.39	0.57	0.34
Channel Routing													
	RD05.07					(2' , n = 0.013)	Pipe	562	0.3				
Channel Routing													
	RD05AALL					(6'x4', n = 0.045)	Box	25	1.1				
	RD06					(6'x4', n = 0.045)	Box	7	3.4				
	RD07ALL					(6'x4', n = 0.045)	Box	149	0.3				
RD5						(6'x4', n = 0.045)	Box	181	0.5				
	D08.14	4.8	0.008	84.78	84.78	Overland	Forest	200	5.4		17.20	0.29	
						Swale	Grassy	1160	5.4	3.8	5.16	0.09	
D08.14											22.35	0.37	0.22
Channel Routing													
	RD08.14					(1.5', n = 0.013)	Pipe	198	3.3				
	RD08.02A					(1.5', n = 0.013)	Pipe	85	3.1				
RD8.14						(1.5', n = 0.013)	Pipe	283	3.2				
	D08.02	63.4	0.099	72.05	72.31	Overland	Forest	200	3.1		21.56	0.36	
	D07.02	7.4	0.012	89.03	92.71	Swale	Grassy	1610	3.1	2.8	9.58	0.16	
		70.8	0.111	73.82	74.44	Channel	Channel	760	1.8	6.1	2.08	0.03	
D8.2											33.22	0.55	0.33
	D08.04	4.5	0.007	76.27	77.37	Overland	Forest	200	3.3		21.00	0.35	
						Swale	Grassy	690	3.3	2.9	3.97	0.07	
D08.04											24.97	0.42	0.25
Channel Routing													
	RD08.04					(1.5', n = 0.013)	Pipe	321	2.8				
Channel Routing													
	RD08.02C1					(2', n = 0.01	Pipe	363	0.1				
Channel Routing													
	RD08ALL					(6'x4', n = 0.045)	Box	338	0.8				
	D09.08	6.9	0.011	80.05	82.26	Overland	Forest	200	3.4		20.73	0.35	
	D09.06	1.4	0.002	89.33	90.10	Swale	Grassy	1050	3.4	3.0	5.83	0.10	
	D09.04	1.5	0.002	74.12	74.11	Swale	Paved	280	1.9	2.8	1.67	0.03	
	D09.02	12.0	0.019	86.58	93.64	Tr	Pipe	35	1.4	5.4	0.11	0.00	
		21.7	0.034	83.86	88.50	Tr	Pipe	55	0.2	2.0	0.45	0.01	
D9											28.79	0.48	0.29
Channel Routing													
	RD09.02C1					(1.25', n = 0.013)	Pipe	225	0.2				

New ID	Name	Area		Curve Number		Type	Character	Length	Slope	Velocity	Time		T lag =
	Basin	[acres]	[miles]	Present	Future			[ft]	[%]	[ft/sec]	[min]	[hrs]	0.6 * Tc
Channel Routing													
	RD09AC					(6'x4', n = 0.045)	Box	48	0.6				
	RD09					(6'x4', n = 0.045)	Box	178	2.2				
RD9						(6'x4', n = 0.045)	Box	227	1.9				
	D10	6.6	0.010	83.65	91.65	Overland	Forest	200	1.1		32.96	0.55	
						Swale	Grassy	550	1.1	1.7	5.39	0.09	
D10											38.35	0.64	0.38
Channel Routing													
	RD10ALL					(6'x4', n = 0.045)	Box	253	1.1				
	RD11A					(6'x4', n = 0.045)	Box	185	1.0				
	RD11					(6'x4', n = 0.045)	Box	86	1.3				
RD10						(6'x4', n = 0.045)	Box	524	1.1				
	D12	1.0	0.002	80.00	80.54	Swale	Grassy	210	1.0	1.6	2.26	0.04	
						Channel	Channel	110	1.5	5.5	0.33	0.01	
D12											2.59	0.04	0.03
	D12.01	8.1	0.013	82.05	84.20	Overland	Forest	200	7.7		14.97	0.25	
						Swale	Grassy	475	7.7	4.5	1.76	0.03	
						Channel	Channel	150	0.4	2.7	0.93	0.02	
D12.01											17.65	0.29	0.18
Channel Routing													
	RD12.01					(1.25', n = 0.013)	Pipe	263	1.1				
Channel Routing													
	RD12ALL					(6'x4', n = 0.045)	Box	266	1.2				
	D13A	4.9	0.008	78.10	79.94	Swale	Grassy	1100	1.3	1.8	10.19	0.17	
D13A											10.19	0.17	0.10
Channel Routing													
	RD13AALL					(6'x4', n = 0.045)	Box	148	0.5				
	RD13ALL					(6'x4', n = 0.045)	Box	51	3.3				
RD13						(6'x4', n = 0.045)	Box	199	1.2				
	D13.06	18.5	0.029	73.16	73.89	Overland	Forest	200	1.3		30.30	0.51	
	D13.01	6.3	0.010	79.13	79.13	Swale	Grassy	650	1.3	1.8	5.99	0.10	
	D13.02	1.0	0.002	68.52	68.52	Swale	Grassy	190	18.3	6.9	0.46	0.01	
		25.8	0.040	74.44	74.96	Channel	Channel	570	0.9	4.8	1.98	0.03	
						Tr	Pipe	88	1.2	6.5	0.22	0.00	
						Tr	Pipe	176	1.2	4.2	0.70	0.01	
						Tr	Pipe	34	7.0	15.8	0.04	0.00	
D13											39.69	0.66	0.40

* Overland flow times were determined based on the SCS Segmental Method.
longest overland flow length used: 200 ft
Tc = 0.007*(n*L)^0.8/(P2^0.5*S^0.4)
whereas: n = 0.24
P2(24hrs) = 3.0 in

** Average velocities for estimating travel time for shallow concentrated flow (paved/unpaved)
Chart at Fig. A-3 (SCS, 1986)

*** Channel flow assumptions: trapezoidal channel with 2:1 side slopes, 10 ft bottom width,
a flow heigth of 4 ft, and a manning's factor of 0.06
v = 1.49/n*((w*h+m*h^2)/(w+2*(1+m^2)^0.5)^(2/3)*S^0.5

**** Channel flow assumptions: according to specific channel/pipe geometry

Table A-7: Comparison of 100-year Design Strom Hydrographs Routed by Volume

DaMon	Time	Ordin.	HSMM	Kinematik Wave (SCS T_c)			Muskingum (SCS T_c)			Muskingum (HSMM T_c)		
			Q	Q	V Diff.	cum.V	Q	V Diff.	cum.V	Q	V Diff.	cum.V
	[hrs]		[cfs]	[cfs]	[1000 ft^3]		[cfs]	[1000 ft^3]		[cfs]	[1000 ft^3]	
30-Aug	0	1	0	0	0		0	0		0	0	
30-Aug	15	2	0	0	0	0	0	0	0	0	0	0
30-Aug	30	3	0	0	0	0	0	0	0	0	0	0
30-Aug	45	4	0	0	0	0	0	0	0	0	0	0
30-Aug	100	5	0	0	0	0	0	0	0	0	0	0
30-Aug	115	6	0	0	0	0	0	0	0	0	0	0
30-Aug	130	7	0	0	0	0	0	0	0	0	0	0
30-Aug	145	8	0	0	0	0	0	0	0	0	0	0
30-Aug	200	9	0	0	0	0	0	0	0	0	0	0
30-Aug	215	10	0	0	0	0	0	0	0	0	0	0
30-Aug	230	11	0	0	0	0	0	0	0	0	0	0
30-Aug	245	12	0	0	0	0	0	0	0	0	0	0
30-Aug	300	13	0	0	0	0	0	0	0	0	0	0
30-Aug	315	14	0	0	0	0	1	1	1	1	1	1
30-Aug	330	15	0	0	0	0	1	1	2	1	1	2
30-Aug	345	16	0	0	0	0	1	1	3	1	1	3
30-Aug	400	17	0	0	0	0	1	1	4	1	1	4
30-Aug	415	18	0	0	0	0	1	1	5	1	1	5
30-Aug	430	19	0	0	0	0	1	1	5	1	1	5
30-Aug	445	20	0	0	0	0	2	2	7	2	2	7
30-Aug	500	21	0	0	0	0	2	2	9	2	2	9
30-Aug	515	22	0	0	0	0	3	3	12	3	3	12
30-Aug	530	23	0	0	0	0	3	3	14	3	3	14
30-Aug	545	24	0	0	0	0	4	4	18	4	4	18
30-Aug	600	25	0	0	0	0	5	5	23	5	5	23
30-Aug	615	26	0	0	0	0	6	5	28	6	5	28
30-Aug	630	27	0	0	0	0	7	6	34	7	6	34
30-Aug	645	28	0	0	0	0	8	7	41	8	7	41
30-Aug	700	29	0	0	0	0	9	8	50	9	8	50
30-Aug	715	30	0	0	0	0	10	9	59	11	10	59
30-Aug	730	31	0	0	0	0	12	11	69	12	11	70
30-Aug	745	32	0	0	0	0	13	12	81	14	13	83
30-Aug	800	33	0	0	0	0	15	14	95	15	14	96
30-Aug	815	34	0	1	1	1	16	14	109	17	15	112
30-Aug	830	35	0	1	1	2	19	17	126	20	18	130
30-Aug	845	36	0	3	3	5	22	20	146	23	21	150
30-Aug	900	37	0	4	4	8	24	22	167	25	23	173
30-Aug	915	38	40	7	-30	-22	27	-12	156	28	-11	162
30-Aug	930	39	80	12	-61	-83	32	-43	113	34	-41	121
30-Aug	945	40	68	16	-47	-130	36	-29	84	39	-26	95
30-Aug	1000	41	55	21	-31	-160	40	-14	70	41	-13	82
30-Aug	1015	42	60	29	-28	-188	45	-14	57	48	-11	71
30-Aug	1030	43	65	41	-22	-210	57	-7	50	63	-2	69
30-Aug	1045	44	75	52	-21	-230	69	-5	44	74	-1	68
30-Aug	1100	45	85	62	-21	-251	75	-9	35	78	-6	62
30-Aug	1115	46	113	223	99	-152	165	47	82	207	85	147
30-Aug	1130	47	140	528	349	197	411	244	326	534	355	501
30-Aug	1145	48	235	782	492	689	683	403	729	806	514	1015
30-Aug	1200	49	330	945	554	1243	830	450	1179	886	500	1516

DaMon	Time	Ordin.	HSMM	Kinematik Wave			Muskingum (own Tc)			Muskingum (HSMM Tc)		
			Q	Q	V Diff.	cum.V	Q	V Diff.	cum.V	Q	V Diff.	cum.V
	[hrs]		[cfs]	[cfs]	[1000 ft^3]		[cfs]	[1000 ft^3]		[cfs]	[1000 ft^3]	
30-Aug	1215	50	368	828	414	1657	806	394	1573	800	389	1904
30-Aug	1230	51	405	542	123	1780	589	166	1739	476	64	1968
30-Aug	1245	52	447	359	-79	1701	331	-104	1634	215	-209	1760
30-Aug	1300	53	489	277	-191	1510	245	-220	1415	229	-234	1526
30-Aug	1315	54	462	220	-218	1292	212	-225	1190	179	-255	1271
30-Aug	1330	55	435	168	-240	1052	152	-255	935	127	-277	994
30-Aug	1345	56	385	137	-223	829	125	-234	701	117	-241	752
30-Aug	1400	57	335	123	-191	638	106	-206	495	92	-219	534
30-Aug	1415	58	280	111	-152	486	101	-161	334	107	-156	378
30-Aug	1430	59	225	97	-115	371	89	-122	212	74	-136	242
30-Aug	1445	60	178	89	-80	291	77	-91	121	78	-90	152
30-Aug	1500	61	130	85	-41	250	75	-50	71	72	-52	100
30-Aug	1515	62	125	80	-41	210	70	-50	22	68	-51	49
30-Aug	1530	63	120	73	-42	167	66	-49	-27	66	-49	0
30-Aug	1545	64	118	69	-44	123	60	-52	-79	54	-58	-58
30-Aug	1600	65	115	67	-43	80	58	-51	-131	63	-47	-104
30-Aug	1615	66	105	64	-37	43	57	-43	-174	51	-49	-153
30-Aug	1630	67	95	60	-32	12	53	-38	-212	56	-35	-188
30-Aug	1645	68	83	57	-23	-12	50	-30	-241	45	-34	-222
30-Aug	1700	69	70	56	-13	-24	48	-20	-261	51	-17	-239
30-Aug	1715	70	68	54	-13	-37	48	-18	-279	46	-20	-259
30-Aug	1730	71	65	51	-13	-49	45	-18	-297	44	-19	-278
30-Aug	1745	72	63	49	-13	-62	43	-18	-315	43	-18	-296
30-Aug	1800	73	60	48	-11	-73	43	-15	-330	41	-17	-313
30-Aug	1815	74	55	47	-7	-80	41	-13	-343	43	-11	-324
30-Aug	1830	75	50	45	-5	-85	40	-9	-352	36	-13	-337
30-Aug	1845	76	45	43	-2	-86	38	-6	-358	40	-5	-341
30-Aug	1900	77	43	43	0	-86	38	-5	-363	35	-7	-348
30-Aug	1915	78	42	42	0	-86	36	-5	-368	39	-3	-351
30-Aug	1930	79	41	40	-1	-87	36	-5	-373	33	-7	-358
30-Aug	1945	80	40	39	-1	-88	34	-5	-378	35	-5	-363
30-Aug	2000	81	39	39	0	-88	34	-5	-383	32	-6	-369
30-Aug	2015	82	38	38	0	-88	33	-5	-387	34	-4	-373
30-Aug	2030	83	37	36	-1	-89	32	-5	-392	31	-5	-378
30-Aug	2045	84	36	36	0	-89	31	-5	-396	31	-5	-383
30-Aug	2100	85	35	36	1	-88	30	-5	-401	31	-4	-386
30-Aug	2115	86	34	36	2	-86	31	-3	-403	29	-5	-391
30-Aug	2130	87	34	35	1	-85	29	-5	-408	30	-4	-394
30-Aug	2145	88	33	35	2	-84	29	-4	-411	27	-5	-400
30-Aug	2200	89	33	35	2	-82	28	-5	-416	30	-3	-402
30-Aug	2215	90	32	34	2	-80	29	-3	-419	27	-5	-407
30-Aug	2230	91	32	34	2	-78	27	-5	-423	28	-4	-410
30-Aug	2245	92	31	33	2	-76	27	-4	-427	25	-5	-416
30-Aug	2300	93	31	32	1	-76	26	-5	-431	28	-3	-419
30-Aug	2315	94	30	32	2	-74	26	-4	-435	25	-5	-423
30-Aug	2330	95	30	31	1	-73	26	-4	-438	26	-4	-427
30-Aug	2345	96	28	30	2	-71	25	-3	-441	24	-4	-430
31-Aug	0	97	23	25	2	-69	25	2	-439	25	2	-428
31-Aug	15	98	15	17	2	-67	21	5	-434	20	5	-424
31-Aug	30	99	8	11	3	-65	13	5	-429	9	1	-423
31-Aug	45	100	4	7	3	-62	5	1	-428	1	-3	-426

Difference in Volume due to different approches of hydrograph computations

```
ID    Project Tributary Strubble Creek
ID    Wetland Ponds, Muskingum Routing
ID    100-yr. Flood, Future Conditions
*FREE
*DIAGRAM
IT    15 30AUG96    0000    100
IO    0  2
IN 60 30AUG96 0000
KK D01
KM SUBAREA D01
PB 7.00
PI 0.077 0.082 0.089 0.097 0.106 0.119 0.136 0.159
PI 0.194 0.257 0.401 3.163 0.598 0.310 0.221 0.175
PI 0.146 0.127 0.112 0.101 0.092 0.085 0.079 0.074
BA 0.170
LS    0  81.34
UD 0.312
KK RD01
KM ROUTING THROUGH D02A
RM 421 0.0006 0.5
KK D02.02
KM SUBAREA D02.02
BA 0.003
LS    0 89.5
UD 0.155
KK RD02.02
KM ROUTE D02.02 THROUGH D02A
RM 196 0.0013 0.5
KK D02A
KM COMBINE RD01 AND RD02.02
HC 2
KK RD02A
KM ROUTING D02A THROUGH D03
RM 32 0.0078 0.5
KK D03
KM SUBAREA D03
BA 0.001
LS 0 94.34
UD 0.02
KK D03C
KM COMBINE RD02A AND D03
HC 2
KK D03.6
KM SUBAREA D03.6
BA 0.005
LS 0 93.18
UD 0.154
KK RD03.6
KM ROUTE D03.6 THROUGH D03.2
RM 880 0.0003 0.5
KK D03.2
KM SUBAREA D03.2
BA 0.001
LS 0 93.50
UD 0.033
KK D03.2C
KM COMBINE D03.2 AND RD03.6
HC 2
KK RD03.2C
KM ROUTE D03.2C THROUGH D03
RM 294 0.0008 0.5
KK D03C1
KM COMBINE RD03.2 AND D03C
HC 2
KK D03.1
```

KM SUBAREA D03.1
BA 0.009
LS 0 85.56
UD 0.032
KK RD03.1
KM ROUTE D03.1 THROUGH D03
RM 37 0.0068 0.5
KK D03ALL
KM COMBINE RD03.1 AND D03C1
HC 2
KK RD03ALL
KM ROUTE D03ALL THROUGH DO4
RM 18 0.014 0.5
KK RD04
KM ROUTE D04 THROUGH D05
RM 252 0.001 0.5
KK RD05
KM ROUTE D05 THROUGH D05A
RM 187 0.0013 0.5
KK D05.07
KM SUBAREA D05.07
BA 0.135
LS 0 85.33
UD 0.344
KK RDO5.07
KM ROUTE D05.07 THROUGH D05A
RM 6 0.0395 0.5
KK D05AALL
KM COMBINE RD05 AND RD05.07
HC 2
KK RD05AALL
KM ROUTE D05AALL THROUGH D06
RM 153 0.0016 0.5
KK RD06
KM ROUTE D06 THROUGH D07
RM 951 0.0003 0.5
KK D07.02
KM SUBAREA D07.02
BA 0.012
LS 0 92.71
UD 0.185
KK RD07.02
KM ROUTE D07.02 THROUGH D07
RM 23 0.0107 0.5
KK D07ALL
KM COMBINE RD06 AND RD07.05
HC 2
KK RD07ALL
KM ROUTE D07ALL THROUGH D08
RM 14 0.0184 0.5
KK D08.14
KM SUBAREA D08.014
BA 0.008
LS 0 84.78
UD 0.224
KK RD08.14
KM ROUTE D08.14 THROUGH D08.02A
RM 49 0.0051 0.5
KK RD08.02A
KM ROUTE D08.02A THROUGH D08.02
RM 111 0.0023 0.5
KK D08.04
KM SUBAREA D08.04
BA 0.007
LS 0 77.37

```
UD 0.250
KK RD08.04
KM ROUTE D08.04 THROUGH D08.02
RM 28 0.009 0.5
KK D08.02C
KM COMBINE RD08.02A AND RD08.04
HC 2
KK D08.02
KM SUBAREA D08.02
BA 0.099
LS 0 72.31
UD 0.332
KK DO8.02C1
KM COMBINE D08.02 AND D08.02C
HC 2
KK RD08.02C1
KM ROUTE D08.02C1 THROUGH D08
RM 6 0.0442 0.5
KK D08ALL
KM COMBINE RD08.02C1 AND RD07ALL
HC 2
KK RD08ALL
KM ROUTE D08ALL THROUGH D09A
RM 10 0.0255 0.5
KK D09.08
KM SUBARES D09.08
BA 0.011
LS 0 82.26
UD 0.282
KK RD09.08
KM ROUTE D09.08 THROUGH D09.06
RM 140 0.0018 0.5
KK D09.06
KM SUBAREA D09.06
BA 0.002
LS 0 90.10
UD 0.058
KK DO9.06C
KM COMBINE D09.06 AND RD09.08
HC 2
KK RD09.06C
KM ROUTE D09.06C THROUGH D09.02
RM 33 0.0075 0.5
KK D09.02
KM SUBAREA D09.02
BA 0.019
LS 0 93.64
UD 0.067
KK D09.02C
KM COMBINE D09.02 AND RD09.06C
HC 2
KK D09.04
KM SUBAREA D09.04
BA 0.002
LS 0 74.12
UD 0.156
KK RD09.04
KM ROUTE D09.04 THROUGH D09.02
RM 96 0.0026 0.5
KK D09.02C1
KM COMBINE RD09.04 AND D09.02C
HC 2
KK RD9.02C1
KM ROUTE D09.02C THROUGH D09A
RM 9 0.0264 0.5
```

```
KK D09AC
KM COMBINE RD09.02C1 AND D09A
HC 2
KK RD09AC
KM ROUTE D09AC THROUGH D09
RM 59 0.0042 0.5
KK RD09
KM ROUTE D09 THROUGH D10
RM 31 0.0081 0.5
KK D10
KM SUBAREA D10
BA 0.01
LS 0 91.65
UD 0.383
KK D10ALL
KM COMBINE RD09 AND D10
HC 2
KK RD10ALL
KM ROUTE D10ALL THROUGH D11A
RM 15 0.0163 0.5
KK RD11A
KM ROUTE D11A THROUGH D11
RM 20 0.0125 0.5
KK RD11
KM ROUTE D11 THROUGH D12
RM 49 0.0051 0.5
KK D12
KM SUBAREA D12
BA 0.002
LS 0 80.54
UD 0.026
KK D12C
KM COMBINE D12 AND RD11
HC 2
KK D12.01
KM SUBAREA D12.01
BA 0.013
LS 0 84.20
UD 0.177
KK RD12.01
KM ROUTE D12.01 THROUGH D12
RM 19 0.0132 0.5
KK D12ALL
KM COMBINE D12C AND RD12.01
HC 2
KK RD12ALL
KM ROUTE D12ALL THROUGH D13A
RM 15 0.0164 0.5
KK D13A
KM SUBAREA D13A
BA 0.008
LS 0 79.94
UD 0.102
KK D13AALL
KM COMBINE D13A AND RD12ALL
HC 2
KK RD13AALL
KM ROUTE D13AALL THROUGH D13
RM 18 0.0142 0.5
KK D13.06
KM SUBAREA D13.06
BA 0.029
LS 0 73.89
UD 0.387
KK RD13.06
```

```
KM ROUTE D13.06 THROUGH D13.04
RM 67 0.0037 0.5
KK RD13.04
KM ROUTE D13.04 THROUGH D13.02
RM 21 0.0117 0.5
KK D13.02
KM SUBAREA D13.02
BA 0.002
LS 0 68.52
UD 0.037
KK D13.02C
KM COMBINE D13.02 AND RD13.04
HC 2
KK RD13.02C
KM ROUTE D13.02C THROUGH D13
RM 423 0.0006 0.5
KK D13ALL
KM COMBINE RD13.02C AND RD13AALL
HC 2
KK RD13AALL
KM ROUTE D13AALL THROUGH D14
RM 131 0.0019 0.5
KK D13.01
KM SUBAREA D13.01
BA 0.01
LS 0 79.13
UD 0.137
KK RD13.01
KM ROUTE D13.01 THROUGH D13.09 (ASSUMPT.: D13.09 = D14)
RM 61 0.0041 0.5
KK D14
KM COMBINE RD13.01 AND RD13ALL
HC 2
KK MATTERN
KM COMPARISON OF MATTERN HYDROGRAPH AND THIS ONE
IN 30 30AUG96 0000
QO 0 0 0 0 0 0 0 0 0 0 0 0
QO 0 0 0 0 0 0 0 80 55 65 85 140
QO 330 405 489 435 335 225 130 120 115 95 70 65
QO 60 50 40 30 20 10 0 0 0 0 0
ZZ
```

Figure A-5: Sample HEC-1 Input File

Table B-1: Geometrical and Hydraulic Parameter of the Proportional Weir

h		y	x	a	Q
[ft]	[ab.WSL]	[ft]	[ft]	[ft]	[cfs]
0.0	2004.9	0.00	11.00	0.00	0.0
0.2	2005.1	0.00	11.00	0.20	3.3
0.4	2005.3	0.00	11.00	0.40	9.2
0.6	2005.5	0.00	11.00	0.60	16.9
0.8	2005.7	0.00	11.00	0.80	26.1
1.0	2005.9	0.00	11.00	1.00	36.4
1.2	2006.1	0.00	11.00	1.20	47.9
1.4	2006.3	0.00	11.00	1.40	60.4
1.6	2006.5	0.00	11.00	1.60	73.8
1.8	2006.7	0.00	11.00	1.80	88.0
2.0	2006.9	0.00	11.00	2.00	103.1
2.2	2007.1	0.00	11.00	2.20	118.9
2.4	2007.3	0.15	9.23	2.25	135.3
2.6	2007.5	0.35	8.37	2.25	151.7
2.8	2007.7	0.55	7.78	2.25	168.1
3.0	2007.9	0.75	7.33	2.25	184.5
3.2	2008.1	0.95	6.96	2.25	200.9
3.4	2008.3	1.15	6.65	2.25	217.3
3.6	2008.5	1.35	6.38	2.25	233.7
3.8	2008.7	1.55	6.15	2.25	250.1
4.0	2008.9	1.75	5.94	2.25	266.5
4.2	2009.1	1.95	5.75	2.25	282.9
4.4	2009.3	2.15	5.58	2.25	299.3
4.6	2009.5	2.35	5.42	2.25	315.7
4.8	2009.7	2.55	5.28	2.25	332.1
5.0	2009.9	2.75	5.15	2.25	348.5
5.2	2010.1	2.95	5.03	2.25	364.9
5.4	2010.3	3.15	4.91	2.25	381.3
5.6	2010.5	3.35	4.81	2.25	397.7
5.8	2010.7	3.55	4.71	2.25	414.1
6.0	2010.9	3.75	4.62	2.25	430.5
6.2	2011.1	3.95	4.53	2.25	446.9
6.4	2011.3	4.15	4.44	2.25	463.3
6.6	2011.5	4.35	4.37	2.25	479.7
6.8	2011.7	4.55	4.29	2.25	496.1
7.0	2011.9	4.75	4.22	2.25	512.5
7.2	2012.1	4.95	4.15	2.25	528.9
7.4	2012.3	5.15	4.09	2.25	545.3
7.6	2012.5	5.35	4.03	2.25	561.7
7.8	2012.7	5.55	3.97	2.25	578.1
8.0	2012.9	5.75	3.91	2.25	594.5
8.2	2013.1	5.95	3.86	2.25	610.9
8.4	2013.3	6.15	3.81	2.25	627.3
8.6	2013.5	6.35	3.76	2.25	643.7
8.8	2013.7	6.55	3.71	2.25	660.1
9.0	2013.9	6.75	3.67	2.25	676.5
9.2	2014.1	6.95	3.62	2.25	692.9

h		y	x	a	Q
[ft]	[ab.WSL]	[ft]	[ft]	[ft]	[cfs]
9.4	2014.3	7.15	3.58	2.25	709.3
9.6	2014.5	7.35	3.54	2.25	725.7
9.8	2014.7	7.55	3.50	2.25	742.1
10.0	2014.9	7.75	3.46	2.25	758.5
10.2	2015.1	7.95	3.42	2.25	774.9
10.4	2015.3	8.15	3.39	2.25	791.3
10.6	2015.5	8.35	3.35	2.25	807.7
10.8	2015.7	8.55	3.32	2.25	824.2
11.0	2015.9	8.75	3.29	2.25	840.6

Governing Equations:

Q = 4.97*a^0.5*b(h-a/3)

x/b = 1 - (2/PI)*(atan(SQRT(y/a)))

Assumptions:

a = Height of the Rectangular Base
a = 2.25 ft
b = Base Width of the Weir
b = 11 ft

0.00 < h < 2.25 ft
-> x = b
-> y = 0

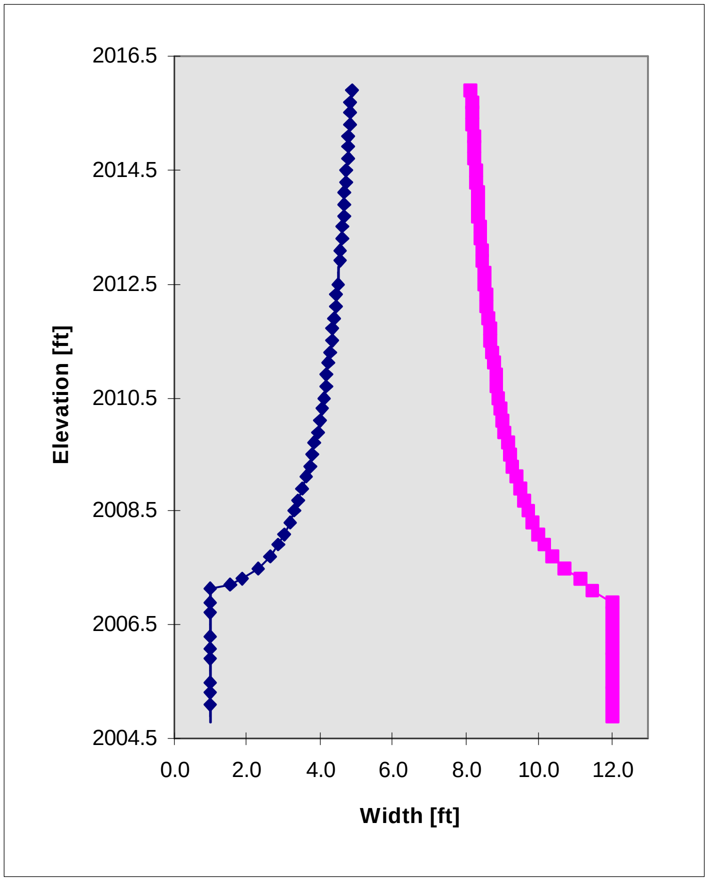


Figure B-1: Design Geometry of the Proportional Weir

Table B-2: Wet Pond Drawdown Time Calculations for 100-yr Design Flood

grate inlet			
Elevation (ft)	Storage [acre-ft)	Outflow [cfs)	Time [hours)
2021.62	13.01	517.06	
2021.36	12.34	382.83	0.02
2021.08	11.64	258.08	0.03
2020.81	10.94	151.42	0.04
2020.54	10.24	66.94	0.08
2020.26	9.53	13.51	0.21
2019.99	8.84	7.12	0.82
2019.72	8.22	6.66	1.09
2019.44	7.59	6.17	1.17
2019.17	6.97	5.63	1.27
2018.89	6.38	5.04	1.34
2018.62	5.84	4.36	1.40
2018.35	5.30	3.56	1.66
2018.10	4.81	1.96	2.15
			11.27

8-in pipe			
Elevation (ft)	Storage (acre-ft)	Outflow (cfs)	Time (hours)
2021.63	13.03	516.47	
2021.29	12.16	344.41	0.02
2020.95	11.29	197.31	0.04
2020.61	10.41	80.64	0.08
2020.26	9.53	9.22	0.24
2019.92	8.68	3.16	1.67
2019.58	7.90	2.97	3.07
2019.24	7.13	2.74	3.29
2018.89	6.38	2.50	3.44
2018.55	5.70	2.24	3.46
2018.21	5.03	1.94	3.93
2018.10	4.82	1.84	1.33
			20.56

perf. riser			
Elevation (ft)	Storage (acre-ft)	Outflow (cfs)	Time (hours)
2021.61	12.98	517.75	
2021.40	12.43	404.53	0.01
2021.07	11.61	256.71	0.03
2020.75	10.79	133.90	0.05
2020.43	9.97	43.02	0.11
2020.11	9.14	7.89	0.39
2019.79	8.38	6.83	1.25
2019.47	7.65	5.80	1.40
2019.15	6.93	4.83	1.66
2018.83	6.25	3.90	1.88
2018.51	5.61	3.03	2.22
2018.18	4.98	2.23	2.92
2018.10	4.82	2.04	0.89
			12.82

Table B-3: Dry Pond Drawdown Time Calculations for 100-yr Design Flood

proportional weir			
Elevation (ft)	Storage (acre-ft)	Outflow (cfs)	Time (hours)
2011.77	12.84	502.24	
2011.33	11.31	465.82	0.04
2010.74	9.30	417.87	0.06
2010.16	7.29	369.93	0.06
2009.57	5.87	321.99	0.05
2008.99	4.66	274.04	0.05
2008.41	3.45	226.10	0.06
2007.82	2.42	170.37	0.06
2007.24	1.80	121.93	0.05
2006.65	1.18	79.22	0.07
2006.07	0.56	43.15	0.12
2005.48	0.24	15.28	0.13
2004.90	0.00	0.00	0.38
			1.14

2-ft pipe			
Elevation (ft)	Storage (acre-ft)	Outflow (cfs)	Time (hours)
2013.87	22.68	505.86	
2013.66	22.07	468.18	0.02
2013.08	19.11	369.75	0.09
2012.49	16.14	280.00	0.11
2011.91	13.32	199.98	0.14
2011.33	11.31	131.10	0.15
2010.74	9.30	75.65	0.24
2010.16	7.29	38.92	0.42
2009.57	5.87	32.07	0.49
2008.99	4.66	29.05	0.48
2008.41	3.45	25.61	0.54
2007.82	2.42	21.66	0.53
2007.24	1.80	17.05	0.39
2006.65	1.18	11.68	0.52
2006.07	0.56	5.57	0.87
2005.48	0.24	1.50	1.10
2004.90	0.00	0.00	3.92
			9.98

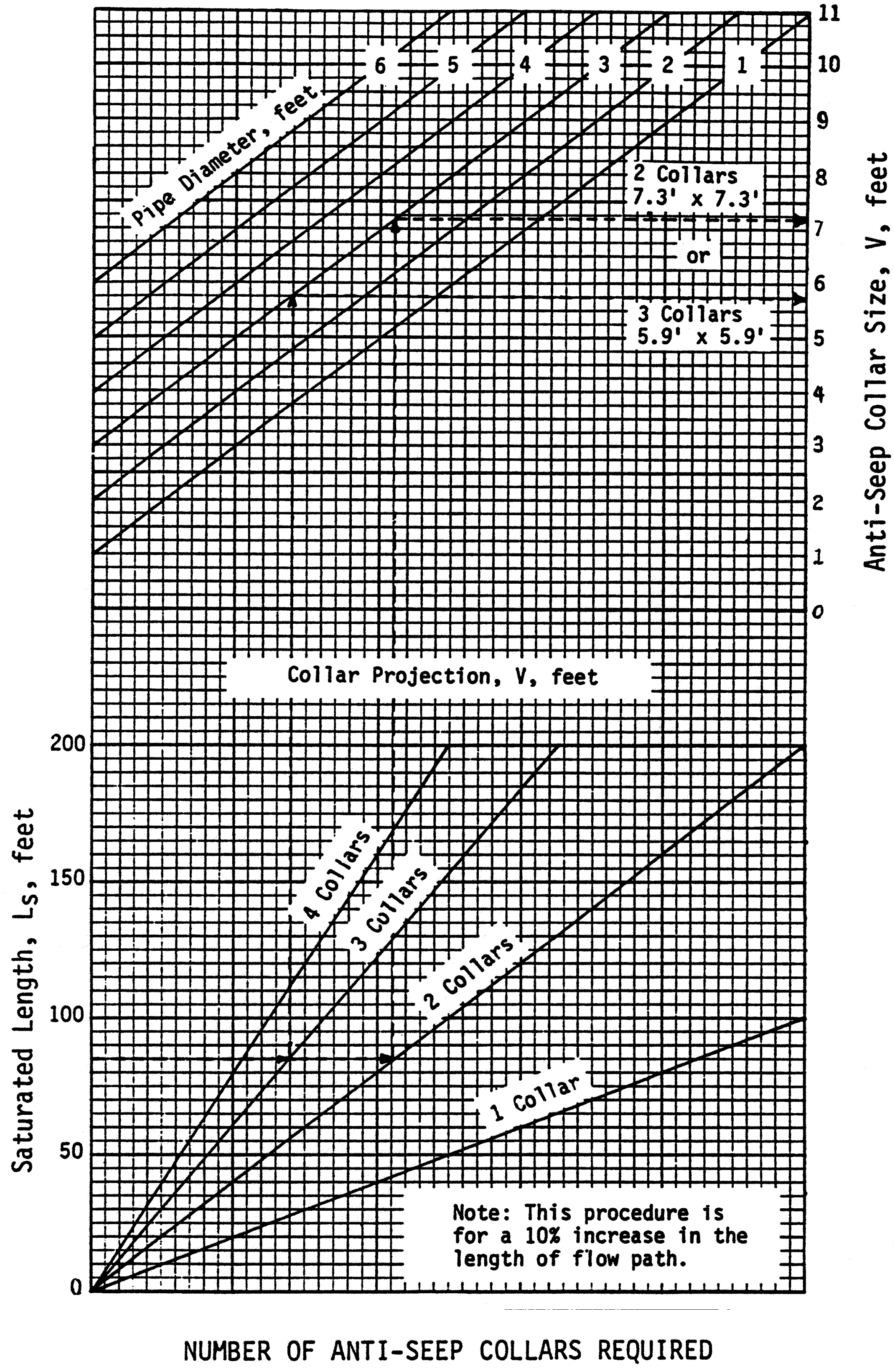


Figure B-2: Procedure for Anti-Seep Collar Design (SCS, 1986)

Table C-1: Dry Pond Site - Summary of Boring Logs (Froehling & Robertson, 1996)

Boring No.	Distance*		Ground Eleveation*		Alluvium*			Decomposed Rock*	
	interv.	cumul.		Surficial Soil*	(ML)	(CL/OL)	(CH)		Auger Refusal*
B-1	0	0	2012.0	2011.5	2010.0		2005.5		2005.5
B-2	100	100	2008.0	2007.5			2004.5	2004.0	2004.0
B-3	100	200	2006.0	2005.5		2004.0	2002.5	1998.0	
B-4	100	300	2008.0	2007.5	2004.5	2003.0	2000.0		
B-5	100	400	2012.0	2011.5	2010.0	2008.5	2004.0		
									Remarks
* All data			B-1	Blows	4/4/5		4/7/8		Auger refusal
are				Material	(ML)		(CH)		
referred			B-2	Blows			4/6/10		Auger refusal
to the				Material			(CH)	(SM)	
unit of			B-3	Blows		2/3/4	5/2/2	6/16/45	Boring terminated
feet.				Material		(CL)	(CH)	(SM)	
			B-4	Blows	4/1/3	2/5/6	3/6/4		Boring terminated
				Material	(ML)	(OL)	(CH)		
			B-5	Blows	2/2/3	5/5/7	4/4/7		Boring terminated
				Material	(ML)	(CL)	(CH)		

Table C-2: Wet Pond Site - Summary of Boring Logs (Froehling & Robertson, 1996)

Boring No.	Distance*		Ground Eleveation*		Alluvium*			Decomposed Rock*	
	interv.	cumul.		Surficial Soil*	(CL)	(ML/OL)	(CH/SC)		Auger Refusal*
B-13		0	2020.0	2019.5		2018.0	2016.5	2016.0	
B-14	75	75	2017.0	2016.5	2015.0	2013.5	2012.0	2010.0	2010.0
B-15	75	150	2018.0	2017.5		2016.0		2014.0	2014.0
* All data									Remarks
are			B-13	Blows		5/4/7	4/4/6		Boring terminated
referred				Material		(OL)	(CH)	(ML)	
to the			B-14	Blows	1/2/1	2/2/7	5/8/21	18/32/*	Auger refusal
unit of				Material	(CL)	(ML)	(SC)	(ML)	
feet.			B-15	Blows				1/5/8	Auger refusal
				Material		(OL)		(SM)	

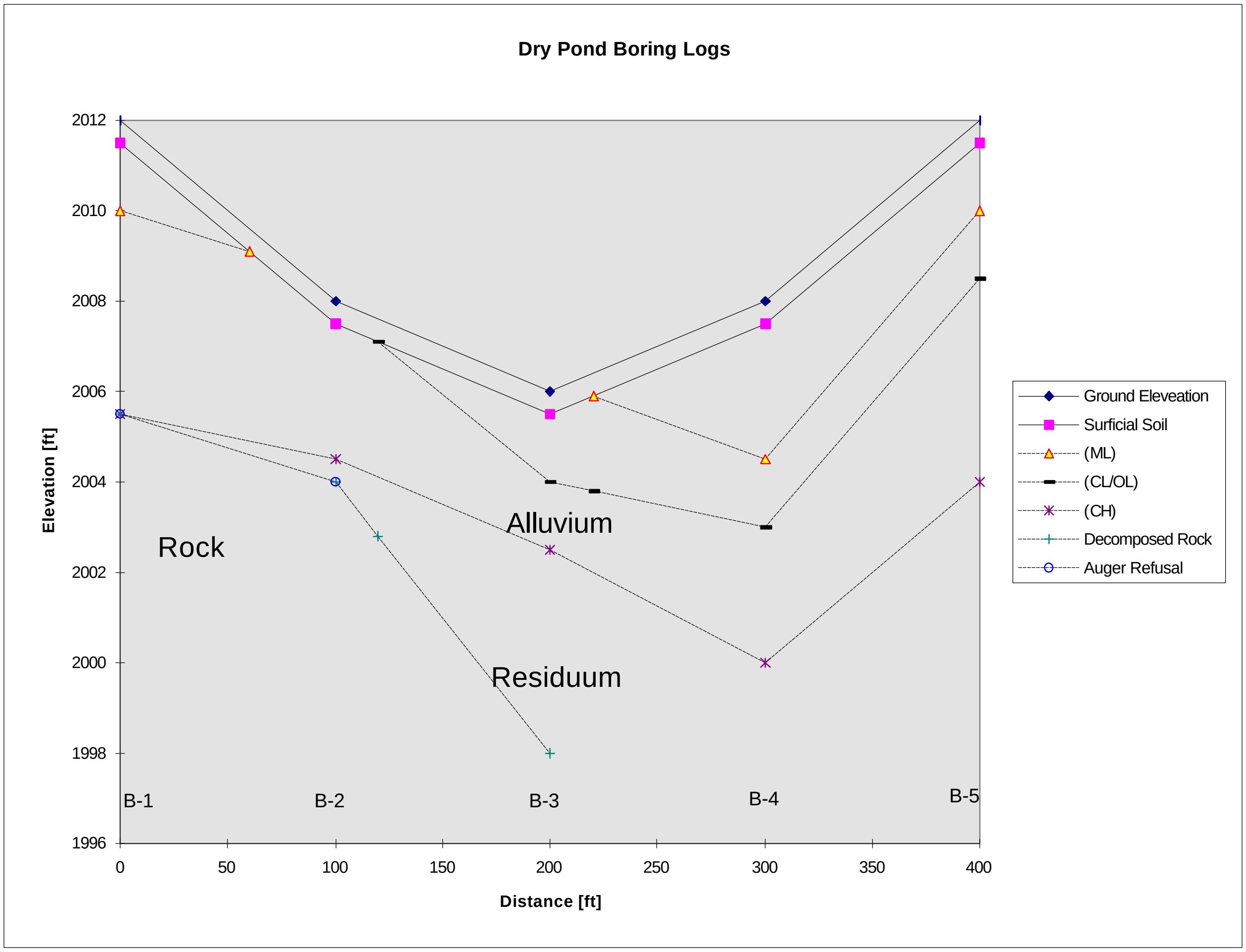


Figure C-1: Dry Pond Boring Logs (Froehling & Robertson, 1996)

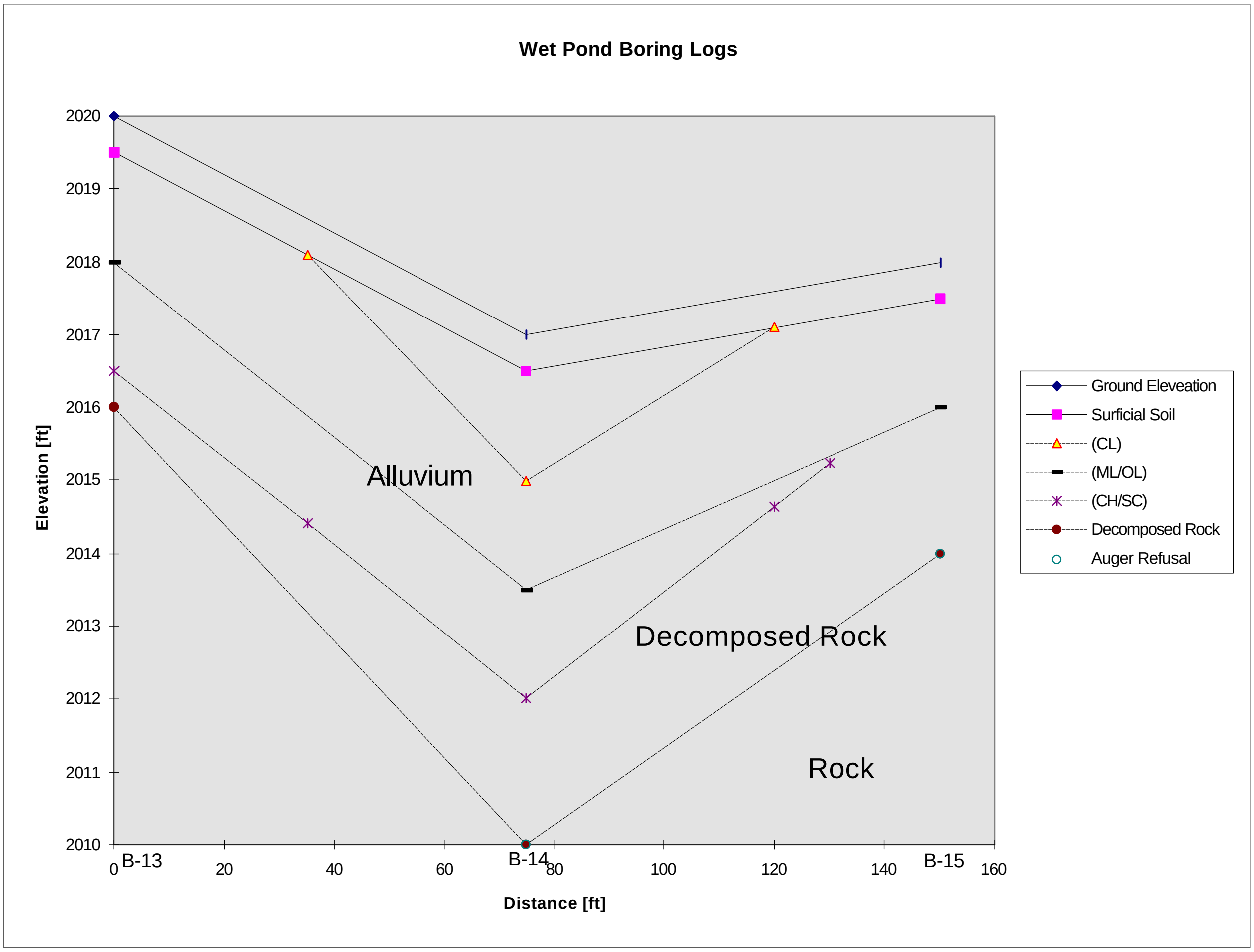


Figure C-2: Wet Pond Boring Logs (Froehling & Robertson, 1996)

Table D-1: Anderson & Associates' Cost Estimation of the Storm Water Management Facility (Anderson & Associates, 1996)

	QUANTITY	UNIT	UNIT PRICE	COST
BORROW	4600	CY	$10.00	$46,000.00
GRADING	33000	CY	$4.50	$148,500.00
ROCK EXCAVATION	900	CY	$40.00	$36,000.00
OUTLET PROTECTION (RIP RAP)	50	SY	$2.00	$100.00
IMPERMEABLE CORE	1	LS	$34,000.00	$34,000.00
SEEDING	9	AC	$2,000.00	$18,000.00
VDOT STD MH-1	2	EA	$2,500.00	$5,000.00
WATER TIGHT F&C	1	EA	$350.00	$350.00
6" D.I.P. (6 L.F.)	1	LS	$300.00	$300.00
12" RCP	106	LF	$28.00	$2,968.00
15" RCP	12	LF	$40.00	$480.00
24" RCP	152	LF	$50.00	$7,600.00
EW-1 FOR 12" RCP	2	EA	$500.00	$1,000.00
EW-1 FOR 24" RCP	1	EA	$2,000.00	$2,000.00
EW-11 FOR 24" RCP	1	EA	$3,000.00	$3,000.00
SOIL STABILIZATION MAT	860	SY	$3.00	$2,580.00
FENCING	780	LF	$6.00	$4,680.00
GUARDRAIL	420	LF	$15.00	$6,300.00
STORM WATER MANAGEMENT SUBTOTAL				**$318,858**
SUBTOTAL CONSTRUCTION				**$318,858**
CONTINGENCY@8%				**$25,509**
TOTAL SITEWORK & UTILITIES				**$344,367**
TOTAL ESTIMATE				**$344,367**

Table D-2: Own Cost Estimation of the Storm Water Management

Task	Construction Description	Masterformat	Unit	Quantity	Total Costs incl. O&P $	Cost (Item) Sum $	Subtotal $
Site Clearing	Clearing Brush with dozer, ball and chain, light clearing	021-108-0300	Acre	3.24	645.0	2089.8	
							2090
Site Grading - Excavation							
Surf. Soil (dam site)	Self propelled scraper 14 C.Y., common earth, 1500' haul	022-246-1300	C.Y.	605.1	2.7	1609.6	
Pond Grading	Self propelled scraper 14 C.Y., common earth, 1500' haul	022-246-1300	C.Y.	6611.0	2.7	17585.3	
Pond Grading	Self propelled scraper 14 C.Y., sand/gravel, 1500' haul	022-246-1050	C.Y.	16681.0	2.3	38366.3	
Pond Grading	Self propelled scraper 14 C.Y., clay, 1500' haul	022-246-1500	C.Y.	6611.0	4.2	28030.6	
Pond Grading	Drilling and blasting only, rock, open face, over 1500 C.Y.	022-234-0100	C.Y.	3336.0	6.0	20016.0	
Hauling	20 C.Y. dump trailerr, 3 miles round trip, 1.7 loads/hr	022-266-1220	C.Y.	33631.0	2.8	95512.0	
Seeding Dry Pond	Mechanical seeding, $2.00/lb, 215 lb/acre	029-304-0010	Acre	1.36	1225.0	1666.0	
Seeding WetPond	Hydro or air seeding for large areas, incl. seed and fertilizer	029-304-1100	S.Y.	2667.0	0.3	906.8	
							203693
Core/Cutoff Trench							
Borrowing	Buy and load, haul 2 miles round trip and spread	022-212-0700	C.Y.	1825.0	12.1	22082.5	
	with 200 H.P. dozer, no compaction - structural fill						
	for 5 mile haul, add	022-212-0900	C.Y.	1825.0	3.0	5438.5	
Compacting	6" to 12" lifts, Sheepsfoot roller	022-204-2200	C.Y.	1825.0	1.9	3485.8	
							31007
Toe Drain Trench							
Material	Buy and load, haul 2 miles round trip and spread	022-212-0100	C.Y.	370.3	7.9	2906.9	
	with 200 H.P. dozer, no compaction - bank run gravel						
	for 5 mile haul, add	022-212-0900	C.Y.	370.3	3.0	1103.5	
Compacting	6" to 12" lifts, vibrating roller	022-204-2200	C.Y.	370.3	1.8	651.7	
Perforated Pipe	PVC, 10' length, 6" diameter	027-168-2040	L.F.	420.0	2.1	898.8	
							5561
Embankment							
Compacting shell	6" to 12" lifts, vibrating roller	022-204-2200	C.Y.	5538.0	1.8	9746.9	
Crest & Upstream Face	Fine grading and seeding incl. lime, fertilizer & seed, w/ equipment	029-304-0310	S.Y.	2674.0	1.4	3850.6	
Downstream Face	Riprap, random broken stone, 9" deep	022-712-0500	S.Y.	442.0	28.5	12597.0	
							26194

Perforated Riser							
Pipe	Corrugated Metal Pipe, 20' length, 30" diameter	027-164-2160	L.F.	58.6	30.5	1787.3	
Compacting	6" lifts, hand tamp	022-204-0300	C.Y.	4.9	8.0	39.1	
Elbow	90 degree, CMP, 30" diameter	estimated	Ea.	1.0	200.0	200.0	
Footing	Concrete in place, over 5 C.Y., forms, reinforcing	033-130-3850	C.Y.	8.3	109.0	904.7	
	Concrete in place, over 5 C.Y., placing	033-172-2600	C.Y.	8.3	10.2	84.2	
Anti-Seep Collars	Concrete foundation material, over 10 C.Y., forms, reinforcing	033-130-4000	C.Y	13.4	166.0	2224.4	
	Concrete foundation material, over 10 C.Y., placing	033-172-2900	C.Y	13.4	3.2	42.7	
Endwall structure	Concrete in place, walls 12" thick, forms, reinforcing	033-130-4260	C.Y.	1.7	325.0	541.7	
	Concrete in place, walls, placing	033-172-5050	C.Y.	1.7	11.2	18.6	
							5843
Proportional Weir							
Walls	Concrete in place, grade walls 12" thick, forms, reinforcing	033-130-4260	C.Y.	23.6	325.0	7672.4	
	Concrete in place, grade walls, placing	033-172-5050	C.Y.	23.6	11.2	263.1	
Slab	Ground Slab	033-130-5700	C.Y.	38.7	102.0	3947.8	
	Slap over 6" thick	033-172-4600	C.Y.	38.7	6.8	263.2	
Weir wings	Concrete in place, average for superstructure forms and reinforcing	033-130-0150	C.Y.	1.1	320.0	355.6	
	Concrete in place, placing, colums, pumped	033-172-0800	C.Y.	1.1	22.5	25.0	
							12527
Manholes							
MH-1	Manhole, precast, 4' I.D., 8' deep	027-152-1130	Ea.	1.0	865.0	865.0	
Slap tops	precast, 8" thick, 4' diameter manhole	027-152-1300	Ea.	1.0	197.0	197.0	
Footing	Concrete in place, under 5 C.Y., forms, reinforcing	033-130-3850	C.Y.	2.7	155.0	413.3	
	Concrete in place, under 5 C.Y., placing	033-172-2400	C.Y.	2.7	20.5	54.7	
Cover	Watertight	estimated	Ea.	1.0	350.0	350.0	
							1880
Pipes							
12" RCP							
Excavate Trench	1' to 4' deep, 3/8 C.Y. tractor loader	022-254-0050	C.Y.	41.0	4.0	162.4	
Bedding for pipe	Sand, dead or bank	026-012-0200	C.Y.	16.1	8.8	141.7	
Compacting bedding	Compactiing bedding in trench	026-012-0500	C.Y.	16.1	2.6	41.1	
Pipes and Placing	12" reinforced concrete pipe	027-162-2010	L.F.	62.0	10.6	654.1	
Backfill Trench	Dozer backfilling, bulk, up to 300' haul	022-204-1300	C.Y.	40.5	0.9	37.7	
Compacting Backfill	6 to 12" lifts, vibrating roler	022-204-1600	C.Y.	16.1	1.5	24.8	
Endwall structure	Concrete in place, walls 12" thick, forms, reinforcing	033-130-4260	C.Y.	4.6	325.0	1504.6	
	Concrete in place, walls, placing	033-172-5050	C.Y.	4.6	11.2	51.6	
15" RCP							
Excavate Trench	1' to 4' deep, 3/8 C.Y. tractor loader	022-254-0050	C.Y.	9.7	4.0	38.4	
Bedding for pipe	Sand, dead or bank	026-012-0200	C.Y.	3.2	8.8	28.2	
Compacting bedding	Compactiing bedding in trench	026-012-0500	C.Y.	3.2	2.6	8.2	
Pipes and Placing	15" reinforced concrete pipe	027-162-2020	L.F.	12.0	12.0	144.0	
Backfill Trench	Dozer backfilling, bulk, up to 300' haul	022-204-1300	C.Y.	8.4	0.9	7.8	
Compacting Backfill	6 to 12" lifts, vibrating roler	022-204-1600	C.Y.	3.2	1.5	4.9	
Endwall structure	Concrete in place, walls 12" thick, forms, reinforcing	033-130-4260	C.Y.	2.3	325.0	752.3	
	Concrete in place, walls, placing	033-172-5050	C.Y.	2.3	11.2	25.8	

24" RCP							
Excavate Trench	1' to 4' deep, 3/8 C.Y. tractor loader	022-254-0050	C.Y.	61.6	4.0	243.9	
Bedding for pipe	Sand, dead or bank	026-012-0200	C.Y.	15.4	8.8	135.5	
Compacting bedding	Compactiing bedding in trench	026-012-0500	C.Y.	15.4	2.6	39.3	
Pipes and Placing	24" reinforced concrete pipe	027-162-2040	L.F.	52.0	23.5	1222.0	
Backfill Trench	Dozer backfilling, bulk, up to 300' haul	022-204-1300	C.Y.	58.5	0.9	54.4	
Compacting Backfill	6 to 12" lifts, vibrating roler	022-204-1600	C.Y.	15.4	1.5	23.7	
Endwall structure	Concrete in place, walls 12" thick, forms, reinforcing	033-130-4260	C.Y.	2.3	325.0	752.3	
	Concrete in place, walls, placing	033-172-5050	C.Y.	2.3	11.2	25.8	
							6124
Fencing	Misc. metal, chicken wire, posts @ 4', 1" mesh, 4' high	028-320-0010	L.F.	780.0	3.5	2698.8	
							2699
Guardtrail	wooden, 3' high, 1" X 6" on 2" X 4" posts	015-302-1001	L.F.	420.0	3.4	1432.2	
Seeding on trail	Mechanical seeding, $2.00/lb, 44 lb/M.S.Y.	029-304-0100	S.Y.	466.7	0.2	112.0	
							1544
					Overall Sum:	$	299162
Regional Adjustment Factor for Roanoke Area (site work): 0.873						$	261168
Index Factor for 1993: 0.962						$	271485
			GRAND TOTAL COST:			$	271485

Masterformat system of classification - according to the Construction Specifications Institute (CSI) and Construction Specifications Canada (CSC) (Waier, 1993)

For detailed calculations of volumes for excavation and backfill and areal computations for seeding, and clearance, refer to Tables D-3 to D-9.

Table D-3: Volumes and Area Sizes for Excavation and Seeding -Wet Pond

Elevation [ft]	Area [ft²]	Average Area [ft²]	Increment [ft]	Increm. Volume [ft³]	Cumulative Vol. [ft³]	Cumulative Vol. [yr³]
2014.5	0				0	0
		23078	0.5	11539		
2015	46156				11539	427
		51360	1.0	51360		
2016	56563				62899	2330
		60332	1.0	60332		
2017	64100				123230	4564
		67750	1.0	67750		
2018	71400				190980	7073
		74450	1.0	74450		
2019	77500				265430	9831
		79200	1.0	79200		
2020	80900				344630	12764
		81950	2.0	163900		
2022	83000				508530	18834
	Clearance:	81950	1.88 acres			

Table D-4: Volumes and Area Sizes for Excavation and Seeding -Dry Pond

Elevation [ft]	Area [ft²]	Average Area [ft²]	Increment [ft]	Increm. Volume [ft³]	Cumulative Vol. [ft³]	Cumulative Vol. [yr³]
2004.9	0				0	0
		2649	0.1	265		
2005.0	5298				265	10
		20284	1.0	20284		
2006.0	35270				20549	761
		43785	2.0	87570		
2008.0	52300				108119	4004
		59050	2.0	118100		
2010.0	65800				226219	8378
		55850	2.0	111700		
2012.0	45900				337919	12516
		27150	2.0	54300		
2014.0	8400				392219	14527
Seeding and Clearance:		59050	1.36 acres			
Clearance Total:		141000	3.24 acres			

Excavation	Total Volume [yr³]:	33361
- Sands and Silts (50%)		16681
- Common Earth (20%)		6672
- Clays (20%)		6672
- Rock (10%)		3336

Table D-5: Seeding at Wet Pond

Elevation [ft]	Area [ft^2]
2018	71400
2019	77500

Area is estimated by interpolation of the contour lir
for elevation 2018.1.

The calculation gives 72010 ft^2
or 2667 yr^2
or 1.65 acres

Table D-6: Excavation, Bedding, and Backfill Volumes

Width (W)= 2 * 3 ft + pipe diameter
Height (H) = Obtained from Drawings No. 1 and 2
Trench Volume (V) = W(i) * (H(i) * Length
Bedding = W(i) * 1 ft * Length
Backfill = V - PI/4 * D^2

Pipe Dia. [ft]	Number i	Length [ft]	Trench Width [ft]	Trench Height [ft]	Trench Volume [ft^3]	Trench Volume [yr^3]	Bedding Volume [yr^3]
1.00	1	6	7.0	3.0	126	4.7	1.6
1.00	1	56	7.0	2.5	980	36.3	14.5
1.25	1	12	7.3	3.0	261	9.7	3.2
2.00	1	52	8.0	4.0	1664	61.6	15.4

The volumes can be further summarized to:

Pipe Dia. [ft]	Number i	Length [ft]	Excavation Trench Volume [ft^3]	Excavation Trench Volume [yr^3]	Bedding Volume [yr^3]	Backfill Volume [yr^3]
1.00	1	62	1106	41.0	16.1	40.2
1.25	1	12	261	9.7	3.2	8.4
2.00	1	52	1664	61.6	15.4	58.5

Endwall Structure: Generally assumed to be 5 ft X 5 ft x 1 ft and 2 wing walls, which are each half the size of the 5X5X1 wall.

Table D-7: Volumes of Embankment and Structure for Wet Pond Facility

Net volume	Elevation	Crest Width	Slope downstr	Slope upstr	Removal	Core/ Cutoff	Additional Embankment Height Total	Additional Embankment Height Core
		25	3:1	4:1		[8 - 12 ft]		
	[ft]	[ft^3/ft]	[ft^3/ft]	[ft^3/ft]	[ft^3/ft]	[ft^3/ft]	[ft^3/ft]	[ft^3/ft]
Max. Embankment	2022.0						45.0	14.4
Crest	2020.2							
Ground	2017.0	80.0	15.4	20.5				
Subsurface Soil	2016.5	12.5	4.8	6.4	23.7	37.0		
Upstream Bottom	2014.5			25.6				
Bottom of Cutoff	2010.5					32.0		
		92.5	20.2	52.5	23.7			
			Sum:	165.1			210.1	

Cross-sectional Area	[ft^3/ft]	[yr^3/yr]		[ft^3/ft]	[yr^3/yr]
Core/Cutoff Trench	69.0	7.7		83.4	9
Shell	96.1	3.6		126.7	14
Toe Drain Trench (4" X 2', 1:1 side slopes)	23.8	0.9		[ft]	[yr]
Seeding				33.4	11
Riprap				29.6	10

Spillway Length [ft]	100	[ft^3]	[yr^3]		[ft^2]	[yr^2]
Core/Cutoff Trench		6900	255.6			
Shell		9614	356.1			
Toe Drain Trench (4" X 2', 1:1 side slopes)		2380	88.2			
Surficial Soil Removal		2370	87.8			
Seeding					3340	371.1
Riprap					2960	328.9
Side Extentions at spillway [ft]	40					
Core/Cutoff Trench		3336	123.6			
Shell		5070	187.8			
Seeding					1336	148.4
Riprap	20				592	65.8
Total						
Core/Cutoff Trench		10236	379.1			
Shell		14684	543.8			
Toe Drain Trench		2380	88.2			
Surficial Soil Removal		2370	87.8			
Seeding					4676	519.6
Riprap					3552	394.7

Table D-8: Volumes of Embankment and Structure for Dry Pond Facility

Net volume	Elevation	Crest Width 25	Slope downstr 3:1	Slope upstr 4:1	Removal	Core/ Cutoff [8 - 12 ft]
	[ft]	[ft^3/ft]	[ft^3/ft]	[ft^3/ft]	[ft^3/ft]	[ft^3/ft]
Crest	2014.9					
Ground	2006.0	222.5	118.8	158.4		
Subsurface Soil	2005.5	12.5	13.35	17.8	43.65	94
Bottom of Cutoff	2002.0					28
		235	132.2	176.2	43.65	
				543.39		

Cross-sectional Area	[ft^3/ft]	[yr^3/yr]	[ft^2/ft]	[yr^2/yr]
Core/Cutoff Trench	122.0	13.6		
Shell	421.4	15.6		
Concrete - Prop. Weir (walls)	515.4	19.1		
Toe Drain Trench (4" X 2', 1:1 side slopes)	23.8	0.9		
Surficial Soil Removal	43.7	1.6		
Seeding			60.6	20.2
Riprap			50.0	16.7

Spillway Length [ft] 320	[ft^3]	[yr^3]	[ft^2]	[yr^2]
Core/Cutoff Trench	39040	1445.9		
Shell	1E+ 05	4994.2		
Concrete - Prop. Weir	1036	38.4		
(2 X 1' walls, 1' X 11' slab, 1' weir wings)				
Toe Drain Trench (4" X 2', 1:1 side slopes)	7617	282.1		
Surficial Soil Removal	13968	517.3		
Seeding			19392	2155
Riprap			425	47

Table D-9: Total Volumes of Embankment and Structure for Facilities

Total	[ft^3]	[yr^3]	[ft^2]	[yr^2]
Core/Cutoff Trench	49276	1825.0		
Shell	149527	5538.0		
Concrete - Prop. Weir	1036	38.4		
Toe Drain Trench	9998	370.3		
Surficial Soil Removal	16338	605.1		
Seeding			24068	2674.2
Riprap			3977	441.9

DRAWINGS

The drawings No. 1 to 4 are attached as file of type DXF.

The drawings attached are:
- DRAWING1.DXF
- DRAWING2.DXF
- DRAWING3.DXF
- DRAWING4.DXF

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