Mitigating Channel Erosion in Developing Watersheds

Anacostia River Study Phase II

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1. SUMMARY

Development of urban and suburban land produces hydrologic changes that have an important impact on flooding, water quality, and channel stability in streams draining the developed land. Stormwater management (SWM) detention facilities may be designed to control one or a number of these impacts. A facility designed to limit peak discharges and control flooding will not, in general, provide control of other impacts, such as water quality and channel erosion impacts of development. This report presents an investigation of SWM design alternatives that provide erosion control for a design storm with a 2 year return period.

The total bed material load occurring during the design storm is used as a quantifiable surrogate for channel erosion. To illustrate the method and constraints of an erosion control design, erosion control SWM plans are developed for two demonstration sites. To provide continuity with current SWM practice, the designs developed here use event-based hydrology developed using U.S. Soil Conservation Service procedures. To provide compliance with existing SWM policy, the designs provide, in addition to erosion control of the 2 year storm, 24 hour extended detention of the 1 year storm, peak discharge control of the 2 year and 10 year storm, and compliance with SCS requirements for a 100 year emergency spillway. Based on the designs developed in this report, general design suggestions are provided that permit erosion control to be explicitly incorporated in the design of SWM facilities. The design suggestions can readily accommodate wet ponds and permanent marshes, in addition to the extended detention dry ponds used to illustrate the erosion control SWM design.

The demonstration sites are located in the Anacostia River Basin, a tributary of the Potomac River. One site is located in the Piedmont Physiographic Province, the other in the Coastal Plain Plain Physiographic Province. The sites were selected to provide a contrast in drainage area, soils, site hydrology, and nature of the receiving stream, thereby allowing the erosion control SWM designs to be developed over a broad range of conditions. In the Anacostia Basin, channel erosion and sedimentation problems related to uncontrolled stormwater runoff are widespread and locally severe and represent an important source of sediment that can severely degrade aquatic habitat and living resources. The governments of Maryland, Prince George's County, Montgomery County, and the District of Columbia are committed to basin-wide management of erosion and sedimentation by the turn of the century.

For the demonstration sites investigated here, a SWM facility designed to provide peak discharge control does not provide erosion control. A SWM facility designed to provide 24 hour extended detention of a storm with a 1 year return period does provide a significant reduction in bed material load for the 2 year storm, although it does not always provide erosion control to

predevelopment levels. To ensure that erosion control at the 2 year level is provided, the total bed material load for the storm must be calculated. In some cases, a range in the size and elevation of the outlet structures must be investigated to locate a combination that provides both erosion control and compliance with other SWM requirements concerning peak discharge and water quality. The methodology to find this optimum solution is presented in this report and a FORTRAN computer program that makes the necessary computations is included.

It was found that erosion control designs are sensitive to the choice of formula used to estimate bed material load in the receiving channel and to the value of discharge where bed material motion is assumed to begin. At each demonstration site, erosion control designs were developed using two different transport formulas. The size and elevation of the pond outlet structures and the need for additional pond storage were found to vary between designs based on the two transport formulas, although the general nature of the designs were similar. However, for different values of the discharge for incipient bed material movement that fall within the range that may be expected in receiving streams, the differences in erosion control design based on different transport formulas can be extreme. The reasons for the large variation of erosion control design with the type of transport formula and estimates of initial-motion discharge are discussed in this report and recommendations are made concerning the choice of transport formulas to be used for erosion control SWM design. Of the two transport formulas used in this study, that of Goncharov (cited in McCuen and Moglen, 1988) is recommended for erosion control design because it is of a class of convex transport functions that provide more stable and reliable designs.

The following summarize the main arguments and results of this report.

- Current stormwater regulations contain no quantitative standard for the control of downstream erosion; this work demonstrates one way such a requirement could be formulated and evaluated.

- Pond designs for erosion control and water quality control both incorporate increased storage for smaller, higher frequency storms. A design that will satisfy one of these criteria will therefore contribute to meeting the other.

- A design meeting 1 year 24 hour extended detention requirements does not necessarily provide erosion control for the 2 year storm. To ensure compliance with both extended detention and erosion control criteria, the SWM design procedure must explicitly examine both detention time and bed material load.

- Erosion control designs are sensitive to the choice of bed load transport formula, as well as the critical discharge at which bed material motion is assumed to begin. For high values of critical discharge, all erosion-controlling designs converge toward similar

erosion control strategies. If the critical discharge is small, however, (corresponding to channels in fine-grained, cohesionless sediments), different transport formulas can yield radically different erosion control designs.

- Transport formulations showing a convex relation between bed material load and discharge appear to provide more reliable erosion control SWM designs than concave transport formulations. Convex relations do not show the extreme range in erosion control design produced by concave relations, and convex relations are less dependent on the estimate of critical shear stress, which is difficult to estimate accurately.

- The apparent difference in estimated bed material load between concave and convex transport formulas may arise in part from the use of flow parameters (velocity, bed shear stress) averaged for the whole channel. If the actual spatial distribution of bed shear stress were used in computing the bed material load, some apparently concave transport functions might actually produce convex transport-discharge relations. However, when mean flow parameters are used to calculate transport, use of convex transport-discharge functions (e.g. the Goncharov or Shields formulas) is recommended.

- Design procedures demonstrated in this work can be used for either new facilities or the retrofit of existing SWM projects.

- Erosion control designs based on the 24-hour storm with a two-year recurrence interval also appear to provide erosion control for more frequent storms.

- Implementation of the erosion control SWM designs developed in this report will serve to decrease entrainment of in-channel sediments, which will contribute to a decrease in channel sedimentation. A decrease in sedimentation is necessary to improve aquatic habitat for benthic organisms and fish spawning and foraging.

2. INTRODUCTION

2.1 PROJECT RATIONALE AND GOALS

Conversion of rural land to urban and suburban use produces important hydrologic changes in developing drainage basins. These changes alter the hydraulics, water quality, sediment movement, and habitat characteristics of stream channels draining developed areas. The principal impact of development on drainage basin hydrology is related to the increase in the speed and volume of storm runoff which results from a decrease in runoff storage capacity in developed areas. Development impacts on the rainfall-runoff pattern of a drainage basin produce an increase in the total discharge in stream channels from storms of a given size, as well as larger peak discharges and a shorter time interval between the precipitation and the peak channel discharge (Leopold, 1968).

A direct result of these changes in the hydrology and hydraulics of developed drainage basins is an increase in the magnitude and frequency of flooding in the channels draining developed areas. The altered hydrology also increases the rates of bed material movement in the stream channels, which can lead to undesirable erosion and deposition in the channels. In addition to increased sediment loads, development impairs water quality in stream channels through the rapid transport through the hydrologic system of urban pollutants such as salt, fertilizers, oils, and solvents. The deterioration of water quality, the erosion of channel bed and banks, and the siltation and contamination of the channel bottom sediments all contribute to habitat degradation for riparian wildlife and vegetation (APWA, 1981).

Over the past two decades, attempts to mitigate the effects on stream channels of hydrologic changes induced by development have become both more widespread and complex. Over this period, much of the effort has focused on the construction of stormwater management (SWM) facilities that reduce the peak discharge in the downstream channels. Typical policies have involved reducing the peak discharge for postdevelopment conditions to the level for predevelopment conditions for a storm or storms of a given return interval. Peak-discharge controlling policies serve to reduce flooding along channels draining developed areas. In the past decade, however, it has been suggested that peak discharge policies do not mitigate the impacts of development on water quality (AWPA, 1981; Schueler, 1987) and channel erosion (McCuen, 1979).

Water quality and channel erosion SWM designs both focus on more frequent, smaller storms than those typically addressed by peak discharge policies, reflecting an emphasis on in-channel processes and conditions, rather than the control of less frequent, out-of-bank flow events. In addition, both water quality and channel erosion criteria lead to designs that require a greater

storage of storm runoff, suggesting that a SWM plan designed to mitigate impacts on either water quality or channel erosion will also contribute to the mitigation of the other. Despite these similarities, however, water quality and channel erosion SWM designs depend on very different processes. Water quality designs seek increased runoff detention times and operate on an assumption that the adsorption and settling of pollutants in a SWM facility will reduce the amount of pollutants released downstream. A greater detention time is presumed to permit more pollutants to settle from the stormwater. Such a design may require the retention of a prescribed volume of runoff (e.g. 0.5 inch), or the detention of the design storm runoff for a prescribed period of time (e.g. 24 hours). In Maryland, the latter design has been adopted as a requirement for new land developments (MD, 1983). A SWM facility designed to provide erosion control must account for an entirely different set of processes related to the magnitude and frequency of flows in the channels draining developed areas. The estimated bed material load in the receiving channel may be used as a surrogate for channel erosion, as done in this report. An erosion control SWM design is then based on limiting postdevelopment bed material load to predevelopment levels for a design storm. Because bed material load varies in a complex and nonlinear fashion with storm discharge, the steps leading to an erosion control SWM design are different from those required for a water quality design.

Because the processes that must be controlled to improve water quality and reduce channel erosion impacts are complex and very different, it is not possible to readily observe that a SWM design that provides control of one (e.g. water quality) will also provide control of the other (e.g. channel erosion). Explicit consideration of the processes controlling water quality and channel erosion is necessary if SWM facilities are to be designed to control more than peak discharge. Because few SWM designs have been developed that explicitly provide erosion control, it can not be clearly demonstrated how much erosion control a water quality SWM plan will provide. The goal of this report is to demonstrate alternative detention pond designs that not only provide downstream erosion control, but also satisfy the existing design standards for SWM facilities. By preparing SWM designs that incorporate erosion control as an explicit design criterion, the relation between erosion control designs and other SMW strategies can be more accurately examined.

2.2 ANACOSTIA RIVER BASIN

The Anacostia River Basin is located in Montgomery and Prince George's Counties, Maryland, and the District of Columbia (Figure 2.1). The Anacostia River is a tributary of the Potomac River and drains an area of 170 square miles. The basin is heavily urbanized; much of the development took place before SWM facilities were in wide use. The goals and design of existing SWM facilities in the Anacostia Basin are being reexamined with the intention of improving water quality and reducing sedimentation in the river channels. Despite the impact of

The Anacostia Basin
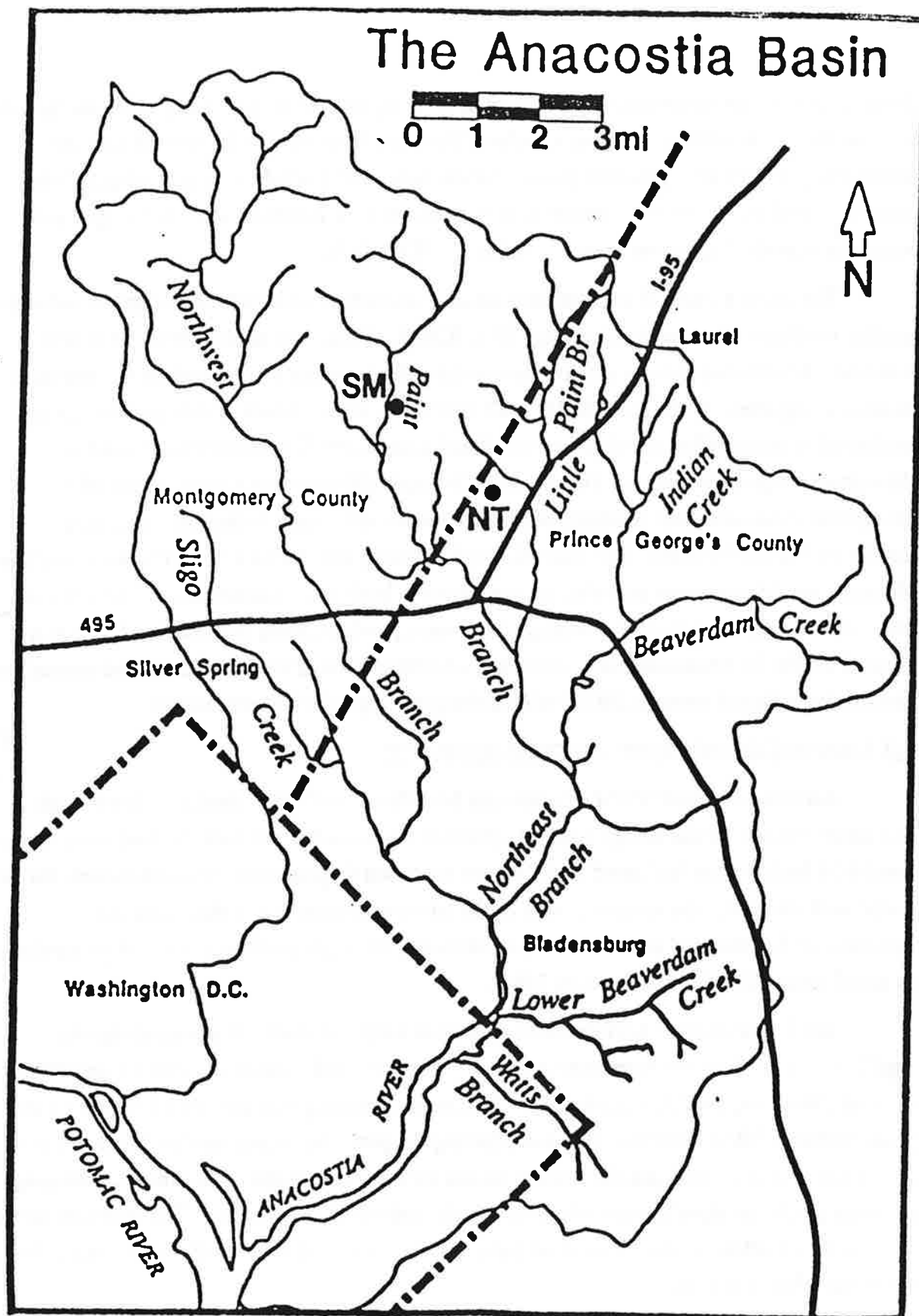
2.1   Map of the Anacostia River Watershed. The locations of the two
       demonstration sites used in this report are shown: SM is Snowdens Mill,
       NT is Newport Towne.

development on the water quality and habitat in the Anacostia River, a diversity of aquatic species may be found. In addition to being a preferred Potomac River nursery for striped bass, the Anacostia provides key spawning grounds for migratory fish (including river herring and white perch), as well as habitat for rainbow trout, largemouth bass, redbreast sunfish, bluegill, and a naturally reproducing brown trout population (ICPRB, 1989).

The lower 8 miles of the Anacostia are tidal and subject to considerable siltation and water quality problems (Century Engineering, 1981; ICPRB, 1988). The remainder of the river is nontidal. Accelerated channel erosion has been identified at many locations along the Anacostia (Century Engineering, 1981; MCDEP, 1987; MWCOG, 1988). Much of this erosion may be attributed to uncontrolled development runoff and poses a significant continuing hazard to downstream aquatic resources. The Anacostia Watershed Restoration Agreement of 1987 recognizes the deteriorated condition of stream channels and aquatic habitat resulting from accelerated rates of erosion. The State of Maryland, along with the District of Columbia and Prince George's and Montgomery counties, are committed to basin-wide management of erosion and sedimentation by the turn of the century. This report provides, using two existing SWM sites located within the Anacostia River watershed, a demonstration of the reduction in bed material load that can be achieved through SWM facilities designed to provide erosion control.

2.3 EROSION CONTROL STORMWATER MANAGEMENT

An erosion control SWM criterion may be defined in a fashion similar to that for peak discharge control: for the design storm, the permissible amount of bed material load downstream of the SWM facility must be limited to the amount estimated for predevelopment conditions. For the purpose of this study, the erosion control SWM criterion adopted is one that states that downstream bed material load for the 2 year storm can be no greater than that which occurred prior to development (McCuen and others, 1987).

The 2 year storm is selected based on general empirical studies that suggest that the equilibrium size of natural river channels tends to correlate with a discharge with a 2 year return interval (Wolman, 1967; Leopold, 1973). This storm frequency was used as a basis for a 2 year peak discharge SWM criterion. However, limiting the peak discharge to predevelopment 2 year levels does not in general provide erosion control for the 2 year storm. In addition to increasing the peak discharge, development also increases the volume of storm runoff. The increased runoff volume may produce increased levels of bed material load even if the peak discharges are limited to the predevelopment case.

The concepts and goals of erosion control SWM have been discussed previously by Booth (1988), MacRae and Wisner (1989), McCuen (1979), McCuen and Moglen (1988), and Schueler

(1987). The case for an erosion control SWM design can be made most clearly in a comparison with a design that provides for peak discharge control for a 2 year design storm. Figure 2.2a presents four different hydrographs for a 2 year storm: predevelopment, postdevelopment with no SWM control, postdevelopment with 2 year peak discharge control, and postdevelopment with 'overcontrol' to limit the total amount of bed material load to predevelopment levels. The figure is based on conditions at Newport Towne, a SWM facility draining 22 acres of townhouse development in Prince George's County. The hydrographs shown in Figure 2.2a are computed using standard SCS methods for a Type II design storm with a 2 yr return interval (SCS, 1982). In Figure 2.2, bed material movement is estimated to begin at a discharge of slightly greater than 2 cfs. The existence of such a discharge level, termed the critical discharge, is a crucial factor in developing an erosion control SWM design. The rate of bed material transport is a nonlinear function of the channel discharge. To determine the total bed material load during a storm, the instantaneous transport rate must be integrated over the entire hydrograph. Figure 2.2b presents the bed material load for the four hydrographs in Figure 2.2a. The role of the critical discharge is evident in this figure as the time when the bed material load becomes nonzero. The total bed material load, integrated over the entire storm hydrograph, is shown in Figure 2.2c for the cases considered. It can be seen that even though the 2 year peak discharge design limits the peak water discharge to predevelopment levels, the increased duration of postdevelopment flow above the critical discharge produces greater total bed material load. The erosion control design limits the postdevelopment bed material load to predevelopment levels by reducing the peak discharge as well as the the duration of flows above the critical discharge for incipient bed material transport.

2.4. REPORT OVERVIEW

In this study, erosion control designs are developed for two demonstration sites in the Anacostia River watershed. In addition to erosion control, these designs comply with current SWM regulations in force in the region. These requirements include peak discharge control of the 2 and 10 year storms to predevelopment levels, 24 hour detention of the 1 year postdevelopment storm (one of several options available for water quality control), and design of an emergency spillway for the 100 year storm according to SCS requirements (SCS, 1981). To provide this compliance with existing SWM policy, the erosion control designs use event-based hydrology (SCS 1982, 1986). To provide an estimate of channel erosion under alternative SWM designs, total bed material load for a design storm is used as a surrogate for channel erosion.

In developing the erosion control designs, a variety of outlet structure alternatives were considered and the size and elevation of the outlet structures were varied to locate the optimum erosion control design. The response of peak water discharge, pond water level, pond drawdown time, and bed material load to the variation in outlet type, size, and elevation was examined. This
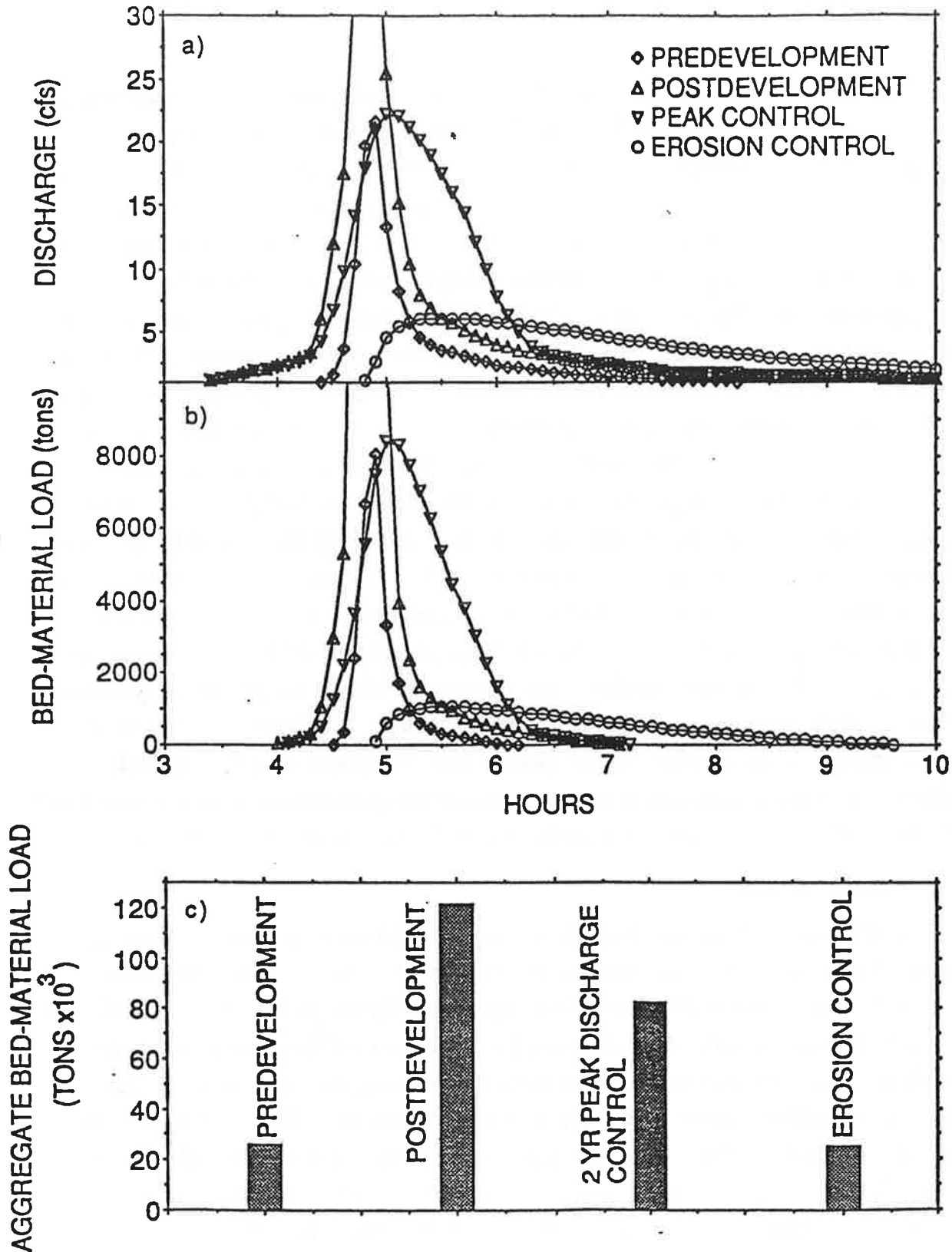
2.2　Relation between channel discharge and sediment load for predevelopment and postdevelopment conditions in a 22 acre drainage area in Prince George's County. Discharge and load are shown for the case with no detention pond, with a pond that provides 2 yr and 10 yr postdevelopment peak discharge control, and with a pond that provides erosion control for the 2 yr postdevelopment storm. The same runoff volume is used for all postdevelopment storms.

investigation provides an opportunity to not only identify an optimal design that satisfies both erosion control and the current SWM criteria, but also to contrast the requirements for an erosion control design with those for designs that satisfy only the existing SWM criteria.

Sections 3 and 4 of this report provide background on the conditions of stream channels in the Anacostia basin and the current SWM regulations in effect in the basin. Section 5 describes the methodology used in this study. The erosion control designs for the two demonstration sites are developed in Section 6 and the amount of bed material load reduction possible with these alternative designs is estimated. Section 7 evaluates the sensitivity of erosion control designs to the choice of pond outlet structures and the choice of formula for computing the bed material load in the receiving channel. Section 7 also provides an estimate of the erosion control provided by the designs developed in Section 6 for storms other than the 2 year design storm. Design suggestions for developing erosion control SWM designs are presented in Section 8.

3. CHANNEL EROSION AND SEDIMENTATION IN THE ANACOSTIA .BASIN

3.1. CAUSES OF CHANNEL EROSION

Channel erosion is the net removal of sediment by the mechanical action of fluid flow and bank processes. Sedimentation is the net deposition of sediment due to mechanical forces. A channel in equilibrium by definition has no net removal or deposition of sediment. An actively eroding channel is a channel that is out of equilibrium. A channel may be out of equilibrium due to changes in either runoff or sediment supply.

Several aspects of an urbanized watershed affect the hydrology in a fashion that can increase channel erosion. First, the percentage of impervious area is increased, primarily due to the addition of pavement and roofs and the removal of vegetation. Second, the urbanized catchment has a constructed drainage system (e.g. a storm sewer system) that speeds the transfer of stormwater to the channel. Third, natural sources of stormwater storage, such as in shallow depressions or in the soil, are lost to grading and compacting of soils in developed areas. An urban drainage basin will respond much more quickly to storms than a rural catchment of the same area, slope, and soils. Additionally, the runoff volume will be greater for an urban catchment. The increase in runoff volume and the decrease in watershed response time combine to produce a peak discharge that is much greater than the predevelopment case. If stormwater in an urban catchment is unmanaged, the increase in runoff volume can lead to channel erosion.

3.2 SURVEY OF CHANNEL EROSION IN THE ANACOSTIA RIVER BASIN

As background for this study on erosion control SWM design in the Anacostia Basin, this section presents a survey of sites at which channel erosion has been identified. The survey is organized into the ten subwatersheds of the Anacostia Basin shown in Figure 2.1. An overall summary of identified erosion sites in the Anacostia is presented first, followed by a brief description and listing of the sites. The detailed listing is organized in an upstream-to-downstream order for each subwatershed.

Information on channel erosion in the Anacostia Basin is available from several sources. The first is a report prepared for the Washington Suburban Sanitary Commission entitled "Anacostia Watershed Erosion-Sedimentation Control Study" by Century Engineering in 1981. This report was commissioned to examine sediment sources upstream of the Bladensburg Marina, which is located near the District of Columbia on the tidal portion of the Anacostia (Figure 2.1). In addition to identifying sites of erosion and deposition, Century Engineering produced estimates for each subwatershed of the amount of sediment delivery by streambank

erosion and bed material load. These estimates were developed using computations of sediment yield from each subwatershed and channel scour/deposition estimates from the HEC-6 program (USCOE, 1977). Significant changes in land uses and management practices since 1981 may affect the value of these estimates for the relative targeting of resources. Additional sites of erosion in the Montgomery County portion of the Anacostia Basin are identified on the Water Resources Map of Montgomery County (MNCPPC, 1988), the Montgomery County Stormwater Management Map (MCDEP, 1987), and the Montgomery County Stormwater Retrofit Inventory (MWCOG, 1988). Further information on channel erosion was collected from personal communications with John Galli of the Metropolitan Washington Council of Governments and Jim Cummins of the ICPRB.

Summary

To provide a general picture of channel erosion in the watershed, a summary of all identified erosion in the Anacostia Basin is presented in Table 3.1. Channel erosion is summarized for each of the ten subwatersheds. In each subwatershed, the total length of all channels (including tributaries) was measured from U.S. Geological Survey 7.5 minute topographic maps. Then the total length of identified eroded channel was computed. To provide an idea of the extent of erosion in mainstem channels and in smaller channels, for which SWM facilities might provide some erosion control, the total length of identified eroded channel was then subdivided into two categories based upon channel length. Small channels are those with a length of less than 1.4 miles from their upstream end as noted on the USGS map. This channel length corresponds to a drainage area of approximately 1 square mile (Hack, 1957). Observed channel erosion in small channels was further subdivided based on the presence or absence of an existing SWM facility. Larger channels are those with lengths greater than 1.4 miles.

The primary locations of channel erosion in the Anacostia are in larger channels (75%), predominantly in the mainstem of the major tributaries, and in small channels downstream of developed areas with uncontrolled stormwater runoff (20%). Although these results do not represent a survey explicitly designed to address the causes of channel erosion, they do suggest that channel erosion in the Anacostia is primarily related to uncontrolled storm runoff, either locally where no SWM is provided, or over a broader area where the integrated effect of insufficient runoff control is observed.

Table 3.1

Summary of Channel Erosion in the Anacostia River Basin

Subwatershed	Total Channel Length (mi)	Identified Eroding Channel Length (mi)	% of Eroding Channel Length: Large Channels	% of Eroding Channel Length: Small Channels with SWM	% of Eroding Channel Length: Small Channels without SWM
UNB	26.0	6.1	66	3	31
LNB	13.5	5	100	0	0
SC	13.0	1.3	8	0	92
PB	30.0	3.5	42	9	49
LPB	9.0	0.8	100	0	0
IC	18.5	2.8	71	29	0
BC	24.0	0.1	100	0	0
NEB	6.8	0	0	0	0
LBC	12.0	3.0	100	0	0
UAR	8.5	2.0	100	0	0
TOTAL	164.3	24.6	75	5	20

Key: UNB: Upper Northwest Branch; LNB: Lower Northwest Branch; SC: Sligo Creek; PB: Paint Branch; LPB: Little Paint Branch; IC: Indian Creek; BC: Beaverdam Creek; NEB: Northeast Branch; LBC: Lower Beaverdam Creek; UAR: Upper Anacostia River.

Upper Northwest Branch Subwatershed

The Upper Northwest Branch flows in a southerly direction in Montgomery County from its headwaters to an old USGS gaging station near Old Randolph Road (USGS No. 01650500). The length of the Upper Northwest Branch is approximately 7 miles and its drainage area is approximately 21 square miles. Rainbow Trout are stocked in the Upper Northwest Branch, which supports a put-and-take fishery, but not Trout reproduction. Trout reproduction appears to be limited because of sedimentation (ICPRB, 1989a). Batchellors Run and Bel Pre Creek are the major tributaries of the Upper Northwest Branch. In 1981, Century Engineering described the channels in the Upper Northwest Branch Subwatershed as stable

natural channels with little evidence of erosion. However, later sources (MCDEP, 1987; MCNPPC, 1988; MWCOG, 1988) identify accelerated stream erosion in several locations.

Streams with identified erosion in the MWGOG inventory (1988) include Batchellors Run (0.5 miles of moderately to severely eroded channel within Batchellors Forest, Site NWB-1, and 0.3 miles of erosion just upstream of Layhill Road, site NWB-2), the Woodlawn Park tributary (0.3 miles of moderate to severe erosion downstream of an existing farm pond, site NWB-3), the mainstem of the Northwest Branch (approximately 1 mile of severe channel erosion within the Northwest Branch Golf Course, about half of which is downstream of a golf course pond, site NWB-4), and a small tributary near the proposed Inter-County Connector Highway crossing in Layhill Square (0.3 miles of moderate to severe channel erosion and adjacent erosion of 0.1 miles of the mainstem of the Northwest Branch, Site NWB-8).

Moderate to severe erosion has been identified in Bel Pre Creek from its confluence with the Upper Northwest Branch to a point approximately 2 miles upstream. Approximately 0.5 miles of eroding channel has also been identified within the Argyle Country Club. Accelerated stream erosion has been identified along approximately 0.5 miles of a small unnamed tributary of Bel Pre Creek draining the Glenmont Metro area. There is also an isolated site of accelerated erosion 0.1 miles long on the Upper Northwest Branch just west of the end of Hawkesbury Terrace (MCDEP, 1987). There is 0.2 miles of slight to moderate channel scouring of a small tributary draining the Dumont Oaks I SWM pond (Site NWB-31 in MWCOG, 1988). Approximately 0.3 miles of a small tributary draining a subdivision near Lockridge Drive suffers from moderate to severe channel erosion (Site NWB-34 in MWCOG, 1988).

Most erosion sites in the Upper Northwest Branch Subwatershed are not directly downstream of existing stormwater management ponds. The location of erosion sites near developed and developing areas suggests that channel erosion in the Upper Northwest Branch results primarily from uncontrolled runoff from urbanized land. Streambank erosion has been estimated to deliver approximately 2,700 tons/year and bed material load about 1,200 tons/year (Century Eng., 1981).

Lower Northwest Branch Subwatershed

The Lower Northwest Branch flows through highly developed residential and commercial areas of Montgomery County and Prince George's County. The stream flows southeasterly for 10.5 miles and has a drainage area of approximately 30 square miles (including Sligo Creek). The Lower Northwest Branch begins at the cut-off point of the Upper Northwest Branch near Old Randolph Road (old USGS gaging station No. 01650500) and ends at the Hyattsville gaging station on Queens Chapel Road (USGS No. 01651000). The two main

tributaries of the Lower Northwest Branch are Sligo Creek and its largest tributary, Long Branch. Sligo Creek and Long Branch are discussed in the Sligo Creek subwatershed section of this report. In the past, the Lower Northwest Branch has been an important spawning area for anadromous fish (shad, herring, and yellow perch), although spawning is now constrained by flood-control barriers (ICPRB, 1988).

The lower part of Northwest Branch flows through a narrow, steep-sided valley. The channel is lined with rocks, both natural and installed riprap. Erosion occurs downstream of Randolph Road for about 2 miles (Century Eng., 1981; MCDEP, 1987, MNCPPC, 1988) and downstream from Riggs Road (near Adelphi Mill) for approximately 3 miles to the boundary with the Upper Anacostia River in Hyattsville, which lies within the Northwest Branch Stream Valley Park (Century Eng., 1981). Streambank erosion, including Sligo Creek, is estimated to be approximately 2,700 tons/year and bed material load is estimated to be 1,700 tons/year (Century Eng., 1981).

Sligo Creek Subwatershed

Sligo Creek is a tributary of the Lower Northwest Branch. It is approximately 7 miles long. Sligo Creek drains one of the most urbanized areas in the Anacostia Basin. It flows mostly through Montgomery County and into Prince George's County just below the county border. The stream originates in Wheaton and is lined with woodlands that are part of a public park. The lower reaches from New Hampshire Avenue to the confluence with the Northwest Branch are channelized. Long Branch is lined by a narrow strip of public parks and urban woods, although the surrounding area is entirely urbanized. Of the four major Montgomery County tributaries to the Anacostia (Sligo Creek, Northwest Branch, Paint Branch, and Little Paint Branch), Sligo Creek has the most degraded water quality, the least diverse fish population, and the most unstable flow regime. The paucity of fish in Sligo Creek may be attributed to toxic contamination of the water and stream bed, unusually high stormwater discharges, or stream blockages (ICPRB, 1989a).

Most of Sligo Creek within Montgomery County is either riprapped or otherwise protected from erosion, or is scheduled to be protected. Those portions that are not protected suffer from accelerated stream erosion (MCDEP, 1987). Most of this erosion may be attributed to uncontrolled stormwater draining urbanized areas.

There is moderate stream channel erosion upstream of the existing SWM pond (located just south of the Kemp Mill Shopping Center) in Sligo Creek Park (Site SL-2 in MWCOG, 1988). The length of this eroded reach is approximately 0.2 miles.

Wheaton Branch is the most severely eroded tributary within the Sligo Creek drainage. This is most evident in the remaining natural portion of channel located below the Dennis Avenue/Wheaton Branch Regional SWM Facility. This lower Wheaton Branch stream reach, which is approximately 0.4 miles long, suffers from extreme channel widening and streambank erosion. The contributing drainage area to this channel is approximately 1000 acres, which is 55% impervious. This reach is typical of many smaller streams in the Maryland Piedmont that have been heavily impacted from uncontrolled urban stormwater runoff. Channel stabilization of this section of Wheaton Branch and a retrofit of the Wheaton Branch SWM facility are presently being conducted by Montgomery County as part of the Coordinated Anacostia Retrofit Program (CARP) to improve water quality and erosion control. This active program provides an opportunity for the designs discussed in this report to be included in ongoing retrofit projects.

Streams with identified erosion in the MWGOG inventory (1988) include a small unnamed tributary near Flora Lane (0.2 miles of moderate to severe channel erosion, site SL-7), the Woodside Tributary off of Edgevale Road (0.2 miles of moderate to severe channel erosion, site SL-9), the mainstem of Sligo Creek (0.1 mile of slight to moderate bottom scouring near Lycoming Street, site SL-10) and the Bennington Tributary (0.2 miles of moderate erosion and channel incision, site SL-11). All of these sites are in areas with no local SWM facilities.

Paint Branch Subwatershed

The Paint Branch originates in the vicinity of Spencerville, Maryland, and flows generally southeast through Montgomery and Prince George's counties for approximately 13.5 miles until it flows into Indian Creek. It has a drainage area of 31 square miles. The Paint Branch and its tributaries are classified by the State of Maryland as Class III, Natural Trout Waters, from their headwaters to the Capital Beltway (I-495). These are the only Class III waters within the Anacostia drainage and contain a reproducing brown trout population. Historical trends in the Trout population show a decline, however, apparently in response to increased development (primarily residential) in the subwatershed (ARC, 1986).

Several isolated sites of accelerated erosion due to uncontrolled stormwater runoff are located on small tributaries of Paint Branch. These include an unnamed tributary just east of the intersection of Lemon Tree Lane and Collingwood Terrace (approx. 0.3 miles of accelerated channel erosion), a small unnamed tributary just north of the intersection of the Paint Branch and Fairland Road (approx. 0.3 miles of accelerated channel erosion), and an unnamed tributary approximately 0.1 miles southeast of the end of Tech Road (approximately 0.3 miles accelerated channel erosion). Additionally, there is accelerated bank erosion and channel incision on the

small tributary (approx. 0.3 miles long) that drains the Snowdens Mill SWM facility, one of the demonstration sites examined in this study. Very little erosion occurs between Fairland Road and Colesville Road on Paint Branch. Substantial deposition occurs downstream of Colesville Road.

Hollywood Branch, a tributary of Paint Branch with a length of approximately 1.5 miles, is extensively eroded as a result of almost no stormwater control in an urbanized area (MCDEP, 1987, MNCPPC, 1988). Accelerated channel erosion occurs for approximately 0.2 miles at the confluence of the Paint and Little Paint Branches. For approximately 0.1 miles of channel, a high unstable cut bank is severely eroding just downstream of University Boulevard. Another site of accelerated erosion is identified further downstream, from Metzerott Road to Route 1 (approx. 0.6 miles).

To relieve flooding on Paint Branch below U.S. Route 1, an overflow cutoff channel was added in the early 1970's by the Corps of Engineers. This channel was intended to give relief to the natural stream during high flows. However, in 1975, when Tropical Storm Eloise caused severe flooding, the spillway that controlled overflow to the cutoff channel was eroded. Since then, the stream has remained in the overflow channel and has abandoned the natural stream course. The overflow channel has eroded in its upstream portion and has deposited sediment downstream of the railroad bridge, approximately 3000 feet below the Route 1 bridge. A failing concrete drop structure approximately 1000 ft upstream of the confluence of Paint Branch and Indian Creek causes accelerated bank erosion for approximately 0.1 miles of channel. Bank erosion on Paint Branch is estimated to be approximately 400 tons/year and bed material load contributions add an estimated 2,700 tons/year (Century Eng., 1981).

Little Paint Branch Subwatershed

Little Paint Branch is a tributary of Paint Branch. Most of Little Paint Branch flows through Prince George's County, although some of its headwaters occur in Montgomery County. High water temperatures and sedimentation contribute to the absence of trout reproduction on Little Paint Branch (ICPRB, 1989a). The stream length from a point east of Route 29 and Greencastle Road to the confluence with Paint Branch is approximately 6.4 miles. The stream has been channelized from Briggs-Chaney Road downstream approximately 2 miles to Interstate 95. A short stretch of alternate erosion and deposition exists just north of Interstate 95. Channel erosion has been accelerated on Paint Branch above the Capital Beltway (I-495) by the use of all terrain vehicles in and near the channel. Approximately 0.8 miles of channel in Little Paint Branch Park from Marie Street to Sellman Road shows moderate to severe channel erosion. Erosion at the confluence of Paint and Little Paint branches is described earlier in this

report, in the Paint Branch subwatershed section. Total channel erosion is estimated to be approximately 400 tons/year with bed material load contributions totalling 2,500 tons/year (Century Eng., 1981).

Indian Creek Subwatershed

The length of Indian Creek from its headwaters to the confluence with Paint Branch is 8 miles. The general direction of Indian Creek is southeasterly. It has a drainage area of approximately 29 square miles and is located entirely in Prince George's County. Land use within this subwatershed varies from large tracts of woodlands in the north, agricultural areas in the central region, and urban-industrial areas to the southeast. Over 1000 acres of the Indian Creek subwatershed consists of mined land, a major sediment source. The stream meanders naturally through relatively wide valleys.

Erosion occurs on the Ammendale Tributary of Indian Creek for approximately 0.8 miles from Industrial Drive downstream to the confluence with the main stem of Indian Creek. Indian Creek is channelized in Beltsville between a point approximately 0.2 miles upstream of its confluence with the Ammendale Tributary and the Old Baltimore Pike. Approximately 0.5 miles downstream of the end of the channelized section is an active beaver dam (located about 0.2 miles west of the intersection of Powder Mill and Edmonston Roads). Approximately 2.5 miles of channel below the beaver dam is a wetland. Moderate channel erosion occurs along approximately 2 miles of channel, beginning 0.2 miles downstream of the beaver dam. This eroding section extends 0.6 miles past the confluence of Indian Creek and Beaverdam Creek. The reach of Indian Creek from Beaverdam Creek to Greenbelt Road (approx. 1.8 miles) is subject to extensive deposition. South of Greenbelt Road, the existing stream is channelized and is developing meanders by depositing material on alternate banks. Many of the tributaries of Indian Creek are quite muddy, largely due to sediment input from construction sites and surface mines. Total bank erosion for Indian Creek to its confluence with Paint Branch is estimated to be 600 tons/year and bed material load is estimated to be 1,500 tons/year (Century Eng., 1981).

Beaverdam Creek Subwatershed

Beaverdam Creek flows approximately 5 miles from its headwaters to its confluence with Indian Creek and is entirely located within Prince George's County. Woodlands and agriculture are the dominant land use in the Beaverdam Creek subwatershed. Most of the land is owned by the Federal Government and is associated with the Beltsville Agricultural Research Center. The very gentle gradient of Beaverdam Creek has resulted in wetland areas and a meandering channel. There is approximately 0.1 miles of streambank erosion on Beaverdam

Creek at the confluence with Indian Creek. There is a population of Brook Lamprey in Beaverdam Creek, an indication of high water quality. Streambank erosion is estimated to be 400 tons/year and annual bed material load is estimated to be 300 tons (Century Eng., 1981).

Northeast Branch Subwatershed

The confluence of Paint Branch and Indian Creek forms the Northeast Branch which drains an area of approximately 75 square miles. This subwatershed includes the Northeast Branch from its headwaters to the Riverdale gaging station on Riverdale Road. The main channel is 4.8 miles long. It flows through urban parkland and is channelized in the vicinity of East-West Highway. Highly developed urban areas cover the southern and eastern portions of this subwatershed, many of which were developed extensively prior to SWM regulations. These include College Park, Riverdale, and Lanham.

This portion of the Anacostia watershed receives flows from Paint Branch, Little Paint Branch, Indian Creek, and Beaverdam Creek. In order to convey this water the Northeast Branch channel is much wider than its tributaries, being up to 200 feet wide. Century Engineering (1981) does not identify any sites of extensive accelerated erosion on the Northeast Branch. An estimated 1,600 tons/year is produced by channel bank erosion and an estimated 1,500 tons/year is contributed by bed material load (Century Eng., 1981).

Lower Beaverdam Creek Subwatershed

Lower Beaverdam Creek enters the main stem of the Anacostia River just upstream of the Maryland/D.C. border. Lower Beaverdam Creek is a separate tributary of the Anacostia and is not a lower portion of Beaverdam Creek. It is approximately 6 miles long and flows entirely within Prince George's County. This subwatershed is extensively urbanized and most of the development took place prior to SWM regulations. This is the most degraded tributary in the Anacostia Basin in terms of water quality, fish habitat, and stream channel erosion (J. Cummins, 1989, personal communication). Approximately 1 mile of Lower Beaverdam Creek between the Capital Beltway (I-95) and Route 50 has been stabilized and is adjacent to the site of a new business park. Approximately 3 miles of the channel has severe bank erosion and many resultant fallen trees. This area of extensive erosion begins near Route 202 and continues downstream until stabilized areas are reached near the confluence with the main stem of the Anacostia.

Upper Anacostia Subwatershed

The Upper Anacostia drainage basin includes an area of approximately 8 square miles. It includes the downstream portions of the Northwest and Northeast Branches, as well as the

section of the Anacostia River to the north of the boundary between the District of Columbia and Prince George's County. Total stream length is approximately 5 miles.

The channel of the Northwest Branch is stable and shows little erosion or deposition. Severe bank erosion occurs along the Northeast Branch downstream of Riverdale for approximately 2 miles. This is a constructed channel flowing between two artificial levees. Some deposition occurs in the upstream reach. Alternating erosion and deposition may be observed between the Bladensburg Marina and the confluence of the Northwest and Northeast Branches. Extensive dredging has been required at the Bladensburg Marina. The total channel erosion is estimated to be 1,700 tons/year and bed material load is estimated to be 700 tons/year (Century Eng., 1981).

4. STORMWATER MANAGEMENT PRACTICE IN THE ANACOSTIA BASIN

This section gives an overview of previous and current SWM practice in the Anacostia River Basin. The section provides background on the regulatory environment in which erosion control SWM design must be considered, and gives an indication of the types of SWM facilities that exist in the basin.

4.1 PREVIOUS AND CURRENT SWM POLICY

The Maryland Sediment Control Act of 1970 (House Bill 1151) requires the appropriate soil conservation district to approve sediment control plans in connection with land clearing, soil movement, and construction (M.A.S.C.D., 1987). The Attorney General of the State of Maryland interpreted the Maryland Sediment Control Act to include control of off-site erosion and subsequent sedimentation resulting from increased stormwater runoff generated by development. From this interpretation has evolved the concept of stormwater management that does not allow the peak rate of discharge after development to exceed the peak rate of discharge prior to development for a particular frequency of storm or storms. This concept was codified into law in 1982 with the passage of the State Stormwater Management Act. The regulations developed pursuant to the act require control of the peak discharge at predevelopment sites and water quality enhancement through the use of infiltration or retention measures wherever feasible.

Using the peak discharge criterion, stormwater management facilities are designed so that the peak discharge after development is less than or equal to the predevelopment peak discharge for the same frequency storm event. For example, present State of Maryland regulations require that SWM facilities limit both the 2 and 10 year storm postdevelopment peak discharges to their respective predevelopment discharges (MD, 1983). This type of peak discharge control may be designated as 2/2, 10/10 peak discharge control.

Peak discharge control is an effective flood-control strategy. In recent years, however, other SWM criteria have been developed, including water quality and erosion control. One peak discharge strategy that has some water quality and erosion control benefits is the attenuation the postdevelopment peak discharge for a given storm to be equal to or less than the predevelopment peak discharge for a more frequent storm. For example, a SWM facility could be designed so that the postdevelopment 10 year storm peak discharge is equal to the predevelopment peak discharge for a 2 year storm (2/10 control).

Several alternatives to peak discharge control have been proposed and are described in Table 4.1. One alternative is bed material load control (McCuen and Moglen, 1988). Bed material load before and after development can be estimated and used as a surrogate for channel erosion in a

SWM design that minimizes downstream channel erosion. The detention facility may be designed so that postdevelopment bed material load does not exceed the predevelopment bed material load for a design storm.

A second alternative to peak discharge control is runoff volume control. Using runoff volume control, a specified volume of runoff is discharged over a specified duration. For example, 0.5 in. of watershed area runoff would be released over the duration of a 2 year 24 hour design storm. The storage capacity and outlet design of the detention pond can be designed to limit the discharge to this specified rate.

A third alternative to peak discharge control is detention time control. By permitting settlement of suspended solids, an increase in detention time provides water quality benefits. Increased detention time may also provide erosion control benefits. Detention time may be defined in different ways. One definition of detention time is a minimum time interval (e.g. 24 hours) between the centroids of the inflow and outflow hydrographs.

A fourth alternative to peak discharge control is retention control. Retention is the holding of a specified volume of runoff within a SWM facility. Water is not discharged from the structure except through evaporation and infiltration. By permitting settlement of suspended solids, retention control provides water quality benefits. Retention control may also provide erosion control benefits.

TABLE 4.1

STORMWATER MANAGEMENT CRITERIA

TYPE	DESCRIPTION
Peak Discharge Control	Most common criteria; effective in limiting out-of-bank flows; consists of attenuating postdevelopment peak discharge to desired value (usually predevelopment level) of storms of selected frequencies.
Bed Material Load Control	Facility designed to limit postdevelopment bed material load to the value of predevelopment bed material load for given storm frequency.
Runoff Volume Control	Specified volume of runoff (e.g. 0.5 in.) released over a specified time (e.g. duration of 24 hour design storm). This control results in extended detention of the runoff with water quality and potential erosion control benefits.
Detention Time Control	Time interval between inflow and outflow hydrographs is specified. Increased detention time is used to improve water quality and may also have erosion control benefits.
Retention Control	A specified runoff volume is retained in the facility, allowing water discharge only through evaporation and infiltration. Retention is used to improve water quality and may also have erosion control benefits.

Since the Sediment Control Act of 1970, the State of Maryland has promulgated more detailed rules and regulations on the management of stormwater. As a result of the Stormwater Management Act of 1982, the Code of Maryland Regulations (MD, 1983) state that the primary goal of state and local stormwater management programs is to "maintain after development, as nearly as possible, the predevelopment runoff characteristics, and to reduce stream channel erosion, pollution, siltation and sedimentation, and local flooding." To implement these goals the State mandated that each county and municipality adopt stormwater management ordinances requiring both peak discharge and water quality control by July 1, 1984. The minimum control requirements established by the State varied somewhat depending upon the county and municipality but are the same for Montgomery and Prince George's counties, in which the Maryland portion of the Anacostia watershed lies.

Prince George's and Montgomery counties and their incorporated municipalities must, as a minimum, require that the postdevelopment peak discharges for a 2 year and 10 year frequency storm event be maintained at a level equal to or less than the respective 2 and 10 year predevelopment discharge rates. This is to be accomplished by stormwater management measures that control the volume, timing, and rate of runoff.

All developments must comply with the State code except for the following: 1) additions or modifications to existing single family detached structures; 2) developments that do not disturb over 5,000 square feet of land area; 3) land development activities that the State Sediment and Stormwater Management Administration determines will be regulated under specific State laws which provide for managing stormwater runoff; or 4) Residential developments consisting of single family houses, each on a lot of 2 acres or greater.

The Regulations adopted by Montgomery County (M.C., 1987) do not differ significantly from the minimum requirements imposed by the State. Prior to 1984, the Montgomery County District Office of the Soil Conservation Service had delegated approval of stormwater management ponds to the County Department of Environmental Protection. After July 1, 1984 the County has required that ponds be built according to Executive Regulation 37-86 (M.C., 1987), which requires 2/2 peak discharge control and mandates water quality control with 24 hour. detention of the 1 year storm or a permanent pool equal to 0.5" of drainage area runoff. The Montgomery County District SCS office still performs dam safety analysis and risk classification, where appropriate. Table 4.2 lists the stormwater management policies for Montgomery County and the approving agencies since the 1970 Sediment Control Act.

In Prince George's County the County District office of the Soil Conservation Service delegated the approval of ponds in Prince George's County to the Washington Suburban Sanitary Commission (W.S.S.C.) until 1984. The Prince George's County's Watershed Protection Branch

of the Department of the Environment assumed responsibility for stormwater management on July 1, 1984. The Prince George's County District SCS office still performs dam safety analysis and risk classification, where appropriate. Table 4.3 lists the stormwater management policies for Prince George's County and the approving agencies since the 1970 Sediment Control Act.

Table 4.2

Montgomery County Stormwater Policy

Date	Responsible Agencies	Regulation	Description
Before 1984	Montgomery Co. SCS District and Mont. Co. Dept. of Env.Protection	Various Design Guidelines & Memos	Policy varies; generally decided upon a case by case basis.
1984-Present	Montgomery Co. Dept. of Environmental Protection	Mont. Co. Exec. Reg. 37-86	2/2 peak discharge attenuation; Water quality may be achieved with 24 hour. detention of a 1 year storm, or with a permanent pool equal to 0.5" of drainage area runoff, or with infiltration of a 1 inch rainfall

Table 4.3

Prince George's County Stormwater Policies

Date	Responsible Agencies	Regulation	Description
Before 1984	Prince George's Co. SCS District and WSSC	Various Design Guidelines & Memos; WSSC Design Manual	Policy varies; decisions generally made upon a case by case basis.
1984-Present	Prince George's Co. Dept. of Watershed Protection	Prince George's Co. Regulations & WSSC Design Manual	2/2 and 10/10 peak discharge control. Water quality may be achieved by 24 hour. detention of 1 year storm, or with a permanent pool equal to 0.5" of drainage area runoff. In addition, infiltration of the first 0.5 inches of rainfall is required unless exempted.

In addition to the peak discharge and water quality SWM requirements, detention ponds in the Anacostia watershed must have emergency spillways designed for the 100 year storm that comply with the Soil Conservation Service Engineering Standard 378 (SCS, 1981). For ponds draining areas smaller than 320 acres with a dam less than 20 feet in height, the invert of the 100 year emergency spillway must be located at an elevation at least 2.0 feet below the top of the settled dam. In addition, the emergency spillway size and elevation must be chosen so that the maximum water level of the 100 year storm is no more than 1 foot below the top of the settled dam.

In attempting to utilize stormwater management ponds to meet multiple goals, the concept of extended detention has recently received a great deal of attention. Extended detention is advocated as a low cost means of removing particulate pollutants and controlling increases in downstream bank erosion (Schueler, 1987). The Metropolitan Washington Council of Governments is presently completing an inventory of stormwater facilities in the Anacostia Basin that would benefit from a retrofit for water quality and erosion control. Most of the recommendations include conversion of existing stormwater facilities to include an extended detention discharge pattern. MWCOG also advocates that new facilities incorporate extended detention as well as a permanent pool for water quality purposes.

4.2. TYPES OF SWM USED IN THE ANACOSTIA

An estimate of the types of SWM facilities currently in use in the Anacostia River watershed is provided in Tables 4.4 and 4.5. Table 4.4 provides a summary of known SWM facilities in Montgomery County (MCDEP, 1987b, 1988). The Anacostia basin includes approximately 11% of the most developed sections of Montgomery County. In Montgomery County, dry ponds and extended detention ponds compose 53% of the known SWM facilities, and 78% of the known SWM ponds. Table 4.5 provides a summary of known SWM facilities in Prince George's County (PGWPB, 1989). The Anacostia basin includes approximately 19% of the most developed sections of Prince George's County. In Prince George's County, detention and extended detention ponds compose 27% of the known SWM facilities, although the total number of SWM facilities listed includes individual underground storage sites with multiple storage locations as more than one facility. In Prince George's County, detention and extended detention ponds compose 78% of the known SWM ponds.

Table 4.6 provides a summary of SWM sites recommended for retrofit consideration in the Anacostia River Basin by MWCOG (1989). The inventory includes 45 dry pond sites: 27 existing dry ponds for which extended detention control has been recommended, and 18 where construction of a new extended detention dry pond is recommended. The inventory also includes 40 wet pond sites: 25 existing ponds for which extended detention control has been recommended, and 18 where construction of a new wet pond is recommended. These sites, including the 27

existing dry ponds, represent all those for which a judgement has been made that water quality or erosion benefits will result from an extended detention facility. Existing ponds not included on the inventory are generally those that drain less than 10 acres or would not accrue sufficient water quality or erosion control benefits to justify a retrofit project (J. Galli, MWCOG, 1989, personal communication).

Table 4.4

Summary of Known SWM Facilities in Montgomery County (MCDEP, 1987b)

Type	Number	Percent
Dry Ponds	387	51
Extended Detention Ponds	12	2
Wet Ponds	113	15
Infiltration	142	19
Underground Storage	74	10
Marsh	5	1
Parking Lot	1	<1
Roof storage	25	3

Table 4.5

Summary of Known SWM Facilities in Prince George's County (PGWPB, 1989)

Type	Number	Percent
Detention Ponds	65	17
Extended Detention	38	10
Infiltration Pond	31	8
Retention Pond	90	23
Infiltration Trench	39	10
Porous Paving	10	3
Underground Storage	116	30

Table 4.6
Summary of MWCOG Retrofit Inventory Stormwater Management Facilities

	Dry Ponds		Wet Ponds		Wetland/Marsh		Infiltration	
	ED Conv.	New	ED Conv.	New	ED Conv.	New	Existing	New
Prince George's County	4	13	9	9	6	21	1	0
Montgomery County	23	5	16	6	5	7	2	1

5. PROJECT METHODOLOGY

To provide a means of quantifying the impact of SWM design alternatives on downstream erosion, flow, and bed material movement must be computed in the channel downstream of the SWM facility. This section provides an explanation of the computational methods developed for this task. In the following section, these methods are then used to develop erosion control SWM designs for two existing SWM facilities in the Anacostia River Basin. The methodology, and the designs produced with it, use event based hydrology. That is, a design storm is used in computing downstream bed material movement and in developing the erosion control designs. The total bed material load for the design storm is used as a quantifiable surrogate for channel erosion.

5.1. TEST PROGRAM FOR EVALUATING DOWNSTREAM HYDRAULICS AND BED MATERIAL TRANSPORT

To evaluate the effect of pond design on downstream flow and bed material load, a FORTRAN program was written to (1) route a runoff hydrograph through a pond with a particular stage-storage relation and suite of outlet structures, (2) compute the flow depth and velocity downstream of the pond, and (3) integrate the bed material load over the hydrograph released from the pond. The organization of the program and its subroutines are briefly described here. The full program listing is included in Appendix A.

The pond routing portion of the program consists of two subroutines. The first subroutine (OUTLET) computes the discharge associated with each outlet structure as a function of pond water level. Input to the subroutine includes the number, type, size, and elevation of the outlet structures. The outlet structures implemented in the program are

> round orifice
> square orifice
> rectangular orifice
> round vertical riser
> square vertical riser
> straight-crested weir
> V-notch weir

In order to accurately compute the total amount of bed material load over a storm, the entire pond outflow hydrograph must be computed. As a result, the outlet hydraulics subroutine must be capable of computing pond outflow discharge at all elevations in the pond. The subroutine OUTLET provides a complete stage-discharge relation for a given set of pond outlet structures. The subroutine incorporates multiple discharge formulas for most outlets, because each outlet may experience either weir or orifice flow, depending on the elevation of the water surface with respect

to the outlet elevation. The formulas used for the outlet discharge computations, and the rationale for their use, are given in Appendix B of this report.

The second subroutine (ROUTE) performs level-pool routing given an inflow hydrograph, a pond stage-storage curve and a set of pond outlets. Input to this subroutine includes the stage-storage curve for the pond, the stage-discharge relation for the outlet structures, and the inflow hydrograph. Output from this subroutine includes the outflow hydrograph, the peak outflow discharge, the peak water surface elevation in the pond, and the time to drawdown for the pond, which is computed as the time in minutes for the water level in the pond to fall from the peak elevation to a prescribed, small elevation above the pond bottom.

Once the outflow hydrograph for the pond is computed, a third subroutine (TRANS) is used to compute the depth, velocity, and bed material load in the channel downstream of the pond. Depth and velocity are computed using power equations of the form:

$$d = \alpha Q^{\beta}$$

$$V = \gamma Q^{\delta}$$

where d is depth in feet, V is flow velocity in ft/s, and Q is outflow discharge in cfs. Values of the coefficients and exponents in these formulas were estimated using HEC-2 runs for measured channel cross sections at each site. The cross-sections for each site are given in Appendix C. Because of a sharp break in channel side slope at each site, it was found that two sets of coefficients and exponents, one for lower discharges and another for higher discharges, provided a significantly better fit between the power equations and the HEC-2 model results. Details of these computations and the general method used for developing the regime coefficients and exponents may be found in Appendix D.

The bed material load in the channel downstream of the pond is computed using two different transport formulas. Both of these formulas can be used to estimate the amount of bed material load in response to a flow over a given width of an erodible channel and a given duration of time. Although the formulas are based on data that are presumed to represent steady, equilibrium transport conditions, they are generally used to estimate bed material load in unsteady, nonequilibrium conditions as well. One of the formulas used is the Goncharov formula (as cited in McCuen and Moglen, 1988), which can be written as

$$q_b = 168,000(DT) \ W \left(\frac{V}{V_c}\right)^3 \left(\frac{D}{d}\right)^{0.1} (V - V_c)$$

where q_b = bed material load (lbf), DT = time increment (minutes), W = width of active bed (ft), V = flow velocity (ft/s), V_c = critical flow velocity for incipient grain motion (ft/s), and D = representative grain size (ft).

The second formula used is that of Meyer-Peter & Muller (1948), which can be written as

$$q_b = 553(DT)\,W\,(\theta d\,S - \tau_c)^{1.5}$$

where θ = specific weight of water (= 62.4 lbf/ft³), S = channel slope, and τ_c = critical shear stress for incipient grain motion (lbf/ft²). A drag-partition term in the original Meyer-Peter and Muller formula has not been used here. This term serves to reduce the total bed shear stress θds by an amount that is lost to form drag over channel irregularities, leaving only the bed shear stress that is considered to actually drive the sediment transport. This term is not used because it cannot be reliably applied to the type and shape of channels investigated here. Instead, the effect of channel irregularities was partly accounted for by deriving the values of flow depth given in the power equations (which are then used to compute θdS) using only the flow above only the center section of each channel cross-section. These sections include the actively transported bed material and tend to be rectangular or prismatic in shape, for which θdS provides a reasonable estimate of the bed shear stress. Details of the development of the depth power equations are given in Appendix D. The transport subroutine computes the bed material load for fixed time increments (6 minutes was used in this study) and sums these loads over the entire outflow hydrograph to produce an estimate of the total integrated load for an individual storm.

Two different transport formulas were used to evaluate the effect of the choice of transport formula on computed downstream load and erosion control designs. A very large number of bed material load formulas exist in the literature (see Vanoni, 1975, Brownlie, 1981, and Gomez and Church, 1989, for summaries). The exact formula that may be most appropriate for any particular small stream channel, such as those examined in this report, is difficult to evaluate. Because all of the formulas are empirical, and because there is considerable overlap in the data to which the formulas are fitted, the most significant difference between the available formulas is the flow parameter used to estimate the transport rate. Bed material transport is actually driven by a force per unit area, or the bed shear stress τ_o (estimated here as θds), which is used in the Meyer-Peter and Muller formula. The Goncharov formula uses mean flow velocity as the flow variable. Flow velocity is related to the bed shear stress, and hence, the transport rate, as a function of the flow depth and the hydraulic roughness of the channel. Although the flow velocity is only indirectly related to the forces actually driving the bed material transport, it can, in general, be estimated more accurately than the bed shear stress, thus making it a reasonable alternative to a shear-based formulation. Although not included in the program, bed material load computed with additional

formulas, including the formulas of Bagnold (1966), Schoklitsch (Shulits, 1935), and Shields (1936) was also investigated (summaries of these formulas may be found in Vanoni, 1975, and Simons and Senturk, 1976).

Most transport formulations in current use require the specification of a velocity or shear stress at which sediment movement begins. This value is termed the critical velocity or the critical shear stress. Bed material transport is then computed in terms of some measure of the excess shear or velocity above the critical level. Bed material transport is a strong, nonlinear function of the driving force. For this reason, an accurate estimate of critical shear stress or velocity is necessary for accurate transport calculations. Unfortunately, the state-of-the-art in estimating critical shear stress or velocity is very crude, particularly for sediments including a variety of different grain sizes, as is typically found in natural channels (Wilcock and Southard, 1988). For sediments containing a range of sizes from silts and clays to gravel, as is the case with the demonstration sites considered here, very little empirical guidance is available. Values of critical velocity used in this report are taken from the work of Fortier and Scobey (1926). Values of critical shear stress used here are taken from a conversion of the Fortier and Scobey critical velocities made by the U.S. Bureau of Reclamation (a recent summary of these is given by McCuen et al., 1987). The Fortier and Scobey values are based on an extensive survey of channels that are observed to be stable. These values have been used in channel designs with some success over a very long period of time and under a wide variety of circumstances. The Fortier and Scobey values are also appropriate for this study because the Fortier and Scobey values of critical shear stress and velocity are typically back-calculated from the same mean flow parameters that are used to compute bed material load in this study.

In this study, the bed material load integrated over a storm event is used as a surrogate for a direct computation of channel erosion. Channel erosion can consist of either or both of bed and bank erosion, which are caused by a variety of processes. These include downcutting of the sediment bed and subsequent channel incision with or without collapse of oversteepened banks; direct erosion of bank materials by the flow; slumping of banks induced by flow undercutting or groundwater seepage; and surface erosion of channel banks by overland flow into the channel. The erosion of channel banks is particularly difficult to predict because (1) bank materials often contain cohesive sediments, for which no generally reliable models of erosion rate are available, (2) the protection against erosion of bank vegetation is extremely difficult to predict, and (3) bank erosion must ultimately respond to previous bed erosion, which is driven by a different suite of processes than bank erosion. Summaries of the difficulty in predicting bed and bank erosion include Hooke (1979), Pizzuto (1984), Wolman (1959), and Wolman and Brush (1961). The difficulty of predicting bank erosion is one of the reasons that bed material load is commonly used

as a surrogate for evaluating the potential of channel erosion in response to hydrologic changes (McCuen and Moglen, 1989; MacRae and Wisner, 1989).

The rationale for using bed material load as a surrogate arises from several sources: (1) bed erosion can only proceed by means of the transport of bed material out of a channel reach; (2) many bank erosion processes can only occur once bed erosion has deepened the channel; (3) bed material load, although still enormously difficult to predict accurately, general involves cohesionless sediments, for which our physical understanding is much better than for cohesive sediments. Even though bed material load is difficult to predict with accuracy, estimates of relative changes in bed material load, when based on the same transport formula, provide accurate estimates of the direction of changes in load.

Because predictions of bed material load are not precise and differences of orders of magnitude are commonly found between bed material loads predicted with different formulas, the approach adopted here in developing erosion control designs is to only compare bed material loads computed with the same transport formula. Thus, an erosion control design for the 2 year storm is one in which the bed material load for the pond-routed discharge is less than or equal to the bed material load computed using the same transport formula and the predevelopment discharge.

5.2. ILLUSTRATING THE RESPONSE OF DOWNSTREAM HYDRAULICS AND BED MATERIAL TRANSPORT TO OUTLET DESIGN ALTERNATIVES

Given an inflow hydrograph and pond stage-storage curve, the outflow hydrograph from a detention pond is controlled by the type, size, and elevation of the pond outlet structures. For the purposes of this study, three properties of the outflow hydrograph are of interest:

(1) Cumulative bed material load downstream of the pond. This is the quantity to be minimized in an erosion control SWM plan.

(2) Peak water discharge. In general, this quantity must meet specific management criteria. In the Anacostia basin, the peak discharge for the 2 year and 10 year storm must not exceed the peak value for the same storm with the drainage area in its predevelopment state. In devising an erosion control SWM plan, it would generally be necessary to meet the current peak discharge requirements, although it appears that peak discharge requirements for the 2 year storm will generally be met by a design that provides erosion control for the 2 year storm.

(3) Time to drawdown. The period of time over which a detention pond detains the storm flow forms an essential part of extended detention SWM plans. A standard extended detention criteria for the Anacostia basin is a 24-hour delay between the centroids of the inflow and outflow hydrographs (Harrington, 1987b). The drawdown

time for a SWM pond also needs to be considered to ensure that the detention period is not longer than desirable. For example, an erosion control SWM plan that entails detention of storm water for 96 hours or more may be undesirable in terms of aesthetic and safety considerations.

Response surface plots.

A convenient and powerful way to visualize and compare the effect of pond design alternatives on downstream hydraulics and bed material load is to use response surfaces that plot contoured values of peak discharge, drawdown time, and bed material load as a function of the type, size, and elevation of different pond outlet structures. Response surfaces have been developed for the demonstration sites that show contoured values of cumulative bed material load, peak discharge, and time to drawdown as a function of outlet size and elevation. A set of four plots (one each for the peak discharge, the drawdown time, the Goncharov load, and the Meyer-Peter and Muller load) may be made for any inflow hydrograph and combination of outlet structures. The bed material load is plotted as a ratio of the computed bed material load to the predevelopment bed material load. Although a set of plots generally represents pond designs incorporating more the one outlet structure, conceptual and graphical clarity require that the size and elevation of only one outlet is varied in any set of plots. While each plot illustrates the effect on the hydrograph variable of different sizes and elevations of outlet structures, the effect of different outlet structure types may be seen by comparing equivalent plots (e.g. of peak discharge) for the same storm, but different outlet structures.

6. DEMONSTRATION OF EROSION CONTROL SWM ALTERNATIVES

Using the methodology described in Section 5, erosion control designs are developed in this section for two existing SWM facilities in the Anacostia River basin. For each site, a general description of the soils, hydrology, and SWM facility is given, an erosion control design is developed, and the reduction in bed material load is demonstrated for the erosion control design, as well as a design meeting 1 year 24 hour extended detention requirements.

6.1. SELECTION CRITERIA FOR DEMONSTRATION SITES

Two SWM sites were selected to demonstrate an erosion control SWM design procedure. The sites are located at Newport Towne (Prince George's County) and Snowdens Mill (Montgomery County). The sites were selected because they provide a contrast in size, soils, site hydrology, and nature of the receiving stream. These contrasts allows us to develop erosion control SWM design over a broad range of conditions. Some of the salient features of the sites are listed below and in Table 6.1.

(1) Both sites are well-maintained and may be expected to perform as designed. Both sites currently use a peak discharge dry pond design.

(2) The two sites provide a contrast in site hydrology (resulting from differences in drainage area, ultimate % impervious area, and soil type) that permit demonstration and evaluation of the erosion control SWM design at different spatial and hydrologic scales.

(3) The two sites are located on different soil types, providing a contrast in both runoff and erosion properties. Newport Towne is located on a coarse-grained coastal plain soil, Snowdens Mill is located on finer-grained soils typical of the Piedmont Physiographic Region.

(5) The two sites provide a contrast in age and condition of the receiving stream. At Newport Towne, the receiving channel is a grass swale with a small eroded gully that has developed over the past 4 years. At Snowdens Mill, the receiving channel is a larger, somewhat incised, stream channel that presumably existed before construction of the SWM facility.

Table 6.1 Characteristics of the Demonstration Sites

	Newport Towne	Snowdens Mill
Location	N. of Cherry Hill Road ADC: 3, K3	N. of East Randolph Road ADC: 34, G3
Watershed	Little Paint Branch	Paint Branch
Drainage Area (acres)	22	82
Ultimate % Impervious	35	28
Soils	Sassafras gravelly sandy loam (Coastal Plain)	Manor Silt Loam (Piedmont)
Construction Date	1985	1978
Receiving Stream Condition	Grass swale with small eroded gully	Small stream with some erosion (incision)

6.2. SITE 1: NEWPORT TOWNE

6.2.1. Site Characteristics

Location. Newport Towne is a townhouse development of 41 acres in Prince George's County, Maryland (Figure 2.1). The site is located on the north side of Cherry Hill Road, 1/4 mile west of Powder Mill Road (P.G. County ADC 3, K3). A dry stormwater detention pond installed during development drains an area of 22 acres, which is slightly more than half the area of the development.

Soils. The Newport Towne development lies within the Atlantic Coastal Plain physiographic province. The Atlantic Coastal Plain is underlain by unconsolidated deposits of gravel, sand, silt and clay that range in age from the Cretaceous to the Holocene. (SCS, 1967) Approximately 85% of the Newport Towne development is underlain by Sassafras gravelly sandy loam. These soils are mapped as moderately eroded, most (>70%) with slopes in the 5% to 10% range (SCS, 1967). Approximately 10% of the site is underlain by an abandoned gravel/borrow pit. The remaining approximately 5% of Newport Towne is underlain by Rumford loamy sand. All of these soils underlying Newport Town fall into the S.C.S. Hydrologic soil group B.

Land Use. Prior to development, the Newport Towne area was primarily forested land and meadows. The development is entirely townhouses on lots of 1/8 or an acre or less. The percent impervious area after development is estimated as 35% (MWCOG, 1989)

Hydrology. Precipitation averages about 42 inches per year, and is evenly distributed throughout the year. Average monthly precipitation ranges from 2.8 inches in February to 4.9 inches in August. Much of the summer precipitation comes in short, high intensity rainfalls from convective

storms, whereas winter precipitation comes mostly in low intensity rainfall from frontal storms (Yorke and Herb, 1978). Table 6.2 presents the hydrologic parameters for the Newport Towne development, computed using standard U.S. Soil Conservation Service procedures (SCS, 1984 and 1986).

TABLE 6.2
HYDROLOGIC PARAMETERS FOR NEWPORT TOWNE

	Predevelopment	Postdevelopment
Drainage Area (acres)	22.08	22.08
Runoff Curve Number	70	86
Time of Concentration (hrs)	0.23	0.22

Table 6.3 provides runoff quantities and peak discharges for the Newport Towne site computed using TR-20 and a 24 hour Type II design storm (SCS, 1982).

TABLE 6.3
SCS TR-20 RUNOFF AND PEAK DISCHARGE FOR NEWPORT TOWNE

Return Period	Precipitation	PREDEVELOPMENT		POSTDEVELOPMENT	
		Runoff	Peak Discharge	Runoff	Peak Discharge
(years)	(inches)	(inches)	(cfs)	(inches)	(cfs)
1	2.7	0.55	13.0	1.41	37.6
2	3.3	0.89	22.2	1.93	51.1
5	4.3	1.54	39.8	2.83	74.2
10	5.2	2.19	57.3	3.66	95.2
100	7.4	3.96	103.5	5.76	146.5

6.2.2. Existing Stormwater Management Facility and Receiving Channel

Stormwater Detention Facility. A dry stormwater detention pond was constructed in 1985 to serve the Newport Towne Development. The pond was designed to attenuate the post development peak runoff values for the 10 year and 100 year storms so that they are less than or equal to pre development values. This pond was designed to comply with the requirements of the Washington Suburban Sanitary Commission (W.S.S.C).

The pond provides a maximum of 133,444 cubic feet (3.063 acre-feet) of stormwater storage. The pond's stage-storage relationship is given in Table 6.4.
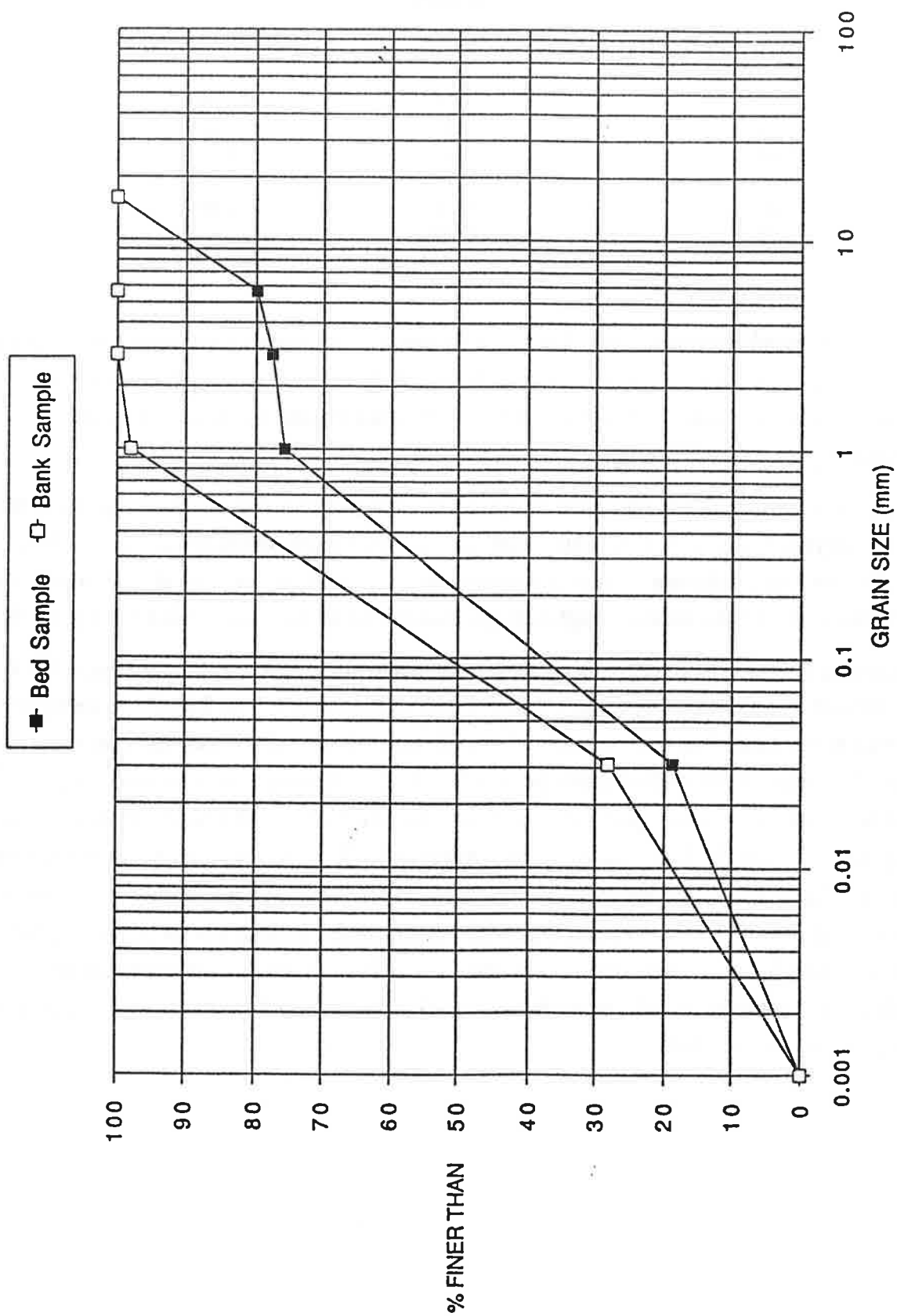
TABLE 6.4
NEWPORT TOWNE STAGE-STORAGE RELATIONSHIP

Elevation (ft)	Storage (cu. ft.)	Storage (acre-ft)
243.5	0	0
244.0	306	0.007
246.0	15390	0.353
248.0	46782	1.074
250.0	85878	1.971
252.0	133444	3.063

The pond has two outlets. The low flow outlet is a square concrete orifice with sides of 1'-10" located at the pond bottom (el. 243.5'). The second outlet is a rectangular concrete weir 5.5 feet long at an elevation of 249.0 feet. The pond is well maintained with little or no debris obstructing the outlet structure.

The existing SWM facility at Newport Towne was designed to control the peak discharge of the 10 year storm. The pond routed discharge for the 10 year storm is 42.2 cfs, which is smaller than the predevelopment peak discharge of 57.3 cfs. The pond routed discharge for the 2 year storm is 25.9 cfs, which is slightly larger than the predevelopment peak discharge of 22.2 cfs.

Receiving Channel. The receiving channel downstream of the pond outlet is rip-rapped for a distance of 37 feet below the control structure with stones ranging from 9 inches to 2 feet in size (intermediate axis). The average channel slope is approximately 0.06. Downstream of the rip-rapped section the channel is a grass swale with a small gully eroded into the soil. Samples of bank and bed material from this gully are composed of sand (60-70%) with a large proportion of silt and clay size particles (20-30%), and some fine gravel (2-25%). Grain size data are shown in Table 6.5 and Figure 6.1. Grain size data were collected to provide input on hydraulic roughness for computing flow in the channel, and to provide input on sediment mobility for computing bed material load in the channel. The bed sample shows a bimodal distribution composed of the residual soil at the site (similar to the bank sample) and small pebbles left as a lag from erosion of the soil overlying the bed.

% FINER THAN

GRAIN SIZE (mm)

- Bed Sample   -□- Bank Sample

6.1   Grain-size distribution of Newport Towne channel sediments.

Table 6.5 NEWPORT TOWNE GRAIN SIZE DISTRIBUTIONS FOR CHANNEL BED AND BANK SAMPLES

GRAIN SIZE (mm)	BED (Wt%)	BANK (Wt%)
<0.0625	18.8	28.2
0.0625-2.00	57.0	69.8
2.00-4.00	1.8	2.0
4.00-8.00	2.3	0.0
>8.00	20.1	0.0

The grassed gully downstream of the Newport Towne pond has an average slope of approximately 0.05. Appendix C presents surveyed cross sections of the swale and its eroded gully. The cross section and hydraulic roughness were used to compute flow velocity and depth as a function of discharge. Mean flow velocity, flow depth, and bed shear stress ($\tau_o = \theta dS$, where θ is the weight of water, d is depth, and S is channel slope) above the center section of the channel are plotted in Figure 6.2.

A representative grain size of the sediment is necessary to predict the sediment transport. A size of 0.3 mm (0.001 ft.), which is the median size of the bed soil/sediment, was used in this study. The values of critical shear stress and critical velocity for incipient motion were taken from the work on critical velocity by Fortier and Scobey (1926) and later conversion of these values to shear stress by the U.S. Bureau of Reclamation (cited in McCuen et al., 1987). The values used for Newport Towne correspond to the sediment category "graded silts to cobbles when colloidal". Because the Newport Towne pond is fully vegetated with a marshy bottom, values of critical shear stress and velocity for the clear water case were used. These are $V_c = 4.0$ ft./s and $\tau_c = 0.43$ lbf/ft^2.

The Newport Towne Development drains into a small unnamed first-order tributary of the Little Paint Branch of the Anacostia River. Wetlands of two basic types are located along these channels (USFWS, 1987). A palustrine wetland with a broadleaf deciduous forest that is temporarily flooded (Fish and Wildlife Service type PF01A) is identified 2500 ft downstream of the SWM facility. This wetland type is common throughout the Little Paint Branch drainage area. The dominant tree species found in this wetland are sycamore, black willow, sweet gum, red maple, river birch, box elder, and pin oak. Riverine lower perennial wetlands are also identified along the Little Paint Branch downstream of Newport Towne (R2OWH: open water/unknown bottom, permanently flooded; R2BBC: Beach/Bar, seasonally flooded).

NEWPORT TOWNE: DOWNSTREAM HYDRAULICS
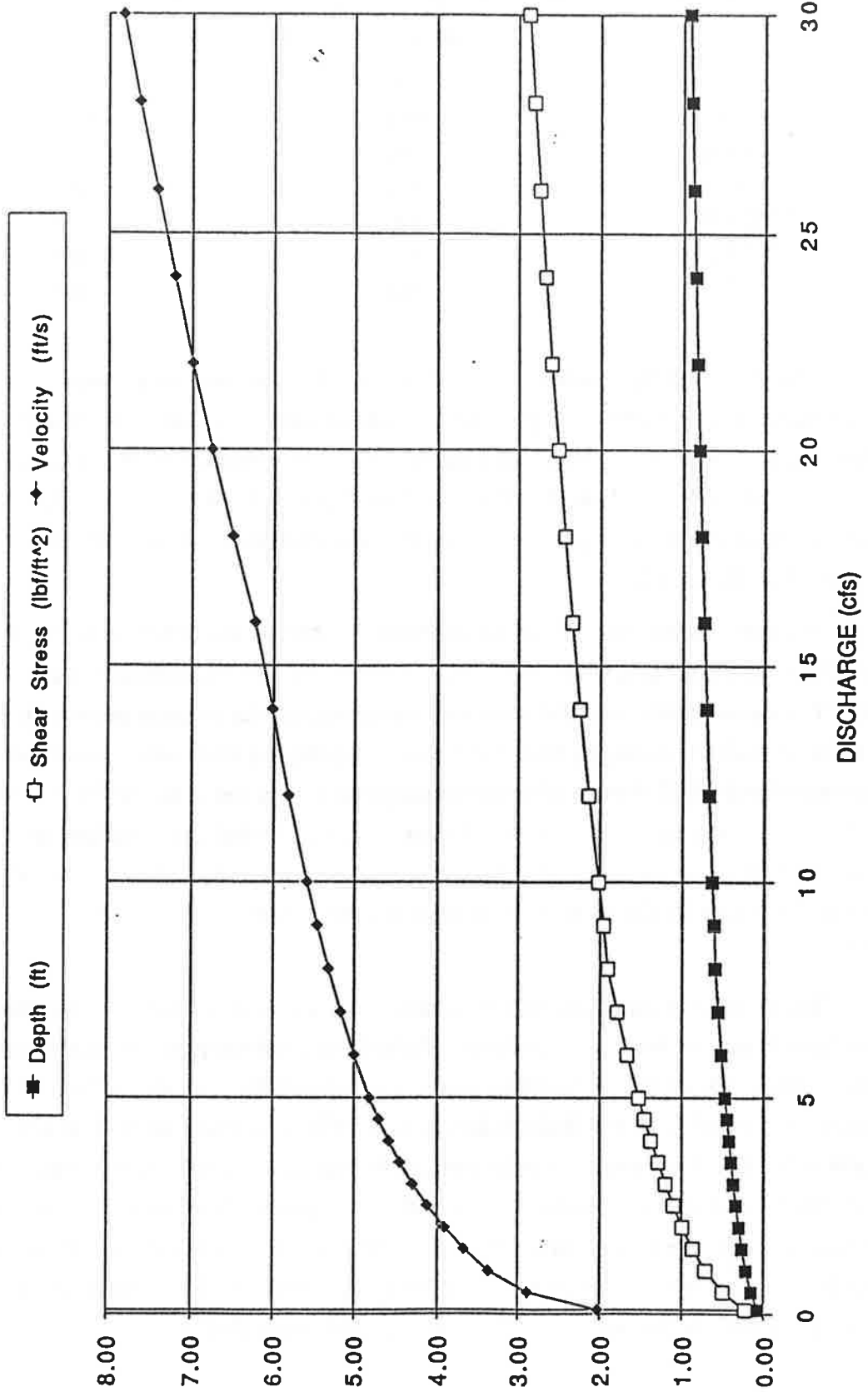
6.2   Flow velocity, depth, and bed shear stress as a function of discharge at Newport Towne.

Largemouth bass, redbreast sunfish, and bluegill sunfish have been observed on Little Paint Branch downstream of the Newport Towne development (ICPRB, 1989a). Largemouth bass and bluegill sunfish spawn in a variety of substrates, but fine gravel-sand is preferred. Excessive stream bed erosion can erode this substrate; excessive sedimentation from tributaries to the Little Paint Branch will degrade the spawning grounds.
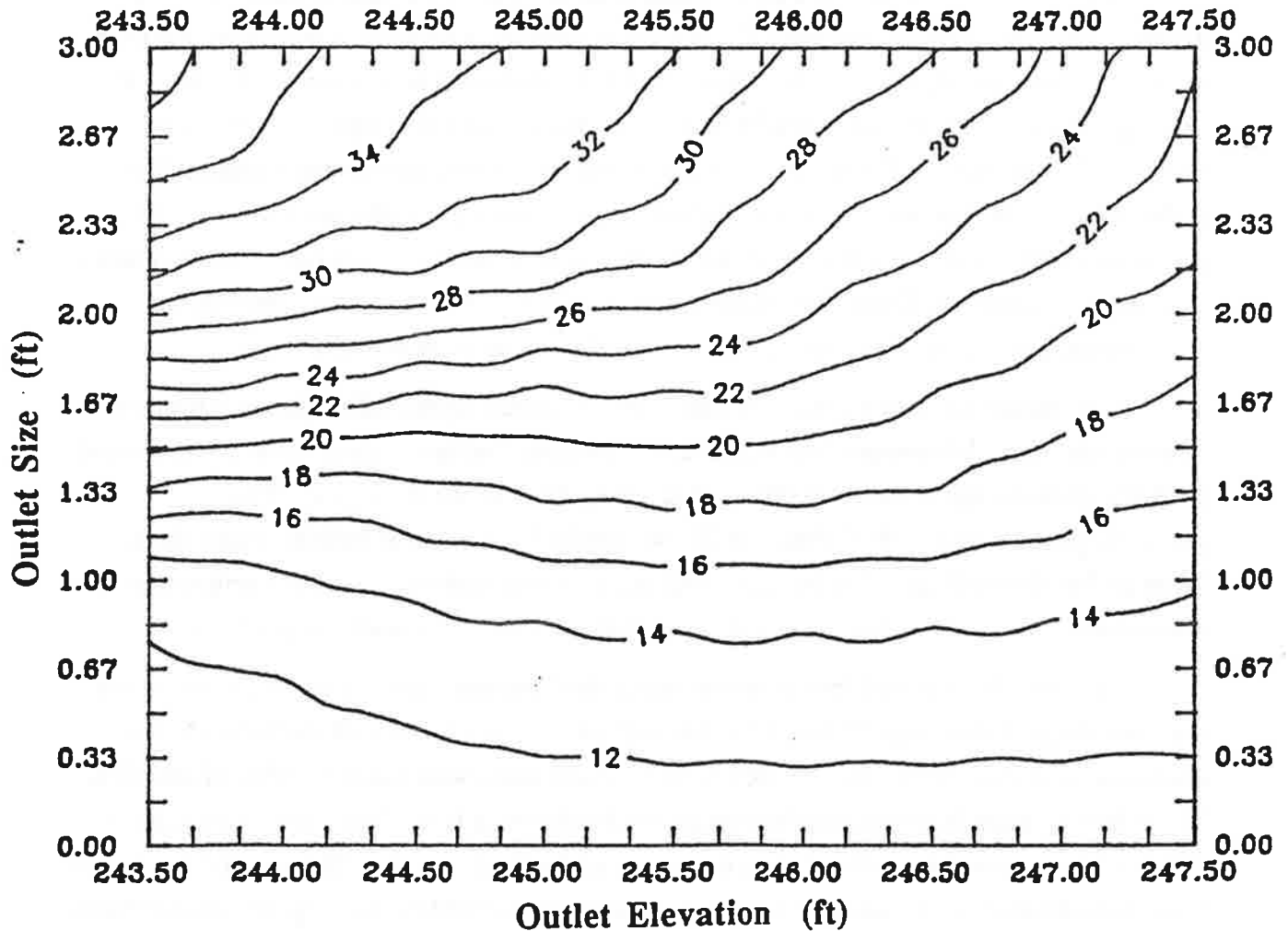
6.2.3 Response Surfaces

Figures 6.3 to 6.6 present a set of response surfaces for the 2 year storm at Newport Towne. The case shown incorporates the existing stage-storage curve and the existing 100 year weir. A 24 hour round, extended detention orifice is located at the base of the pond to provide drainage. This orifice was sized at 0.38 ft using the procedure of Harrington (1987b). The surfaces show the effect of varying the size and elevation of an additional square orifice. The orifice is varied in size from 0 to 3 feet on a side and in elevation from the pond bottom to 4 ft above the pond bottom. These outlet structures represent a convenient suite for evaluating retrofit alternatives. Using a different type of outlet (round orifice, box riser, circular riser) for the variable-sized outlet gave response surfaces nearly identical to those presented here.

The maximum peak discharge is found with the largest square orifice placed at the bottom of the pond (upper left corner of the Figure 6.3). The peak discharge values fall to a minimum of 11.5 cfs when the square orifice is of zero size (along the abscissa of the plot). The predevelopment 2 year peak discharge of 22.2 is satisfied by a range of variable orifice sizes. Time to drawdown (Figure 6.4) increases with increasing elevation of the square orifice over the entire range of elevations, but varies with the orifice size only in the lower range of sizes.

The total bed material load computed using the Goncharov formula follows a pattern that matches the peak discharge (Figure 6.5). The highest loads are found with the largest square orifice located at the pond bottom. The lowest loads are found with a square orifice of zero size. These loads are not, however, smaller than the predevelopment loads, with a minimum value of 131% of the predevelopment load when the square orifice is of zero size. This result states that the extended detention orifice provides the minimum bed material load for the range of square orifices examined, but that erosion control at the 2 year level is not provided. Similar results were obtained using a 48 hour and 72 hour extended detention orifice, which produced Goncharov loads of 135% and 138% of the predevelopment loads, respectively. The computed bed material loads increase slightly with decreasing size of the extended detention orifice because the smaller orifices produce higher pond water levels and a greater peak flow over the 100 year weir, thus producing a greater transport during the peak discharge period.

Newport Towne: 2 year Peak Discharge (cfs)

Outlet Elevation (ft)

6.3   Response surface for a 24 hr extended detention orifice (0.38 ft round orifice) at pond bottom, the existing 100 year weir, and a square orifice varying in size from 0 to 3 ft and in elevation from 243.5 ft to 247.5 ft: Response of peak discharge (cfs) for the 2 yr storm at Newport Towne.
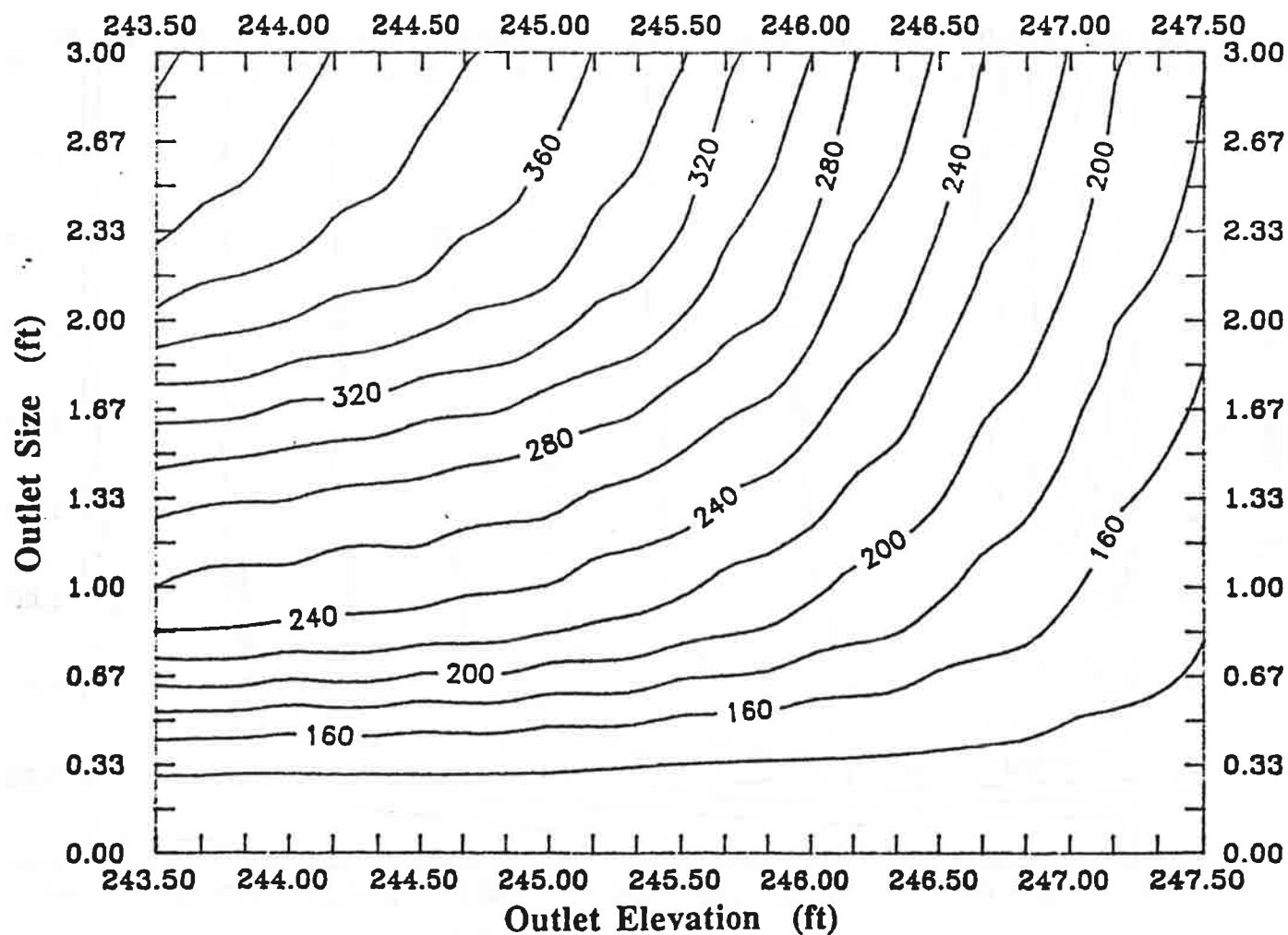
Newport Towne: 2 year Time to Drawdown (minutes)
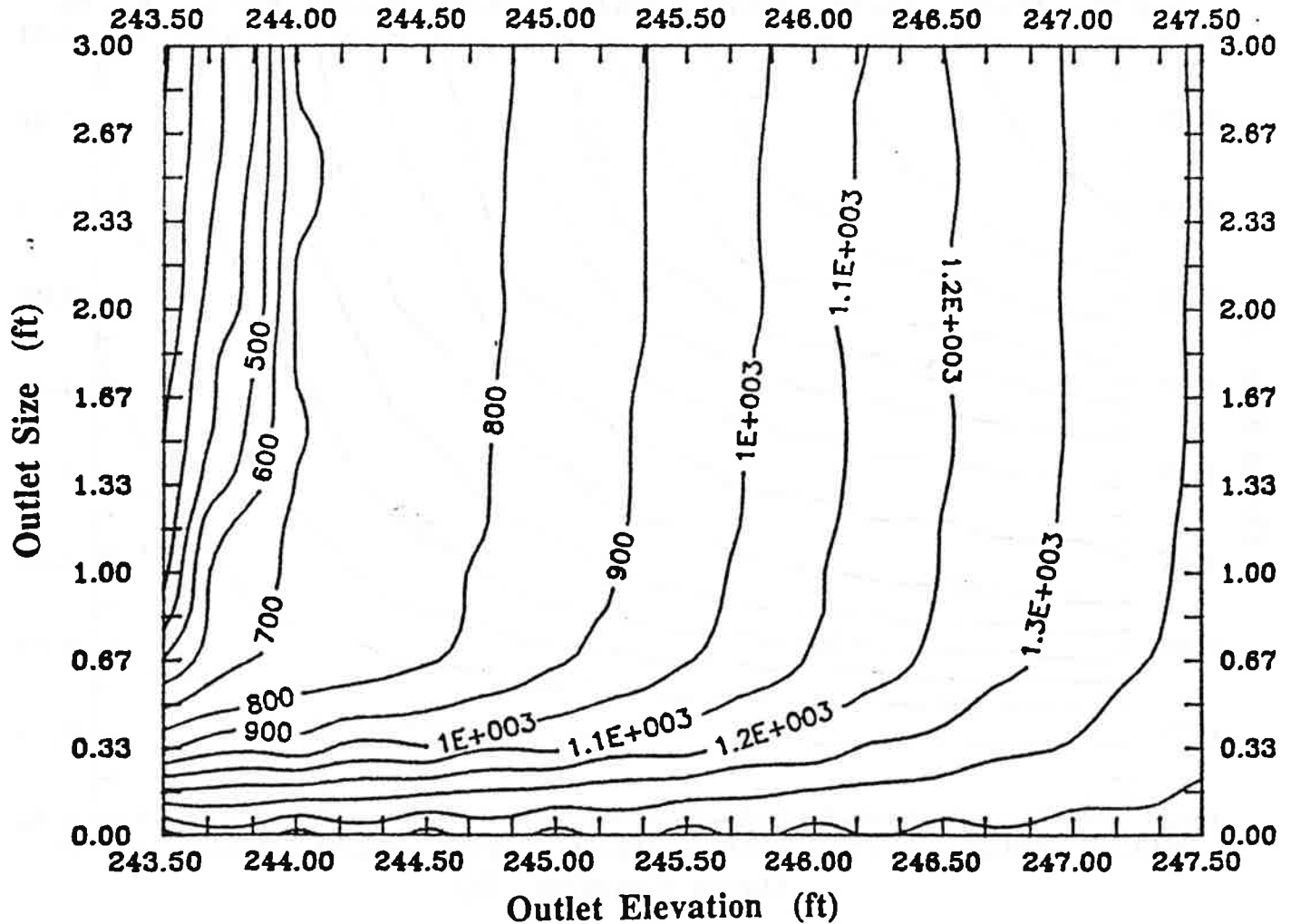
6.4   Response surface for a 24 hr extended detention orifice (0.38 ft round orifice) at pond bottom, the existing 100 year weir, and a square orifice varying in size from 0 to 3 ft and in elevation from 243.5 ft to 247.5 ft: Response of drawdown time (minutes) for the 2 yr storm at Newport Towne.
The symbol E signifies scientific notation (e.g. 1.1E+003 = 1.1 x 10^3).

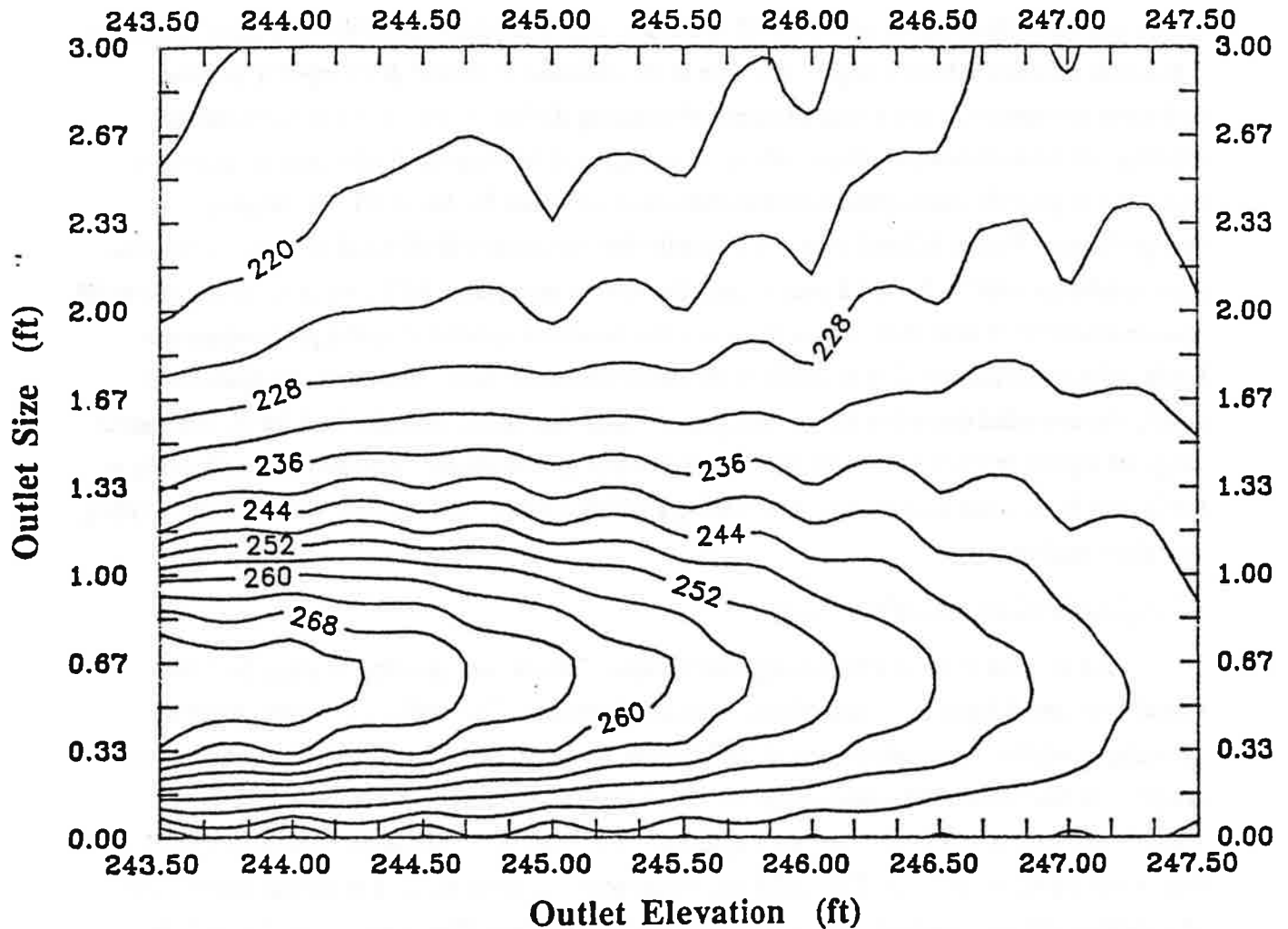# Newport Towne: 2 year Goncharov Load
(% of predevelopment load)

6.5   Response surface for a 24 hr extended detention orifice (0.38 ft round orifice) at pond bottom, the existing 100 year weir, and a square orifice varying in size from 0 to 3 ft and in elevation from 243.5 ft to 247.5 ft: Response of Goncharov bed material load (% of predevelopment load)) for the 2 yr storm at Newport Towne.

Newport Towne: 2 year Meyer-Peter & Muller Load
(% of predevelopment load)

6.6   Response surface for a 24 hr extended detention orifice (0.38 ft round orifice) at pond bottom, the existing 100 year weir, and a square orifice varying in size from 0 to 3 ft and in elevation from 243.5 ft to 247.5 ft: Response of Meyer-Peter and Muller bed-material load (% of predevelopment load)) for the 2 yr storm at Newport Towne.

The total bed material load computed using the Meyer-Peter and Muller (MPM) formula (Figure 6.6) follows a different pattern than that given by the Goncharov formula. Like the Goncharov results, the MPM results show a local minimum for a square orifice of zero size. However, the MPM results also show a minimum for the largest orifice located at the bottom of the pond (upper left-hand corner of Figure 6.6). As explained in section 7 of this report, this second minimum results from the shape of the relation between the MPM load and flow discharge. The slope of this function decreases with increasing discharge so that a second transport minimizing solution exists: releasing all of the runoff in the shortest time period possible. We argue in section 7 that this solution, which is highly sensitive to the estimate of critical discharge for incipient sediment movement, is not a true transport-minimizing design, based on the observation that existing SWM facilities (which provide some storage and lower peak discharges) are generally observed to provide more erosion control than no stormwater facility at all. In addition, comparison of Figure 6.3 and Figure 6.6 reveals that the upper MPM solution does not provide peak discharge control for the 2 year storm. The lowest acceptable MPM loads are found when the square orifice is of zero size. These loads are not, however, smaller than the predevelopment loads, with a minimum value of 236% of the predevelopment load. Similar to the Goncharov result, the extended detention orifice does provide a minimum bed material load for the acceptable range of square orifices examined, but erosion control for the Meyer-Peter and Muller formula at the 2 year level is not provided by substituting a 24 hour extended detention orifice for the existing low flow orifice.

6.2.4. Erosion Control SWM Design

An erosion control SWM design for Newport Towne was developed using both the Goncharov and Meyer-Peter and Muller transport formulas. The designs were developed to provide, in addition to erosion control (EC) for the 2 year storm, 2/2 and 10/10 peak discharge control, 24 hour extended detention (ED) of the 1 year storm, and compliance with SCS requirements for a 100 year emergency spillway. For clarity, the design process is presented below in step-by-step form. The outlet structures were designed starting at the bottom of the pond. The design process involved determining which of the two low flow alternatives (EC or ED) produced the smaller orifice size. This orifice was then placed at the bottom of the pond so that both EC and ED would be provided. The maximum water level with only this orifice and the appropriate design storm (1 year storm for an ED orifice, 2 year storm for an EC orifice) was determined to provide the minimum elevation for the next higher outlet (10 year peak discharge outlet). Using this minimum, the size and elevation of the 10 year outlet were varied to find the optimum solution that provided erosion control, 10 year peak discharge control, and a minimum elevation for the 100 year emergency spillway.

DESIGN PROCEDURE: NEWPORT TOWNE (Goncharov)

REQUIREMENTS:

        a) 24 hour extended detention for the 1 year storm

        b) erosion control for the 2 year storm

        c) peak discharge control for the 2 year and 10 year storms

        d) minimize elevation of the 100 year spillway & top of dam

1)   Calculate bed-material load for the 2 year predevelopment storm using the Goncharov formula, yielding a total load of 5.14×10^7 lbs.

2)   Compute bed material load for 2 year postdevelopment storm using only one outlet of varying size at the base of the pond.

3)   For the 2 year storm, plot bed material load (expressed as a percentage of predevelopment load) as a function of orifice size (expressed as the design detention time for a 1 year storm using the method of Harrington, 1987b). (Figure 6.7). Compare the size of outlet required for 2 year erosion control with the size of outlet required for 1 year 24 hour extended detention outlet.
Diameter of a circular orifice for 1 year, 24 hour ED control: 0.38 ft
Diameter of a circular orifice for EC: <4.0 ft.
Because the ED outlet is smaller than the EC outlet, use the 24 hour ED outlet to provide both 24 hour extended detention and erosion control.

Place a 0.38 ft. circular orifice at 243.5 ft (pond bottom)

4)   Find the maximum pond water level for the ED outlet and 1 year storm, which is 249.2 ft. This is the minimum elevation for the 10 year weir that will provide both 2 year erosion control and 1 year, 24 hour extended detention. It is assumed here that any design that provides 2 year erosion control will also provide 2 year peak discharge control.

        Note: using the rule that the 100 year spillway must be at least 1 ft above the 10 year outlet (Harrington, 1987a), a minimum elevation of 249.2 ft for the 10 year outlet requires that the 100 year spillway invert must be at an elevation of at least 250.2 ft. But the 100 year spillway must also be at least 2.0 ft. below the top of the dam (SCS Engineering Code 378), which is at 252.0 ft. Therefore, there is not a solution for extended detention that will not involve extending the pond storage.

5)   Check if all SWM requirements are met by changing only the existing low flow weir to the ED outlet (using the existing stage-storage curve and upper outlet, which is the most economical solution). For the 2 year storm, erosion control is not satisfied (exceeds predevelopment load by

NEWPORT TOWNE WITH ONLY ONE OUTLET AT POND BOTTOM

■ MPM % OF PREDEVELOPMENT BED-MATERIAL LOAD ▫ GON % OF PREDEVELOPMENT BED-MATERIAL LOAD

DESIGN EXTENDED DETENTION TIME
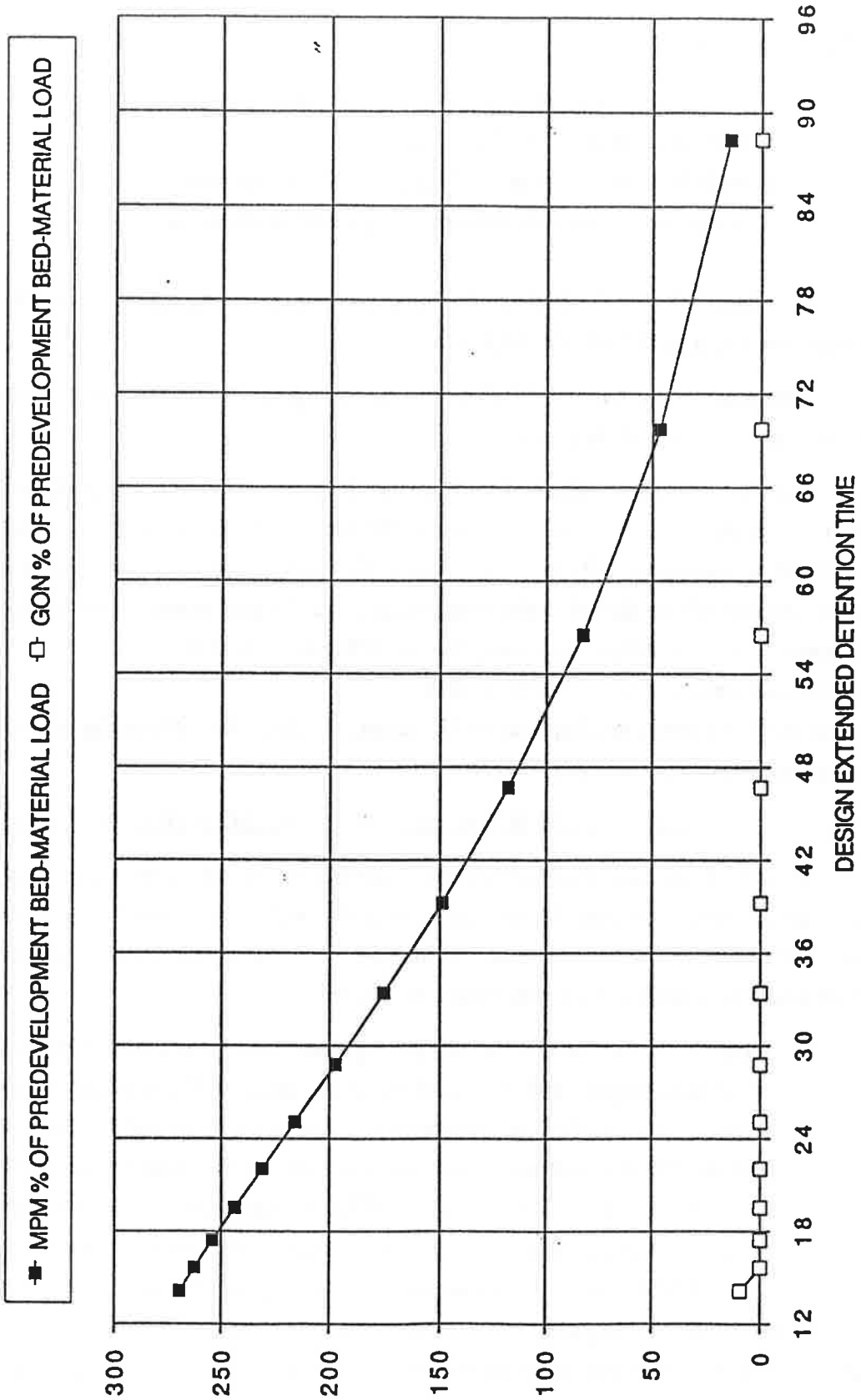
6.7 Variation of bed-material load (expressed as a percent of predevelopment load) as a function of low flow outlet size (expressed as design detention time for a 1 yr storm) at Newport Towne.

31%) because of significant flow over the existing upper (100 year) weir. From step 4, peak elevation for the 1 year storm is 249.2 ft, which exceeds the existing weir height by 0.2 ft, so extended detention is not satisfied by current pond configuration with an ED orifice.

6)   Determine the feasible range of sizes and elevations for the 10 year outlet. Constraints:
- erosion control for the 2 year storm (Figure 6.8)
- peak discharge control for the 10 year storm (Figure 6.9)
- minimize the elevation of the 100 year emergency spillway invert. Emergency spillway invert to be set at an elevation equal to the maximum water level for the 10 year storm, or 1 ft above the 10 year outlet elevation, whichever is higher (Harrington, 1987a). Maximum pond water levels for the 10 year storm are shown in Figure 6.10.

7)   The combination of elevation and size for the 10 year outlet that satisfies all three constraints is:

         Elevation: 249.6 ft
         Size: 7.5 ft
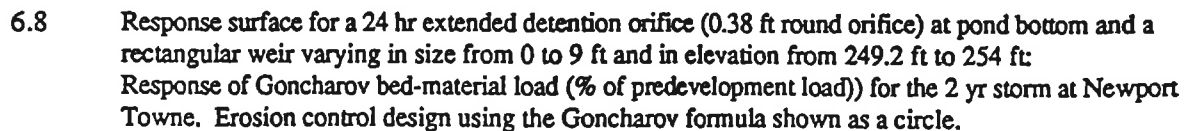   Maximum 10 year water level: 251.2 ft.

This solution is marked as circles on Figures 6.8, 6.9, and 6.10.

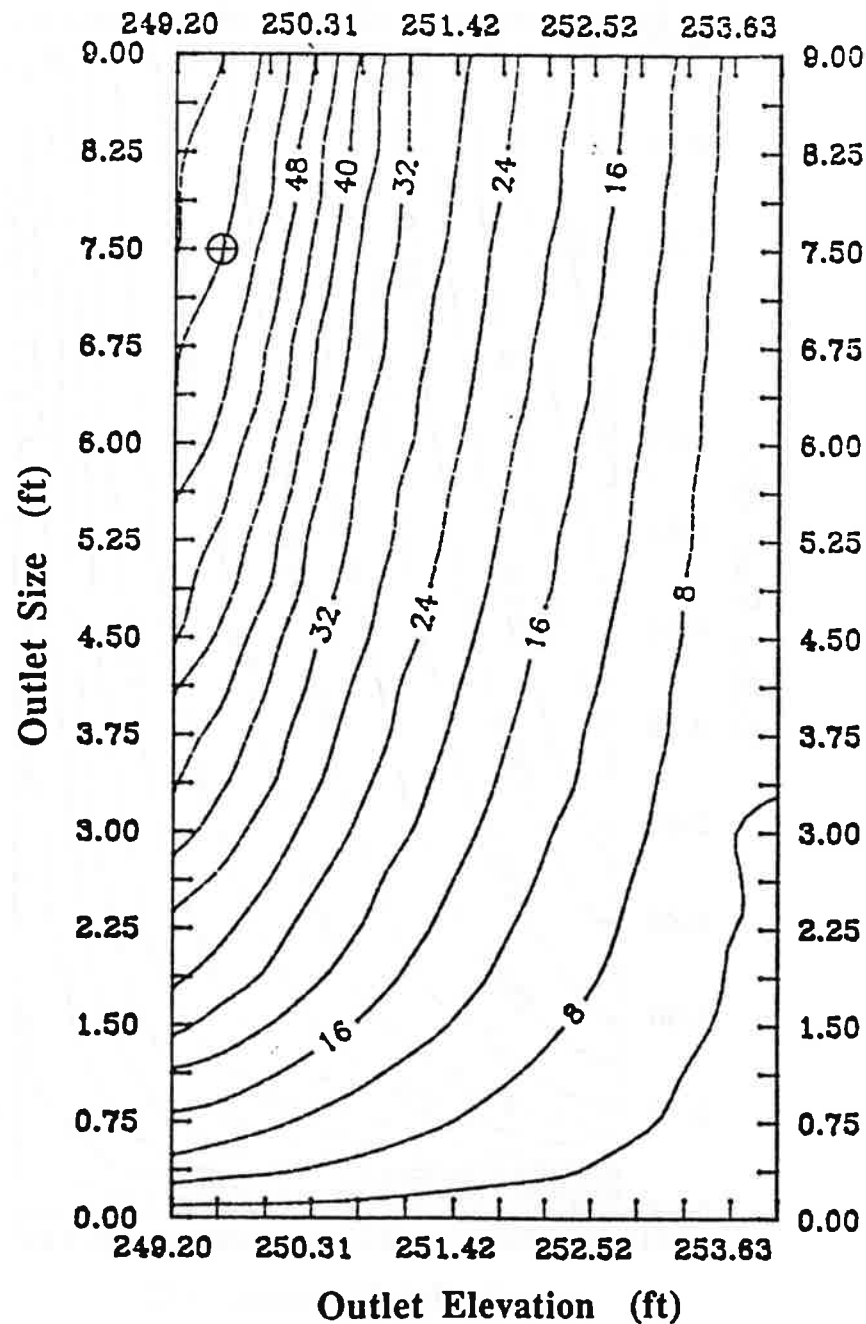**Place a 7.5 ft wide straight-crested weir at 249.6 ft.**

Place a 100 year weir at 251.2 ft.

Increase dam height to 253.2 ft.

8)   Size the 100 year weir to pass the 100 year storm at a head of 1 ft., taking into account the storage of the pond and the discharge through the lower weirs.

Newport Towne: 2 year Goncharov Load
(% of predevelopment load)

6.8   Response surface for a 24 hr extended detention orifice (0.38 ft round orifice) at pond bottom and a rectangular weir varying in size from 0 to 9 ft and in elevation from 249.2 ft to 254 ft:
Response of Goncharov bed-material load (% of predevelopment load)) for the 2 yr storm at Newport Towne. Erosion control design using the Goncharov formula shown as a circle.
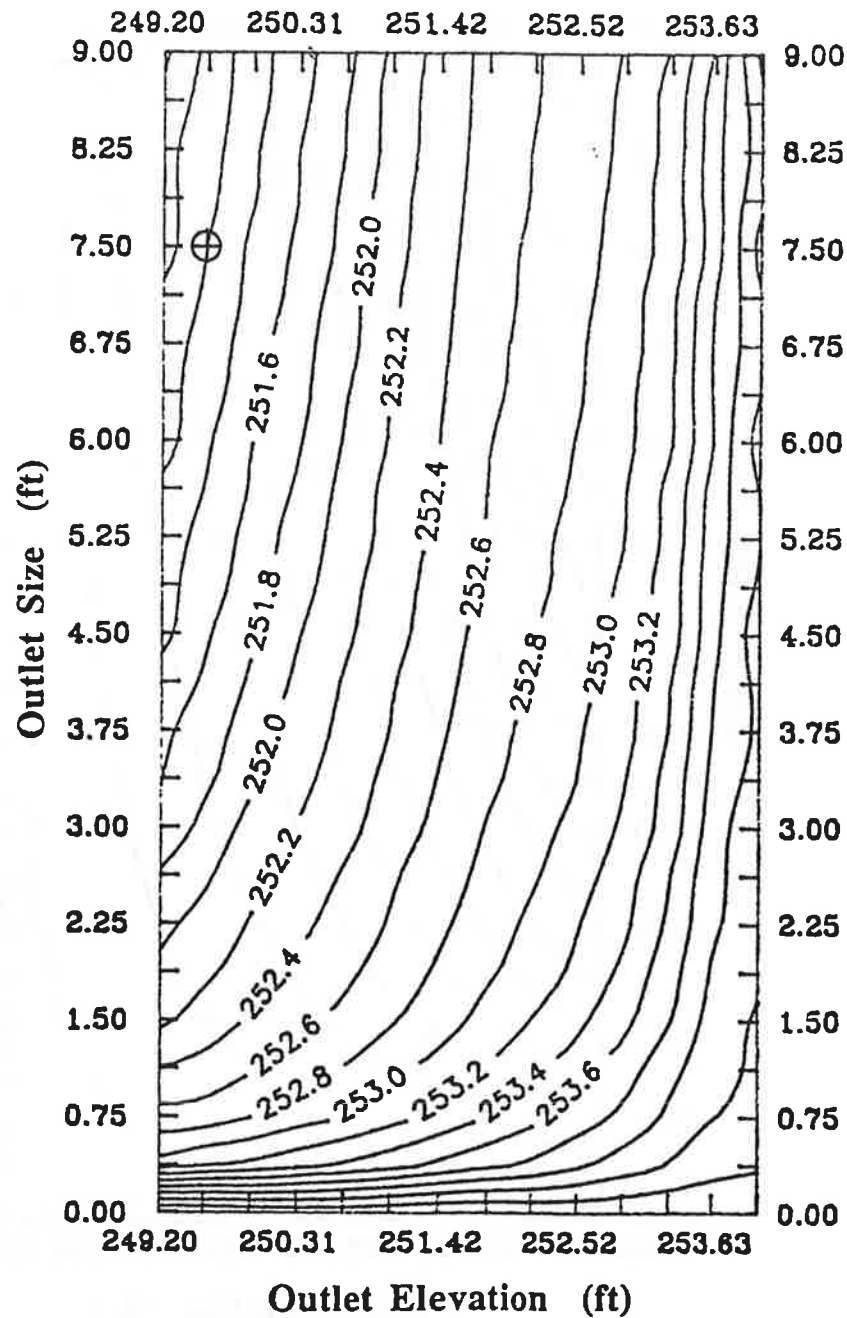
Newport Towne: 10 year Peak Discharge (cfs)

6.9   Response surface for a 24 hr extended detention orifice (0.38 ft round orifice) at pond bottom and a
       rectangular weir varying in size from 0 to 9 ft and in elevation from 249.2 ft to 254 ft.
       Response of peak discharge (cfs) for the 10 yr storm at Newport Towne. Erosion control design using the
       Goncharov formula shown as a circle.

Newport Towne: 10 year Maximum Pond Water Level (feet elevation)

6.10   Response surface for a 24 hr extended detention orifice (0.38 ft round orifice) at pond bottom and a rectangular weir varying in size from 0 to 9 ft and in elevation from 249.2 ft to 254 ft: Response of peak pond water level for the 10 yr storm at Newport Towne. Erosion control design using the Goncharov formula shown as a circle.

DESIGN PROCEDURE: NEWPORT TOWNE (Meyer-Peter and Muller)

REQUIREMENTS:

        a) 24 hour extended detention for the 1 year storm

        b) erosion control for the 2 year storm

        c) peak discharge control for the 2 year and 10 year storms

        d) minimize elevation of the 100 year spillway & top of dam

1)   Calculate bed material load for the 2 year predevelopment storm using the Meyer-Peter and Muller formula, yielding a total load of 2.16 X 10^5 lbs.

2)   Compute bed material load for 2 year postdevelopment storm using only one outlet of varying size at the base of the pond.

3)   For the 2 year storm, plot bed material load (expressed as a percentage of predevelopment load) as a function of orifice size (expressed as the design detention time for a 1 year storm using the method of Harrington, 1987b). (Figure 6.7). Compare the size of outlet required for 2 year erosion control with the size of outlet required for 1 year 24 hour extended detention outlet.
Diameter of a circular orifice for 1 year, 24 hour ED control = 0.38 ft
Diameter a circular orifice for EC is 0.26 ft.
Because the EC outlet is smaller than the ED outlet, use the EC outlet to provide both 24 hour extended detention and erosion control.

<u>Place a 0.26 ft. circular orifice at 243.5 ft (pond bottom)</u>

4)   Check drawdown time for the EC outlet and the 2 year storm: 66.5 hours.

5)   Find the maximum pond water level for the EC outlet and 2 year storm, which is 251.5 ft. This is the minimum elevation for the 10 year weir that will provide both 2 year erosion control and 1 year, 24 hour extended detention.

        Note: using the rule that the 100 year spillway must be at least 1 ft above the 10 year outlet (Harrington, 1987a), a minimum elevation of 251.5 ft for the 10 year outlet requires that the 100 year spillway invert must be at an elevation of at least 252.5 ft. But the 100 year spillway must also be at least 2.0 ft. below the top of the dam (SCS Engineering Code 378), which is at 252.0 ft. Therefore, there is not a solution for erosion control that will not involve extending the pond storage.

6)   Check if all SWM requirements are met by changing only the existing low flow weir to the EC outlet (using the existing stage-storage curve and upper outlet, which is the most economical solution). From step 4, peak elevation for the 2 year storm is 251.5 ft, which exceeds the existing

weir height by 1.5 ft, and is only 0.5 ft below the top of the pond. Erosion control cannot be satisfied by current pond configuration with an EC orifice.

7)   Determine the feasible range of sizes and elevations for the 10 year outlet. Constraints:

        • peak discharge control for the 10 year storm (Figure 6.11)

        • minimize the elevation of the 100 year emergency spillway invert. Emergency spillway invert to be set at an elevation equal to the maximum water level for the 10 year storm, or 1 ft above the 10 year outlet elevation, whichever is higher (Harrington, 1987a). Maximum pond water levels for the 10 year storm are shown in Figure 6.12.

8)   The combination of elevation and size for the 10 year outlet that satisfies these constraints is:

      Elevation: 251.5 ft

      Size: 7.25 ft

      Maximum 10 year water level: 252.5 ft.

This solution is marked as circles on Figures 6.11 and 6.12.

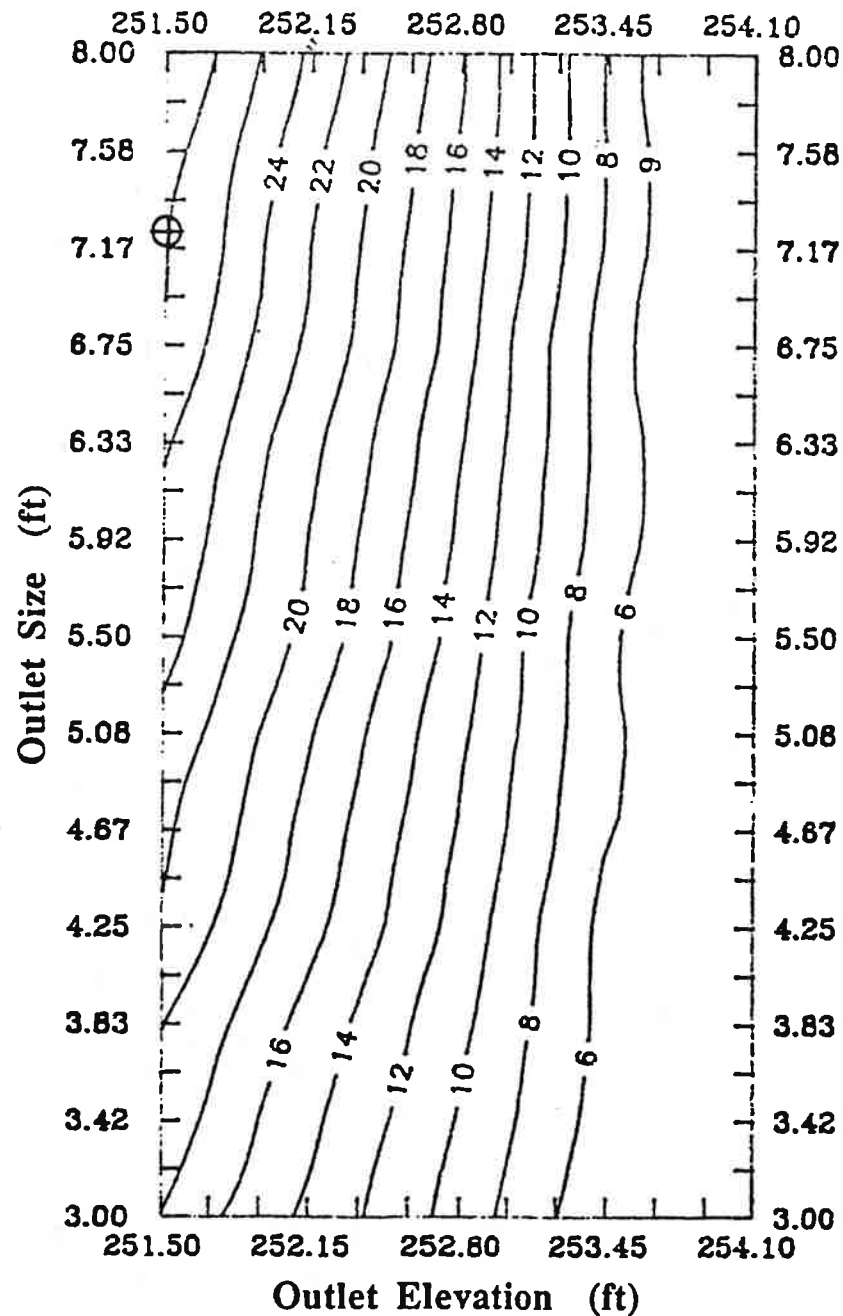<p align="center"><u>Place a 7.25 ft wide straight-crested weir at 251.5 ft.</u></p>

<p align="center"><u>Place a 100 year weir at 252.5 ft.</u></p>

<p align="center"><u>Increase dam height to 254.5 ft.</u></p>

9)   Size the 100 year weir to pass the 100 year storm at a head of 1 ft., taking into account the storage of the pond and the discharge through the lower weirs.

Newport Towne: 10 year Peak Discharge (cfs)

6.11   Response surface for an erosion control orifice (0.26 ft round orifice) at pond bottom and a rectangular weir varying in size from 3 to 8 ft and in elevation from 251.5 ft to 254.1 ft:
Response of peak discharge (cfs) for the 10 yr storm at Newport Towne. Erosion control design using the Meyer-Peter and Muller formula shown as a circle.

Newport Towne: 10 year Maximum Pond Water Level (feet elevation)
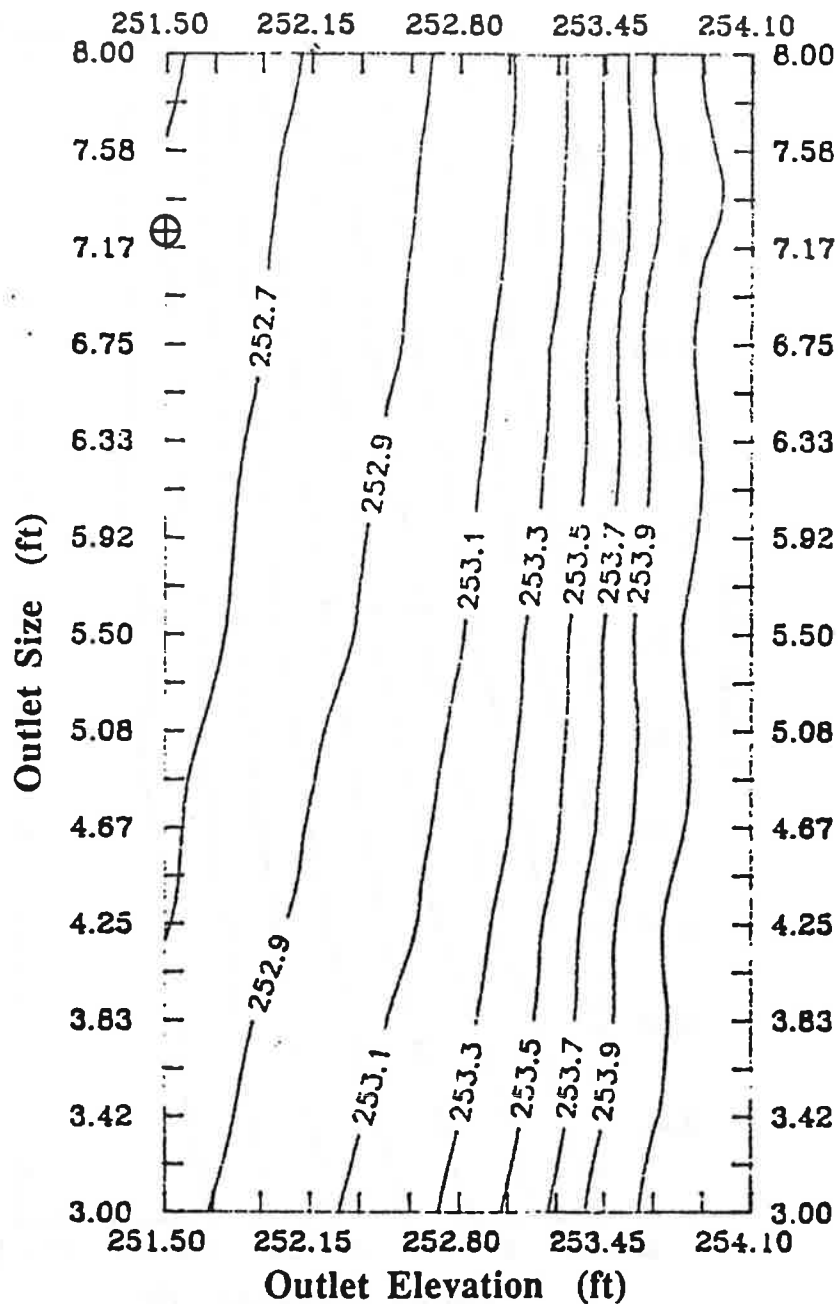
6.12 Response surface for an erosion control orifice (0.26 ft round orifice) at pond bottom and a rectangular weir varying in size from 3 to 8 ft and in elevation from 251.5 ft to 254.1 ft.
Response of peak pond water level for the 10 yr storm at Newport Towne. Erosion control design using the Meyer-Peter and Muller formula shown as a circle.

Table 6.6 summarizes the erosion control design results for Newport Towne. The Meyer-Peter and Muller formula requires a smaller low flow orifice to obtain erosion control. The smaller low flow orifice, in turn, forces the 10 year and 100 year outlets to be higher. Both transport solutions require a larger pond. To increase pond size, we have increased the elevation of the dam and extrapolated the existing stage-storage curve to the new elevation. An alternative approach would be to excavate the pond, providing increased storage at elevations that do not require an increase in dam height.

Table 6.6 Erosion Control Designs for Newport Towne

| Transport Formula | Low Flow Orifice | | | | 10 Year Outlet | | | 100 Year Spillway | Dam Height |
	Type	Elevation (ft)	Size (ft)	Detention Time (1 yr storm)	Type	Elevation (ft)	Size (ft)	Elevation (ft)	Elevation (ft)
Goncharov	round orifice	243.5	0.38	24 hr	straight-crested weir	249.6	7.5	251.2	253.2
Meyer-Peter and Muller	round orifice	243.5	0.26	52 hr	straight-crested weir	251.5	7.25	252.5	254.5

The storage volume required by the erosion control designs is equivalent to 1.43 inches and 1.85 inches over the 22 acre drainage area for the Goncharov and Meyer-Peter and Muller solutions, respectively. For the Goncharov solution, this storage depth corresponds to 101% of the 1 year design storm runoff and 74% of the 2 year runoff. For the Meyer-Peter and Muller solution, this storage depth corresponds to 131% of the 1 year design storm runoff and 96% of the 2 year runoff. In the case of the Goncharov solution, the erosion control criterion is not solely responsible for the amount of storage volume required. A 24 hour extended detention orifice is used in this solution. Review of Figures 6.8, 6.9, and 6.10 shows that the constraints on pond storage due to the 10 year and 100 year requirements alone produce essentially the same design as that for erosion control. The greater storage required for the Meyer-Peter and Muller solution results from the use of an erosion control low flow orifice that is smaller than the 24 hour extended detention orifice.

6.2.5. Sediment Yields under different SWM Plans

Table 6.7 presents the calculated bed material loads for the 2 year storm and a variety of different postdevelopment conditions. The postdevelopment condition with no pond produces bed material loads significantly in excess of the predevelopment loads. The existing pond, which was designed for 10 year peak discharge control, does little to alter bed material load for the 2 year storm. The values for an extended detention pond were computed for a case with a 1 year 24 hour extended detention orifice (0.38 ft diameter) at the bottom of the pond and, for 2 year peak discharge control, a 4 ft straight-crested weir at an elevation of 249.2 ft. The elevation of the 2 year weir was set at the maximum pond elevation for the 1 year storm and the ED orifice. The 2/2 peak discharge requirement was satisfied by all sizes (including zero) of weir at elevation 249.2 ft, so the weir size of 4 ft was chosen to provide a tradeoff between the height of the 2 year peak water level and a convenient size of the weir. Both the Goncharov and Meyer-Peter and Muller bed material loads were found to be insensitive to the width of the 2 year weir.

The Goncharov bed material load for the extended detention case is considerably lower than for the existing pond, as might be expected from a substantial increase in runoff storage for the 2 year storm. In contrast, the Meyer-Peter and Muller solution shows a slight (though probably insignificant) increase in bed material load for the extended detention case. This estimate of the bed material load is apparently incorrect and is related to the general shape of the Meyer-Peter and Muller transport function. Meyer-Peter and Muller estimates of total bed material load are primarily sensitive to the amount of discharge that falls below the level necessary to initiation sediment motion. Unlike the erosion control design, the extended detention design does not increase the amount of discharge below this critical level to the level necessary to bring the pond-routed bed material load to predevelopment conditions. The sensitivity of Meyer-Peter and Muller estimates of bed material load to the value of critical discharge for incipient sediment motion presents serious problems when used in an erosion control SWM design. Further discussion of this problem is delayed until Section 7 of this report. The reduction in load estimated by the Goncharov formulation is considered to be the more reliable estimate.

TABLE 6.7

Total Bed Material Loads for Postdevelopment Conditions
at Newport Towne and the 2 year storm

	Goncharov Load (% of predevelopment)	Meyer-Peter & Muller Load (% of predevelopment)
Postdevelopment	471	212
Existing Pond	350	223
Extended Detention Pond	58	234
Erosion Control Pond	46	100

6.3. SITE 2: SNOWDENS MILL

6.3.1 Site Characteristics

Location. Snowdens Mill is a residential development in Montgomery County, Maryland (Figure 2.1). The site is located on Serpentine Way approximately 1/4 mile south of Fairland Road (Mont. County ADC 34, G3). A dry stormwater detention pond installed during development drains an area of 82 acres.

Soils. The Snowdens Mill development lies within the Piedmont physiographic province. The Piedmont is underlain by soils weathered from igneous and metamorphic rocks. Approximately 91% of the Snowdens Mill drainage is underlain by Manor silt loam. These soils are mapped as moderately eroded, most with slopes ranging from 3% to 25% (SCS, 1961). The remaining soils at Snowdens Mill are Worsham silt loam. The Manor silt loam is in S.C.S. hydrologic soil group B and the Worsham silt loam is in group D.

Land Use. Prior to the development for which the SWM facility was designed, the Snowdens Mill area was primarily forested land (~29%) and pasture (~40%) with single family homes on lots of 1/4 acre (~24%), townhouses (~3%) and open space (~4%). The development replaces the forested land and pasture with residences on 1/2 acre lots (~69% of total area). The percent impervious area after development is estimated as 28% (MWCOG, 1989)

Hydrology. The design precipitation regime for the Snowdens Mill site is identical to that for the Newport Towne site. Table 6.8 presents the hydrologic parameters for Snowdens Mill computed using standard U.S. Soil Conservation Service procedures (SCS, 1984 and 1986).

TABLE 6.8
HYDROLOGIC PARAMETERS FOR SNOWDENS MILL

	Predevelopment	Postdevelopment
Drainage Area (acres)	82.13	82.13
Runoff Curve Number	65	73
Time of Concentration (hrs)	0.40	0.30

Table 6.9 provides runoff quantities and peak discharges for the Snowdens Mill site computed using TR-20 and a 24 hour Type II design storm (SCS, 1982).

TABLE 6.9
SCS TR-20 RUNOFF AND PEAK DISCHARGE FOR SNOWDENS MILL

		PREDEVELOPMENT		POSTDEVELOPMENT	
Return Period (years)	Precipitation (inches)	Runoff (inches)	Peak Discharge (cfs)	Runoff (inches)	Peak Discharge (cfs)
1	2.7	0.38	19.7	0.68	55.5
2	3.3	0.65	40.6	1.05	89.7
5	4.3	1.21	84.8	1.75	153.7
10	5.2	1.79	132.4	2.44	216.2
100	7.4	3.42	262.1	4.28	379.0

6.3.2. EXISTING STORMWATER MANAGEMENT FACILITY AND RECEIVING CHANNEL

Stormwater Detention Facility. A dry stormwater detention pond was constructed in 1978 to serve the Snowdens Mill Development. The pond provides a maximum of 225,585 cubic feet (5.17 acre-feet) of stormwater storage. The pond's stage-storage relationship is shown in Table 6.10. The top of the pond embankment is at an elevation of 318.0 feet. The embankment is a Class A earthen dam.

TABLE 6.10
SNOWDENS MILL STAGE-STORAGE RELATIONSHIP

Elevation (ft)	Storage (cu. ft.)	Storage (acre-ft)
303.5	0	0
304.0	1305	0.03
306.0	18945	0.43
308.0	46755	1.07
310.0	80865	1.86
312.0	121815	2.80
314.0	. 170055	3.90
316.0	225585	5.17

The pond has twin 66 inch CMP risers at an elevation of 313.5 feet which serve as both the principal and emergency spillway for the pond. Low flow is passed through a single 15 inch CMP outlet at the pond base (el. 303.5 feet). This pond was designed to comply with the requirements of the Montgomery County District Office of the Soil Conservation Service and the Dam Safety Division of the State of Maryland Department of Natural Resources. The circular risers are designed to provide a peak water level of 316.0 ft for the 50 year storm, which leaves a freeboard of 2.0 feet below the dam top. The low flow orifice provides 2/2 peak discharge control. The pond is well maintained with little or no debris obstructing the outlet structures.

The SWM facility at Snowdens Mill is designed to provide 2/2 peak discharge control and the circular risers provide an emergency spillway for the 50 year storm. The pond routed discharge for the 2 year storm at Snowdens Mill is 15.9 cfs, which is significantly smaller than the predevelopment peak discharge of 40.6 cfs. The pond routed discharge for the 10 year storm is 192 cfs, which is larger than the predevelopment peak discharge of 132 cfs.

Receiving Channel. The receiving stream downstream of the pond outlet is stable with slight channel erosion. There is sparse riprap downstream of the outlet for approximately 30 feet. The channel has a slope of 0.012 for 200 ft downstream of the pond. The main channel is 6 to 10 feet wide; inside the main channel is a smaller low flow channel 3 to 4 feet wide. The stream bed is composed primarily of gravel and sand. Grain size data for bulk samples taken from the channel bottom at four locations are given in Table 6.11 and Figure 6.13. These data do not include armored layers of coarse gravel observed to mantle emergent bars and areas of the channel bottom. The mean size of the gravel layer is 36 mm (0.12 ft). Appendix C presents surveyed cross-sections of the channel downstream of the pond.
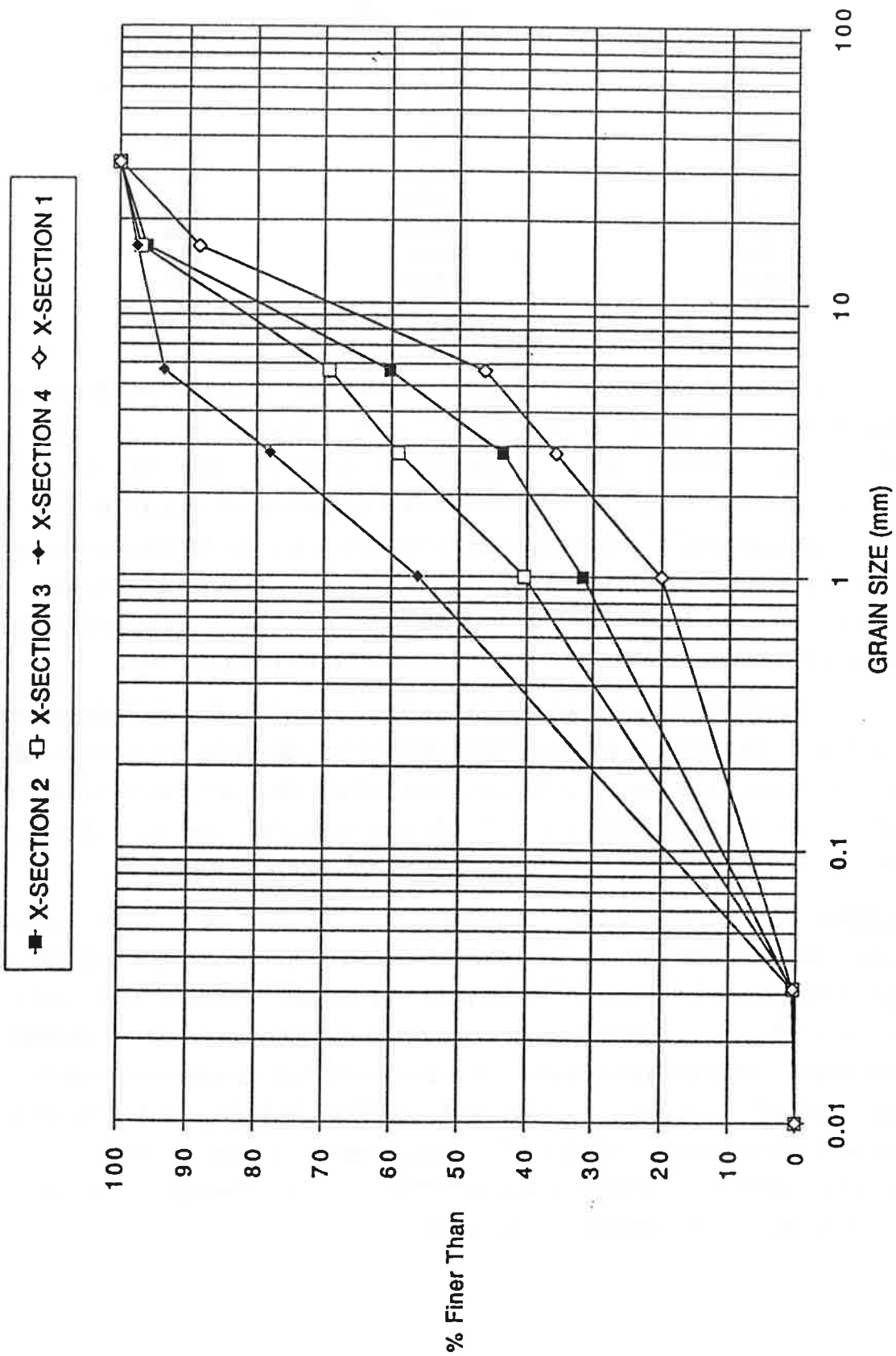
6.13   Grain-size distribution of Snowdens Mill channel sediments.

Table 6.11 Snowdens Mill Grain Size Data

GRAIN SIZE (mm)	CROSS-SECTION #1 (Wt%)	CROSS-SECTION #2 (Wt%)	CROSS-SECTION #3 (Wt%)	CROSS-SECTION #4 (Wt%)
<0.0625	0.5	0.7	0.4	0.7
0.0625-2.00	19.6	30.9	40.0	55.4
2.00-4.00	15.6	12.1	18.6	22.0
4.00-8.00	10.6	16.6	10.3	15.5
>8.00	53.7	39.7	30.9	6.4

Mean flow velocity, flow depth, and bed shear stress above the center section of the downstream channel are plotted in Figure 6.14

A representative grain size of the sediment is necessary to predict the bed material transport rate. The mean size of the gravel layer on the channel bed of 36 mm (0.12 ft..) was used in this study. This value was chosen because of the abundance of this material on the channel surface and because this material is transported at a slower rate than the finer bed material and can therefore accumulate and protect the bed surface before significant amounts of bed erosion can occur. The values of critical shear stress and critical velocity for incipient motion were taken from the work on critical velocity by Fortier and Scobey (1926) and later conversion of these values to shear stress by the U.S. Bureau of Reclamation (cited in McCuen et al., 1987). The values used for Snowdens Mill correspond to the sediment category "graded loam to cobbles when colloidal". This category was chosen because of the broad range of grain sizes evident in the bed samples, and because a fine-grained, cohesive mantle—composed of silts, clays, and organic material—was observed on the bed surface. Such a mantle provides greater resistance to erosion than the cohesionless sediment below. Because this material is present only on the bed surface, only a small proportion of the material is evident in the bulk grain size distributions presented in Table 6.11. Because the Snowdens Mill pond is fully grassed, values of critical shear stress and velocity for the clear water case were used. These are $V_c = 3.75$ ft./s and $\tau_c = 0.38$ lbf/ft^2.

The Snowdens Mill development drains into a small unnamed first-order tributary of the Paint Branch of the Anacostia River. Wetlands of two basic types are located along these channels (USFWS, 1987). A palustrine wetland with a broadleaf deciduous forest (temporarily flooded, Fish and Wildlife Service types PF01A) is identified along the Paint Branch within 2000 ft of the SWM facility. These wetlands, and palustrine open water wetlands (semipermanently flooded, or permanently flooded/intermittently exposed water regimes types POWZ, POWF) are common along the Little Paint Branch channel. The dominant tree species found in this wetland are sycamore, black willow, sweet gum, red maple, river birch, box elder, and pin oak. Riverine
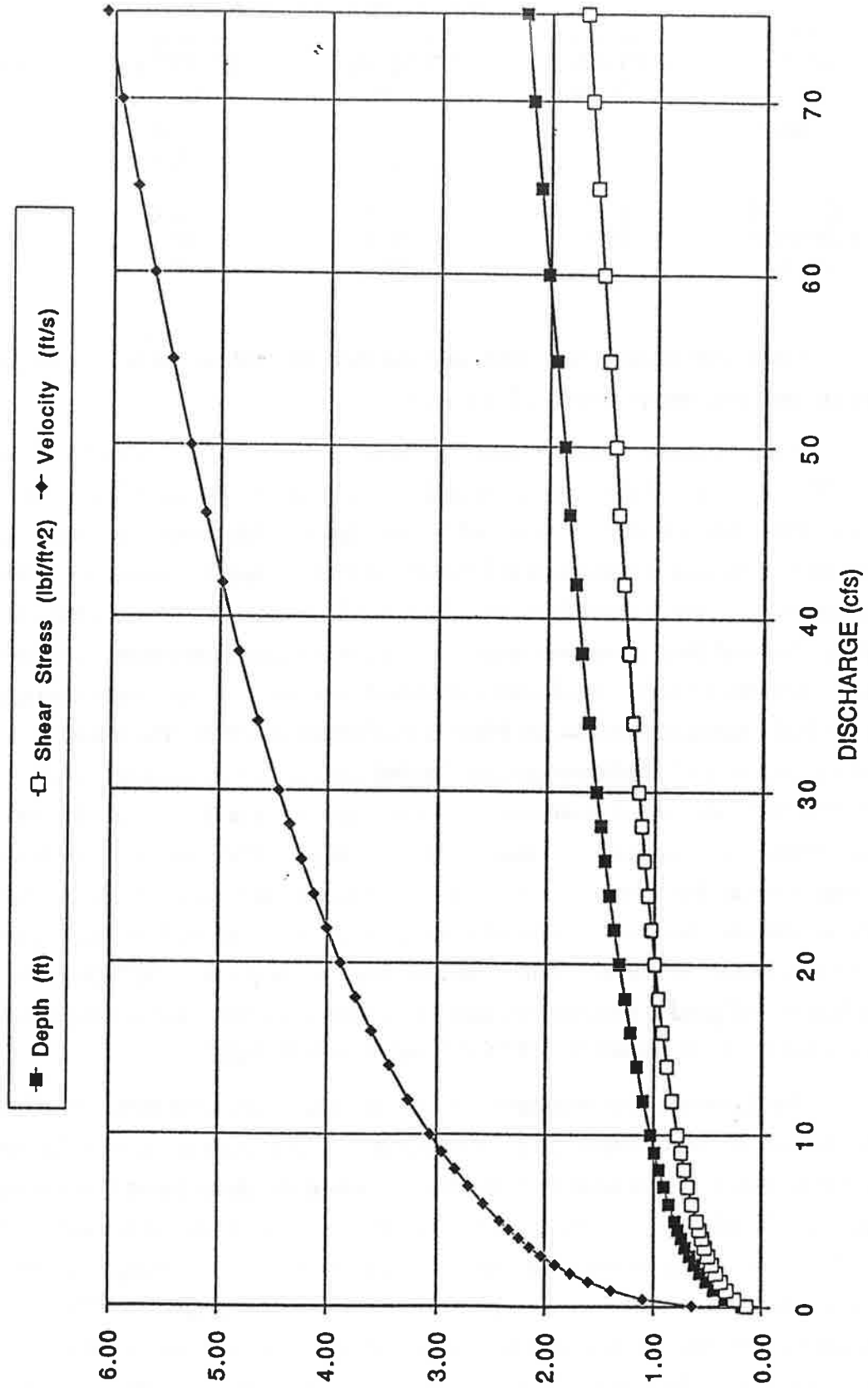
6.14   Flow velocity, depth, and bed shear stress as a function of discharge at Snowdens Mill.

lower perennial wetlands and riverine upper perennial wetlands are also commonly identified along the Paint Branch downstream of Snowdens.Mill (types R2OWH and R3OWH: open water/unknown bottom, permanently flooded).

Reproducing populations of Brown Trout have been identified in portions of the Paint Branch upstream of Capital Beltway (ICPRB, 1989a). Brown Trout spawn in the gravel areas of streams preferring sediment with a mean size of 7-15mm. Largemouth bass, redbreast sunfish, and bluegill sunfish have been observed on Paint Branch downstream of the Snowdens Mill development . Largemouth bass and bluegill sunfish spawn in a variety of substrates, but fine gravel-sand is preferred. Excessive stream bed erosion can erode this substrate; excessive sedimentation from tributaries to the Paint Branch will degrade the spawning grounds.

6.3.3 Response Surfaces

Figures 6.15 to 6.18 present a set of response surfaces for the 2 year storm at Snowdens Mill. The case shown uses the existing stage-storage curve and the existing 50 year circular risers. A 24 hour round, extended detention orifice is added at the base of the pond. This orifice was sized at 0.46 ft using the procedure of Harrington (1987b). The surfaces show the effect of varying the size and elevation of an additional square orifice. The orifice is varied in size from 0 to 4 feet on a side and in elevation from the pond bottom to 5 ft above the pond bottom. These outlet structures represent a convenient suite for evaluating retrofit alternatives. Using a different type of outlet (round orifice, box riser, circular riser) for the additional outlet gave response surfaces nearly identical to those shown in the Figures.

The maximum peak discharge is found with the largest square orifice placed at the bottom of the pond (upper left corner of the Figure 6.15). The peak discharge values fall to a minimum of 14.5 cfs when the square orifice is of zero size (along the abscissa of the plot). The predevelopment 2 year peak discharge of 40.6 is satisfied by a range of variable orifice sizes. Time to drawdown (Figure 6.16) increases with increasing elevation of the square orifice over the entire range of elevations, but varies with the orifice size only in the lower range of sizes.

The total bed material load computed using the Goncharov formula follows a pattern that matches the peak discharge (Figure 6.17). The highest loads are found with the largest square orifice located at the pond bottom. The bed material loads drop to zero at a square orifice size of approximately 1.25 ft to 1.75 ft. This result, combined with Figure 6.15, indicates that a 24 hour extended detention orifice provides erosion control at the 2 year level, as well as peak discharge control of the 2 year storm.

The total bed material load computed using the Meyer-Peter and Muller formula (Figure 6.18) shows a minimum where the square orifice is reduced to zero size (along the plot
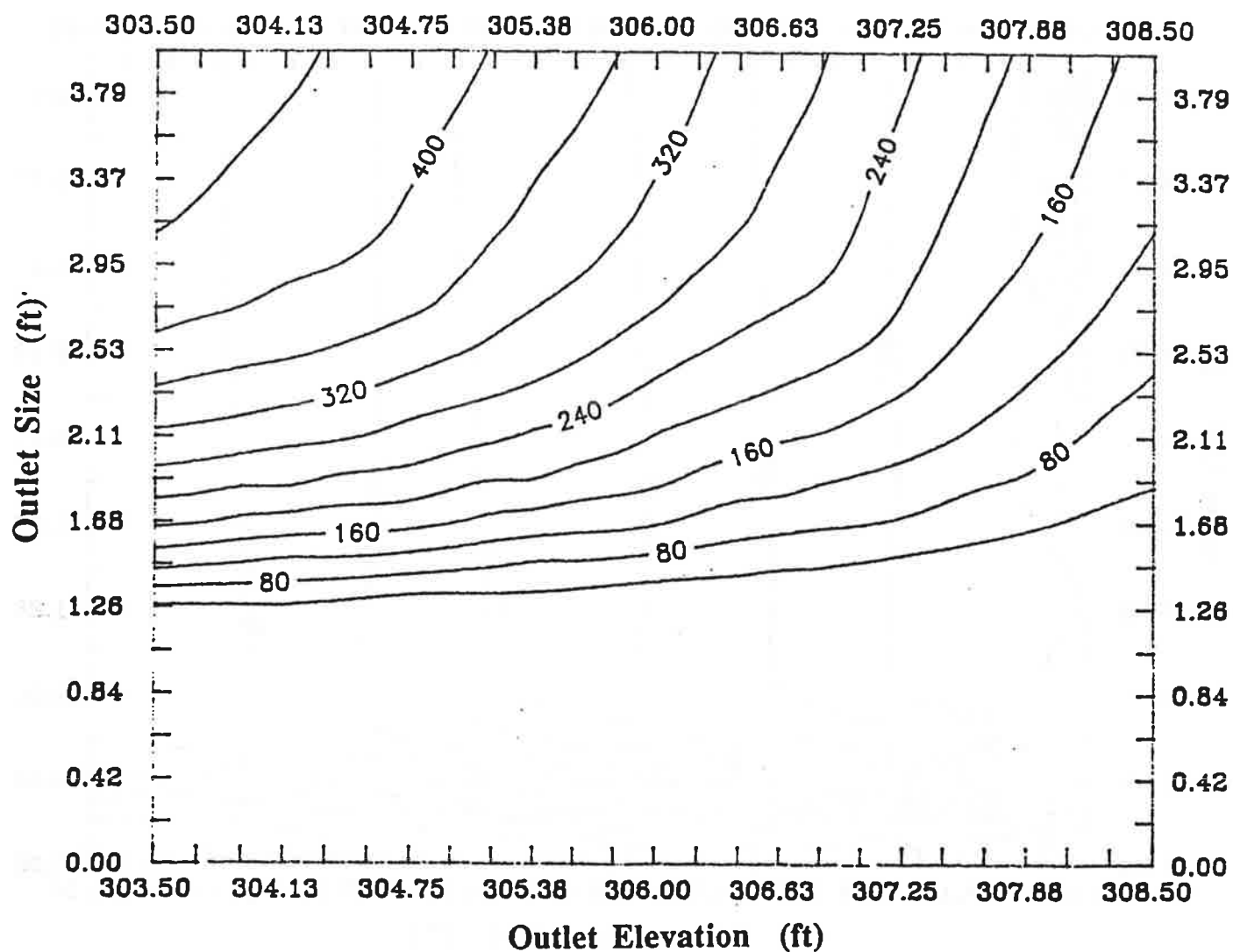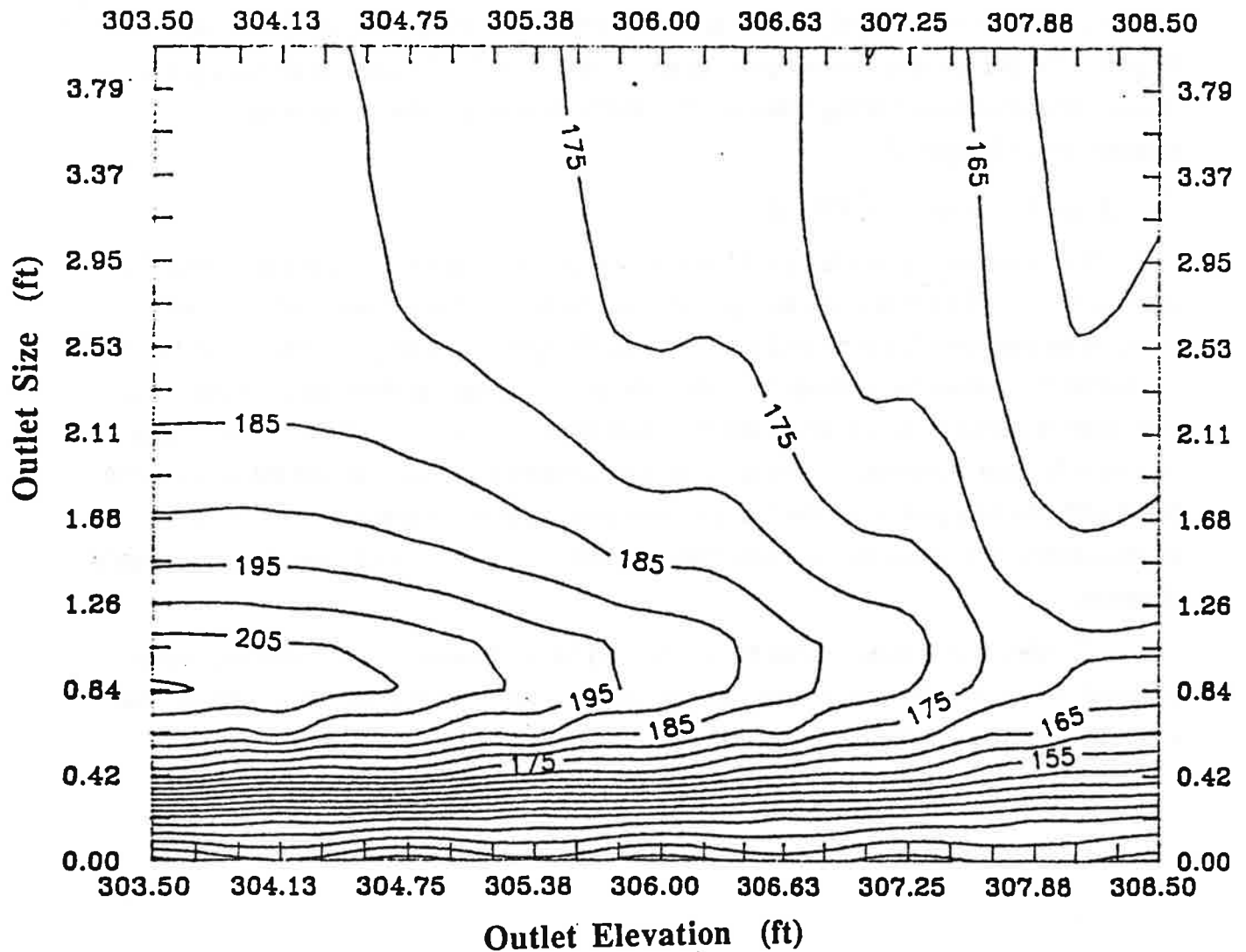
Snowdens Mill: 2 year Peak Discharge (cfs)
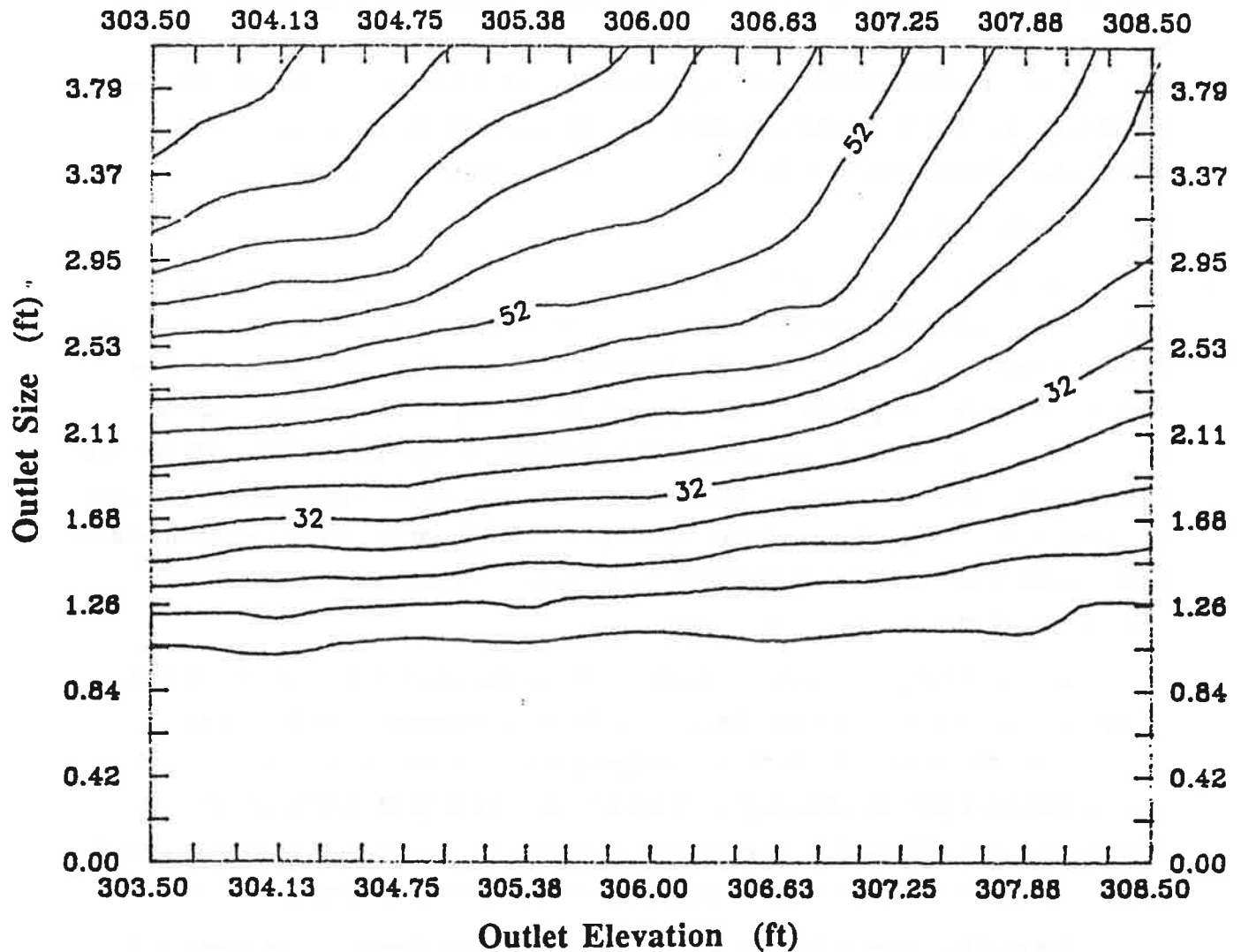
6.15   Response surface for a 24 hr extended detention orifice (0.46 ft round orifice) at pond bottom, the existing 50 year circular risers, and a square orifice varying in size from 0 to 4 ft and in elevation from 303.5 ft to 308.5 ft:
Response of peak discharge (cfs) for the 2 yr storm at Snowdens Mill.

Snowdens Mill: 2 year Time to Drawdown (minutes)
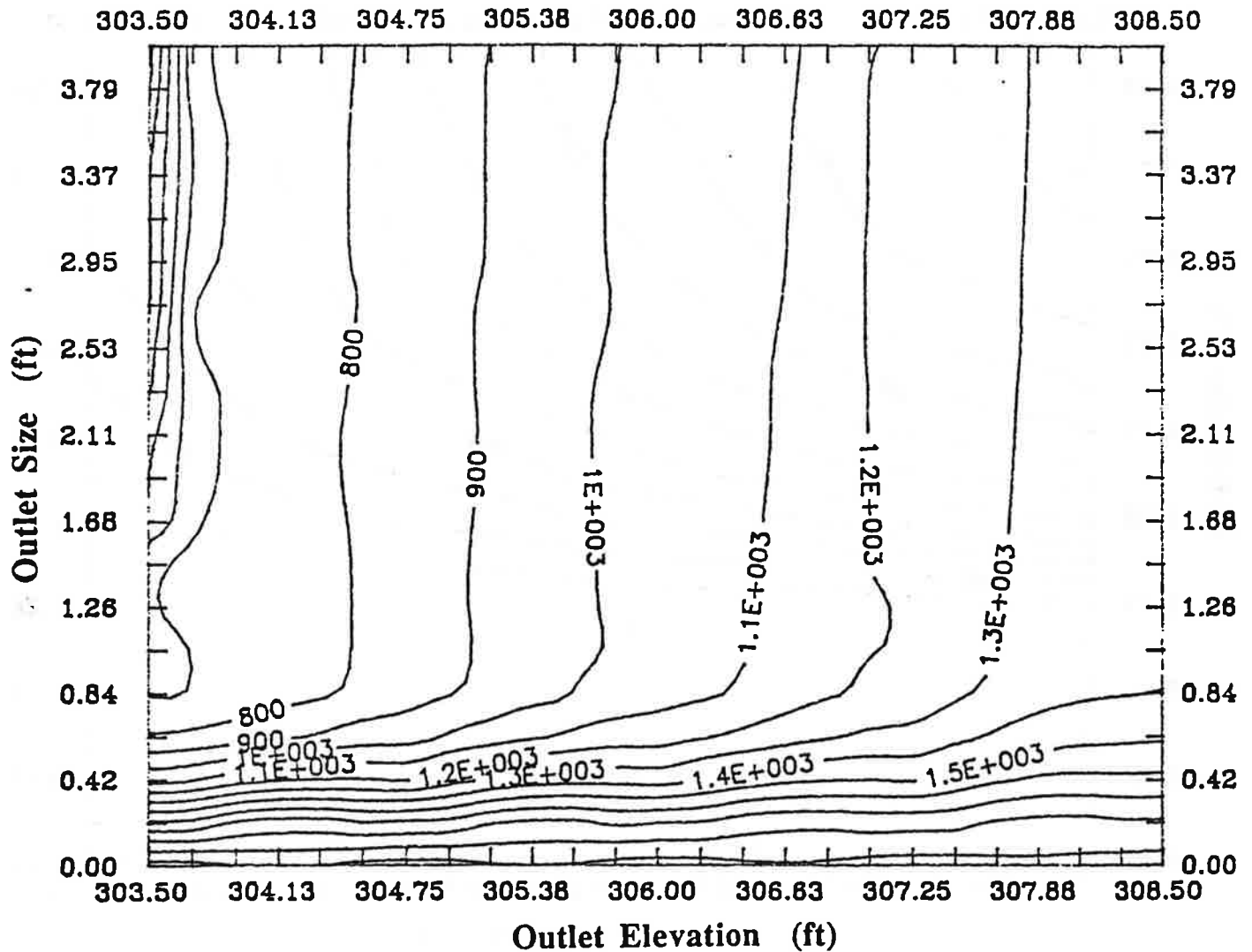
6.16   Response surface for a 24 hr extended detention orifice (0.46 ft round orifice) at pond bottom, the existing 50 year circular risers, and a square orifice varying in size from 0 to 4 ft and in elevation from 303.5 ft to 308.5 ft:
Response of drawdown time (minutes) for the 2 yr storm at Snowdens Mill.
The symbol E signifies scientific notation (e.g. 1.1E+003 = 1.1 x 10^3).

Snowdens Mill: 2 year Goncharov Load
(% of predevelopment load)

6.17 Response surface for a 24 hr extended detention orifice (0.46 ft round orifice) at pond bottom, the existing 50 year circular risers, and a square orifice varying in size from 0 to 4 ft and in elevation from 303.5 ft to 308.5 ft:
Response of Goncharov bed material load (% of predevelopment load)) for the 2 yr storm at Snowdens Mill.

Snowdens Mill: 2 year Meyer-Peter & Muller Load
(% of predevelopment load)

6.18   Response surface for a 24 hr extended detention orifice (0.46 ft round orifice) at pond bottom, the existing 50 year circular risers, and a square orifice varying in size from 0 to 4 ft and in elevation from 303.5 ft to 308.5 ft:
Response of Meyer-Peter and Muller bed-material load (% of predevelopment load)) for the 2 yr storm at Snowdens Mill.

abscissa). The bed material load at this level is just under the predevelopment load (98% of predevelopment). Thus, the Meyer-Peter and Muller formula and the Goncharov formula lead to the same conclusion: a 24 hour extended detention orifice provides erosion control at the 2 year level, as well as peak discharge control of the 2 year storm. The Meyer-Peter and Muller formula also shows a local minimum for large values of the square orifice located at an elevation well above the pond bottom, although this local minimum is significantly greater than that found along the abscissa. This pattern is roughly similar to the equivalent plot for the Newport Towne site (Figure 6.6). For the same reasons given in section 6.2.3, this local minimum does not provide a reasonable transport-minimizing solution. This local minimum also does not provide 24 hour extended detention control.

6.3.4. Erosion Control SWM Design

The results of Figures 6.15, 6.17, and 6.18 show that an orifice designed to provide 24 hour detention of the 1 year storm also provides erosion control. Even though an ED orifice provides erosion control, a variety of solutions exist that provide not only erosion control, but also 2/2 and 10/10 peak discharge control and meet 100 year emergency spillway requirements. The peak water level for the 1 year storm and the 1 year 24 hour ED orifice is 311.8 ft. The 10 year outlet can be located anywhere above this elevation; the combination of sizes and elevations for this outlet will affect the peak water level for the 10 year storm, which in turn has an effect on the allowable elevations of the 100 year emergency spillway and possible need for additional storage in the pond.

The design procedures for the Goncharov and Meyer-Peter and Muller transport formulas are nearly identical. However, because the optimum solution found in each case is different, the complete set of design steps is given below.

DESIGN PROCEDURE: SNOWDENS MILL (Goncharov)

REQUIREMENTS:

    a) 24 hour extended detention for the 1 year storm

    b) erosion control for the 2 year storm

    c) peak discharge control for the 2 year and 10 year storms

    d) minimize elevation of the 100 year spillway & top of dam

1)   Calculate bed material load for the 2 year predevelopment storm using the Goncharov formula, yielding a total load of 2.83 X 10^7 lbs.

2)   Compute bed material load for 2 year postdevelopment storm using only one outlet of varying size at the base of the pond.

3)   For the 2 year storm, plot bed material load (expressed as a percentage of predevelopment load) as a function of orifice size (expressed as the design detention time for a 1 year storm using the method of Harrington, 1987b). (Figure 6.19). Compare the size of outlet required for 2 year erosion control with the size of outlet required for 1 year 24 hour extended detention

Diameter of a circular orifice for 1 year, 24 hour ED control = 0.46 ft ; Diameter for EC is 1.7 ft. Because the ED outlet is smaller than the EC outlet, use the 24 hour ED outlet to provide both 24 hour extended detention and erosion control.

Place a 0.46 ft. circular orifice at 303.5 ft (pond bottom)

4)   Find maximum pond water level for ED outlet and 1 year storm, which is 311.8 ft. This is the minimum elevation for the 10 year weir that will provide both 2 year erosion control and 1 year, 24 hour extended detention. It is assumed here that any design that provides 2 year erosion control will also provide 2 year peak discharge control.

5)   Check if all SWM requirements are met by changing only the existing low flow orifice to the ED outlet (using the existing stage-storage curve and upper outlets, which is the most economical solution). For the 2 year storm, erosion control is satisfied (estimated pond-routed bed material load is zero), as is the 2 year peak discharge requirement (10.0 cfs compared to 40.6 cfs). Maximum water level for the 1 year storm is 311.8 ft, which is below the existing circular risers, so 1 year, 24 ED is satisfied. For the 10 year storm, peak discharge is 193 cfs, which is greater than the 10 year predevelopment peak of 132.4 cfs. Therefore, the 10 year peak discharge requirement is not satisfied. Furthermore, the maximum pond water level for the 10 year storm is

SNOWDENS MILL WITH ONLY ONE OUTLET AT POND BOTTOM

■ MPM % OF PREDEVELOPMENT BED-MATERIAL LOAD   □ GON % OF PREDEVELOPMENT BED-MATERIAL LOAD

DESIGN EXTENDED DETENTION TIME
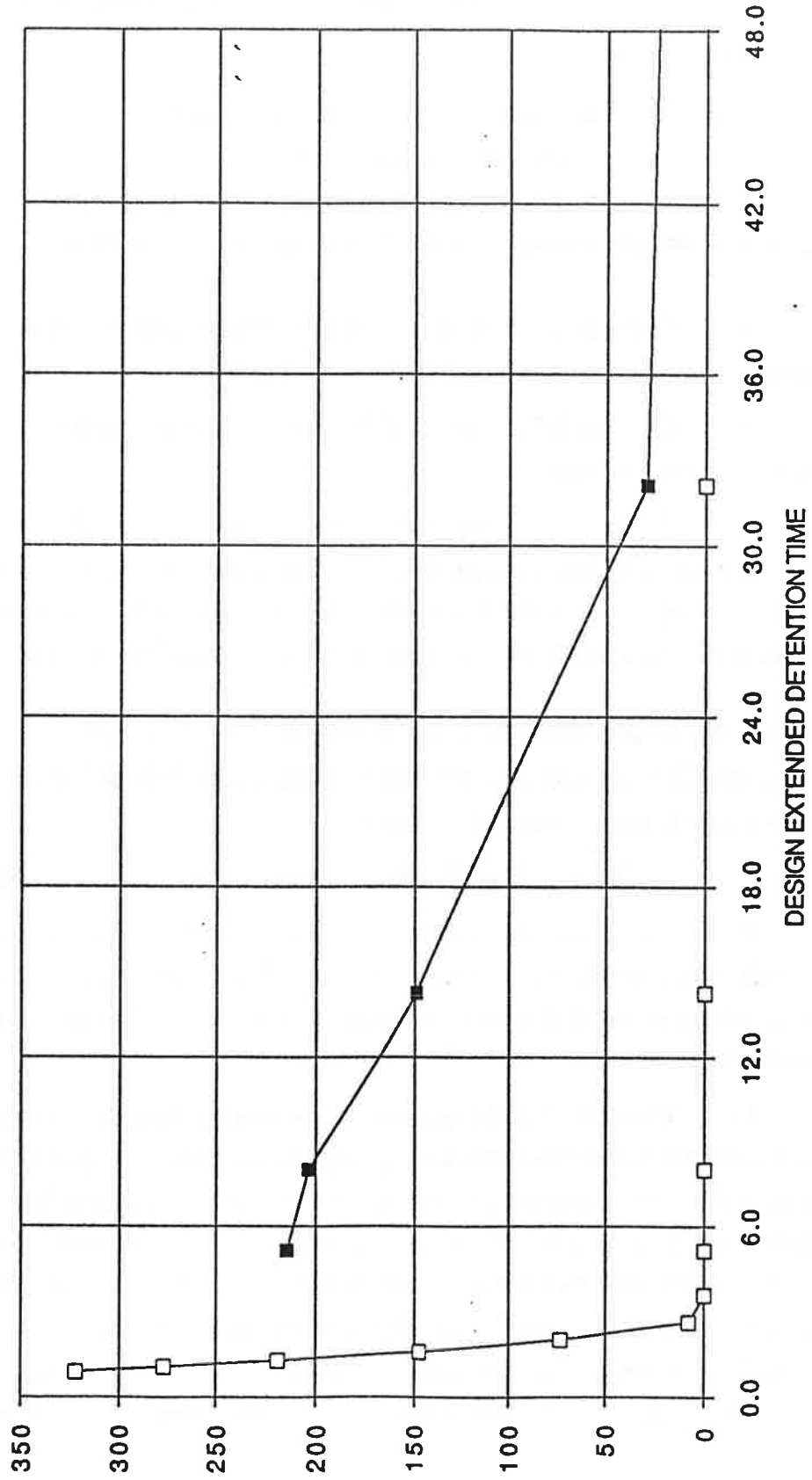
6.19   Variation of bed-material load (expressed as a percent of predevelopment load) as a function of low flow outlet size (expressed as design detention time for a 1 yr storm) at Snowdens Mill.

315.0 ft, which is only 3 ft below the top of the current dam, which suggests that a 10 year outlet must be designed.

6)   Determine the feasible range of sizes and elevations for the 10 year outlet. Constraints:

• erosion control for the 2 year storm (all outlets placed above 311.8 ft. provide erosion control for the 2 year storm).

• peak discharge control for the 10 year storm (Figure 6.20)

• minimize the elevation of the 100 year emergency spillway invert. Emergency spillway invert to be set at an elevation equal to the maximum water level for the 10 year storm, or 1 ft above the 10 year outlet elevation, whichever is higher (Harrington, 1987a). Maximum pond water levels for the 10 year storm are shown in Figure 6.21.

7)   The combination of elevation and size for the 10 year outlet that satisfies the last two constraints is:

Elevation: 312.2 ft

Size: 4.4 ft (diameter of circular riser)

Maximum 10 year water level: 315.3 ft.

This solution is marked as circles on Figures 6.20, and 6.21.

Place a 4.4 ft diameter circular riser at 312.2 ft.

Place a 100 year weir at 315.3 ft.

8)   Size the 100 year weir to pass the 100 year storm at a head of no more than 1.7 ft., taking into account the storage of the pond and the discharge through the lower weirs.

Snowdens Mill: 10 year Peak Discharge (cfs)
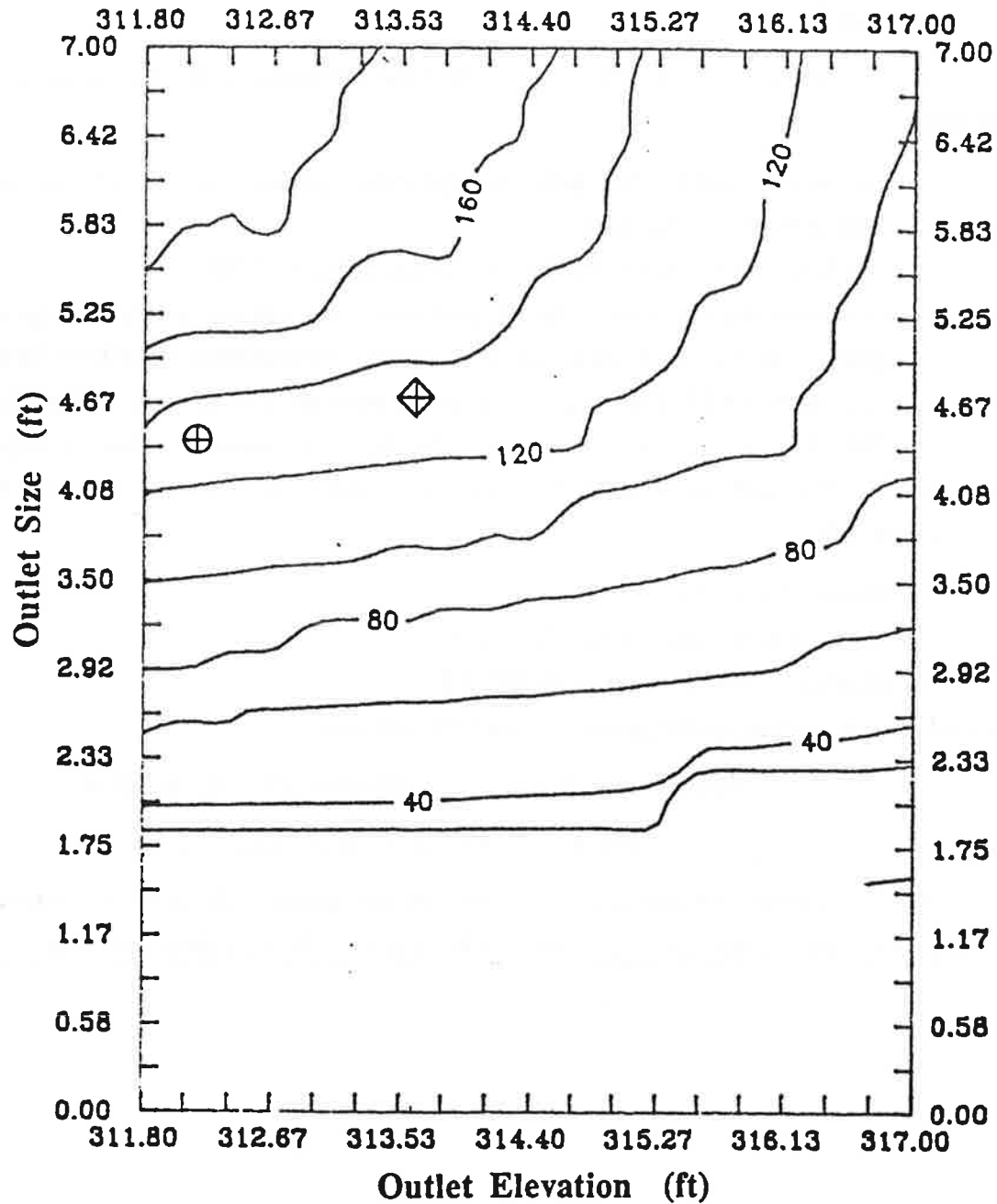
6.20 Response surface for a 24 hr extended detention orifice (0.46 ft round orifice) at pond bottom and a circular riser varying in diameter from 0 to 7 ft and in elevation from 311.8 ft to 317.0 ft:
Response of peak discharge (cfs) for the 10 yr storm at Snowdens Mill. Erosion control designs:
Goncharov formula shown as a circle; Meyer-Peter and Muller formula shown as a diamond.

Snowdens Mill: 10 year Maximum Pond Water Level (feet elevation)
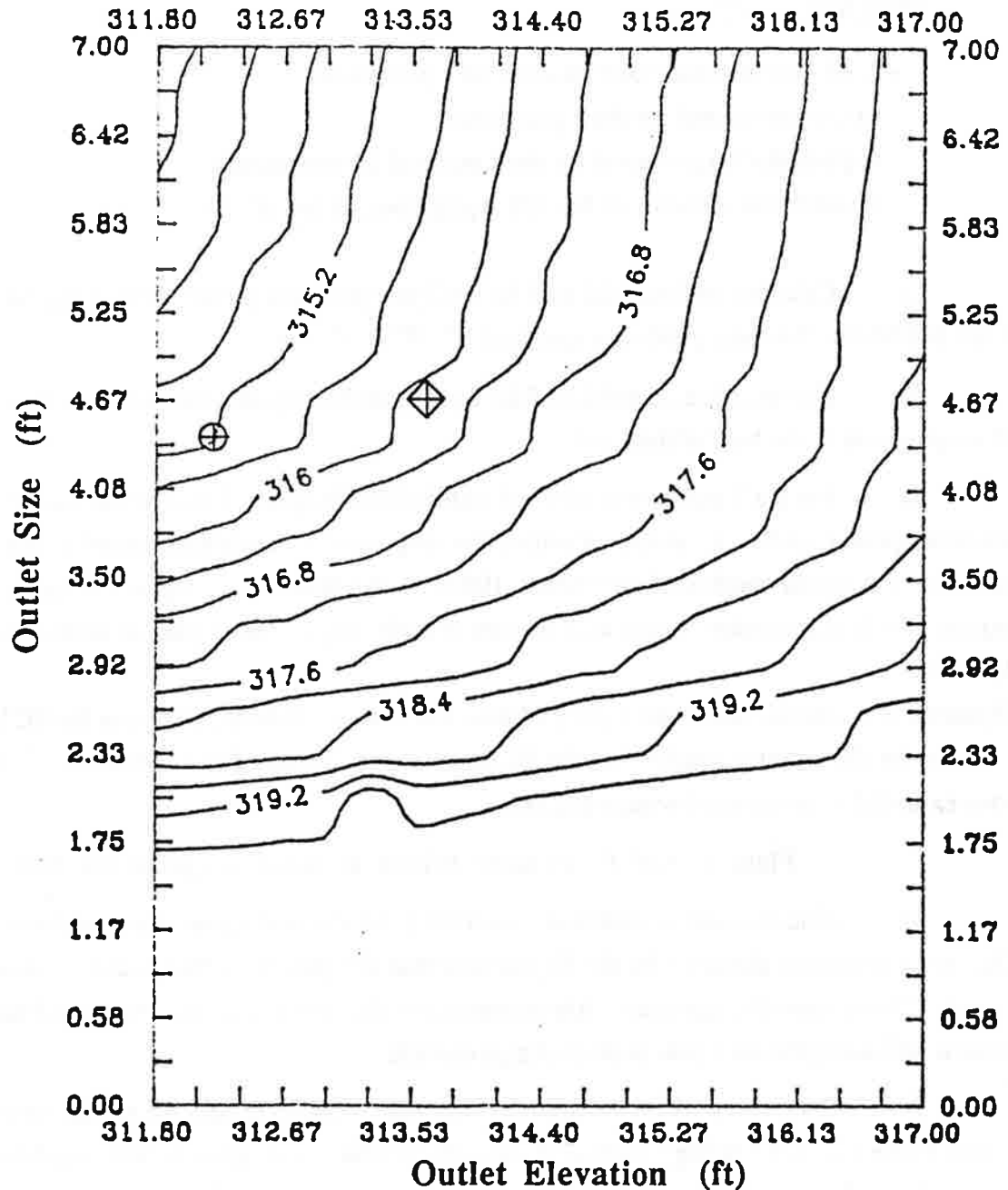
6.21 Response surface for a 24 hr extended detention orifice (0.46 ft round orifice) at pond bottom and a circular riser varying in diameter from 0 to 7 ft and in elevation from 311.8 ft to 317.0 ft:
Response of peak pond water level for the 10 yr storm at Snowdens Mill. Erosion control designs: Goncharov formula shown as a circle; Meyer-Peter and Muller formula shown as a diamond.

DESIGN PROCEDURE: SNOWDENS MILL (Meyer-Peter and Muller)

REQUIREMENTS:

     a) 24 hour extended detention for the 1 year storm

     b) erosion control for the 2 year storm

     c) peak discharge control for the 2 year and 10 year storms

     d) minimize elevation of the 100 year spillway & top of dam

1)   Calculate bed material load for the 2 year predevelopment storm using the Meyer-Peter and Muller formula, yielding a total load of 1.87×10^5 lbs.

2)   Compute bed material load for 2 year postdevelopment storm using only one outlet of varying size at the base of the pond.

3)   For the 2 year storm, plot bed material load (expressed as a percentage of predevelopment load) as a function of orifice size (expressed as the design detention time for a 1 year storm using the method of Harrington, 1987b). (Figure 6.19). Compare the size of outlet required for 2 year erosion control with the size of outlet required for 1 year 24 hour extended detention

Diameter of a circular orifice for 1 year, 24 hour ED control = 0.46 ft ; Diameter for EC is 0.48 ft. Because the ED outlet is smaller than the EC outlet, use the 24 hour ED outlet to provide both 24 hour extended detention and erosion control.

Place a 0.46 ft. circular orifice at 303.5 ft (pond bottom)

4)   Find maximum pond water level for ED outlet and 1 year storm, which is 311.8 ft. This is the minimum elevation for the 10 year weir that will provide both 2 year erosion control and 1 year, 24 hour extended detention. It is assumed here that any design that provides 2 year erosion control will also provide 2 year peak discharge control.

5)   Check if all SWM requirements are met by changing only the existing low flow orifice to the ED outlet (using the existing stage-storage curve and upper outlets, which is the most economical solution). For the 2 year storm, erosion control is satisfied (estimated pond-routed bed material load is zero), as is the 2 year peak discharge requirement (10.0 cfs compared to 40.6 cfs). Maximum water level for the 1 year storm is 311.8 ft, which is below the existing circular risers, so 1 year, 24 ED is satisfied. For the 10 year storm, peak discharge is 193 cfs, which is greater than the 10 year predevelopment peak of 132.4 cfs. Therefore, the 10 year peak discharge requirement is <u>not</u> satisfied. Furthermore, the maximum pond water level for the 10 year storm is

315.0 ft, which is only 3 ft below the top of the current dam, which suggests that design alternatives for the 10 year outlet are worth considering.

6) Determine the feasible range of sizes and elevations for the 10 year outlet. Constraints:

- erosion control for the 2 year storm (Figure 6.22).
- peak discharge control for the 10 year storm (Figure 6.20)
- minimize the elevation of the 100 year emergency spillway invert. Emergency spillway invert to be set at an elevation equal to the maximum water level for the 10 year storm, or 1 ft above the 10 year outlet elevation, whichever is higher (Harrington, 1987a). Maximum pond water levels for the 10 year storm are shown in Figure 6.21.

7) The combination of elevation and size for the 10 year outlet that satisfies the last two constraints is:

Elevation: 313.6 ft

Size: 4.7 ft (diameter of circular riser)

Maximum 10 year water level: 316.0 ft.

This solution is marked as diamonds on Figures 6.20, 6.21, and 6.22.

Place a 4.7 ft diameter circular riser at 313.6 ft.

Place a 100 year weir at 316.0 ft.

8) Size the 100 year weir to pass the 100 year storm at a head of no more than 1.0 ft., taking into account the storage of the pond and the discharge through the lower weirs.

Snowdens Mill: 2 year Meyer-Peter & Muller Load
(% of predevelopment load)
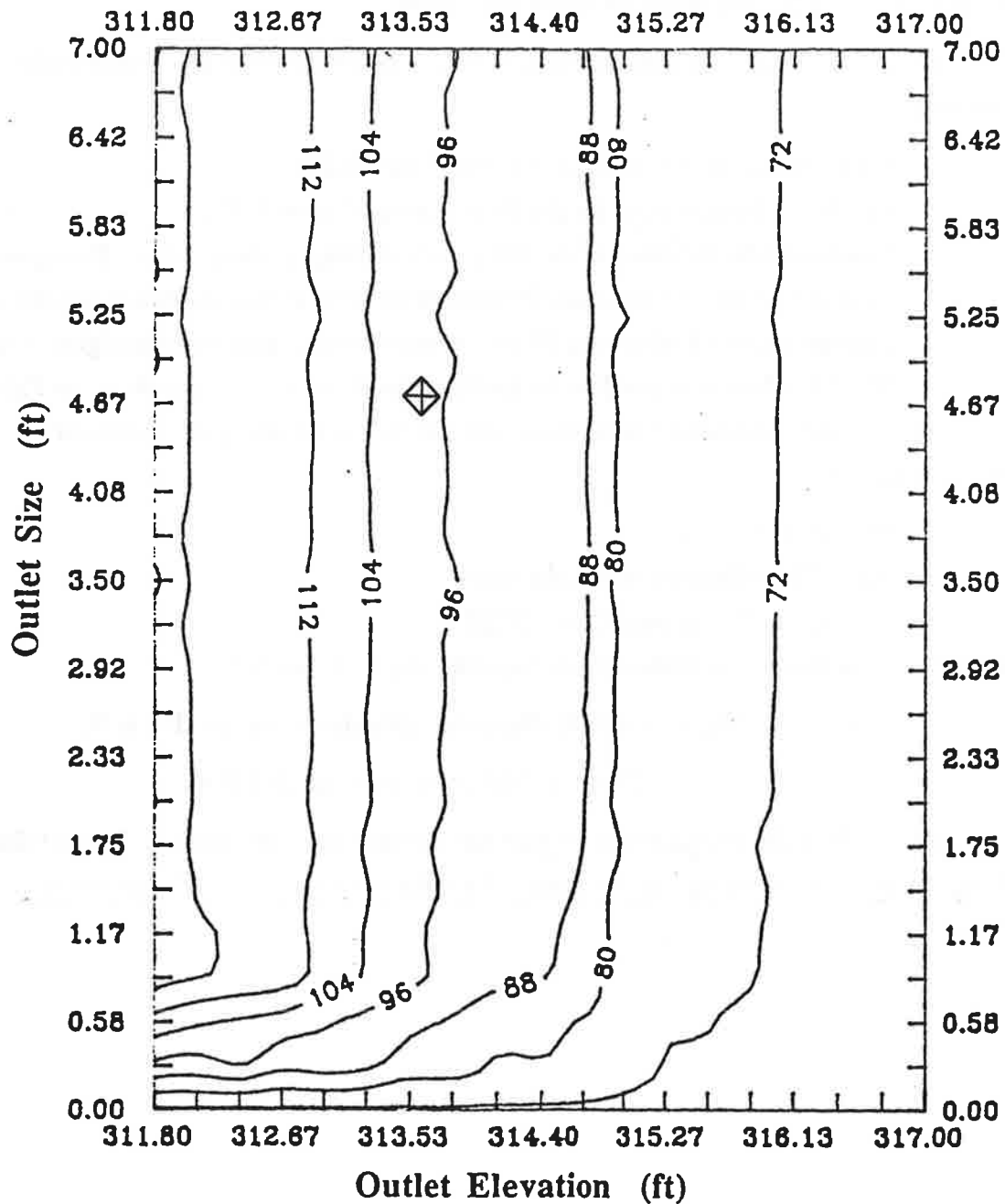
6.22 Response surface for a 24 hr extended detention orifice (0.46 ft round orifice) at pond bottom and a circular riser varying in diameter from 0 to 7 ft and in elevation from 311.8 ft to 317.0 ft:
Response of Meyer-Peter and Muller bed-material load (% of predevelopment load)) for the 2 yr storm at Snowdens Mill. Erosion control design using the Meyer-Peter and Muller formula shown as a diamond.

Table 6.12 summarizes the erosion control design results for Snowdens Mill. The Meyer-Peter and Muller solution requires a greater elevation for the 10 year outlet in order to decrease the amount of flow over the 10 year orifice during the 2 year storm, thereby maximizing the amount of runoff released below the discharge for incipient sediment motion and bringing 2 year bed material loads below the predevelopment levels. The higher elevation of the 10 year outlet then forces the 100 year outlet to be located exactly 2 ft below the top of the existing dam, which is the maximum allowable elevation for the current dam height and SCS regulations for the 100 year emergency spillway.

Table 6.12 Erosion Control Designs for Snowdens Mill

		Low Flow Orifice				10 Year Outlet		100 Year Spillway	Dam Height
Transport Formula	Type	Elevation (ft)	Size (ft)	Detention Time (1 yr storm)	Type	Elevation (ft)	Size (ft)	Elevation (ft)	Elevation (ft)
Goncharov	round orifice	303.5	0.46	24 hr	CMP riser	312.2	4.4	315.3	318.0
Meyer-Peter and Muller	round orifice	303.5	0.46	24 hr	CMP riser	313.6	4.7	316.0	318.0

The storage volume required by the erosion control designs is equivalent to 0.94 inches over the 82 acre drainage area. This storage depth corresponds to 138% of the 1 year design storm runoff and 90% of the 2 year runoff. In the case of the Goncharov solution, the erosion control criterion is not responsible for the amount of storage volume required. A 24 hour extended detention orifice is used in this solution. Because any size and elevation of the 10 year orifice above the 1 year pond level produces an outflow hydrograph that satisfies the 2 year bed material load constraint using the Goncharov equation, the constraints on pond storage result entirely from the 10 year and 100 year requirements when using the 24 extended detention orifice. The Meyer-Peter and Muller equation forces a different design because the 2 year bed material load constraint using the Meyer-Peter and Muller equation requires a 10 year outlet that is slightly larger and 1.4 ft higher than that required by the Goncharov equation. In both cases, however, the existing pond storage is sufficient to provide erosion control and meet the existing SWM requirements, although in the case of the Meyer-Peter and Muller solution, the freeboard for the 100 year spillway may prove to be too small in a final design, and therefore increased pond storage may prove necessary.

6.3.5. Sediment Yields under different SWM Plans

Table 6.13 presents the bed material loads for a 2 year storm and a variety of different postdevelopment conditions. The values for an extended detention pond were computed for a case with a 1 year 24 hour extended detention orifice (0.46 ft diameter) at the bottom of the pond and, for 2 year peak discharge control, a 4 ft circular riser at an elevation of 311.8 ft. The elevation of the 2 year weir was set at the maximum pond elevation for the 1 year storm and the ED orifice. The 2 year peak discharge requirement was satisfied by all sizes (including zero) of riser at elevation 311.8 ft, so the riser diameter of 4 ft was chosen to provide a tradeoff between the height of the 2 year peak water level and a convenient size of the riser. Both the Goncharov and Meyer-Peter and Muller bed material loads were found to be insensitive to the diameter of the 2 year riser.

The postdevelopment drainage with no pond produces bed material loads significantly in excess of the predevelopment loads, particularly for the Goncharov solution. The Goncharov formula predicts zero bed material load for the existing pond, and a value of 4% of the predevelopment load for the extended detention pond. The difference between the existing and extended detention pond loads occurs because some water flows over the 2 year weir for the ED design, while no discharge flows over the much higher 50 year riser for the existing pond. Both values of Goncharov load are highly dependent on the estimate of the critical mean channel velocity at which sediment motion begins. A value of 3.75 ft/s was used for the computations presented here. For the extended detention example, total loads of 45%, 30%, and 13% of predevelopment were computed for critical velocity values of 2.5 ft/s, 3.0 ft/s, and 3.5 ft/s respectively. The Meyer-Peter and Muller bed material load estimate for the existing pond shows a slight increase over the no-pond case. The Meyer-Peter and Muller load for the extended detention pond drops to 130% of predevelopment values.

TABLE 6.13
Total Bed Material Loads for
Postdevelopment Conditions at Snowdens Mill for the 2 year storm.

	Goncharov Load (% of predevelopment)	Meyer-Peter & Muller Load (% of predevelopment)
Postdevelopment	519	180
Existing Pond	0	206
Extended Detention Pond	4	130
Erosion Control Pond	0	98

7. EVALUATION OF EROSION CONTROL SWM METHODOLOGY

7.1. SENSITIVITY ANALYSES

Computation of the erosion control provided by different SWM alternatives requires choices to be made among the different computational schemes available. This section provides an evaluation of the sensitivity of our erosion control estimates to the choice of detention pond outlet structure and the choice of transport formulation.

7.1.1. Type of pond outlet

Because a variation in hydraulic performance exists among different pond outlet structures, the effect on downstream hydraulics (and therefore erosion control design) of different pond outlet types was examined. The response surfaces used for developing the erosion control designs provide a ready means for evaluating the effect of outlet type on downstream hydraulics. Four different outlet structures (round orifice, square orifice, circular riser, box riser) were examined using the 2 year storm and the Snowdens Mill site. Each structure was varied in both size and elevation. Each structure was considered in combination with a 24 hour extended detention orifice located at the bottom of the pond and the two existing 50 year circular risers. It was found that the type of structure had no significant effect on the variation with outlet size and elevation of peak discharge, time to drawdown, or bed material load. The flexibility suggested by these results indicates that a variety of outlet structures may be considered in the design process, providing opportunities to reduce the costs associated with a modified pond design.

7.1.2. Transport Formulation

A very large number of sediment transport formulas for stream bed material exist in the literature (see Vanoni, 1975, Brownlie, 1981; Gomez and Church, 1989 for summaries) and the exact formula that may be applicable to any particular small stream channel, such as those examined in this report, is difficult to evaluate. The salient features of a transport formulation that affect estimates of erosion control are (1) the shape of the relation between transport rate with discharge ($\partial q_b/\partial q_t$ and $\partial^2 q_b/\partial q_t^2$, where q_b is bed material load from a channel of given width and over a time period t, and q_t is the average water discharge during the time period t) and (2) the estimate of the critical discharge for initiation of sediment transport (q_c). To illustrate the relation of transport function to erosion control, Figure 7.1 presents a plot of sediment transport rate as a function of discharge for six different transport formulas. These include, in addition to the Goncharov and Meyer-Peter and Muller formulas, the formulas of Schoklitsch and Shields, and the bed load and total load formulas of Bagnold (a convenient summary of these formulas may be found in Vanoni, 1975 or Simons and Senturk, 1976). The transport formulas are plotted for the channel downstream of the Snowdens Mill demonstration site.

SNOWDENS MILL: DOWNSTREAM TRANSPORT

Legend:
- SHIELDS (lbf)
- Goncharov (tons)
- Schoklitsch (lbf)
- M-P&M (lbf)
- Bagnold Bed (lbf)
- Bagnold total (lbf)

X-axis: DISCHARGE (cfs) — 0.0, 10.0, 20.0, 30.0, 40.0, 50.0, 60.0, 70.0, 80.0, 90.0

Y-axis: 0,000; 5,000; 10,000; 15,000; 20,000; 25,000; 30,000; 35,000; 40,000; 45,000; 50,000
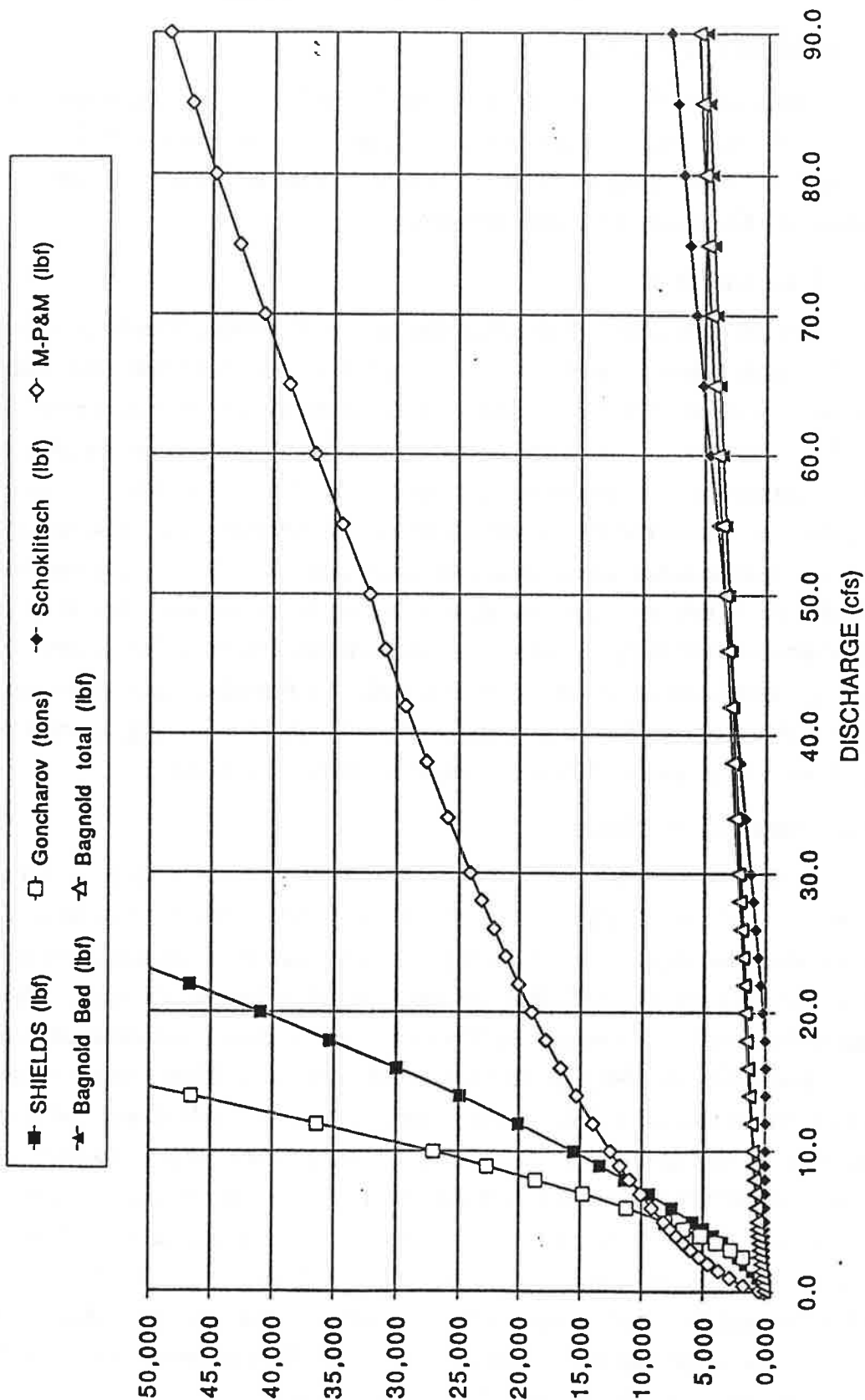
7.1   Variation of bed material transport with discharge using six different transport formulas and the Snowdens Mill channel.

BED MATERIAL LOAD MINIMIZATION WITHOUT A CRITICAL DISCHARGE

The effect of the shape of transport function ($\partial^2 q_b / \partial q_t^2$) on an erosion control design will first be considered independent of the value of q_c. As may be seen in Figure 7.1, three of the formulas (Meyer-Peter & Muller and both Bagnold formulas) are concave. Two of the formulas (Shields and Goncharov) are convex. The Schoklitsch formula is linear, once the critical shear stress is exceeded. The shape of the transport function can have a profound effect on the nature of an erosion control design. The optimum erosion control solution for a concave function would involve passing a maximum amount of transport in a minimum amount of time. In other words, the optimum solution for such a function discharges as much flow as possible at levels where the marginal increase in transport (for a given increase in discharge) is the smallest. The erosion control solution for a convex transport function is exactly the opposite; erosion control would be achieved by storing the maximum amount of water and releasing it at the lowest possible discharge. This again corresponds to allocating runoff to the discharge at which the marginal increase in transport is smallest. The Schoklitsch solution is linear in discharge, and therefore any discharge pattern will provide exactly the same amount of bed material load (assuming $q_c = 0$).

The effect of transport function shape on erosion control design can be developed explicitly. To identify the transport minimizing release strategy, consider the stormwater facility as if it were an actively operated reservoir. In this case the operating problem would be to find a set of discharges that minimize the aggregate downstream transport while releasing the entire volume of runoff. Suppose that $(q_t)^a$ represents the bed material load associated with the average discharge q_t during the time interval t, and that the entire runoff volume R must be released in no more than T time periods. The problem is to identify the set of discharges $(q_1, q_2, ...q_T)$ that minimizes the aggregate transport:

$$\text{Min} \sum_{t=1}^{T} (q_t)^a$$

subject to the constraint

$$\sum_{t=1}^{T} q_t = R$$

The optimal discharge sequence can be found by solving the lagrangian:

$$\text{Min } f(q_t) = \sum_{t=1}^{T} (q_t)^a - L (\sum_{t=1}^{T} q_t - R)$$

The first order conditions for a minimum require

$$1. \quad \frac{\partial f(q_t)}{\partial q_t} = a \, q_t^{(a-1)} - L = 0 \qquad\qquad t = 1, 2, \dots T$$

$$2. \quad \frac{\partial f(q_t)}{\partial L} = \sum_{t=1}^{T} q_t - R = 0$$

Condition 2 effectively enforces the runoff volume constraint.

Solving (1) for qt requires

$$q_t = [a/L]^{(1-a)} \qquad\qquad t = 1,2,\dots T$$

i.e. the discharge is constant for all time periods. Substituting a constant discharge in necessary condition (2) yields $q_t = R/T$, t = 1,2,...T. This constant release strategy is most closely approximated in passively controlled stormwater management facilities by an extended detention release strategy.

The second order necessary conditions require that

$$3. \quad \partial^2 f(q_t)/\partial(q_t)^2 = a\,(a-1)\,q_t^{(a-2)} \geq 0$$

These are the necessary conditions for a minimum when $f(q_t)$ is a continuous differentiable function. The transport minimizing strategy will depend upon which of three possible cases are identified as representing transport in the channel.

Case 1: a=1 (Linear transport function)

If a = 1 transport is a linear function of discharge. The aggregate transport is directly proportional to the total discharge (which is fixed). If transport is a linear function of discharge, as is the case with the Schoklitsch transport formulation, no optimization of downstream transport is possible. Any discharge pattern that is feasible (discharging the entire runoff volume after T time periods) will produce the same aggregate transport.

Case 2: a>1 (Convex transport function)

For a > 1 the transport rate increases at an accelerating rate with discharge. The transport-discharge function is convex, as is the case with the Goncharov and Shields formulas. The second order necessary conditions (3) will be satisfied, and the constant release $q_t = R/T$ will minimize the aggregate transport downstream. The transport minimizing strategy in this case can be described as minimizing the maximum discharge.

Case 3: 0<a<1 (Concave transport function)

In this case transport increases at a decreasing rate with discharge. The transport-discharge function is concave, as is the case with the Meyer-Peter and Muller and Bagnold formulas. The second derivatives, $\partial^2 f(q_t)/\partial(q_t)^2$, are negative, indicating the constant release $q_t = R/T$ maximizes transport downstream. In this case no interior solution exists; the transport minimizing discharge pattern will be an extreme point (e.g. $q_t = R$, $t=j$; $q_t = 0$ $t \neq j$; $t=1,2,...T$). For this case transport will be minimized by the discharge pattern that maximizes the minimum discharge.

MINIMIZATION WITH A CRITICAL DISCHARGE

The level of critical shear stress plays an important role in determining the erosion control design and, to some extent, reduces the differences in erosion control designs described in the preceding section. The critical discharge influences the erosion control design because, for a given runoff volume, any flows below q_c will produce no sediment transport. Any transport function (regardless of its shape) will predict a reduction in bed material load if an erosion control design releases enough of the discharge below the q_c. Consider Figure 7.1 and an outflow hydrograph of a fixed size (say, for the 2 year postdevelopment storm). In the extreme case, if the entire runoff volume can be released at a rate smaller than q_c, (a solution involving maximum storage), all transport functions, whether convex or concave, would predict zero transport in the downstream channel. In a less extreme case, SWM designs that allow a large portion of the hydrograph to be released at discharges below q_c will still have low cumulative bed material loads compared to the uncontrolled runoff hydrograph, regardless of the shape of the transport function. It is only when much of the discharge occurs at levels greater than q_c that the concave and convex solutions begin to yield dramatically different solutions. Thus, the level of q_c significantly effects the performance of the different transport formulations in determining erosion control designs.

The explicit mathematical treatment of transport function shape given in the previous section assumed the aggregate transport function is continuously differentiable. The effect of q_c is to make the transport function only piecewise continuous. Any transport minimizing strategy should clearly seek to maximize the runoff volume discharged at or below q_c. Extended detention strategies will tend to maximize the fraction of the runoff volume discharged at or below q_c. If all the runoff cannot be discharged below q_c the transport minimizing discharge pattern for the remaining runoff will again be determined from the three cases described above.

If q_c is small, compared to the average discharge, the effect of the critical discharge will be negligible and the three cases described above will determine the transport minimizing discharge pattern. As the detention time and the critical discharge increase, the fraction of the runoff volume that can be discharged below q_c will also increase. For sufficiently large values of q_c and T, an

extended detention strategy may dramatically reduce downstream transport, irrespective of the form of the transport-discharge function.

As an example, the sensitivity of aggregate transport to both detention time and critical shear stress was assessed using the Meyer-Peter and Muller transport equation, one of the concave transport functions. Transport calculated with the Meyer-Peter and Muller equation will be a concave function of discharge corresponding to Case 3 described above. An extended detention discharge strategy will therefore be expected to maximize downstream transport for the portion of the runoff volume that must be released above q_c.

Channel hydraulics downstream of the Newport Towne demonstration site were used to calculate the aggregate transport resulting from releasing the post development runoff volume for the 2 year storm at a constant discharge. Values of critical shear stress from 0.02 lb/ft^2 to 1.0 lb/ft^2 were used to calculate transport for detention times up to 120 hours. Figure 7.2 is calculated for a constant discharge equal to the ratio of the runoff volume and the detention time. As shown in the figure, for low values of critical shear stress (0.02 lb/ft^2 roughly corresponds to coarse unconsolidated sand, Lane, 1955) the critical discharge is low compared to the average discharge. As expected for a concave transport function the aggregate transport increases with detention time even though the average discharge rate is declining.

Increasing the critical shear stress increases the fraction of the runoff volume that can be discharged at a rate that will not cause transport. Extended detention releases still result in a transport maximizing release pattern for the volume discharged above the critical threshold. However, as q_c and T increase, the volume released below the critical discharge is large enough to cause a net reduction in transport under an extended detention release pattern. Put another way, the aggregate transport shown in Figure 7.2 could be further reduced if the volume of runoff that cannot be discharged below the critical discharge was passed through the channel over a shorter time.

CHOICE OF TRANSPORT FORMULATION

The discussion above suggests that the effect of transport formulation on erosion control SWM design will be most important at sites having the smallest values of critical discharge. The lowest values of q_c are generally associated with the smallest sizes of sediments, provided they are cohesionless (cohesive forces make the very smallest silts and clays much harder to erode), which primarily occurs in the sand size range (0.06 to 2.0 mm). In these cases, convex transport formulations will lead toward an erosion control design that is similar to those developed here for the demonstration sites, namely, increased storage for the 2 year storm. Other transport formulations, however, will lead to exactly the opposite conclusion, that an erosion control design

EFFECT OF CRITICAL SHEAR
Tc – Critical Shear (lbs/sq–ft)

Tc=0.02

Tc=0.2

Tc=0.4

Tc=0.6

Tc=0.8

Tc=1.0

AGGREGATE LOAD (LBS)

8000000.00
6000000.00
4000000.00
2000000.00
0.00

0.00   20.00   40.00   60.00   80.00   100.00   120.00

DETENTION TIME (HRS)
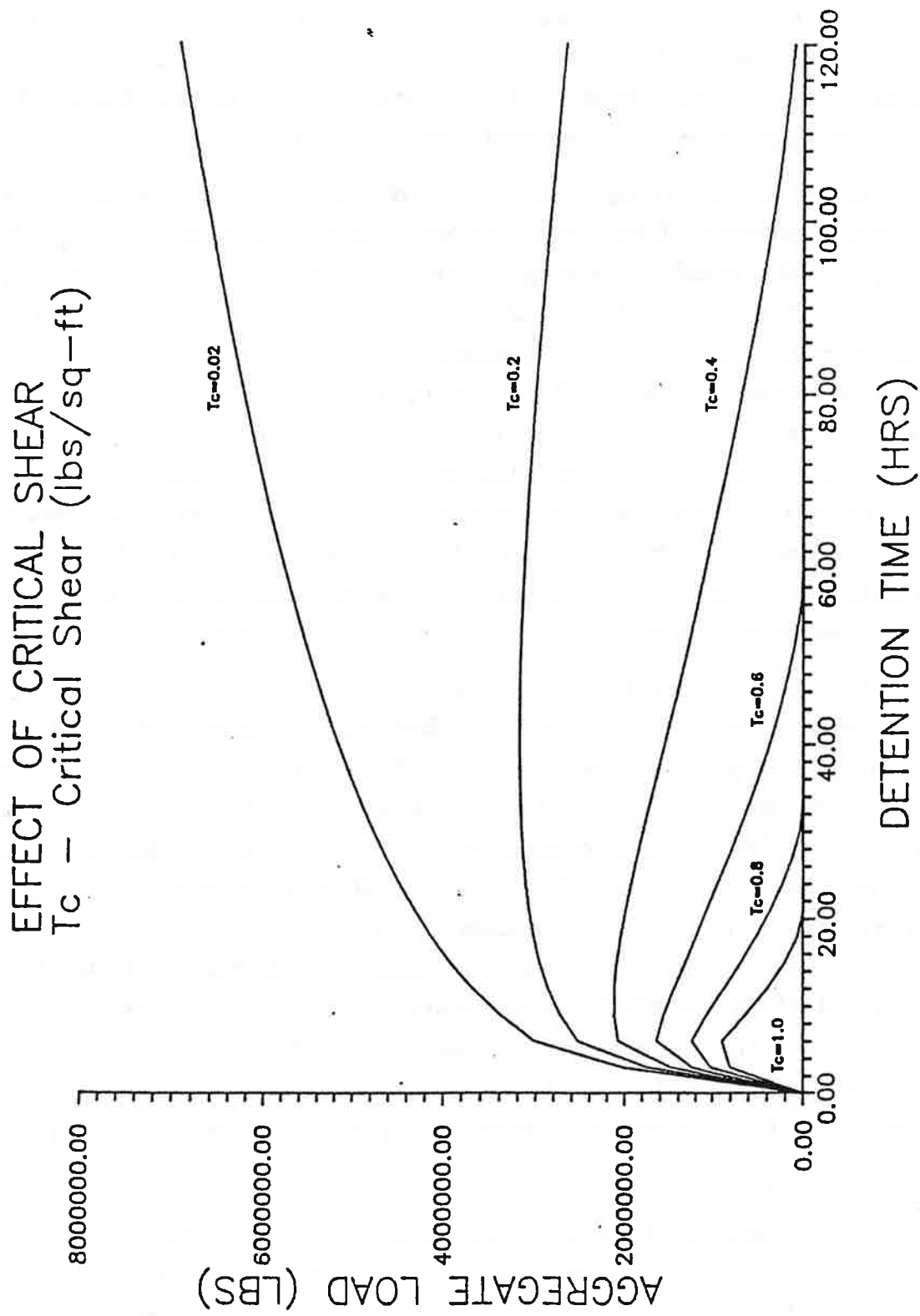
7.2   Effect of critical shear stress on aggregate bed-material load. Bed material load is computed using the Meyer-Peter and Muller formula. Calculations are based on a constant discharge equal to the ratio of the runoff volume and the detention time.

would require the largest possible discharges over the shortest possible time periods. This conclusion is in contrast to that made by McCuen and Moglen (1988) and MacRae and Wisner (1989), who suggest that the type of transport formulation does not influence the erosion control design. This difference may result from the fact that most transport functions considered by these workers were convex, and that the values of q_c are relatively high.

An erosion control design based on a concave transport formula and a channel with small, noncohesive sediments will lead toward the recommendation of no stormwater storage, short of finding methods of actually accelerating the storm runoff from developed areas. This result is not consistent with observations of eroding channels in the Anacostia basin, however. The most serious cases of channel erosion in the Anacostia are found in areas with insufficient stormwater management (either no SWM control in local areas, or insufficient control over the much larger areas above the main stems of the major Anacostia tributaries). One may conclude, then, that the existing SWM facilities in the Anacostia Basin do provide some measure of erosion control relative to those areas where little or no SWM is provided. This suggests that storage of stormwater does provide some reduction in bed material load relative to no storage at all. This observation supports the use for erosion control SWM design of transport formulations, such as the Goncharov or the Shields formulations, that yield a convex relation to channel discharge.

This conclusion is also supported by the difficult problem of estimating q_c. Erosion control designs based on convex transport functions are inherently less sensitive to the choice of q_c, while erosion control designs based on concave transport functions are highly sensitive to the choice of q_c. It must be borne in mind, however, that these conclusions are based on the use of mean bed shear stress and flow velocity in computing the bed material load. Because bed material load is highly sensitive to bed shear stress, the total bed material load integrated across a stream channel will be very sensitive to the spatial distribution of bed shear stress in the channel. It is entirely possible that transport functions which show a concave relation with q_t when the bed shear stress is computed as θds would actually produce a convex relation with q_t, if the actual distribution of bed shear stresses in a channel section were used to compute the bed material load. Thus, the difficulty in predicting the aggregate bed material load may be as much a function of using estimated mean flow parameters as it is related to the choice of q_c or transport formula.

7.2 Project hydrology formulation

The erosion control SWM designs were formulated in Section 6 for the 2 year storm at both demonstration sites. Because bed material load is a nonlinear function of channel discharge, simple extrapolations of the change in bed material load for other design storms cannot be made. Figure 7.3 presents, for each demonstration site, an estimate of bed material load for the 1, 2, and

Newport Towne

Precipitation of 24-hr Storm (inches)

Snowden's Mill

Precipitation of 24-hr Storm (inches)

7.3   Bed-material load for the erosion control designs and the 1 yr, 2 yr, and 10 yr storms.
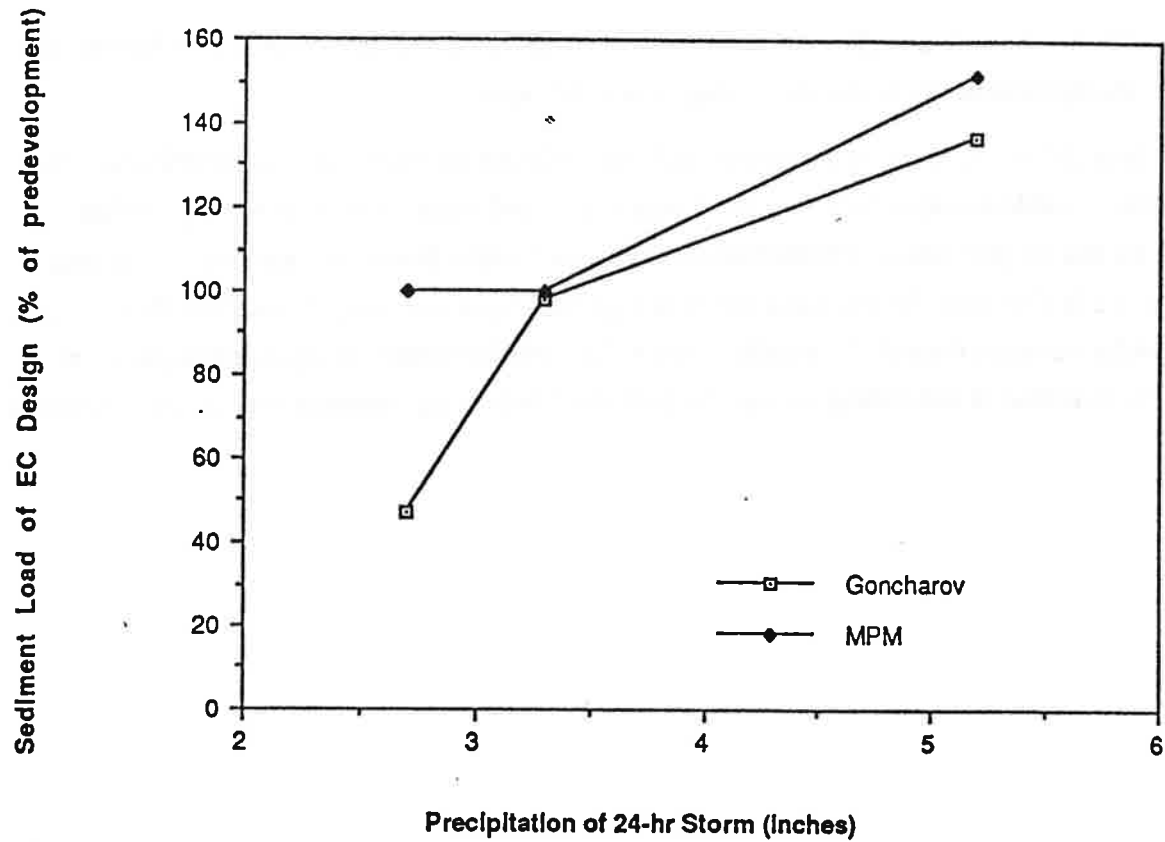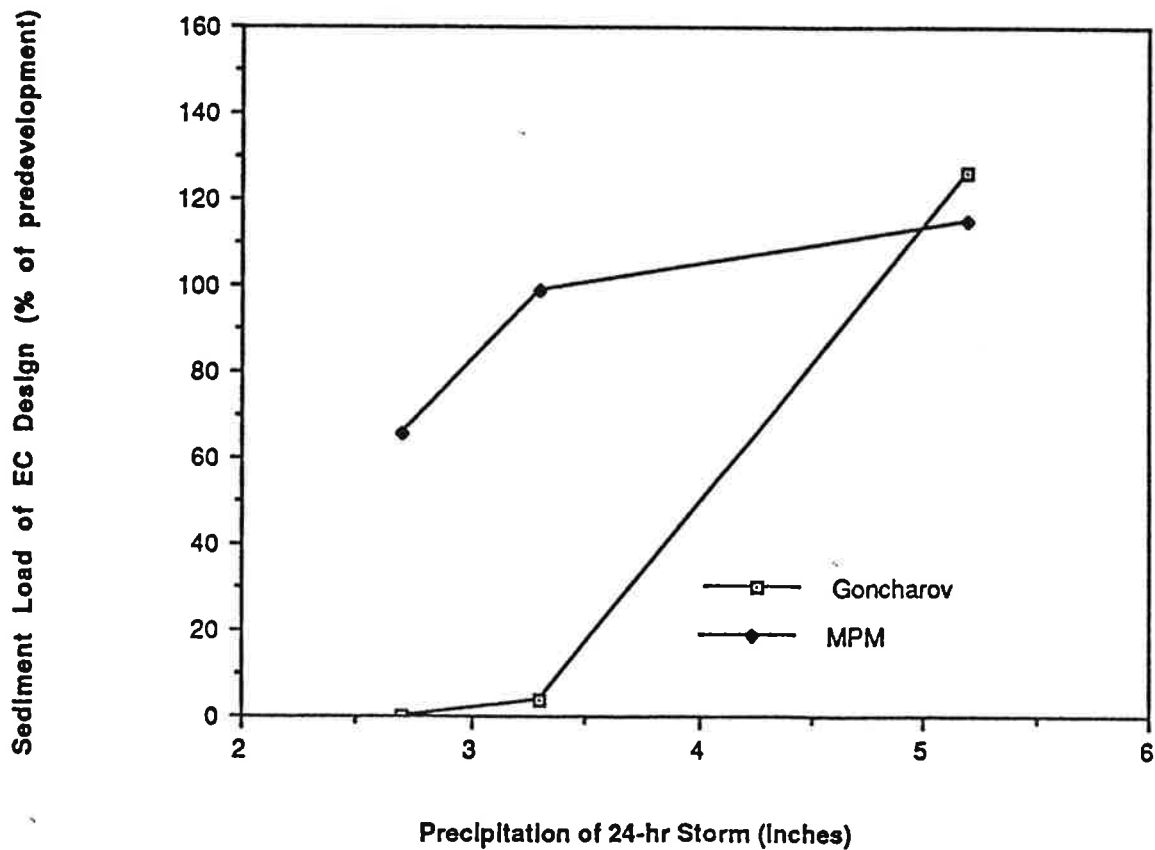
10 year storms. In each case, the bed material load is expressed as a percentage of the bed material load for the equivalent storm and predevelopment conditions.

In addition to providing erosion control for the 2 year storm, the designs developed in this report also provide erosion control for the 1 year storm, and come close to achieving erosion control for the 10 year storm. The former result is an important feature of these erosion control designs. Peak discharge designs for a storm of a given frequency often provide little flow control, and therefore erosion control, for smaller storms. The erosion control designs developed here appear to be robust in controlling erosion for both the 2 year design storm as well as more frequent storms.

8. DESIGN SUGGESTIONS

The design of an erosion control stormwater facility involves additional steps in comparison with the design of a facility that controls only peak discharge. General design suggestions are presented here for a case where erosion control for the two year storm is to be achieved, while also satisfying the current SWM requirements in Montgomery and Prince George's Counties. The design suggestions are based on the procedures used to develop the erosion control designs for the demonstration sites, as described in Section 6 of this report. The SWM requirements that are addressed by the design suggestions are:

- 24 hour extended detention for the 1 year postdevelopment storm (Harrington, 1987b).

- Bed material load for the 2 year storm limited to 2 year predevelopment levels.

- Peak discharge for the 2 and 10 year storms limited to predevelopment levels for the same storms.

- Emergency spillway designed for the 100 year storm. (Harrington, 1987a; SCS 1981).

In preparing the design suggestions, we have assumed that a stage-storage relation for the pond has been developed based on topographic conditions and grading plans at the site. If a design elevation for the top of the dam exists, provision is made in the design suggestions to evaluate whether sufficient storage is available to satisfy both erosion control and extended detention requirements. In this way, these design suggestions may be applied to both new pond design and the evaluation and retrofit of an existing facility. When used as part of a new pond design, the design suggestions are intended to be used in connection with standard design procedures currently in practice (e.g. Harrington, 1987a and 1987b; WSSC, 1987; SCS, 1981). Use of the term "standard design procedures" in the accompanying flow chart (Figure 8.1) and following text refers to a pond design that would result from using the standard references. When the design suggestions are used to evaluate a retrofit of an existing pond, the existing pond design should be substituted for the term "standard design procedures".

The first step in designing a pond to match all the SWM requirements under consideration here is to determine the necessary size of the erosion control orifice. This may be done by running the erosion control program in Appendix A for a case with only one outlet located at the bottom of the pond. The size of the outlet is varied while holding its elevation at the pond bottom constant. The program output permits one to determine the size of the low flow outlet that produces a bed material load that just equals that of the predevelopment case. If this orifice is larger than the 24 hour extended detention orifice, the erosion control orifice will not provide 24 hour extended detention and the extended detention design should be used to size the low flow orifice. If the

FIGURE 8.1

Design Flowchart for Pond That Provides 24 hr Extended Detention for the 1 Year Storm,
Erosion Control for the 2 Year Storm, and Peak Discharge Control for the 2 and 10 Year Storms
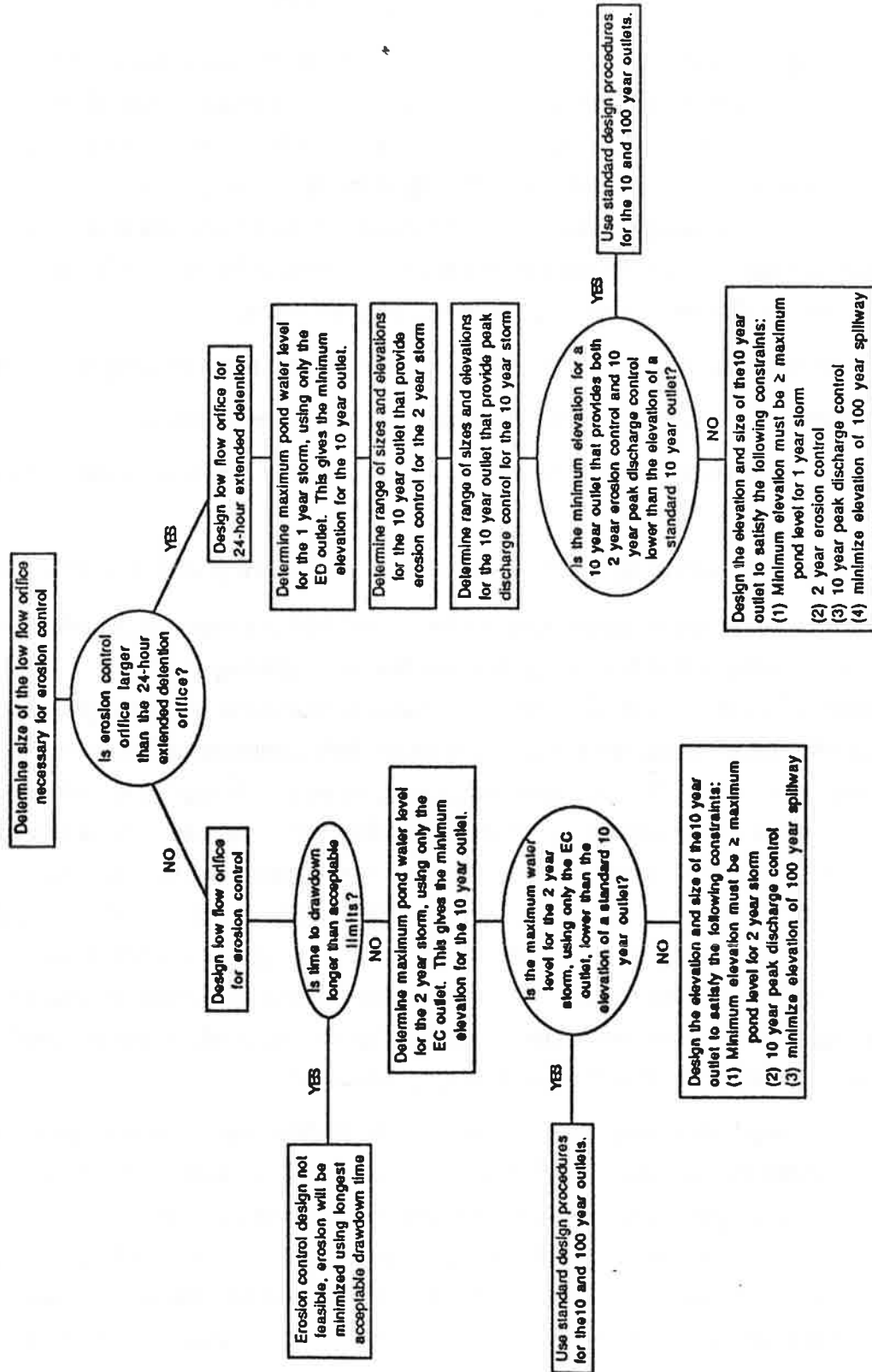
Notes: If the design solution for the 10 year outlet permits the elevation of the 100 year spillway to be lower than 2 ft below the top of the dam, the existing pond storage may be used by sizing the 100 year spillway so that the maximum pool elevation for the 100 year storm is at least 1 ft below the top of the dam. If the elevation of the 100 year spillway must be greater than 2 feet below the top of the dam, pond storage must be increased. This may be accomplished by raising the elevation of the dam to 2 ft above the 100 year spillway design elevation, and sizing the 100 year spillway to carry the design storm with 1 ft of head.

8.1   Flow chart for an erosion control SWM facility that meets existing peak discharge and extended detention requirements.

erosion control size is used for the low flow orifice, the program output should also be checked for the period of time required to drain the pond. If the drawdown time is unacceptably large, an erosion control design is not possible. However, downstream erosion will be reduced if the orifice is sized to provide the longest acceptable drawdown time, provided a convex transport function (such as the Goncharov formula) is used to estimate the bed material load.

Once the low flow orifice size is determined, a maximum low flow water level must be determined for the appropriate design storm. This low flow level will be used to determine the minimum elevation allowable for the higher outlets. · If the low flow orifice is sized according to extended detention guidelines, this maximum water level is computed for the one year storm using only the ED orifice. If an erosion control orifice size is used, the maximum water level is computed for the 2 year storm using only the erosion control orifice. If any other outlets are located lower than the computed maximum low flow water level, flow through the upper outlets will prevent the limiting criterion (erosion control or extended detention) from being satisfied.

Once the maximum low flow water level is defined, its elevation may be compared to any predetermined elevation of a 10 year outlet. It is presumed that a low flow outlet that provides erosion control for the two year storm will also provide peak discharge control for that storm (2/2 control). If the maximum low flow water level is below the standard elevation of the 10 year outlet, both extended detention and erosion control will be achieved using standard design procedures for the 10 and 100 year outlets.

If the maximum low flow water level is greater than the elevation of the standard 10 year outlet, the elevation and size of the 10 year outlet should be designed to meet the following constraints:

(1) The minimum allowable elevation of the 10 year outlet is the maximum pond surface elevation of the low flow storm, as determined above.
(2) The 10 year outlet must provide 2 year erosion control (this step is not necessary if the low flow device is sized to provide erosion control).
(3) The 10 year outlet must provide peak discharge control.
(4) The elevation of the 100 year emergency spillway is to be minimized.

The design solution for the 10 year outlet may be found by examining a range of elevations and sizes for the 10 year outlet and examining the variation with outlet size and elevation of (i) the integrated bed material load for the 2 year storm (in the case where an ED size is used for the low flow outlet), (ii) the peak discharge for the 10 year storm, and (iii) the maximum water level for the 10 year storm. Combinations of elevation and size that provide both 2 year erosion control and 10

year peak discharge control may be examined to find the outlet design that yields a minimum elevation for the 100 year emergency spillway. The emergency spillway elevation is taken to be at the maximum water level for the 10 year storm, or 1 foot above the crest of the 10 year outlet, whichever is higher (Harrington, 1987a). The process of finding a design solution is facilitated by using contoured response surfaces, as done in Section 6 for the demonstration sites.

If a predetermined elevation for the top of the dam is to be evaluated along with a stage-storage relation, determination of whether this configuration can satisfy all of the SWM criteria used here (extended detention; erosion control, 2 and 10 year peak discharge control) can be made based on the requirement that the elevation of the 100 year emergency spillway is at least 2.0 ft below the top of the dam (requirement from SCS Engineering Code 378 for drainage areas smaller than 320 acres and dams less than 20 ft in height). If the minimum elevation for the emergency spillway is higher than 2 ft below the top of the dam, pond storage must be increased. The storage may be increased by increasing the dam height to an elevation 2 feet above the minimum elevation of the emergency spillway determined above, or by excavating the pond so that the minimum elevation for the emergency spillway is 2 feet below the top of the existing dam. In either case, the emergency spillway must be sized to carry the 100 year storm with 1 ft of head above the spillway invert (SCS Engineering Code 378 requires 1 ft freeboard above the 100 year design storm). If the minimum required elevation for the emergency spillway is lower than 2 ft below the top of the dam, erosion control and extended detention may be achieved using the predetermined pond storage and dam height. The 100 year emergency spillway would then need to be designed so that the maximum water level for the 100 year storm reached an elevation no higher than 1 ft below the top of the dam.

9. CONCLUSIONS

In this report, we have developed erosion control designs for two existing stormwater management (SWM) facilities located in the Anacostia River Basin. These designs are based on a design storm with a 2 year return interval and limit pond-routed bed material loads for postdevelopment conditions to predevelopment levels. In this study, bed material load is used as a quantifiable surrogate for channel erosion. In addition to erosion control, the designs also incorporate the SWM regulations currently in effect in the Anacostia: peak discharge control of the 2 year and 10 year storm, 24 hour extended detention of the 1 year storm (one of two options for water quality control), and an emergency spillway designed for the 100 year storm according to SCS requirements. In addition to providing erosion control for the 2 year storm, the designs developed in this report also provide erosion control for storms smaller than the two year storm and come close to achieving erosion control for larger storms. The former result is a robust feature of the erosion control design. Peak discharge designs for a storm of a given frequency often provide little flow control, and therefore erosion control, for smaller storms. The erosion control designs developed here appear to be effective in controlling erosion for storms smaller than the 2 year design storm.

An erosion control design depends on the formula used to compute sediment transport. For this reason, we have developed designs for both sites using two different transport formulas, one based on the mean flow velocity in the receiving stream (the Goncharov formula) and the other based on the mean bed shear stress in the receiving stream (the Meyer-Peter and Muller formula). The two formulas lead to designs that modify the runoff hydrograph in similar ways. At one site, Snowdens Mill, the erosion control designs are similar, although the design based on the Meyer-Peter and Muller formula requires higher elevations for the 10 year and 100 year outlets which just meet design requirements using the existing pond storage. At the other site, Newport Towne, the two formulas result in a slightly different low flow outlet. The design based on the Goncharov formula uses a 1 year 24 hour extended detention low flow orifice, whereas the design based on the Meyer-Peter and Muller formula requires a smaller low flow orifice that corresponds to 52 hour extended detention of the 1 year storm. At Newport Towne, the erosion control design based on the Goncharov formula requires an 11% increase in pond storage over the existing conditions; the design based on the Meyer-Peter and Muller formula requires a 20% increase in pond storage.

Although some of the erosion control designs require additional pond storage, this result is only partly related to the erosion control requirement. In three of the four cases, the size of the low flow outlet, which was designed to satisfy both erosion control and extended detention requirements, was set at the size necessary for extended detention. In general, the need for increased pond volume can result from different combinations of the following requirements: (1)

locating the elevation of the 10 year outlet at a level above the maximum pond level for the design storm of the low flow outlet (the 1 year storm for extended detention, the 2 year storm for erosion control), (2) satisfying the 10 year peak discharge requirement (10/10 control), and (3) satisfying the requirement for 2 ft of freeboard between the top of the dam and the invert of the 100 year emergency spillway. The optimum elevation and size of the 10 year outlet must satisfy four requirements: (1) elevation greater than or equal to a minimum fixed by the low flow design storm, (2) erosion control for the two year storm, (3) peak discharge control of the 10 year storm, and (4) minimize the elevation of the 100 year emergency spillway. The optimum solution to these constraints was, in general, not the minimum possible elevation for the 10 year outlet. The designs that met all of these requirements required a pond storage of 90% of the runoff from the 2 year design storm at Snowdens Mill, and 74% and 96% of the 2 year runoff at Newport Towne for the Goncharov and Meyer-Peter and Muller solutions, respectively.

The nature of an erosion control design is sensitive to the choice of transport formula. This dependence is related to the shape of the relation between bed material load and discharge, and the estimate of the critical discharge at which sediment movement begins. If the function is convex (e.g. the Goncharov or Shields formulas), the erosion control result is relatively independent of the estimate of critical discharge. In this case, an erosion control design will be one that minimizes the peak discharge and maximizes the amount of discharge released at subcritical levels. If the function is concave (e.g. the Meyer-Peter and Muller and Bagnold formulas), and the value of critical discharge is large enough so that a substantial amount of discharge may be released at subcritical levels, a design that maximizes storage and minimizes the release discharge may also reduce bed material load to predevelopment levels. However, if the function is concave and the value of critical discharge is very small, bed material load will be minimized by a design that releases the largest amount of runoff to the channel in the shortest period of time. In the Anacostia basin, the most serious cases of channel erosion are found in areas with insufficient stormwater management (either no SWM control in local areas, or insufficient control over the much larger areas above the main stems of the major Anacostia tributaries). Thus, it appears that SWM facilities generally do provide some measure of erosion control relative to those areas where little or no SWM is provided. If any stormwater detention provides a reduction in bed material load relative to uncontrolled runoff, a SWM design pointing to maximum release of stormwater is incorrect. This observation supports the use, for erosion control SWM design, of convex transport-discharge functions, such as the Goncharov or the Shields formulations. Erosion control SWM designs based on concave transport-discharge functions (such as the Meyer-Peter and Muller or Bagnold formulas) are highly sensitive to the estimate of the critical discharge for incipient sediment motion. Because estimates of critical discharge are extremely difficult to make with any

accuracy, it is recommended that concave transport functions not be used to design erosion control SWM facilities, especially when mean flow parameters are used to calculate the sediment transport.

Pond designs for erosion control and water quality control both incorporate increased storage for smaller, higher frequency storms. A design that will satisfy one of these criteria will therefore contribute to meeting the other. Both erosion control and extended detention requirements depend on the size of the low flow outlet. If both criteria are to be met, the criterion that requires the smaller low flow outlet will be limiting. In three of the four designs in this report, the outlet for 24 hour detention of the 1 year storm was smaller than the outlet for 2 year erosion control. In the fourth case, the size of the erosion control outlet was smaller and corresponded to an extended detention period of 52 hours. In all cases, an extended detention design should decrease the bed material load in the receiving stream. However, in some of the cases presented in this report, a design meeting the 1 year 24 hour extended detention criterion did not provide erosion control for the 2 year storm. To ensure compliance with both extended detention and erosion control criteria, a SWM design must explicitly examine both detention time and bed material load.

Impact on the Aquatic Resources of the Anacostia River Basin

Erosion and sedimentation in the streams of the Anacostia Basin have been important factors in decreasing the quality of the channels as an aquatic habitat (ICPRB, 1988). Development induced increases in sediment supply to the stream channels through soil erosion, construction, and mining, and increased erosion, sediment movement, and deposition within the channels, have led to the accumulation and infiltration of fine sediments in coarse-grained beds and the filling of pools in pool-and-riffle sequences. This sedimentation leads to a direct loss of spawning grounds and a decrease in benthic populations upon which fish feed (Diplas and Parker, 1985; Shea and Mathers, 1978; Phillips, 1965).

The carrying capacity of adult trout in a stream is determined by the amount of suitable foraging and refuge sites. A study of the Paint Branch and its tributaries found that the principal factors regulating the standing crop of older trout are increased sedimentation, which reduces the amount of refuge and foraging habitat, and diminished base flows associated with decreased permeability of the basin (ARC, 1986) . The study on the Paint Branch and its tributaries found the relative abundance of Brown Trout to be inversely related to the degree of silt deposition. To maintain a quality pool-and-riffle habitat and clean gravel spawning areas for trout, it is essential that both the flow regime and the bed material load be maintained at or near natural levels (MNCPPC, 1983).

Sources of fine sediment include both terrestrial sediments delivered to the stream system and entrainment of sediments within the channel or forming the channel bed and banks. An erosion control SWM design involves extended detention of storm runoff and thereby provides increased settling times which can reduce the transfer to the receiving stream of fine sediments delivered to the pond. In addition, an erosion control SWM designs explicitly limits the entrainment of channel bed sediment to predevelopment levels. The assumption can then be made that by limiting bed material load to predevelopment levels, the entrainment of channel bank sediments may also be limited to natural conditions.

In the past two decades, improved land and water use practices have contributed to the reemergence of more diverse fish communities in the Anacostia Basin (ICPRB, 1989b). Implementation of erosion control SWM would further improve the aquatic habitat by reducing the input of fine sediment into the river system and decreasing the supply and movement of fine sediment within the channel itself. The Anacostia Watershed Restoration Agreement of 1987 recognizes the deteriorated condition of stream channels and aquatic habitat resulting from accelerated rates of erosion. The State of Maryland, along with the District of Columbia and Prince George's and Montgomery counties, are committed to basin-wide management of erosion and sedimentation by the turn of the century. This report has provided, using two existing SWM sites located within the Anacostia River watershed, a demonstration of the reduction in bed material load that can be achieved through SWM facilities designed to provide erosion control.

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APPENDIX A
FORTRAN COMPUTER CODE

```
C THE PROGRAM CONTROL CALLS SUBROUTINES TO CALCULATE BED LOAD TRANSPORT
C DOWNSTREAM OF POND OUTLETS
C THE PROGRAM IS PRESENTLY SET UP TO USE ENGLISH UNITS

      PROGRAM CONTROL
      PARAMETER(PI=3.141592654, GRAV=32.174, SWEIGHT=62.4,
     &          IA=50, IB=600, IC=4)

      REAL Y(IA), S(IA), T(IB), INFLW(IB), OUTFLW(IB),
     &     QBGON(IB), QBMPM(IB), GONSUM, MPMSUM,
     &     DT, CW(IC), CO(IC), YO(IC), LA(IC), SO, Q0,
     &     D, VC, ALPHALO, BETALO, GAMMALO, DELTALO,
     &     WIDTH, VCUT, DCUT,ALPHAHI, BETAHI, GAMMAHI, DELTAHI,
     &     YOMIN(IC), YOMAX(IC), LAMIN(IC), LAMAX(IC), HORIF(IC),
     &     SL, TC, QPEAK, TD, LAINC(IC), YOINC(IC)

      INTEGER NY, NSTRUCT, NT, TOP(IB), TOPSUM, NPND, NTD, OUTYPE(IC),
     &        INCYO, INCLA, NVARY


C OPEN FILES CONTAINING DATA
C FILES CONTAINING DATA MUST NOT USE TABS TO SEPARATE VALUES
C USE ONE SPACE TO SEPARATE VALUES
C FILE 'PARAMETERS' SHOULD BE ENTERED AS ONE ROW SEPARATED BY SPACES WITH
C EOF TYPED AT THE END OF THE FILE
C ALL OTHER FILES SHOULD BE ENTERED IN COLUMNS WITH EOF AT THE END

      OPEN (10,FILE='PARAMETERS')
      OPEN (11,FILE='STAGE-STORAGE')
      OPEN (12,FILE='INFLOW')
      OPEN (14,FILE='OUTLETS')

C OPEN OUTPUT FILES

      OPEN (20,FILE='RESULTS')
      OPEN (23,FILE='SUMFLW')
      OPEN (24,file='SUMQB')

C READ IN DATA

C READ THE MISC. PARAMETERS
      READ (10,*) NY,NSTRUCT,NT,SO,Q0,D,VC,DT,SL,TC,NPND
      READ (10,*) NVARY,INCYO,INCLA,WIDTH,VCUT,DCUT
      READ (10,*) ALPHALO,BETALO,GAMMALO,DELTALO,
     &            ALPHAHI,BETAHI,GAMMAHI,DELTAHI

C STAGE-STORAGE CURVE SHOULD USE STAGE INCREMENTS NO LARGER THAN
C 0.5 FT TO MINIMIZE INTERPOLATION ERRORS IN THE
C STORAGE-INDICATION PROCEDURE
C READ THE STAGE-STORAGE CURVE
      READ(11,*) (Y(I),S(I),I=1,NY)

C READ THE INFLOW HYDROGRAPH
      DO 20 I=1,NT
      READ (12,*) INFLW(I)
      T(I)=(I-1)*DT
 20   CONTINUE

C READ THE OUTLET STRUCTURE PARAMETERS
```

```fortran
      DO 30 I=1,NSTRUCT
      READ (14,*) OUTYPE(I),CW(I), CO(I), YOMIN(I), YOMAX(I), LAMIN(I),
     &            LAMAX(I)
      IF (OUTYPE(I).EQ.1) THEN
          READ (14,*) HORIF(I)
      ENDIF
      YOINC(I) = (YOMAX(I)-YOMIN(I))/FLOAT(INCYO)
      IF (YOINC(I).LT.0.001)  YOINC(I) = 0.0
      LAINC(I) = (LAMAX(I)-LAMIN(I))/FLOAT(INCLA)
      IF (LAINC(I).LT.0.001)  LAINC(I) = 0.0
      YO(I) = YOMIN(I)
 30   CONTINUE

C IF THERE IS NO POND, GOTO NOPOND, THEN TRANS

      IF (NPND.EQ.0) THEN
        CALL NOPOND(INFLW,NT,DT,OUTFLW,QPEAK,TD)
        CALL TRANS (NT, OUTFLW,VCUT,DCUT,WIDTH, D, VC,
     &              ALPHALO, BETALO, GAMMALO, DELTALO,
     &              ALPHAHI, BETAHI, GAMMAHI, DELTAHI,
     &              QBGON, QBMPM, GONSUM, MPMSUM, SL, TC,
     &              DT, TOPSUM)
        WRITE (*,*) 'YOU HAVE NO POND'
        DO 700 I=1, NT
          WRITE (20,702) T(I), INFLW(I), OUTFLW(I),
     &      QBMPM(I),QBGON(I)
 702      FORMAT (F6.1,2X,2(F8.3,2X),2(E12.4,2X))
 700    CONTINUE

        WRITE (23,704) QPEAK, TD, MPMSUM, GONSUM
 704    FORMAT (2(F7.2,2X),2(E12.4,2X))
C       WRITE (*,*) 'QPEAK(cfs) TD(min) MPMSUM(lbf)'
C       WRITE (*,*) QPEAK, TD, MPMSUM

      ELSE

C IF THERE IS A POND, GOTO (OUTLET,ROUTE,TRANS) VIA POND
        DO 46 I=1,INCYO
          DO 44 J=1,NSTRUCT
            YO(J)=YO(J)+YOINC(J)
            LA(J) = LAMIN(J)
 44       CONTINUE
          DO 42 K=1,INCLA
            DO 45 L=1,NSTRUCT
              LA(L)=LA(L)+LAINC(L)
 45         CONTINUE
            WRITE (*,*) 'YO, LA', YO(NVARY), LA(NVARY)
            CALL POND(NY,NT,Y,S,T,INFLW,OUTFLW,GONSUM2,MPMSUM2,
     &                DT,CW,CO,HORIF,YO,LA,SO,QO,D,VC,SL,TC,OUTYPE,
     &                QPEAK2,TD2,NSTRUCT,
     &                WIDTH,NVARY,VCUT,DCUT,
     &                ALPHALO,BETALO,GAMMALO,DELTALO,
     &                ALPHAHI,BETAHI,GAMMAHI,DELTAHI)
 42       CONTINUE
 46     CONTINUE

      ENDIF


      STOP
```

```
      END

C*******************************************************************
C*******************************************************************


      SUBROUTINE POND(NY,NT,Y,S,T,INFLW,OUTFLW2,GONSUM2,MPMSUM2,
     &               DT,CW,CO,HORIF,YO,LA,SO,QO,D,VC,SL,TC,OUTYPE,
     &               QPEAK2,TD2,NSTRUCT,
     &               WIDTH,NVARY,VCUT,DCUT,
     &               ALPHALO,BETALO,GAMMALO,DELTALO,
     &               ALPHAHI,BETAHI,GAMMAHI,DELTAHI)
      PARAMETER(PI=3.141592654, GRAV=32.174, SWEIGHT=62.4,
     &          IA=50, IB=600, IC=4)
      REAL Y(IA), S(IA), T(IB), INFLW(IB), OUTFLW2(IB),ELEV(IB),
     &     ELEVMX,VCUT,DCUT,
     &     QBGON2(IB), QBMPM2(IB), GONSUM2, MPMSUM2,HORIF(IC),
     &     DT, CW(IC), CO(IC), YO(IC), LA(IC), QOUT(IA), SO, QO,
     &     D, VC, SL, TC, QPEAK2, TD2,WIDTH,
     &     ALPHALO,BETALO,GAMMALO,DELTALO,
     &     ALPHAHI,BETAHI,GAMMAHI,DELTAHI

      INTEGER NY, NSTRUCT, NT, TOP(IB), TOPSUM, NTD, OUTYPE(IC),
     &        NVARY

      CALL OUTLET (Y, NY, NSTRUCT, CW, CO, YO, LA, QOUT, OUTYPE,
     &             HORIF)
      CALL ROUTE (T, NT, INFLW, OUTFLW2, NY, y,S, SO, QOUT, QO,
     &            DT, QPEAK2, TOPSUM, TD2,elev,elevmx)
      CALL TRANS (NT, OUTFLW2,VCUT,DCUT,WIDTH, D, VC,
     &            ALPHALO, BETALO, GAMMALO, DELTALO,
     &            ALPHAHI, BETAHI, GAMMAHI, DELTAHI,
     &            QBGON2, QBMPM2, GONSUM2, MPMSUM2, SL, TC,
     &            DT, TOPSUM)
C     DO 800 I=1, NT
C     WRITE (20,4024) T(I), INFLW(I), OUTFLW2(I), ELEV(I), QBMPM2(I)
C    & ,QBGON2(I)
C4024 FORMAT (F6.1, 2X, 3(F7.3,2X), 2(E12.4,2X))
C800  CONTINUE

C     WRITE (*,2022) YO(NVARY),LA(NVARY),QPEAK2, TD2, ELEVMX,
C    &               MPMSUM2, GONSUM2
C2022 FORMAT (5(F8.3,2X),2(E12.4,2X))

      WRITE (23,4026) YO(NVARY),LA(NVARY),QPEAK2,TD2,ELEVMX
 4027 FORMAT(2(F7.3,3X),2E12.4)

      WRITE (24,4027) YO(NVARY),LA(NVARY),MPMSUM2,GONSUM2
 4026 FORMAT(5(F8.3,2X))

      RETURN
      END

C*******************************************************************
C*******************************************************************


      SUBROUTINE NOPOND(INFLW,NT,DT,OUTFLW,QPEAK,TD)
      PARAMETER(PI=3.141592654, GRAV=32.174, SWEIGHT=62.4,
     &          IA=50, IB=600, IC=4)
      REAL INFLW(IB),OUTFLW(IB),DT,TD,QPEAK
```

```
      INTEGER NT

      DO 55 I=1, NT
         OUTFLW(I)=INFLW(I)
C THE FOLLOWING COMPUTES PEAK DISCHARGE (QPEAK) & TIME OF DRAWDOWN (DT)
C WHEN THE FLOW IS NOT ROUTED THROUGH THE POND
      QPEAK=OUTFLW(1)
      NTD=0
      DO 53 II=2,NT
        IF(OUTFLW(II).GT.QPEAK) QPEAK=OUTFLW(II)
        IF(OUTFLW(II).LT.QPEAK.AND.OUTFLW(II).GT.0.1) NTD = NTD + 1
  53  CONTINUE

      TD=(DT*NTD)

  55  CONTINUE

      RETURN
      END


C********************************************************************
C********************************************************************
C THIS SUBROUTINE CALCULATES THE TOTAL OUTLET DISCHARGE
C USING WEIR AND ORFICE EQUATIONS AND THE POND STAGE

      SUBROUTINE OUTLET (Y, NY, NSTRUCT, CW, CO, Y0, LA, QOUT, OUTYPE,
     &                   HORIF)
      PARAMETER(PI=3.141592654, GRAV=32.174, SWEIGHT=62.4,
     &          IA=50, IB=600, IC=4)
      REAL Y(IA),CW(IC), CO(IC), Y0(IC), LA(IC), QOUT(IA),QTEMP,MIDPT,
     &     HORIF(IC)
      INTEGER NY, NSTRUCT, OUTYPE(IC)

      DO 4007 I0=1, NY
        QOUT(I0)=0.0
      DO 4008 I1=1, NSTRUCT
       IF (Y(I0).GT.Y0(I1)) THEN
          TDEPTH=Y(I0)-Y0(I1)
          RDEPTH=(Y(I0)-Y0(I1))/LA(I1)

       IF (OUTYPE(I1).EQ.1) THEN
C********************** RECTANGULAR ORIFICE***************
         IF (Y(I0).LE.(Y0(I1)+HORIF(I1))) THEN
              QTEMP = CW(I1)*LA(I1)*(TDEPTH)**1.5
         ELSE
            MIDPT = Y0(I1)+0.5*HORIF(I1)
            QTEMP = CO(I1)*LA(I1)*HORIF(I1)*SQRT(2.*GRAV*(Y(I0)-MIDPT))
     &             *(1-(1./96.)*(HORIF(I1)/(Y(I0)-MIDPT))**2.)
         ENDIF

       ELSEIF (OUTYPE(I1).EQ.2) THEN
C********************** CIRCULAR ORIFICE*******************
          IF (Y(I0).LE.(Y0(I1)+LA(I1))) THEN

C RAMPONI FORMULA FOR XPHI
C-------------------------
             XPHI=(10.12*(RDEPTH)**1.975)-(2.66*(RDEPTH)**3.78)
              IF(LA(I1).LE.0.98.AND.RDEPTH.LT.1.0) THEN

C STAUSS & JORISEN FORMULA FOR XMU WITH D<0.3m. AND h/D<1
```

```
C-----------------------------------------------------------
                  XMU=0.555+(LA(I1)/(110.*TDEPTH))+0.041*(TDEPTH/LA(I1))
                ELSE

C   RAMPONI APPROXIMATION OTHERWISE
C----------------------------------
                  XMU=(0.350+0.002*LA(I1)/TDEPTH)
                ENDIF

C STAUSS(1931) GENERAL FORMULA
C-----------------------------
                  QTEMP = XPHI*XMU*LA(I1)*(Y(IO)-Y0(I1))**2.5
                ELSE
                  MIDPT = Y0(I1)+0.5*LA(I1)
                  QTEMP =
     &          CO(I1)*PI/4.*(LA(I1)**2.0)*SQRT(2.*GRAV*(Y(IO)-MIDPT))
     &          *(1-(1./128.)*(LA(I1)/(Y(IO)-MIDPT))**2.)
                ENDIF

          ELSEIF (OUTYPE(I1).EQ.3) THEN
C********************** CIRCULAR RISER**********************
          IF (Y(IO).LE.(Y0(I1)+0.4*LA(I1))) THEN
               QTEMP = CW(I1)*PI*LA(I1)*(Y(IO)-Y0(I1))**1.5
          ELSE
               QTEMP =
     &          CO(I1)*PI/4.*(LA(I1)**2.0)*SQRT(2.*GRAV*(Y(IO)-Y0(I1)))
          ENDIF

          ELSEIF (OUTYPE(I1).EQ.4) THEN
C********************** SQUARE RISER**********************
          IF (Y(IO).LE.(Y0(I1)+0.4*LA(I1))) THEN
               QTEMP = CW(I1)*4.0*LA(I1)*(Y(IO)-Y0(I1))**1.5
          ELSE
               QTEMP =
     &          CO(I1)*(LA(I1)**2.0)*SQRT(2.*GRAV*(Y(IO)-Y0(I1)))
          ENDIF

          ELSEIF (OUTYPE(I1).EQ.5) THEN
C********************** STRAIGHT-CRESTED WEIR**************
          QTEMP = CW(I1)*LA(I1)*(Y(IO)-Y0(I1))**1.5

          ELSEIF (OUTYPE(I1).EQ.6) THEN
C********************** 90 DEGREE VEE-NOTCH WEIR***********
C         LA(I1)=ANGLE OF THE V-NOTCH
               QTEMP = CW(I1)*TAN(LA(I1)/2.)*(Y(IO)-Y0(I1))**2.50

          ELSEIF (OUTYPE(I1).EQ.7) THEN
C********************** CULVERT***********************************
                  QTEMP = 0.0
          WRITE (*,*) 'CULVERT COMPUTATIONS REQUESTED'
          ELSEIF (OUTYPE(I1).EQ.8) THEN
C********************** SQUARE ORIFICE****************
          IF (Y(IO).LE.(Y0(I1)+LA(I1))) THEN
                  QTEMP = CW(I1)*LA(I1)*(TDEPTH)**1.5
          ELSE
          MIDPT = Y0(I1)+0.5*LA(I1)
           QTEMP = CO(I1)*LA(I1)*LA(I1)*SQRT(2.*GRAV*(Y(IO)-MIDPT))
     &             *(1-(1./96.)*(LA(I1)/(Y(IO)-MIDPT))**2.)
          ENDIF
```

```
C WRAP UP THE IF BLOCK FOR DIFFERENT OUTLET TYPES
      ELSE
          WRITE (*,*) 'YOU HAVE NO OUTYPE'
          QTEMP = 0.0
      ENDIF
C WRAP UP THE IF BLOCK FOR Y < YO
      ELSE
          QTEMP = 0.0
      ENDIF
      QOUT(IO) = QOUT(IO) + QTEMP
 4008    CONTINUE
 4007 CONTINUE
      RETURN
      END
C********************************************************************
C********************************************************************
C THIS SUBROUTINE ROUTES AN INFLOW HYDROGRAPH THROUGH THE POND USING THE
C STORAGE-INDICATION METHOD

      SUBROUTINE ROUTE (T, NT, INFLW, OUTFLW, NY, y,S, SO, QOUT, Q0,
     &           DT, QPEAK, TOPSUM, TD,ELEV,ELEVMX)
      PARAMETER(PI=3.141592654, GRAV=32.174, SWEIGHT=62.4,
     &          IA=50, IB=600, IC=4)
      REAL T(IB),INFLW(IB),OUTFLW(IB),S(IA),ELEV(IB),ELEVMX,BOTTOM,
     &     QOUT(IA), SI(IA), TD, SO, Q0, RHS, DT, TEMP, QPEAK,Y(IA)
      LOGICAL OK, SPILL,DUM
      INTEGER NT, NY, TOP(IB), TOPSUM, NTD

      DO 1020 N=1, NY
      SI(N) = ((2*S(N)/DT)*726) + QOUT(N)
C FACTOR OF 726 CONVERTS S/DT FROM ACRE-FT PER MINUTE TO CFS

 1020   CONTINUE
      TEMP = ((2*SO)/DT) - Q0
      OUTFLW(1) = Q0
      TOPSUM=0
      DO 1040 N=1, NT-1
          RHS = (INFLW(N) + INFLW(N+1)) + TEMP
          IF (RHS.GT.0.001) THEN
             OUTFLW(N+1) = XINTERP(NY, QOUT, SI, RHS, OK, SPILL)
             elev(N+1) = XINTERP(NY,Y,QOUT,OUTFLW(N+1),OK,DUM)

             TEMP = RHS - 2*OUTFLW(N+1)
             IF (OUTFLW(N+1).LE.0.01) OUTFLW(N+1) = 0.0

C THE FOLLOWING ALERTS THE USER THAT THE POND HAS OVERTOPPED
             TOP(N+1) = 0
             IF (SPILL) THEN
                WRITE(*,*) 'POND HAS OVERTOPPED'
                TOP(N+1) = 1
             ENDIF
             TOPSUM=TOPSUM + TOP(N+1)
          ELSE
             OUTFLW(N+1) = 0.0
             ELEV(N+1) = XINTERP(NY,Y,QOUT,OUTFLW(N+1),OK,DUM)
          ENDIF

 1040   CONTINUE

C THE FOLLOWING COMPUTES PEAK DISCHARGE (QPEAK) & TIME OF DRAWDOWN (DT)
```

```
      ELEVMX=0.0
      QPEAK=OUTFLW(1)
      NTD=0
      BOTTOM = Y(1) + 0.5
      DO 1090 I=2,NT
         IF(OUTFLW(I).GT.QPEAK) QPEAK=OUTFLW(I)
         IF(ELEV(I).GT.ELEVMX) ELEVMX=ELEV(I)
         IF(OUTFLW(I).LT.QPEAK.AND.ELEV(I).GT.BOTTOM) NTD = NTD + 1
 1090 CONTINUE

      TD=(DT*NTD)
      IF(ELEV(NT).GT.BOTTOM) TD = 5760.

C THE FOLLOWING LINE MAKES QPEAK AND DRAWDOWN TIME (TD)
C NEGATIVE IF POND HAS OVERTOPPED
      IF(TOPSUM.NE.0) THEN
        QPEAK=-QPEAK
        TD=-TD
      ENDIF

      RETURN
      END
C*******************************************************************
C*******************************************************************
C THIS SUBROUTINE CALCULATES BEDLOAD TRANSPORT USING THE GONCHAROV
C & MEYER-PETER & MULLER EQUATIONS
      SUBROUTINE TRANS (NT, OUTFLW,VCUT,DCUT,WIDTH, D, VC,
     &                ALPHALO, BETALO, GAMMALO, DELTALO,
     &                ALPHAHI, BETAHI, GAMMAHI, DELTAHI,
     &                QBGON, QBMPM, GONSUM, MPMSUM, SL, TC,
     &                DT, TOPSUM)
      PARAMETER(PI=3.141592654, GRAV=32.174, SWEIGHT=62.4,
     &          IA=50, IB=600, IC=4)
      REAL OUTFLW(IB), QBGON(IB), QBMPM(IB), GONSUM, MPMSUM,
     &     VC, D, SL, TC, DT,VCUT,DCUT,
     &     WIDTH,DEPTH,VELOCTY,
     &     ALPHALO,BETALO,GAMMALO,DELTALO,
     &     ALPHAHI,BETAHI,GAMMAHI,DELTAHI
      INTEGER NT, TOPSUM
      GONSUM=0.0
      MPMSUM=0.0

      DO 2010 N=1,NT

C COMPUTE DEPTH AND VELOCITY DOWNSTREAM OF THE POND

      IF (OUTFLW(N).LE.0.01) THEN
          DEPTH=0.0
      ELSEIF (OUTFLW(N).LE.DCUT) THEN
          DEPTH=ALPHALO*(OUTFLW(N)**BETALO)
      ELSE
          DEPTH=ALPHAHI*(OUTFLW(N)**BETAHI)
      ENDIF

      IF (OUTFLW(N).LE.0.01) THEN
          VELOCTY=0.0
      ELSEIF (OUTFLW(N).LE.VCUT) THEN
        . VELOCTY=GAMMALO*(OUTFLW(N)**DELTALO)
      ELSE
          VELOCTY=GAMMAHI*(OUTFLW(N)**DELTAHI)
```

```
      ENDIF


C THE FOLLOWING COMPUTES BEDLOAD USING THE GONCHAROV EQUATION
      TEST = VC+0.02
      IF(VELOCTY.LE.TEST) THEN
          QBGON(N)=0.0
      ELSE
          QBGON(N)=168000*DT*WIDTH*((VELOCTY/VC)**3.0)
     &*((D/DEPTH)**0.1)*(VELOCTY-VC)
      ENDIF

C THIS SECTION CALCULATES BEDLOAD TRANSPORT USING THE MPM EQUATION
      IF((SWEIGHT*DEPTH*SL/TC).LE.1.001) THEN
          QBMPM(N)=0.0
      ELSE
          QBMPM(N)=553.0*DT*WIDTH*(((SWEIGHT*DEPTH*SL)-TC)**1.5)
      ENDIF
C THE FOLLOWING COMPUTES THE TOTAL LOAD COUNTING NEGATIVE VALUES AS ZERO
C TRANSPORT
      IF(QBGON(N).GT.0.0) GONSUM=GONSUM+QBGON(N)
      IF(QBMPM(N).GT.0.0) MPMSUM=MPMSUM+QBMPM(N)
 2010 CONTINUE

C THE FOLLOWING MAKES LOADS NEGATIVE IF POND OVERTOPS

      IF(TOPSUM.NE.0) THEN
        GONSUM = -GONSUM
        MPMSUM = -MPMSUM
      ENDIF

      RETURN
      END

C*******************************************************************************
C*******************************************************************************

C  INTERPOLATION FUNCTION
      FUNCTION XINTERP(NUM, A, B, X, OK, SPILL)
      REAL A(IA), B(IA), X, FRACTION
      INTEGER NUM
      LOGICAL OK, LAST, SPILL
      OK = .TRUE.
      LAST = .TRUE.
      SPILL = .FALSE.
      IF (X .LT. B(1)) THEN
        OK = .FALSE.
        XINTERP = A(1)
        RETURN
      END IF
      IF (X .GT. B(NUM)) THEN
        SPILL = .TRUE.
        XINTERP = A(NUM)
        RETURN
      END IF
      DO 3010 I=1, NUM-1
        IF (B(I+1) .GT. X) THEN
          LAST = .FALSE.
          GOTO 3020
        END IF
```

```
3010   CONTINUE
3020   IF (X .EQ. B(I)) THEN
          XINTERP = A(I)
          RETURN
       END IF
       FRACTION = (X-B(I))/(B(I+1)-B(I))
       XINTERP = A(I) + (FRACTION * (A(I+1) - A(I)))

       RETURN
       END
```

APPENDIX B

HYDRAULIC FORMULAS FOR POND OUTLET STRUCTURES

This appendix lists the formulas used in this study to compute the discharge from the pond outflow structures. A discussion of the background and rationale for each formula is given where appropriate. In addition to parameters defined for each outlet type, the following parameters are used throughout the appendix.

Q = outlet discharge (cfs)
Y = pond water surface elevation (ft)
Y0 = elevation of outlet invert (ft).
g = acceleration gravity (ft/s^2)

RECTANGULAR ORIFICE

The flow discharge through a rectangular orifice is given by the following equations:

If $(Y \leq Y0 + h)$: $Q = C_w\, b\, (Y-Y0)^{1.5}$

If $(Y > Y0 + h)$: $Q = C_o\, bh\sqrt{2g(Y-m)}\ (1 - \frac{1}{96}(\frac{h}{Y-m})^2)$

h = orifice height (ft)
b = orifice width (ft)
m = midpoint elevation of orifice (Y0 + h/2) (ft)
$C_w = 3.1$ Weir coefficient
$C_o = 0.6$ Orifice coefficient

For $Y < Y0 + h$ the orifice works as a weir with a discharge equation equivalent to that of a straight crested weir.

If $Y > Y0 + h$ orifice flow is obtained. The term $(1-(1/96)*(h/(Y-m))^2)$ represents a correction factor (Sotelo, 1982) which is obtained by integrating an element of discharge dQ across the orifice. Alternatively, Rouse (1946) obtained an expression for the coefficient of discharge Co as a function of the orifice dimensions, and the mean head (Y-m). The correction term shown above takes values smaller than 1 (i.e. reduction in discharge) for h/(Y-m) values greater than unity. Therefore, this correction is mainly applied to orifices under low heads with relatively large dimensions compared to this head. For these cases, the discharge should be reduced by this factor because the mean velocity through the orifice can not be computed from the mean head measured from the center of the orifice. The difference between the velocity computed with the head (Y-m) and the actual mean velocity through the orifice, increases with increasing values of h/(Y-m) (Rouse, 1946).

CIRCULAR ORIFICE

The flow discharge through a circular orifice is given by the following equations:

If $(Y \leq YO + D)$: $Q = \phi\mu D(Y-YO)^{2.5}$
where

$\phi = (10.12R^{1.975}) - (2.66R^{3.78})$

$\mu = 0.555 + \dfrac{D}{110(Y-YO)} + 0.041\dfrac{Y-YO}{D}$ for $D < 0.98$ ft and $\dfrac{Y-YO}{D} < 1$

$\mu = 0.35 + \dfrac{0.002D}{Y-YO}$ for all others

If $(Y > YO + D)$: $Q = C_O \dfrac{\pi}{4} D^2 \sqrt{2g(Y-m)}\ \left(1 - \dfrac{1}{128}\left(\dfrac{D}{Y-m}\right)^2\right)$

$R = \dfrac{Y-YO}{D}$

D = orifice diameter (ft)
m = midpoint elevation of orifice ($YO + D/2$)
$C_O = 0.6$ Orifice coefficient

For $Y < YO + D$ weir flow conditions are obtained. The discharge formula used is the theoretical formula derived by Stauss (Sotelo, 1982). The coefficient μ was determined by the Stauss and Jorissen formula (Smetana, 1957) for values of $D < 0.98$ ft and $Y-YO/D < 1$. For other conditions, μ is given by Ramponi's formula (Sotelo, 1982) which is valid, in the form written above, for large approach flow areas compared with the flow area over the weir. The ϕ factor is determined using the approximation of Ramponi (Sotelo,1982; Schlag, 1963) given by the equation shown above.

For $Y > YO + D$ orifice flow is obtained, and the correction factor $(1-(1/128(D/(Y-m))^2))$ is used for the same reasons described in the rectangular orifice.

CIRCULAR RISER

The discharge equations for the circular riser with inlet control are given by:

If $(Y \leq YO + 0.4D)$: $Q = C_W \pi D(Y-YO)^{1.5}$

If $(Y > YO + 0.4D)$: $Q = C_O \dfrac{\pi}{4} D^2 \sqrt{2g(Y-YO)}$

D = riser diameter (ft)
$C_W = 3.1$ Weir coefficient
$C_O = 0.6$ Orifice coefficient

Generally, for the circular as well as for the square riser, the barrel is designed to carried out the design storm in a way that the flow over the weir is not submerged under normal flow

conditions (i.e. the water level in riser is less than the critical depth at the weir). The flow through the barrel is computed using the culvert equations.

For Y > YO + 0.4 D the flow at the top of the riser changes from weir flow to orifice flow. The change from weir flow to orifice flow is achieved when (Y-YO)/D reaches 0.4 as assumed by Harrington (1987a). From studies at Cornell University (Sotelo, 1982), it has been reported that a transition flow region exists for values of (Y-YO)/D between 0.4 and 1.0. However, for simplicity, orifice flow is assumed for values of (Y-YO)/D greater that 0.4 due to the uncertainties in describing the flow characteristics within the transition region.

SQUARE RISER

For the square riser the following equations are considered:

If $(Y \leq YO + 0.4b)$: $\qquad Q = C_w 4b(Y-YO)^{1.5}$

If $(Y > YO + 0.4b)$: $\qquad Q = C_O b^2 \sqrt{2g(Y-YO)}$

b = length of riser side (ft)
$C_w = 3.1$ Weir coefficient
$C_O = 0.6$ Orifice coefficient

The same assumptions as in the circular riser are considered. The weir flow at the top of the riser is equal to the sum of the discharges of four straight crested weirs. This is a good assumption provided that there is no mutual flow interference among the weirs.

STRAIGHT-CRESTED WEIR

$$Q = C_w b(Y-YO)^{1.5}$$

b = length of weir (ft)
$C_w = 3.1$ Weir coefficient

VEE-NOTCH WEIR

$$Q = C_w \tan \left(\frac{A}{2}\right) (Y-YO)^{2.5}$$

A = angle of v-notch (degrees)
$C_w = 2.57$ Weir coefficient

NOTES

It is important to point out that for all the weir and orifice equations described above, the discharge coefficients are the ones corresponding to approach flow areas considerable larger than the flow area at the outlet. Furthermore, the effects of the walls and bottom of the pond on the flow discharge is assumed negligible.

CULVERT

Different flow characteristics through a culvert structure are obtained as the water level in the pond changes during a storm event. The different types of flow that may occur are depicted in Fig. B-1 (French, 1985) according to the classification of culvert flow given by Bodhaine (1976). The flow characteristics can be simplified to types 1, 2 and 5 for water levels in the downstream channel smaller than the critical depth in the culvert (i.e. downstream channel considerably lower than the culvert inlet).

In type 1 flow, the critical depth is located at the entrance of the culvert and therefore, the flow is inlet control. However, if the critical depth is at the outlet, the flow is under type 2 outlet control conditions. Type 5 is basically a transition flow region between types 2 or 3 and type 6 (provided the water level in the downstream channel is lower than the crown of the culvert). In general, the flow in type 5 is contracted by the top edge of the inlet, and the culvert flows partially full at a depth below the critical. However, type 5 flows are usually obtained for culvert length to diameter ratios smaller than 6 and culvert slopes greater than 3% (Bodhaine, 1976). Therefore, eventually type 5 flow will change to type 6 flow with outlet control.

For the current analysis, flow types 1 and 2 are good representations of typical culvert flow characteristics of a 2-yr storm. This is a valid assumption because a culvert that is conservatively designed to carry the 5-yr or 10-yr storms (under type 4 outlet control flow) is unlikely to flow full under the 2-yr storm. Furthermore, if the culvert is designed so that the barrel slope S_o is greater than the critical slope S_c, only inlet control type 1 flows are obtained. If $(h_1-z)/D$ reaches values greater than 1.5 for the 2-yr storm, the computer program will compute the culvert discharge for either type 5 or 6 flows. However, this is unlikely to happen. To demonstrate this, an example taken from Harrington (1987a) was selected. In this example, a storm-water detention pond with a designed outlet culvert structure was able to pass the 5 yr storm of 65.4 cfs under type 4 flow conditions with $(h_1-z)/D = 1.97$. For the 2-yr storm of 12.9 cfs, the same culvert (diameter = 3 ft, length = 60 ft, and slope = 1%) is able to pass the 2-yr flow under type 1 flow conditions with $(h_1-z)/D$ equal to 0.8.
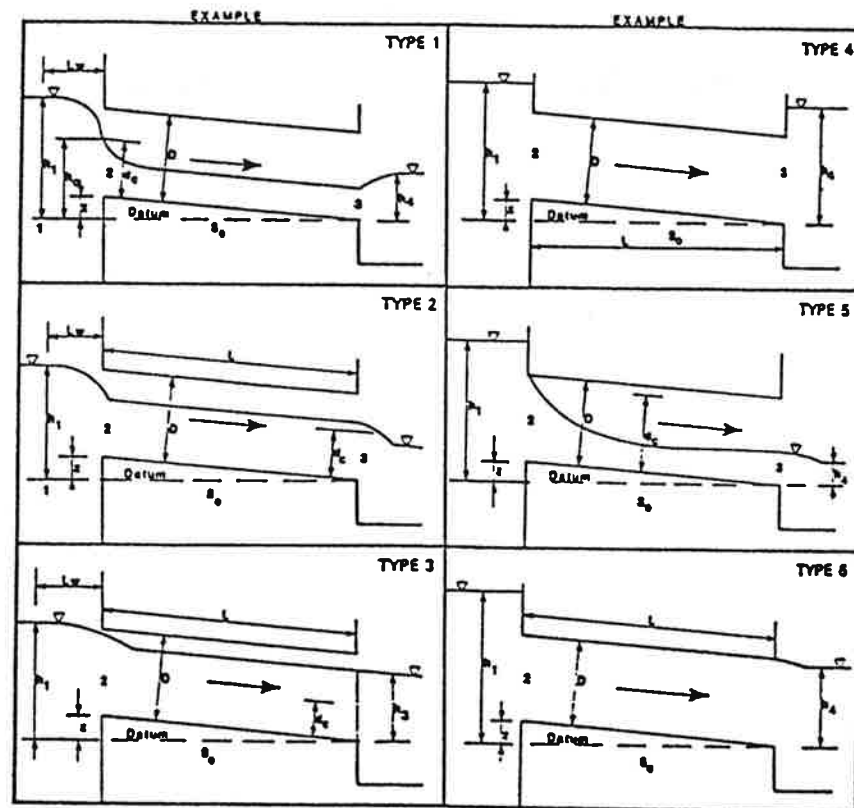
Culvert type flow	Discharge equation
Type 1. Critical depth at inlet $(h_1 - z)/D < 1.5$ $h_4/h_c < 1.0$ $S_0 > S_c$	$Q = C_D A_c \sqrt{2g\left(h_1 - z + \alpha_1 \dfrac{u_1^2}{2g} - y_c - h_{f1,2}\right)}$
Type 2. Critical depth at outlet $(h_1 - z)/D < 1.5$ $h_4/h_c < 1.0$ $S_0 > S_c$	$Q = C_D A_c \sqrt{2g\left(h_1 + \alpha_1 \dfrac{u_1^2}{2g} - y_c - h_{f1,2} - h_{f2,3}\right)}$
Type 3. Tranquil flow throughout $(h_1 - z)/D < 1.5$ $h_4/h_c \leq 1.0$ $h_4/h_c > 1.0$	$Q = C_D A_3 \sqrt{2g\left(h_1 + \alpha_1 \dfrac{u_1^2}{2g} - h_3 - h_{f1,2} - h_{f2,3}\right)}$
Type 4. Submerged outlet $(h_1 - z)/D > 1.0$ $h_4/D > 1.0$	$Q = C_D A_0 \left[\dfrac{2g(h_1 - h_4)}{1 + (29C_D^2 n^2 L/R_0^{4/3})}\right]^{1/2}$
Type 5. Rapid flow at inlet $(h_1 - z)/D \geq 1.5$ $h_4/D \leq 1.0$	$Q = C_D A_0 \sqrt{2g(h_1 - z)}$
Type 6. Full flow free outlet $(h_1 - z)/D \geq 1.5$ $h_4/D \leq 1.0$	$Q = C_D A_0 \sqrt{2g(h_1 - h_3 - h_{f2,3})}$

B.1   Types of culvert flow.

APPENDIX C

CROSS SECTIONS OF DEMONSTRATION SITE CHANNELS
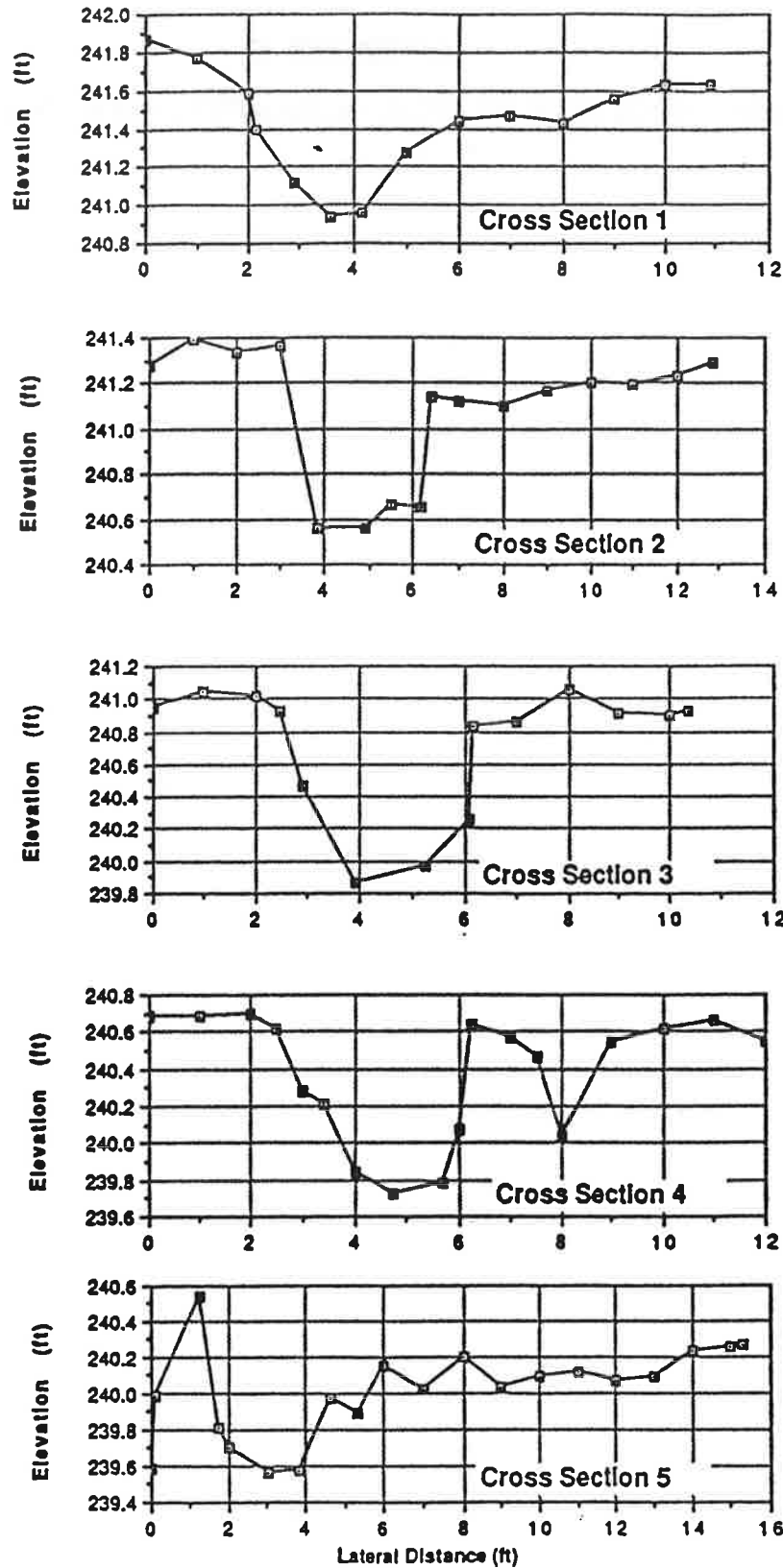
C.1   Cross-sections of channel downstream of the Newport Towne detention pond. Cross-sections viewed looking downstream; cross-section 1 is closest to SWM facility.
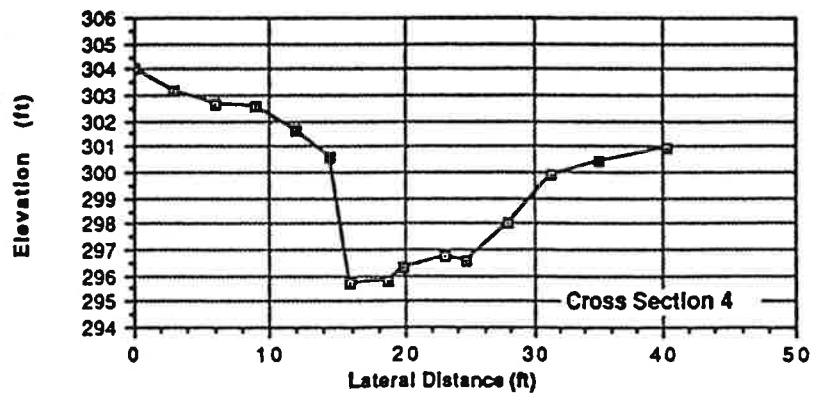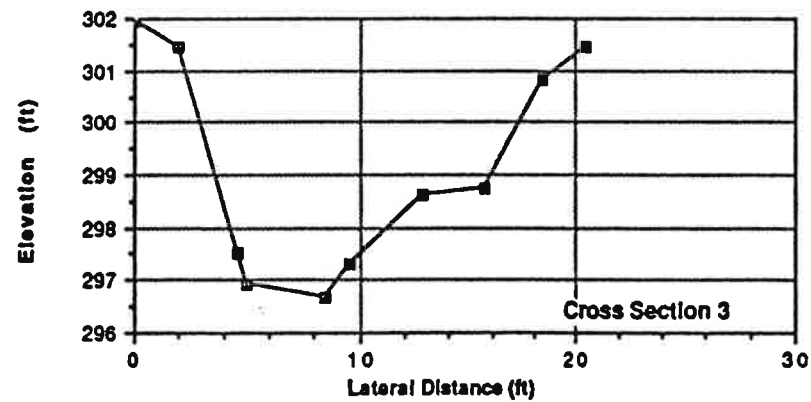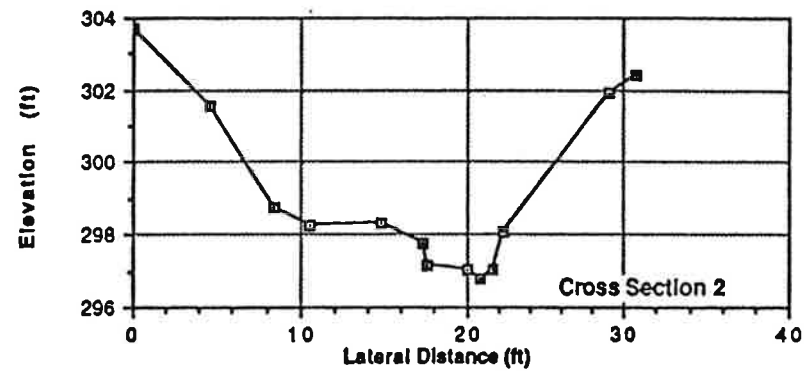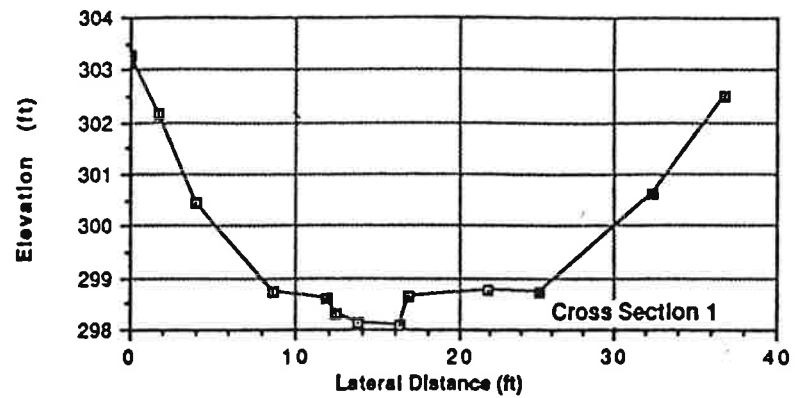
C.2 Cross-sections of channel downstream of the Snowdens Mill detention pond. Cross-sections viewed looking downstream; cross-section 1 is closest to SWM facility.

APPENDIX D

## DEVELOPMENT OF DEPTH AND VELOCITY POWER EQUATIONS

The power relationships for depth and velocity were developed from the computed water surface profiles for the channel downstream of each demonstration site. Using the surveyed cross-sections (Appendix C), water surface profiles were obtained using the computer program HEC-2 (USCOE, 1982). For the Newport Towne channel, the program was run for a series of flows between 1 and 10 cfs (flows approximately within the banks of the channel) and for flows between 10 and 80 cfs for overbank flows. For each cross-section, depth and velocity versus discharge relationships were obtained using HEC-2 for the range of flows analyzed. Similar relationships were obtained for Snowdens Mill for flows between 2 and 20 cfs (inbank), and between 20 and 200 cfs (overbank).

In order to implement these relationships in the computer program, depth and velocity versus flow discharge were fitted using nonlinear regression. The regressions were performed for each cross-section and a range of discharges (including inbank and overbank discharge) using the computer program STATGRAPHICS (1988). The correlation coefficients from these regressions were generally equal to 0.98 and 0.99. STATGRAPHICS uses an algorithm developed by Marquardt (1963). Nonlinear regression (NLR) is a better approach than the linear regression of log-transformed values (LTR). The prediction obtained by the inversion of the estimated Log Y or Log V is biased (Koch and Smillie, 1986). This bias is proportional to the error variance and, therefore, only for perfect fits (error variance equal to zero) the bias is unity. On the other hand, using the NLR model, unbiased estimates of the coefficients and exponents are obtained. In this case, the expected value of the nonlinear prediction model error is equal to zero, thereby reducing the bias of the LTR model (Singh, et. al, 1987).

Average relationships (depth versus discharge and velocity versus discharge) for the channel downstream of each pond were obtained by weighting the coefficients and exponents from each cross-section. The weighting was done according to the length of the channel segment represented by each cross-section. Figures D.1 to D.4 show the average relationships for the Newport Towne and Snowdens Mill channels. In general, the average % error or variation (defined as $|E-C|/C$, where E is the estimated velocity V or depth Y from the regime equations and C the computed velocity or depth at a particular cross-section from the HEC-2 results) was found to be around 10%.
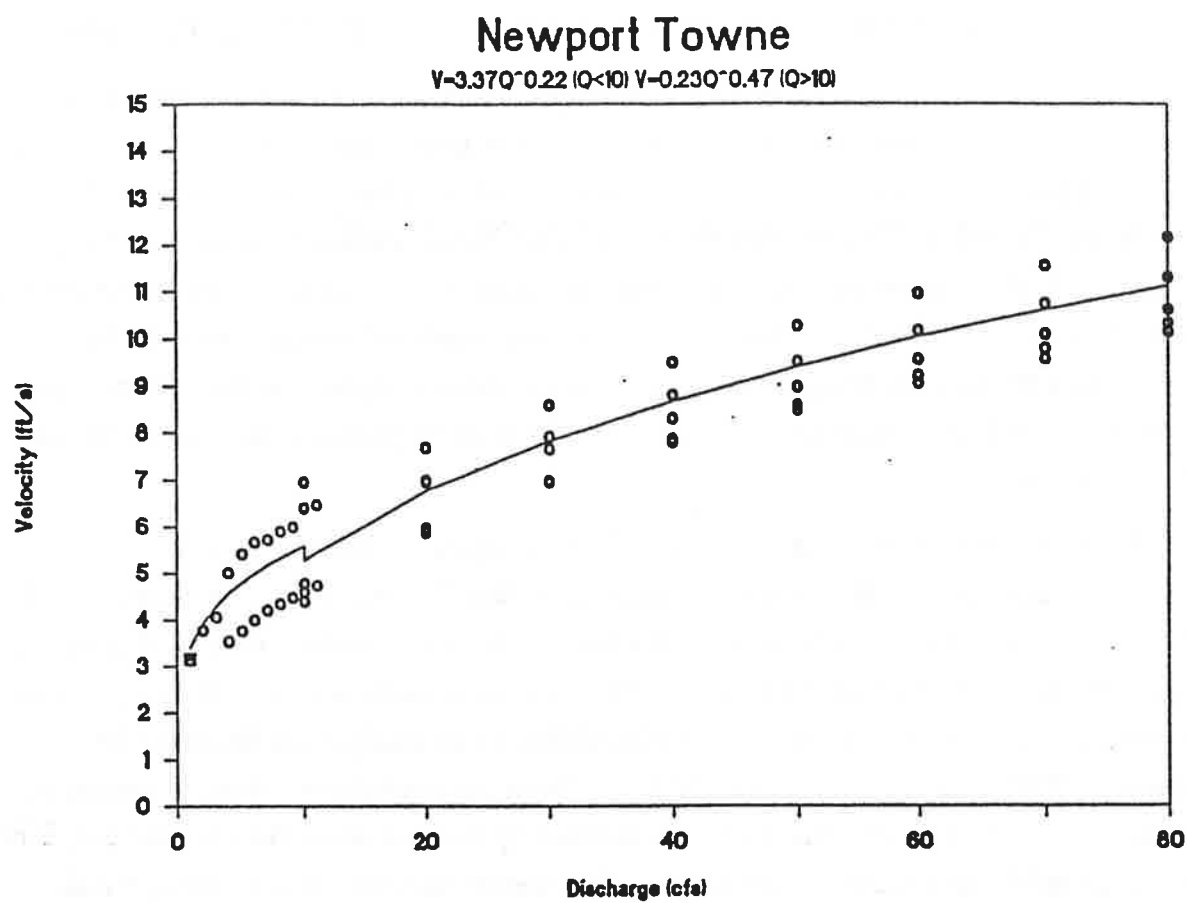
Newport Towne

V=3.37Q^0.22 (Q<10) V=0.23Q^0.47 (Q>10)

D.1   Velocity as a function of discharge for channel downstream of the Newport Towne detention pond.

Newport Towne

Y=0.23Q^0.47 (Q<10) Y=0.31Q^0.32 (Q>10)

Flow depth (ft)

Discharge (cfs)

D.2   Depth as a function of discharge for channel downstream of the Newport Towne detention pond.

Snowdens Mill
V=1.42Q^0.32 (Q<20) V=1.40Q^0.34 (Q>20)

D.3   Velocity as a function of discharge for channel downstream of the Snowdens Mill detention pond.

Snowdens Mill

Y=0.44Q^0.37 (Q<20) Y=0.32Q^0.45 (Q>20)
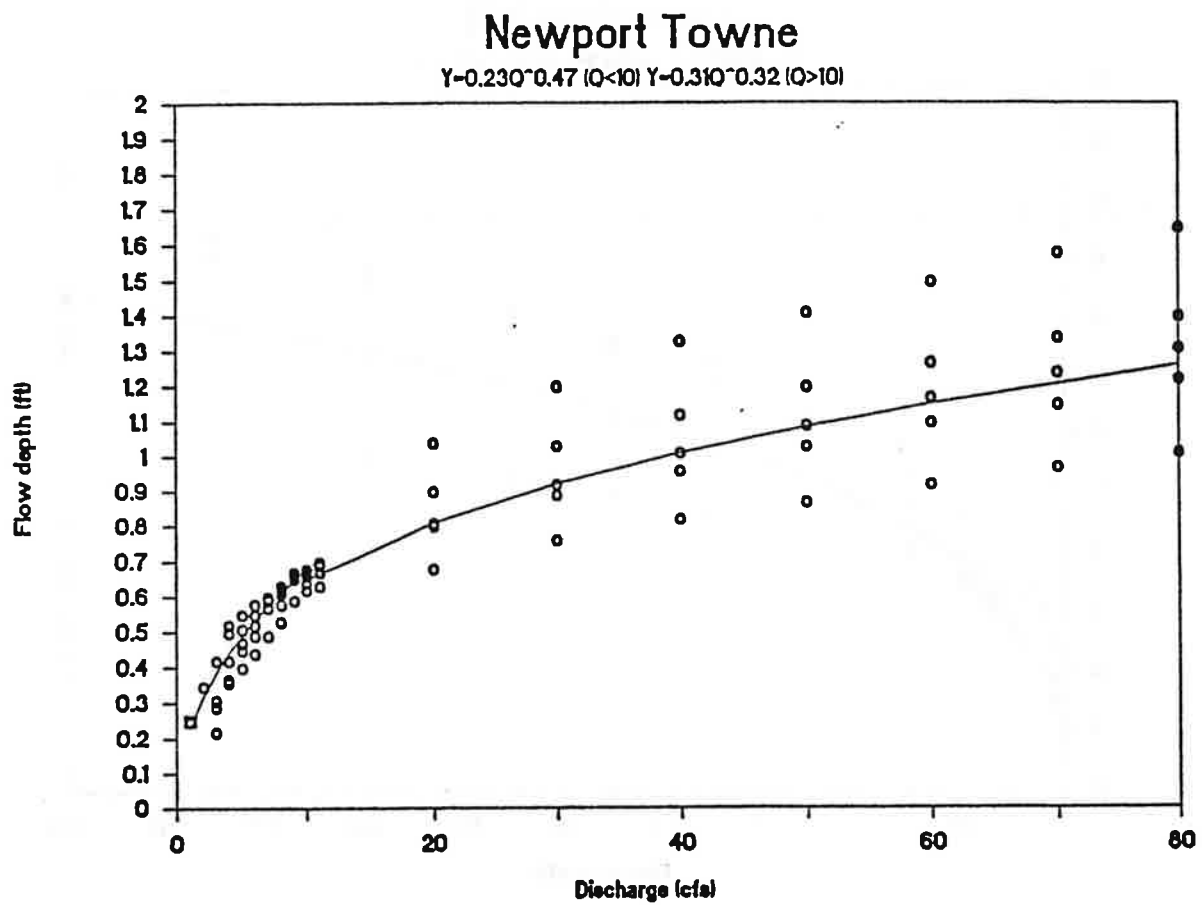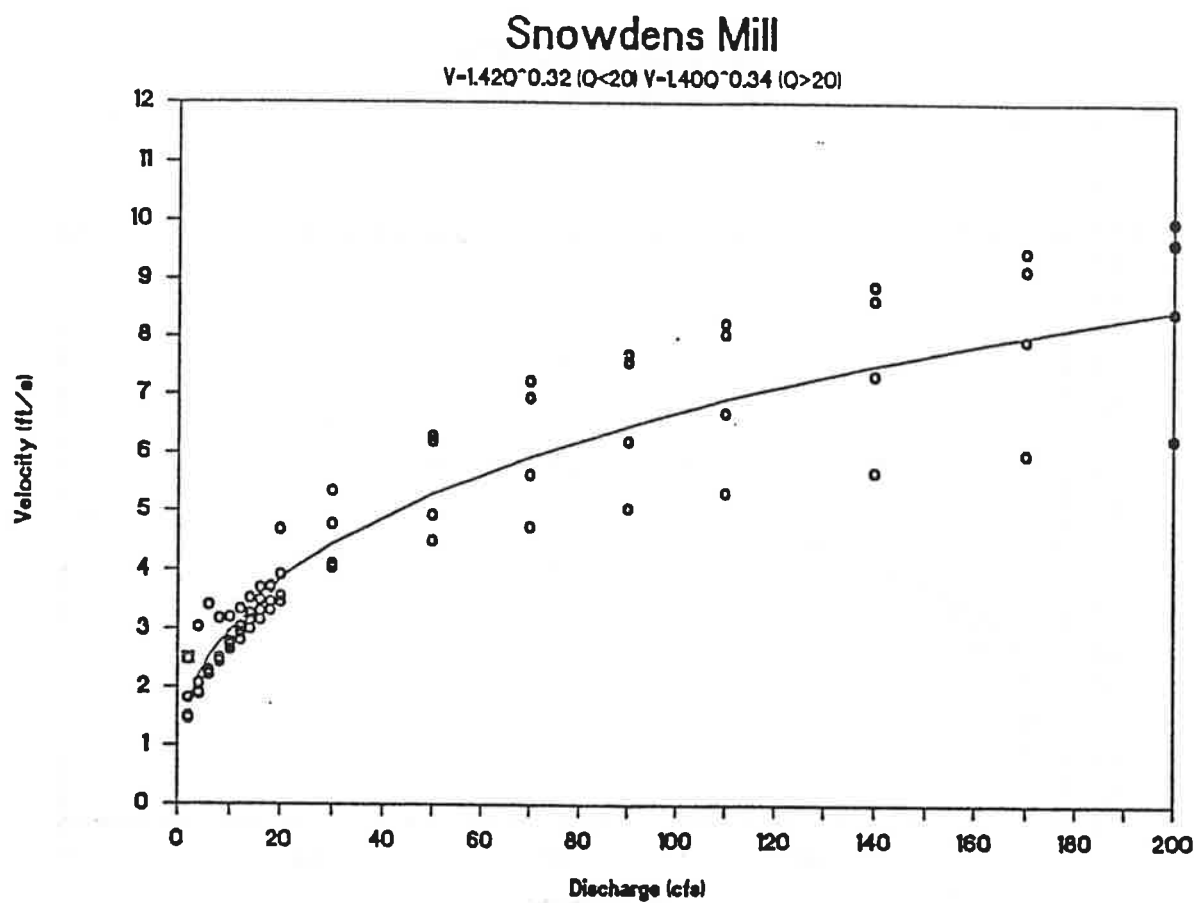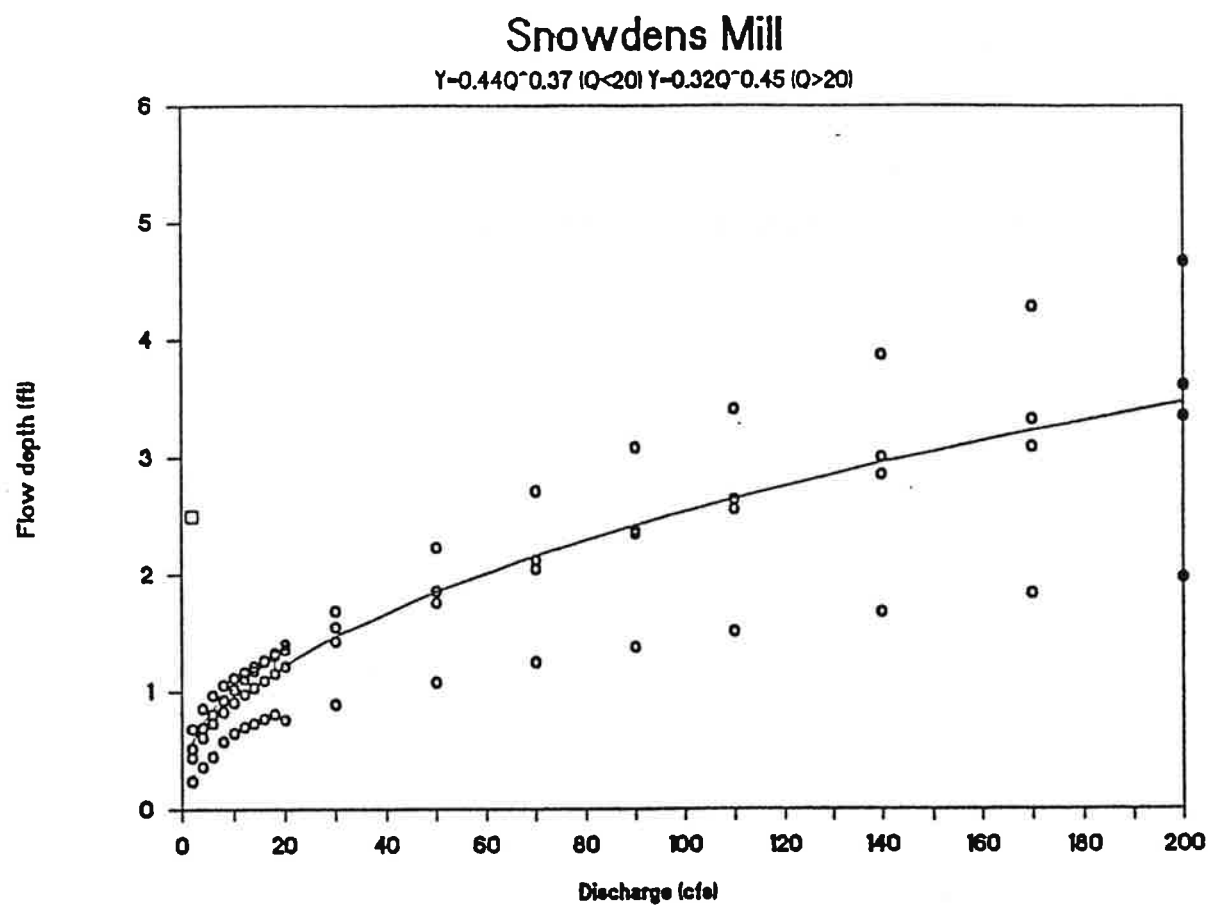
D.4   Depth as a function of discharge for channel downstream of the Snowdens Mill detention pond.

APPENDIX E

⹂ GLOSSARY *

bed load The sediment load in a stream channel that is transported on or near the stream bed by rolling, sliding, or hopping. Bed load and sediment load may be expressed as the mass of sediment transported through a channel section per unit time (e.g. lbs/second).

bed material load That portion of the sediment load in a stream channel that consists of particles that are found in the bed material. The sediment load that is not bed material load is wash load. Total bed material load is used in this report as the total mass of bed material transported through a channel section over the duration of the design storm.

dead storage The portion of a pond or other stormwater management facility that is below the elevation of the lowest outlet structure.

design storm A rainfall event of specified size and return period that is used to generate the runoff hydrograph used for design purposes.

detention The temporary storage of storm runoff in a pond or other stormwater management facility.

detention time The amount of time a parcel of water is actually present in a pond or other stormwater management facility. Theoretical detention time for a runoff event is the average time parcels of water reside in the basin over the period of release from the pond or other stormwater management facility.

discharge The rate of flow expressed as a volume of water per unit time (e.g. ft^3/sec = cfs).

drawdown The gradual reduction in water level in a pond due to the combined effect of outlet discharge, infiltration, and evaporation.

freeboard The space from the top of an embankment to the maximum water elevation expected for the largest design storm. This space is required as a safety margin in a pond.

hydrograph A graph depicting the discharge of a stream as a function of time.

invert elevation The vertical elevation of a pipe or orifice which defines the water level in a pond.

peak discharge The maximum instantaneous rate of flow during a storm, usually in reference to a specific design storm event.

peak discharge control Controlling post-development peak discharge rates to pre-development levels.

release rate The rate of discharge released from a stormwater management facility.

retention The holding of runoff in a basin without release except by means of evaporation, infiltration, or emergency bypass.

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* Adapted from Schueler, 1987.

retrofit To install a new stormwater management facility or improve an existing facility in a previously developed area.

riser A vertical pipe extending from the bottom of a pond that is used to control the discharge rate from a pond for a specified design storm.

stormflow The portion of flow which reaches the stream shortly after a storm event.

stormwater management facility A facility designed to control stormwater discharge, usually for a design storm. These include detention and retention ponds, infiltration basins, and porous pavement. Stormwater management facilities are a type of best management practice (BMP).

stream power The rate of potential energy expenditure in a stream channel. This energy expenditure is the energy available to transport sediment and produce heat. Steam power may be expressed per unit channel length as the product of the specific weight of the fluid, the fluid discharge, and the channel slope (γQS).

wash load The finest portion of the sediment load, usually silt and clay, which is washed through the channel. Wash load is all the transported sediment that is not bed material load. An insignificant amount of the wash load sediment is found in the channel bed. Wash load plays a negligible role in the erosion and deposition of stream channels.