4014

REVISED FINAL DESIGN
CALCULATION PACKAGE

ON-SITE DISPOSAL FACILITY
PHASE IV
PROJECT NUMBER 20104

United States Department of Energy
Fernald Environmental Management Project
Fernald, Ohio

prepared by

GEOSYNTEC CONSULTANTS

1100 Lake Hearn Drive, NE
Suite 200
Atlanta, Georgia 30342

under

Fluor Fernald, Inc.
Contract 95PS005028
Document No. 20100-CA-0001

INFORMATION ONLY

DEPARTMENT OF ENERGY • UNITED STATES OF AMERICA •

REVISION 0
VOLUME VI of VI
AUGUST 2001

000001

REVISED FINAL DESIGN CALCULATION PACKAGE ON-SITE DISPOSAL FACILITY

Volume VI

August 2001
Revision 0

United States Department of Energy

Fernald Environmental Management Project
Fernald, Ohio

Prepared by

GeoSyntec Consultants
1100 Lake Hearn, NE, Suite 200
Atlanta, Georgia 30342

Under

Fernald Environmental Restoration Management Corporation
Subcontract 95PS005028

REVISED FINAL DESIGN PACKAGE
LIST OF CALCULATIONS
On-Site Disposal Facility
Fernald Environmental Management Project

VOLUME I

LIST OF CALCULATIONS (continued)

LIST OF CALCULTIONS (continued)

LIST OF CALCULATIONS (continued)

LIST OF CALCULATIONS (continued)

LIST OF CALCULATIONS (continued)

12.4 OSDF-PHASE IV-SURFACE WATER MANAGEMENT SYSTEM DESIGN (SEE VOLUME VI)

COMPUTATION COVER SHEET

SUBJECT OF COMPUTATIONS PHASE IV SURFACE-WATER MANAGEMENT SYSTEM DESIGN

Computations By:
(Cognizant Engineer)

Signature Dana Beth Mehlman — Date 22 Aug 2001

Printed Name and Title: Dana Beth Mehlman, Staff Engineer

Assumptions and Procedures Checked By:
(Checker)

Signature — Date 24 Aug. 2001

Printed Name and Title: Ganesh Gopalakrishnan, Project Engineer

Computations Checked by:

Signature — Date 24 Aug 2001

Printed Name and Title: Ganesh Gopalakrishnan, Project Engineer

Computations Backchecked by:
(Cognizant Engineer)

Signature Dana Beth Mehlman — Date 24 Aug 2001

Printed Name and Title: Dana Beth Mehlman, Staff Engineer

Approved by:
(DTL)

Signature — Date 24 AUG 2001

Printed Name and Title: Daniel G. Pass, Assistant Project Engineer

Approved by:
(TETL)

Signature — Date 27 Aug 2001

Printed Name and Title: J.F. Beech, Principal

Record of Revision (Number and initial all revisions)

Rev. No.	Reason	Date	By	Checked	Approval

PHASE IV SURFACE-WATER MANAGEMENT SYSTEM DESIGN

EXECUTIVE SUMMARY

PURPOSE OF ANALYSES

The purpose of this calculation package is to design the OSDF surface-water management (SWM) structures to be constructed as part of the Phase IV development of the OSDF. These SWM structures are designed to satisfy the requirements of the Surface-Water Management and Erosion Control (SWMEC) Plan [GeoSyntec, 2000d] and the OSDF Design Criteria Package (DCP) [GeoSyntec, 2000a] and are shown on the Construction Drawings. Additionally, the adequacy of existing SWM structures is assessed. Required modifications or additions to existing structures are incorporated into the Construction Drawings and this calculation package.

METHODS OF ANALYSES

For the purpose of hydrologic analyses and routing through OSDF Sedimentation Basin 1 (basin), the total drainage area anticipated to drain into the basin is modeled as the OSDF Design Scenario. The OSDF Design Scenario presented here considers the actual conditions anticipated to exist when both Cell 4 and Cell 5 have been constructed and lined and Cell 2 has been capped. This represents the "worst case" conditions for the structures being evaluated. SWM structures for the condition where only Cell 4 is constructed have been evaluated, and assessed to be adequate, as a part of the OSDF Phase III package [GeoSyntec, May 2000]. Design Case "A" represents a channel and a culvert at the southeast corner of Cell 5 constructed as part of the SWM system for Cell 5. Neither the channel nor the culvert is incorporated into the OSDF Design Scenario. Design Case "B" represents multiple culverts located along the Emergency Access Road. None of these culverts are incorporated into the Design Scenario or the other Design Case.

For Phase IV SWM structures (i.e., new structures), analyses are performed to design channels and culverts. These structures are designed to safely convey peak flow rates from the 25-year, 24-hour storm event. Additional analyses are performed for the selection of channel lining (channel bed stability) and required height of cover over culverts (structural stability). Analyses for channels along the East and West perimeters of the OSDF was performed as part of the OSDF Final Design Package [GeoSyntec, 1997c]. Phase IV construction will not adversely impact these channels; therefore, no analyses are performed as part of this Calculation Package to assess the adequacy of these channels.

For existing SWM structures, analyses are performed to assess the adequacy of the modified OSDF Sedimentation Basin 1 (basin). The basin is considered adequate if it complies with the design criteria for the 10-, 25-, and 100-year, 24-hour storm events, outlined in the DCP for construction, filling, and closure of the OSDF. Hydrologic and basin routing analyses are performed using methodologies presented in TR-20 [SCS, 1982] and TR-55 [SCS, 1986], as coded into the computer program HydroCAD™. Hydraulic analyses for channels are performed using Manning's equation as coded into a computer spreadsheet. Hydraulic analyses for culverts are performed using methodologies presented in USDOT [1985], as coded into the computer program CulvertMaster®.

CONCLUSIONS

All Phase IV (i.e., new) channels and culverts are designed in accordance with the requirements of the SWMEC Plan [GeoSyntec, 2000d] and the DCP [GeoSyntec, 2000a], in particular as follows:

New Channels

- Calculated peak flow velocities for grass-lined channels are less than the permissible flow velocity of 5.0 fps.
- Calculated freeboards for channels equal or exceed the minimum required of 0.5 ft.

New Culverts

- Calculated headwater elevations at the inlet of culverts allow for 0.5 ft or greater freeboard.
- For CMP culverts cover provided at road crossings equal or exceed required thickness of cover for structural stability.

The existing basin (with modifications shown in the Construction Drawings) has adequate capacity for applicable design storm events, in particular as follows:

OSDF Sedimentation Basin 1

- The available minimum storage volume exceeds the calculated runoff volume (10-year, 24-hour storm event) and the calculated volume of 0.125 acre-ft/year per acre (disturbed upstream drainage area multiplied by the basin cleanout frequency).
- The calculated peak water elevation for the 25-year, 24-hour storm event is below the elevation of emergency spillway.
- The calculated peak water elevation for the 100-year, 24-hour storm event allows more than 1 ft of freeboard from the minimum embankment crest elevation.

The required cleanout frequency for the basin is one year per cleanout.

PHASE IV SURFACE-WATER MANAGEMENT SYSTEM DESIGN

TABLE OF CONTENTS

PHASE IV SURFACE-WATER MANAGEMENT SYSTEM DESIGN

PROCEDURES

INTRODUCTION AND PURPOSE

The major elements of the OSDF Phase IV project include construction and lining of the Cell 4 and Cell 5 liner systems and construction of the Cell 2 cover system. These and other construction elements are listed and their relationships to the design scenario and the design cases are shown in Table A-1. The layout of the surface-water management (SWM) system is shown on Construction Drawings.

The purpose of this Calculation Package is to design the OSDF SWM structures to be constructed as part of the Phase IV development of the OSDF. Additionally, the adequacy of the existing OSDF Sedimentation Basin 1 with new modifications is assessed. Required modifications or additions to existing structures are incorporated into the Construction Drawings and this Calculation Package. Analyses for channels along the East and West perimeters of the OSDF were performed as part of the OSDF Final Design Package [GeoSyntec, 1997c]. Phase IV construction will not adversely impact these channels; therefore, no analyses are performed as part of this Calculation Package to assess the adequacy of these channels.

DESIGN CRITERIA

The OSDF Phase IV SWM system is designed to satisfy the requirements of the Surface-Water Management and Erosion Control (SWMEC) Plan [GeoSyntec, 2000b] and the OSDF Design Criteria Package (DCP) [GeoSyntec, 2000a]. Section 2.8, *Surface-Water Management*, of the DCP contains requirements for the OSDF runon/runoff structures. This section of the DCP is included as Attachment A-1.

The design of Phase IV structures includes analyses for channels and culverts. These structures are designed to safely convey peak flow rates from the 25-year, 24-hour design storm event. Additional analyses are performed for the selection of channel lining (i.e., channel bed stability) and required height of cover over culverts (i.e., structural stability).

The assessment of existing structures includes analyses to assess the adequacy of the modified OSDF Sedimentation Basin 1. The OSDF Basin 1 is considered adequate if it complies with the design criteria for the 10-, 25-, and 100-year, 24-hour design storm events, outlined in the DCP for construction, filling and closure of the OSDF.

DESIGN SCENARIOS

The design of Phase IV SWM structures, and assessment of hydraulic capacity for existing structures, is performed using hydrologic and basin routing analyses in which the total drainage area anticipated to drain into the basin is modeled in this calculation package as the "OSDF Design Scenario". The OSDF Design Scenario presented in this package considers the actual conditions anticipated to exist when both Cell 4 and Cell 5 have been constructed and lined and Cell 2 has been capped. This represents the "worst case" conditions for the SWM structures being evaluated for this phase of construction. SWM structures for the condition where only Cell 4 is constructed have been evaluated, and assessed to be adequate, as a part of the OSDF Phase III package (GeoSyntec, May 2000). Design Case "A" represents a channel and a culvert at the southeast corner of Cell 5 constructed as part of the SWM system for Cell 5. Neither the channel nor the culvert is incorporated into the OSDF Design Scenario. Design Case "B" represents multiple culverts located along the Emergency Access Road. None of these culverts are incorporated into the Design Scenario or the other Design Case. The OSDF Design Scenario, Design Case "A", and Design Case "B" are further described in Table A-1, and below.

OSDF Design Scenario

The OSDF Design Scenario incorporates existing and new conditions, including the construction of the enhanced permanent leachate transmission system (EPLTS) project, Cell 4, Cell 5, the Cell 1 and Cell 2 final cover systems, and the Access Control Facility Road. For this design scenario the Cell 1 final cover system is vegetated, the Cell 2 final cover system is unvegetated, and Cells 3, 4 and 5 are receiving impacted material (i.e., runoff from within the perimeter berm of cells receiving impacted material is contained and does not enter the Phase IV SWM system). The capacity of the OSDF Basin 1 is reduced due to the relocation of the access control facility road, which has been relocated to just south of Valve House 6. The bottom of the OSDF Basin 1 has been excavated to increase the capacity of the basin and OSDF Basin 1 is also expanded east to the perimeter of the impacted material access facility road berm. A schematic layout of the SWM system represented in the OSDF Design Scenario is provided in Attachment A-2.

Design Case "A"

Design Case "A" incorporates a channel and a culvert at the southeast end of Cell 5, which is not a part of the Phase IV SWM system for the Cell 5 liner system. Design Case "A" considers existing conditions in the runon area to the east of the OSDF, and includes the runoff from the easternmost section of Cells 4 and 5. Neither the channel nor the culvert is represented in the OSDF Design Scenario. A schematic layout of the SWM system in Design Case "A" is provided in Attachment A-2.

Design Case "B"

Design Case "B" incorporates multiple culverts along the Emergency Access Road, which are not a part of the Phase IV SWM system for the Cell 5 liner system. Design Case "B" considers existing conditions in the runon area to the east of the OSDF. These culverts are not represented in the OSDF Design Scenario or the other Design Case. A schematic layout of the SWM system in Design Case "B" is provided in Attachment A-2.

SOFTWARE

Hydrologic and Basin Routing Analyses

Surface-water runoff peak flow rates and other hydrologic and hydraulic information are estimated using the computer program "HydroCAD™" [HydroCAD™, 2001]. This program uses hydrologic procedures presented in U.S. Soil Conservation Services' TR-20 [SCS, 1982] and TR-55 [SCS, 1986]. Hydrographs for individual subcatchments are routed through a user specified nodal network for each design scenario and design case (see Attachment A-8), using standard hydrologic routing techniques. Runoff hydrographs from subcatchments are developed using input parameters which describe the storm event for which the calculation is being performed and characteristics of the subcatchment. Built-in subroutines allow the user to route the hydrographs through a sediment basin. The HydroCAD™ Technical Reference Manual is provided as Attachment A-3.

Channel Hydraulic Analyses

Hydraulic analyses for channels are performed using computations based on Manning's equation, as coded into a computer spreadsheet.

Culvert Hydraulic Analyses

Hydraulic analyses for culverts are performed using methodologies presented in USDOT [1985], as coded into the computer program CulvertMaster® [Haestad Methods, 2000]. Built-in subroutines allow the user to perform calculations for inlet and outlet control. The CulvertMaster® User's Guide [Haestad Methods, 2000] Theory Section is provided as Attachment A-4.

HYDROLOGIC AND BASIN ROUTING ANALYSES

Hydrologic and basin routing analyses are performed using HydroCAD™ for 10-, 25-, and 100-year, 24-hour storm events. Input parameters are presented in the Data Verification section of this calculation package. HydroCAD™ output reports are presented in Attachment C-1 of the Calculation Section of this calculation package. Major input used in the HydroCAD™ subroutines includes parameters for subcatchments, reaches, and basins. These parameters are introduced and calculation methods described in the following sections.

Subcatchment Runoff

Rainfall Distribution

This parameter characterizes the assumed distribution of the design rainfall depth over a 24-hour duration and is selected based on the location of the Fernald site within the United States. Selection of this parameter is further described in the Data Verification section of this calculation package.

Rainfall Depth

This parameter is the total rainfall for a 24-hour design storm event. The design rainfall depths for return periods of 2, 10, 25, and 100 years are obtained as described in the Data Verification section of this calculation package.

Hydrologic Soil Groups (HSG)

This parameter classifies surficial soils at the site based on hydrologic characteristics, as presented in the Soil Surveys of Butler and Hamilton Counties, Ohio [USDA-SCS, 1980 and 1992]. The soil types and hydrologic soil groups found in the vicinity of the OSDF watershed are shown on a plan view of the OSDF area in Attachment A-5 [USDA-SCS, 1980 and 1992] and soil hydrologic data in Attachment A-6 [USDA-SCS, 1980 and 1992]. The soil names identified for the OSDF area and their respective HSG classification are summarized below.

SCS Map Label	Soil Name	Hydrologic Soil Group
FcA, FdA	Fincastle	C
Rda	Raub	C
RwB2	Russell	B
XeB, XeB2, XfA, XfB2	Xenia	B
MoE2	Miamian	C

An HSG classification of B/C is selected for final cover system and liner system runout areas. The final cover system and liner system runout will be constructed using soil from the borrow area. This soil is anticipated to include B and C soils, with approximately 85% of the soil being classified as B.

Runoff Curve Numbers (CNs)

CNs are selected based on ground cover type, land use, cover condition, and HSG classification of site soils. A list of land use categories characterizing drainage areas for the Phase IV SWM system is presented in the following table. This list is developed based on characterization of the ground cover and land use for anticipated runoff area conditions during Phase IV construction. Land use categories are associated with TR-55 categories of land use and ground cover (See Attachment A-7 from [SCS, 1986]) and are presented in the following table:

Land Use Category	TR-55 Land Use/ Ground Cover Category	HSG	CN
Unvegetated final cover system and liner system runout	Newly graded areas	B	86
		C	91
		B/C	89
Vegetated final cover system	Open space in poor condition - corresponds to 50% grass cover	B	79
		C	86
		B/C	83
Runon Areas East of the OSDF	Pasture in fair condition	B	69
		C	79
Disturbed area used for construction support activities	Farmsteads – buildings, lanes, driveways, and surrounding lots	C	82
Sedimentation basin (SB) or direct runoff to SB	Impervious surfaces	-	98

For areas indicated in the above table as having an HSG of "B/C", the CN was calculated based on a 1:1 ratio of B and C soils. A weighted CN for each subcatchment (CN_{S-i}) is calculated as a weighted average of CNs present within the subcatchment. The following equation is used as coded into a spreadsheet:

$$CN_{S-i} = \sum_{i=1}^{n} P_i CN_i$$

Where, P_i is the percentage of total subcatchment area for CN_i.

Subcatchment Time of Concentration

The time of concentration for each subcatchment is calculated as the sum of the travel times for sheet, shallow concentrated, and channel flows, for a flow path from the most hydraulically distant location in the subcatchment to the outlet of the subcatchment. These calculations are performed within HydroCAD™ using specified input parameters for flow regimes. Methods and equations used for these calculations are presented in the HydroCAD™ Technical Reference Manual presented as Attachment A-3. Input parameters include the following:

- For sheet flow travel time: surface description or n = Manning's roughness coefficient (dimensionless); L = flow length (ft); P = 2-year; 24-hour rainfall depth (inches); and S = land slope (rise/run).
- For shallow concentrated flow travel time: surface description; L = flow length (ft); and S = land slope (rise/run).
- For channel flow travel time: L = flow length (feet); s = longitudinal slope (rise/run); n = Manning's roughness coefficient (dimensionless); D = flow depth (feet); W = bottom width (ft); and sideslopes (rise/run).

Manning's roughness coefficients (for sheet flow calculation referenced above) are selected based on a correlation of ground description to TR-55 surface descriptions [SCS, 1986], as shown in the following table:

Ground Description	TR-55 Surface Description	Manning's n for Sheet Flow
Unvegetated final cover system and liner system runout	Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Vegetated area, construction support area and vegetated final cover system	Grass: short grass prairie	0.15

A further description of input parameters for time of concentration computations is provided in the Data Verification section of this calculation.

Reaches

Hydrologic analyses, for estimation of peak flow rates and runoff volumes (for evaluation of SWM structures), are performed by routing the hydrographs for individual subcatchments through reaches. Reaches include channels and culverts, as described below.

Channels

For hydrologic analyses, channel parameters are input into HydroCAD™ based on information from the Construction Drawings. Channels consist of either "V" or trapezoidal-shaped sections, with either grass or riprap lining. Input parameters include: (i) bottom width (ft); (ii) sideslopes (rise/run); (iii) length (ft); (iv) longitudinal slope (rise/run); (v) Manning's roughness coefficient (dimensionless); and (vi) depth (ft). The adequacy of channels is assessed as described in a later section.

Culverts

For hydrologic analyses, culverts are input into HydroCAD™ as one or multiple 72-in. diameter pipes. Input parameters include: (i) length (ft); (ii) slope (rise/run); and (iii) Manning's roughness coefficient (dimensionless). The culvert sizes selected in the computer model (i.e., one or multiple 72-in. dia. pipes) are fictitious values used solely for ease of computation. Design of culverts (i.e., selection of size and number of pipes) is performed using the computer program CulvertMaster® which uses peak flow rates obtained from hydrologic analyses and accepted methodologies described in a later section.

Sedimentation Basin

Surface-water runoff is routed through an individual basin for the scenario noted below.

- OSDF Basin 1: routed for the OSDF Design Scenario.

Routing through the basin is performed utilizing the Storage-Indication Method coded into HydroCAD™. For the storage-indication method, the inflow runoff hydrograph at a basin and the stage-storage and stage-discharge relationships of the basin are utilized.

The stage-storage relationship of a basin is developed in HydroCAD™ using the Surface Area – Conic Volume Determination Method where cumulative storage is calculated based on areas within elevation contour lines from the outlet pipe invert elevation to the minimum embankment crest elevation..

The stage-discharge relationship of a basin is modeled in HydroCAD™ using the size and inlet elevations of the principal spillway riser and outlet pipes and the size, shape, and inlet elevation of the emergency spillway. Principal spillways consist of a pair of vertical riser and horizontal discharge pipes. Riser pipes are modeled as sharp-crested weirs in low flow conditions and as orifices in high flow conditions. Horizontal outlet pipes are modeled as culverts. The OSDF Basin 1 includes an emergency spillway, which is modeled as broad crested rectangular weir. Also, the low-flow orifices (8-in. diameter) located at the bottom of the risers are neglected (conservatively) for this analysis. Additional information concerning development of stage-discharge relationships for these control structures is included in the HydroCAD™ Technical Reference Manual provided as Attachment A-3.

HYDRAULIC ANALYSES

The design of new SWM structures and assessment of existing SWM structures includes hydraulic analyses for channels, culverts, and the basin. Peak flow rates from HydroCAD™ output reports are utilized.

Channels

The hydraulic capacity of each new channel is evaluated for conveyance of peak flow rates from the 25-year, 24-hour design storm event. In addition, analyses are performed to evaluate lining stability. For the purpose of these analyses, channels are divided into segments of relatively uniform cross-section and longitudinal slope.

Hydraulic Capacity

The adequacy for the hydraulic capacity of channels is evaluated in terms of an available freeboard for peak flow rates. The available freeboard is calculated as the difference between the minimum available flow depth in the channel and the peak flow depth. The peak flow depth is calculated based on Manning's equation as coded into a computer spreadsheet. Manning's equation is as follows:

$$Q = \frac{1.49}{n} A R^{2/3} S_0^{1/2}$$

Where, Q is the peak flow rate (cfs), n is Manning's roughness coefficient (dimensionless), A is cross-sectional area of flow (ft^2), R is hydraulic radius (ft) defined as R = A/P where P is wetted perimeter (ft), and S_o is longitudinal slope (rise/run). The minimum required freeboard (from the peak flow level to the level of overtopping of the channel sideslopes) is 0.5 ft. The minimum available flow depth is obtained from the Construction Drawings.

Channel Lining

For each new channel, a peak flow velocity is calculated and compared to the permissible flow velocity for the selected channel lining. Permissible flow velocities for grass and riprap linings are <5 fps and <12 fps, respectively. Peak flow velocities are calculated based on Manning's equation, using a computer spreadsheet.

Culverts

The hydraulic capacity of each new culvert is evaluated for conveyance of peak flow rates from the 25-year, 24-hour design storm event. New culverts are made of corrugated metal pipe (CMP).

Hydraulic Capacity

The adequacy of culverts is evaluated for inlet and outlet control conditions in terms of headwater elevation, using the computer program CulvertMaster®. An allowable headwater elevation is calculated as the elevation of overtopping of the channel sideslopes or road crossing minus 0.5 ft freeboard. The higher of the inlet or outlet headwater elevations is taken as the controlling elevation.

The following input parameters are required: (i) material type and culvert shape; (ii) inlet configuration; (iii) number of culverts and diameter (ft); (iv) length (ft); (v) inlet and outlet invert elevations (ft MSL); (vi) 25-year, 24-hour peak flow rate (cfs); (vii) tailwater elevation; and (viii) overtopping elevation. In cases where culverts discharge into well-defined channels, tailwater elevations are assumed as the elevation of the normal flow depth in the downstream channel. In cases where the downstream channels are not well-defined, a tailwater depth of 1 foot is assumed arbitrarily. Tailwater elevations for the two culverts discharging into the sedimentation basin are assumed as the peak water level in the basin for the 25-year, 24-hour storm event.

Outlet Protection

For each new culvert, riprap outlet protection is recommended using a design chart from USDA-SCS [1987], for the 25-year, 24-hour storm event. This design chart is presented in Attachment C-4B. Outlet protection is recommended in terms of downstream riprap length and average riprap particle size (d_{50}). For the purposes of designing outlet protection, the flow depth just downstream of the culvert is assumed as less than one half the diameter of the culvert (i.e., a minimum tailwater condition). Criteria for the selection of downstream riprap length and thickness are listed below.

- The downstream riprap length (at a culvert outlet) is based on the design chart, but at a minimum will be four times the culvert diameter (hereafter referred to as "minimum riprap length").
- If riprap is not required based on the design chart but the outlet velocity for the culvert is 5 fps or greater, the minimum riprap length is recommended.
- If riprap is not required based on the design chart and outlet velocity for the culvert is less than 5 fps, no outlet protection is recommended.
- If the direction of discharge (i.e., at culvert outlet) is not aligned with the longitudinal direction of the receiving channel, the minimum riprap length is recommended.
- Riprap thickness as described in DCP (GeoSyntec, 2000a).

Downstream channel peak flow depths are obtained from the results of hydrologic analyses. Outlet velocities are obtained from CulvertMaster® output reports. Where riprap is recommended for outlet protection, inlet protection is also recommended. The upstream length is two times the culvert diameter with a d_{50} equal to that for the outlet protection riprap.

Structural Stability

For each new culvert, structural stability is evaluated. For culverts crossing roads used by heavy construction vehicles (off-highway vehicle road) a Caterpillar D400E articulated truck (D400E) [Caterpillar, 1998] is selected as the design vehicle. For culverts crossing roads used only by highway vehicles (on-highway road), an HS-20 axle load is selected for design. The design axle load for off-highway vehicle roads of 95 kips is for a fully loaded truck and the combination of both rear axles (See specification for D400E in Attachment A-9). The design axle load for on-highway vehicle roads of 32 kips corresponds to an HS-20 loading [ACPA, 1998].

For CMP culverts, structural stability is evaluated using manufacturers guidelines [Contech, 1999 – See Table in Attachment A-9] for unpaved roads. In accordance with these guidelines, the minimum required cover for culvert diameters of 42 in. and less for off-highway and on-highway vehicle roads is 3.0 ft and 2.0 ft, respectively.

Sedimentation Basin

The existing basin is evaluated for conveyance of runoff from applicable design storm events. A cleanout frequency of once per year is assumed consistent with the SWMEC Plan.

OSDF Sedimentation Basin 1 (See DCP Section 2.8 in Attachment A-1)

- The minimum available storage volume (below the principal spillway inlet elevation) should exceed the calculated runoff volume from the 10-year, 24-hour design storm event and the calculated volume of 0.125 acre-ft per year per acre (disturbed upstream drainage area multiplied by the basin cleanout frequency). Total drainage area is conservatively used in place of disturbed upstream drainage area.
- The principal spillway should discharge runoff from the 25-year, 24-hour design storm event with no flow entering the emergency spillway.
- The principal and emergency spillways should discharge runoff from the 100-year, 24-hour design storm event, allowing for 1 ft of freeboard from the minimum embankment crest elevation.

REFERENCES

ACPA. *Concrete Pipe Handbook.* American Concrete Pipe Association, Irving, Texas, 1998.

Caterpillar. *Caterpillar Performance Handbook.* Caterpillar Inc., Peoria, Illinois, October 1998.

Contec. *Corrugated Steel Pipe and Pipe Arch.* Contech Construction Products, Inc., Middleton, Ohio, 1999.

U.S. Department of Transportation (USDOT). *Hydraulic Design of Highway Culvert* - Hydraulic Design Series No. 5, Federal Highway Administration, McLean, Virginia, 1985.

GeoSyntec Consultants. *OSDF Final Design Criteria Package*, United States Department of Energy, Fernald Environmental Management Project, Fernald, Ohio, prepared by GeoSyntec Consultants, 2000a.

GeoSyntec Consultants. *OSDF Final Design Package*, United States Department of Energy, Fernald Environmental Management Project, Fernald, Ohio, prepared by GeoSyntec Consultants, May 1997c.

GeoSyntec Consultants. *OSDF Surface-Water Management and Erosion Control Plan*, United States Department of Energy, Fernald Environmental Management Project, Fernald, Ohio, prepared by GeoSyntec Consultants, May 2000d.

GeoSyntec Consultants. Calculation Package On-Site Disposal Facility Phase III, United States Department of Energy, Fernald Environmental Management Project, Fernald, Ohio, prepared by GeoSyntec Consultants, May 2000.

Haestad Methods. *Users Guide, CulvertMaster® for Windows, Version 2.0.* Haestad Methods, Inc., Waterbury, Connecticut, 2000.

HydroCAD. *HydroCAD™, Stormwater Modeling System, Version 5.96.* Applied Microcomputer Systems, Chocorua, New Hampshire, 2001.

U.S. Department of Agriculture (USDA-SCS). *Urban Hydrology for Small Watersheds, Technical Release 55 (TR-55).* 2nd ed. Washington, D.C., Soil Conservation Service, 1986.

U.S. Department of Agriculture (USDA-SCS). *Technical Release Number 20 (TR-20)*, Soil Conservation Service, 1982.

U.S. Department of Agriculture (USDA-SCS). *Soil Survey of Hamilton County, Ohio,* Soil Conservation Service, in cooperation with Ohio Department of Natural Resources and Ohio Agricultural Research and Development Center, August 1992.

U.S. Department of Agriculture (USDA-SCS). *Soil Survey of Butler County, Ohio*, Soil Conservation Service, 1980.

U.S. Department of Agriculture (USDA-SCS). *Water Management and Sediment Control for Urbanized Areas*, Soil Conservation Service, Washington. D.C., 1987.

TABLE A-1

OSDF PHASE IV CONSTRUCTION ELEMENTS AND DESIGN SCENARIO

Analysis	Feature	Status	Considered in Design of Structures	Comments
OSDF DS	Cell 1, 2, 3 liner system	Existing	Yes	
OSDF DS	Cell 4, 5 liner system	New	Yes	
OSDF DS	OSDF Basin 1	Existing	Yes	Basin configuration as modified as part of Phase IV construction activities.
OSDF DS	Channels and culverts	Existing	Yes	
OSDF DS	Channels and culverts	New	Yes	
OSDF DS	EPLTS Project	Existing	Yes	To be constructed prior to Phase IV. Therefore, considered existing for the purpose of this analysis.
OSDF DS	Cell 1 final cover system (vegetated)	Existing	Yes	To be constructed prior to Phase IV. Therefore, considered existing for the purpose of this analysis.
OSDF DS	Cell 2 final cover system (unvegetated)	New	Yes	
OSDF DS	Access Control Facility Road	Existing	Yes	To be constructed prior to Phase IV. Therefore, considered existing for the purpose of this analysis.
DC A	South East Channel	New	Yes	
DC B	Emergency Access Road	New	Yes	

OSDF DS = OSDF Design Scenario
DC A = Design Case "A"
DC B = Design Case "B"

000030

4014

ATTACHMENT A-1

DCP SECTION 2.8
SURFACE-WATER MANAGEMENT

Daniel, D.E. and Benson, C.H., "Water Content-Density Criteria for Compacted Soil Liners", *ASCE Journal of Geotechnical Engineering*, 1990, Vol. 116, No. 12, 1990, pp. 1811-1830.

U.S. Environmental Protection Agency (USEPA), "*Seminar Publication - Requirements for Hazardous Waste Landfill - Design, Construction, and Closure*, EPA/625/4-89/022, U.S. Environmental Protection Agency, 1989.

U.S. Environmental Protection Agency (USEPA), "*Technical Guidance Document - Quality Assurance and Quality Control for Waste Containment Facilities*", EPA/600/R-93/182, U.S. Environmental Protection Agency, 1993.

2.8 Surface-Water Management

2.8.1 Categories of Surface Water

Surface-water management for the OSDF must consider three categories of surface water:

- ~~stormwater~~surface-water runon from outside the battery limit ~~into~~ to within the battery limit;
- ~~stormwater~~surface-water runoff, which includes all runoff from disturbed areas within the battery limit, except for wastewater explicitly identified below; and
- wastewater, which includes all waters that must be contained, collected, and conveyed to the biosurge lagoon or the FEMP former production area storm drainage control system.

Wastewater generated as a result of development of the OSDF area includes:

- leachate and runoff from impacted material within the OSDF; these wastewaters will be contained in the OSDF, allowed to percolate into the leachate collection system, and then conveyed by gravity through the leachate collection system pipe to the OSDF ~~leachate transmission system~~ EPLTS (as discussed in Section

2.5 of this DCP)~~;~~. Surface-water collected in the OSDF cell catchment area may be conveyed to the FEMP former production area storm drainage control system or other on-site wastewater collection/conveyance point acceptable to DOE and OEPA/USEPA.

- runoff from impacted-material staging areas; these are self-contained units; liquid generated in these units will be conveyed to the FEMP former production area storm drainage control system, or other on-site wastewater collection/conveyance point acceptable to DOE and OEPA/USEPA;

- runoff from impacted-material haul roads; this water will be contained, collected, and conveyed to the FEMP former production area storm drainage control system, or other on-site wastewater collection/conveyance point acceptable to DOE and OEPA/USEPA; and

- perched ground water that seeps into excavations; this water will be contained, collected, and conveyed to the FEMP former production area storm drainage control system, or other on-site wastewater collection/conveyance point acceptable to DOE and OEPA/USEPA.

The remainder of this section of the DCP presents design criteria for management of stormwaters and wastewaters.

2.8.2 General Design Criteria

The functions of the surface-water management system are to: (i) route surface water to designated locations where it can be appropriately managed; (ii) protect the OSDF from damage caused by precipitation and ~~stormwater~~surface-water runon and runoff; and (iii) discharge surface water to existing watercourses in accordance with applicable regulatory and DOE requirements.

The surface-water management system should perform in a manner that meets the project requirements for both temporary conditions (i.e., during construction, filling, and closure of the OSDF) and long-term conditions (i.e., after closure of the OSDF).

The system should prevent ~~stormwater~~surface-water runon to the OSDF and uncontrolled stormwater and wastewater runoff from the OSDF. Features of the permanent surface-water management system should be designed to require minimal monitoring and maintenance. The system should be integrated, to the extent possible, with existing topography, features, and facilities (design considerations).

2.8.3 ~~Stormwater~~ Surface-Water Management During OSDF Construction/Filling/Closure

A. Design Criteria

- Temporary surface-water control structures for the OSDF ~~shall~~will be designed for the 25-year, 24-hour storm event (ARAR: EPA 40 CFR §258.26 and OAC 3745-27-08(C)(6)(a) and (b)). For the FEMP property, this event has a rainfall intensity of 4.7 in. ~~(120 mm)~~ [Parsons, 1995a].

- Temporary surface-water control structures ~~shall~~will be designed to minimize silting and scouring (ARAR: OAC 3745-27-08(C)(6)(c)).

- Temporary runon control measures should meet the following criteria (design considerations).

  - Upgradient runon should be prevented from entering active working areas. Such runon should be diverted around work areas using berms, dikes, or channels as appropriate. This runon should not be allowed to mix with wastewater.

  - Runon to temporary excavations should be prevented using berms, ditches, or other surface-water control features.

  - Runon to impacted material stockpiles should be prevented using berms, ditches, or other surface-water control features.

- Prior to placement of impacted material into an OSDF cell, permanent runon controls must be in place. The requirements for permanent runon control are described in more detail in Section 2.8.4 of this DCP.

- Runoff from disturbed areas should be routed to the appropriate temporary sediment basin or managed using other appropriate erosion control practices. There must be no mixing of ~~stormwater~~surface-water runoff and wastewaters (functional requirements).

- Temporary sediment basins ~~shall~~will meet the following criteria of OEPA (ARAR: OAC 3745-27-08(C)(6)(d)):

  - the minimum acceptable basin storage ~~shall~~will be established as the larger of the calculated runoff volume from a 10-year, 24-hour storm event, or, 0.125 acre-ft ~~(0.015 ha-m)~~ per year (for each acre ~~(ha)~~ of upgradient disturbed area) multiplied by the scheduled frequency of basin cleanout (in years) ((6)(d)(i)); for the FEMP property, the 10-year, 24-hour storm event has a rainfall intensity of 4.1 in. ~~(103 mm)~~ [Parsons, 1995a];

  - the principal spillway ~~shall~~will be capable of safely discharging the flow from a 10-year, 24-hour storm event; the inlet elevation of the emergency spillway ~~shall~~will be designed to provide flood storage, with no flow entering the emergency spillway during a 25-year, 24-hour storm event, with allowance provided for the flow passed by the principal spillway during the event ((6)(d)(ii)); as previously noted, for the FEMP property, the 25-year, 24-hour storm event has a rainfall intensity of 4.7 in. ~~(120 mm)~~ [Parsons, 1995a];

  - the combination of principal and emergency spillways should be capable of safely discharging the flow from a 100-year, 24-hour storm event; the basin embankment design should provide for no less than 1 ft ~~(0.3 m)~~ of net freeboard when flow is at the design depth, after allowance for embankment settlement ((6)(d)(iii)); for the FEMP property, the 100-

year, 24-hour storm event has a rainfall intensity of 5.6 in. ~~(142 mm)~~ [Parsons, 1995a]; ~~and~~

- the basin ~~shall~~will be constructed using a compacted soil liner, a geomembrane, or a combination thereof ((6)(d)(iv~~).~~)); and

- sediment basins will be equipped with ring buoys and other safety/drowning equipment in accordance with USOSHA 1926.106.

- With respect to the last ARAR ((6)(d)(iv)), on 24 February 1992, the OEPA DSIWM issued the following guidance on the need for lining sediment basins:

  "The sole purpose of a liner in a sediment basin is water retention. Therefore, a design capable of ponding water, whether or not it contains a liner, will be acceptable to the Director. In areas with predominantly in-situ low permeability clay, a liner may be unnecessary (it would be wise to scarify and recompact the clay surface). The landfill engineer is responsible for meeting the "ponding" standard. In areas with more permeable soils a recompacted clay liner is necessary, but the QA/QC standards can be minimal and certainly do not need to follow the landfill liner standards."

  The foregoing requirement is interpreted as allowing the development of unlined sediment basins in the low-permeability tills underlying the FEMP. To assure compliance with the intent of this guidance, the construction specifications for sediment basins associated with the OSDF should require scarification and recompaction of the till exposed in the sediment basin excavation, and overexcavation of any observed granular soil zones, followed by backfilling with till and recompaction (design consideration).

- ~~Stormwater~~ Surface-water runoff from the FEMP watersheds ~~in the OSDF~~ to the receiving water course (e.g., Paddys Run) should be discharged at a rate no greater than the predevelopment runoff discharge rate [ODNR-~~DSWC~~, 1996] (design consideration).

- Temporary channels for stormwater runoff should be designed to meet the following criteria (design considerations).

    - Channel lining:
        - peak flow velocity in riprap-lined channels should be less than 12 ft per second ~~(3.7 m/s)~~, unless it is demonstrated that greater velocities will not cause erosion or malfunction of the surface-water management feature; and
        - peak flow velocities in grass-lined channels should be less than 5 ft per second ~~(1.5 m/s)~~.

    - Channel sideslopes should be no steeper than 3 horizontal to 1 vertical.

    - Channel bottom widths may be zero.

    - The channel freeboard should be at least 0.5 ft ~~(0.15 m)~~ under the design storm event.

    - Channels should be sloped at no less than 0.5 percent to prevent sediment buildup and clogging, unless it can be established by calculation that a lesser slope will not clog or build up sediment that will cause loss of flow capacity in the design storm event. Channel slopes should be no steeper than 5 percent unless it can be established by calculation that a steeper slope will not cause unacceptable erosion or other malfunction.

- Temporary culverts should be designed according to the following criteria (design considerations).

    - Culverts may be used in locations as needed and where cost-effective.

- Channels should be protected from erosion using riprap or erosion mats for a ~~distance~~ length of at least two culverts diameters upstream and a width of at least three culvert diameters ~~diameters downstream~~ of the culvert inlet ~~or outlet, respectively~~. The length and width, of riprap lining and average particle size downstream of the culvert outlet should meet criteria for permanent outlet protection provided in USDA-SCS, 1987.

- Minimum thickness of riprap lining will be two times D_{50}, but not less than 6 in. and will be underlain by geotextile filters.

- Riprap ~~shall~~will be designed according to the following criteria (design considerations).

  - For channel lining, riprap should be sized to meet the following criteria [ODNR, 1996]:
    - D_{50} = 62.4 pcf x d x S/4

    where: D_{50} = theoretical spherical diameter of average stone size; d = peak flow depth for the design storm event (ft); and S = channel slope (rise/run).

  - Riprap should meet the following particle size criteria [ODNR, 1996]:
    - D_{max} = 1.5 x D_{50}
    - D_{15} = 0.5 to 0.75 x D_{50}

    where: ~~D_{50} = theoretical spherical diameter of average stone size;~~ D_{max} = theoretical spherical diameter of largest stone size; and D_{15} = theoretical spherical diameter of the stone size for which 15 percent of the material is smaller.

  - For channels, the minimum thickness of the riprap lining should be two times D_{50}, but not less than 6 in. ~~(150 mm)~~.

- Riprap used at channel transitions should extend upstream and downstream of the transition a distance of five times the downstream channel depth; the minimum extension should be 15 ft ~~(4.5 m)~~.

- Geotextile filters may be used to control piping and erosion beneath riprap in temporary facilities. Granular soils should be used for filters in permanent structures containing riprap, if required to prevent undermining of the riprap.

- Rock, grade control structures should be designed according to the following criteria (design considerations).

  - Rock, grade control structures may be used in temporary facilities. They should be designed in accordance with standard design procedures.

  - The minimum height of rock, grade control structures should be 1.5 ft ~~(0.45 m)~~ and the minimum top width should be 2 ft ~~(0.6 m)~~.

- Temporary erosion control measures should include the items listed below (design considerations).

  - Runoff from all disturbed areas should be routed to sediment basins, or managed using other appropriate sediment control practices, prior to discharge to natural watercourses, except for wastewaters which should be managed as described in Section 2.8.5 of this DCP.

  - The size of any excavated or disturbed area should be as small as possible to minimize the potential for erosion (design consideration). Disturbed areas should be revegetated at the earliest possible time.

  - Temporary erosion control may be achieved using geosynthetic materials, vegetation, crusting agents, check dams, straw bales, silt fences, or other appropriate structures.

- The use of erosion control materials should be minimized in impacted soils requiring OSDF disposal. Preference should be given to runon control, surface grading, and the selective use of erosion resistant impacted materials to control erosion of impacted areas.

- Maintenance and upkeep procedures for temporary erosion control features should be specified in the *Surface-Water Management and Erosion Control Plan*.

It is noted that ~~stormwater~~ surface-water routing and ~~stormwater~~ surface-water management system design for watercourses and structures beyond the battery limit will be addressed in other design packages being prepared as part of the integrated FEMP remediation.

B. Calculations

Calculations should be performed to size the sediment basins for each contributory drainage area for each representative phase of the OSDF development. The calculations should be performed as described below.

- The amount of surface-water runon and runoff should be calculated for each contributory drainage area.

- The size of the drainage control structures (e.g., channels) should be calculated for each contributory drainage area.

- The size of the sediment basin, including outlet structures, should be calculated for each contributory drainage area.

The above calculations should be performed using the design storm events previously identified. Runon/Runoff routing and sediment basin sizing may be evaluated using the procedures described in USDA-SCS Technical Releases 20 and/or 55 [USDA-SCS, 1975, 1986a]; an acceptable tool for performing these calculations is the computer program "*HydroCAD™ Stormwater Modeling System*" [Applied

Microcomputer Systems, 1993]. The above evaluations should be based on the information and guidance contained in USDA-SCS manuals [1985, 1986b, and 1988] and ODNR-~~DSWC~~ [1996].

Culverts should be sized in accordance with U.S. Federal Highway Administration guidelines [USDOT, 1985] and meet the structural design criteria contained in applicable design references such as the Concrete Pipe Design Manual [America Concrete Pipe Association, 1970].

In the event that a channel bottom grade is less than 0.5 percent, an analysis should be performed to establish that the channel does not clog or build up sediment that will cause loss of flow capacity in the design storm event.

2.8.4 ~~Stormwater~~ Surface-Water Management After OSDF Closure

A. Design Criteria

- Permanent runon control structures for the OSDF ~~shall~~will be designed to limit interruption and damage (i.e., washout) of the OSDF in the 2,000-year, 24-hour storm event (design criterion for assumption of a DOE Performance Category 2 facility). For the FEMP property, this event has a rainfall intensity of 13.0 in. ~~(330 mm)~~ [Parsons, 1995a]. Runon should be controlled and diverted away from and around the OSDF using channels or diversion berms (design consideration).

- Permanent runoff control structures for the OSDF ~~shall~~will be designed to limit interruption and damage (i.e., washout) of the OSDF in the 2,000-year, 24-hour storm event (design criterion for assumption of a DOE Performance Category 2 facility).

- Permanent runoff control measures should be designed according to the following criteria (design considerations).

- Runoff from the 2,000-year, 24-hour storm event should be allowed to sheet flow to the toe of the OSDF final cover.

- Runoff from the toe of the OSDF final cover should either sheet flow away from the facility or to a drainage channel beyond the toe.

- Any drainage channels beyond the OSDF final cover system toe should outlet to existing drainage features at the battery limit. The location of the outlets should progress from north to south concurrent with the progressive development of the OSDF. The final outlet location for runoff from the eastern portions of the OSDF should be immediately south of the southern limit of the OSDF.

- Permanent drainage channels ~~shall~~will be designed to meet the following criteria (design considerations).

  - The dimensions of the channel should accommodate both normal low flows and peak precipitation runoff flows.

  - The final grades of the channel should be no less than 0.5 percent to prevent sediment buildup and clogging, unless it can be established by calculation that a lesser slope will not clog or buildup sediment that will cause loss of flow capacity in the design storm event. Channel slopes should be no steeper than 5 percent unless it can be established by calculation that a steeper slope will not cause unacceptable erosion or other malfunction.

  - Peak flow velocity in the channel should not initiate channel gully erosion or scour.

  - Erosion potential should be minimized at channel transitions by utilizing smooth, rounded, and graded transitions wherever possible (preferred) and erosion control structures only when needed.

- Flow velocity in the channel during high frequency (e.g., 2-year return frequency) and low-intensity (i.e., approximately 1 in. ~~(125 mm)~~ in 24 hours) storm events should be large enough to limit sedimentation in the channels, to the extent possible.

- Channel sideslopes should be no steeper than 3 horizontal to 1 vertical.

- The freeboard in the drainage channel should be at least 0.5 ft ~~(0.15 m)~~ during the design storm event.

- Permanent drop inlets and culverts may be used downgradient of the OSDF if necessary and if failure of the drop inlet and culvert would not result in damage to, or interruption of, the OSDF. Permanent drop inlets and culverts should be designed to meet the following criteria (design considerations).

  - Culverts beneath roads or access corridors where traffic is limited to highway vehicles should be designed for American Association of State Highway and Transportation Officials (AASHTO) HS-20 live loads and applicable dead loads.

  - Culverts beneath haul roads or access roads used for construction traffic should be designed for vehicle live loads and applicable dead loads.

  - Channels should be protected from erosion using riprap for a length of at least two culvert diameters upstream and a width of at least three culvert diameters upstream and downstream. ~~and four culvert diameters downstream~~ of the culvert inlet ~~or outlet, respectively.~~ The length and thickness of riprap lining and average particle size downstream of the culvert outlet should meet criteria for outlet protection provided in ODNR.

  - Permanent culverts should not be used upgradient of the OSDF.

- Riprap, if needed, should be designed as described in Section 2.8.3 of this DCP (design consideration).

- Riprap should consist of field stone or rough unhewn quarry stone of approximately rectangular shape. The stone should be hard and angular and of a good quality, consistent with the UMTRA Technical Approach Document [DOE, 1989] (design consideration).

- Granular soils should be used as filters and bedding for permanent riprap features where necessary to prevent undermining of the riprap (design consideration).

- Rock grade control structures, if used, should be designed to meet the criteria listed in Section 2.8.3 of this DCP (design consideration).

- Stormwater runoff from watersheds in the FEMP to the receiving water course (e.g., Paddys Run) should be discharged at a rate no greater than the predevelopment runoff discharge rate [ODNR-~~DSWC~~, 1996] (design consideration).

It is noted that stormwater routing and stormwater management system design for watercourses and structures beyond the battery limit will be addressed in other design packages being prepared as part of the integrated FEMP remediation.

B. Calculations

Calculations should be performed to size the drainage channels for each contributory drainage area. For these areas where a permanent drainage channel is not needed, the amount of surface-water runoff should be calculated. The calculations that should be performed are described below.

- The amount of surface-water runon and runoff should be calculated for each contributory drainage area.

- The size of the permanent drainage channel should be calculated for each contributory drainage area.

The above calculations should be performed using the design storm events previously identified. Runon/runoff routing and sediment basin sizing may be evaluated using the procedures described in USDA-SCS Technical Releases 20 and 55 [USDA-SCS, 1975, 1986a]; an acceptable tool for performing these calculation is the computer program "*HydroCAD*^{TM} *Stormwater Modeling System*" [Applied Microcomputer System, 1993]. The above evaluations should be based on information and guidance contained in USDA-SCS manuals [1985,1986b,1988].

In the event that a channel bottom grade is less than 0.5 percent, an analysis should be performed to establish that the channel does not clog or build up sediment that will cause loss of flow capacity in the design storm event.

The erosion resistance of the permanent drainage channel at the north and east toes of the OSDF should be evaluated as follows:

- obtain the allowable tractive force on the channel vegetation and topsoil using methods established by Temple et al. [1987], as described in the DOE Technical Approach Document [1989] and referenced documents;

- establish the actual tractive force on the channel vegetation and the "effective" tractive force on the channel topsoil using methods established by Temple et al. [1987], as described in the DOE Technical Approach Document [1989] and referenced documents;

- determine the potential for erosion of the drainage channel by comparing the allowable tractive force on the topsoil to the "effective" actual tractive force on the topsoil; and

- evaluate the potential for the riprap portion of the channel lining to erode using the Safety Factors Method as described in the DOE Technical Approach Document [1989] and referenced documents.

2.8.5 Wastewater Management

Wastewaters that will be encountered in development of the OSDF were identified in Section 2.8.1 of this DCP. These wastewaters should be managed as follows.

- *Leachate* - ~~All precipitation or other water that falls or runs into an active OSDF cell will be considered leachate and allowed to percolate into the cell leachate collection system~~ Liquid that has percolated through, or been released from, the impacted material that has been disposed in the OSDF (functional requirement). Placement of impacted material in OSDF cells ~~shall~~will be performed such that runoff from active and open portions of a cell resulting from the 25-year, 24-hour storm event can be managed within the cell (ARAR: EPA 40 CFR §258.26 and OAC 3745-27-08(C)(6)(a)). Leachate should be managed as described in Section 2.5 of this DCP.

- *Impacted Runoff* – Precipitation that comes in contact with impacted material and runs off rather than percolating. Impacted runoff collected in the cell catchment areas may be conveyed as described in this Section. ~~Impacted material staging areas for demolition debris and soils~~ An OMTA will be constructed for the staging of impacted material for subsequent disposal in the OSDF. To the extent possible, the OMTA should be located within the former production area ~~whenever possible~~. Runoff from these areas should drain to stormwater control structures within the former production area storm drainage control system (design consideration). Runoff from any staging area located within the OSDF battery limit should also be directed to the FEMP former production area storm drainage control system if possible, or to other on-site wastewater collection/conveyance points (if necessary) acceptable to DOE and OEPA/USEPA (design consideration). Additional discussion of the ~~impacted material staging area~~ OMTA is presented in Section 2.11 of this DCP. Runoff from impacted material ~~access~~ haul roads should be contained within the haul road boundary and allowed to flow by gravity to the FEMP former production area storm drainage control system, or to other on-site wastewater collection/conveyance points (if necessary) acceptable to DOE and

USEPA/OEPA. Drainage control structure for impacted material haul roads should be designed for the 25-year, 24-hour storm event. (design consideration).

- *Perched Ground Water* - Perched ground water that enters the OSDF excavation should be collected in a toe drain, or other suitable sump, and pumped to the FEMP former production area storm drainage control system (including pumpage to the impacted-material haul road, where the water will be allowed to flow by gravity to the FEMP former production area storm drainage control system), or to other on-site wastewater collection/conveyance points (if necessary) acceptable to DOE and OEPA/USEPA (design consideration). The management of perched ground water that enters the borrow area excavation is not wastewater; management of this latter runoff is discussed in Section 2.10 of this DCP.

2.8.6 References

FEMP property data and information required to design the surface-water management system should be obtained from the references cited in Section 1.5 of this DCP. References from the general technical literature that may be used to design these systems are given below.

American Concrete Pipe Association, "*Concrete Pipe Design Manual*", American Concrete Pipe Association, Arlington, VA, February, 1970.

Applied Microcomputer Systems, "*HydroCAD™ Stormwater Modeling System*", Version 3.10, Chocorua, NH, 1993.

Chow, V.T., "*Open-Channel Hydraulics*", McGraw-Hill, Inc., 1959.

Department of Labor, OSHA Construction Standard, 29 CFR 1926.106 "*Working Over or Near Water.*"

Fernald Environmental Management Project, "*Fluor Daniel Fernald Safety Performance Requirements Manual*", RM-0021.

Ohio Department of Natural Resources - Division of Soil and Water Conservation (~~ODNR-DSWC~~ODNR), "*Rainwater and Land Development*", 2nd Edition, 1996.

Parsons, "*2,000-Year Flood and Probable Maximum Flood Sitewide Flood Plain Determination*", CERCLA/RCRA Unit 2, Project Order 148, Fernald Environmental Management Project, Rev. A, Fairfield, OH, August 1995a.

Richardson, E.V., et al., "*Highways in the River Environment - Hydraulic and Environmental Design Considerations*, U.S. Department of Transportation, Available from Publications Office, Engineering Research Center, Colorado State University, Fort Collins, CO, 1975.

Temple, D.M., Robinson, K.M., Ahring, R.M., and Davis, A.G., *Stability Design of Grass-Lined Open Channels*", U.S. Department of Agriculture, Agriculture Research Service, Agriculture Handbook Number 667, 1987.

U.S. Department of Agriculture-Soil Conservation Service (USDA-SCS), "*Computer Program for Project Formulation, Hydrology*", Technical Release 20 (TR20), U.S. Department of Agriculture, Soil Conservation Service, Washington, D.C., 1975.

U.S. Department of Agriculture-Soil Conservation Service (USDA-SCS), "*National Emergency Handbook, Section 4 - Hydrology*", U.S. Department of Agriculture, Soil Conservation Service, Washington, D.C., 1985.

U.S. Department of Agriculture-Soil Conservation Service (USDA-SCS), "*Urban Hydrology for Small Watersheds*", Technical Release 55 (TR55), U.S. Department of Agriculture, Soil Conservation Service, Washington, D.C., 2nd Edition, 1986a.

U.S. Department of Agriculture-Soil Conservation Service (USDA-SCS), *Engineering Field Manual for Conservation Practices*, U.S. Department of Agriculture, Soil Conservation Service, Washington, D.C., 1986b.

U.S. Department of Agriculture-Soil Conservation Service (USDA-SCS), "*Water Management and Sediment Control for Urbanizing Areas*", U.S. Department of Agriculture, Soil Conservation Service, Washington, D.C., 1987.

U.S. Department of Agriculture-Soil Conservation Service (USDA-SCS), *Ponds - Planning, Design Construction*, Agricultural Handbook Number 590, U.S. Department of Agriculture, Soil Conservation Service, Washington, D.C., November 1988.

U.S. Department of Transportation (USDOT), "*Hydraulic Design of Highway Culverts*", Hydraulic Design Series No. 5, Federal Highway Administration, McLean, VA, September 1985.

U.S. Department of Energy (DOE), "*Technical Approach Document, Revision II*", Uranium Mill Tailings Remedial Action Project, December 1989.

2.9 Support Facilities and Utilities

2.9.1 General Design Criteria

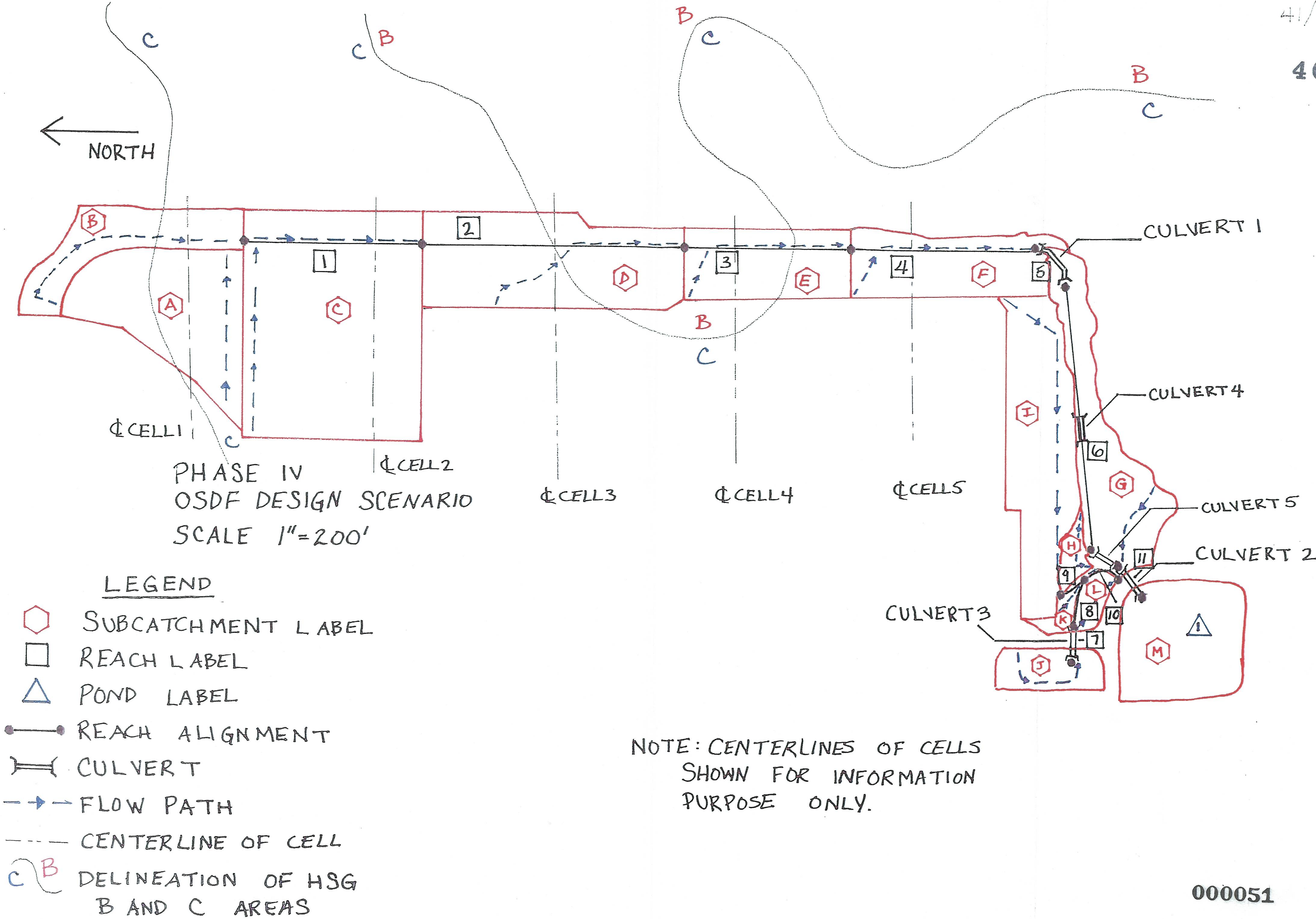
The function of support elements and utilities is to provide support for and enhance the performance of, the OSDF during construction, filling, closure and post-closure care. As identified in Section 1.3 of this DCP, the support elements will include survey benchmarks, construction support area, equipment ~~decontamination~~ wash facility, materials storage areas, access control features, construction haul roads and leachate transmission system access corridor. Utilities will include electricity, water, and wastewater systems. Design criteria are presented separately in this section for each of these elements.

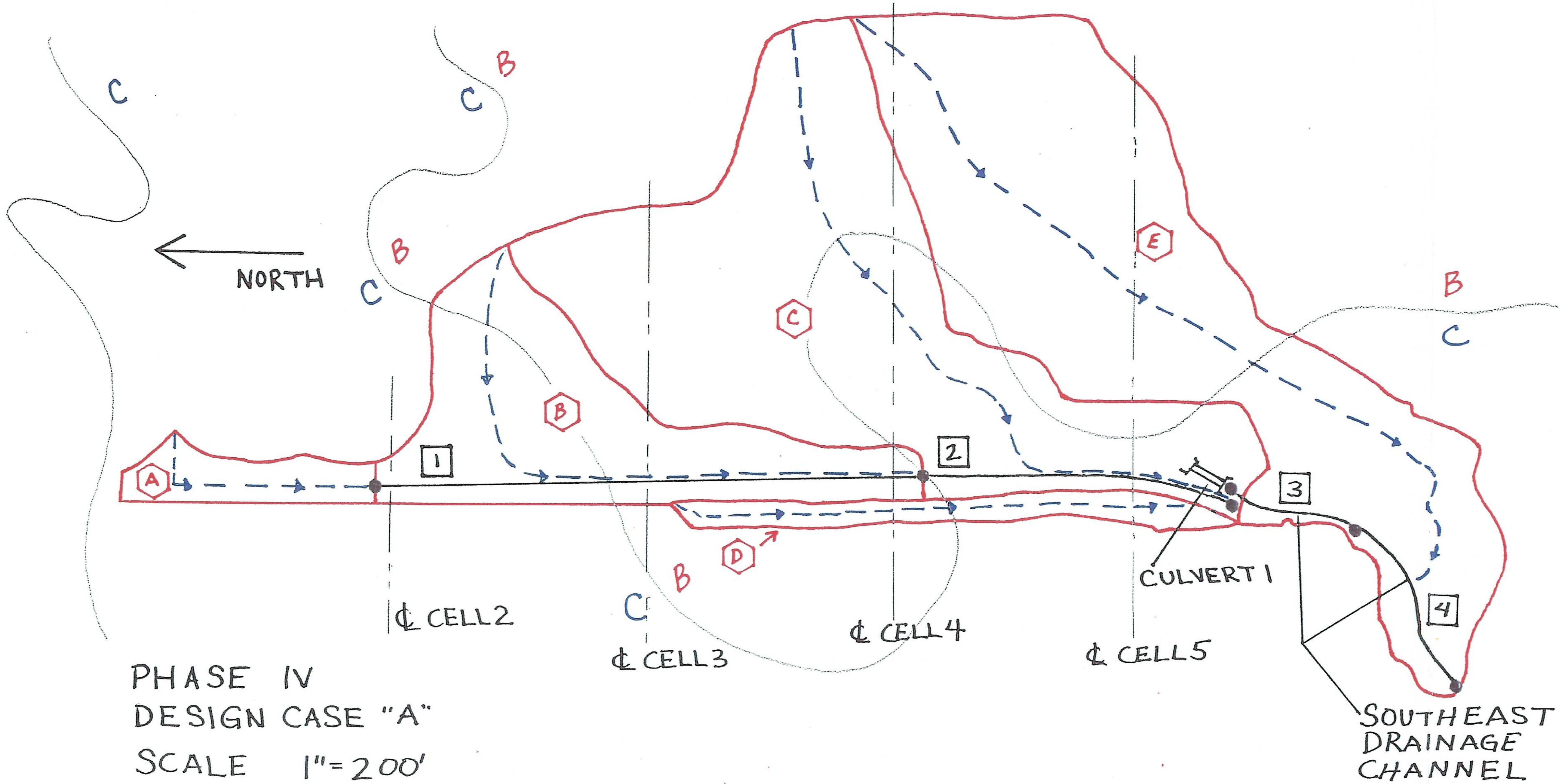
The support elements must provide adequate and reliable support for the activities that will be performed for the OSDF. Utilities must provide reliable service to the support elements for each type of utility. The support elements and utilities should be developed in a manner that is consistent with the requirements of applicable utility codes at the FEMP and with applicable health and safety requirements for the FEMP.

ATTACHMENT A-2

LAYOUT OF DESIGN SCENARIO AND DESIGN CASE SWM SYSTEMS

4014


NORTH
C
C B
B C
B C
B C
B C
CULVERT 1
CULVERT 4
CULVERT 5
CULVERT 2
CULVERT 3
1
2
3
4
5
6
7
8
9
10
11
A
B
C
D
E
F
G
H
I
J
K
L
M
1
℄ CELL 1
℄ CELL 2
℄ CELL 3
℄ CELL 4
℄ CELL 5
PHASE IV
OSDF DESIGN SCENARIO
SCALE 1"=200'
LEGEND
SUBCATCHMENT LABEL
REACH LABEL
POND LABEL
REACH ALIGNMENT
CULVERT
FLOW PATH
CENTERLINE OF CELL
C B DELINEATION OF HSG
B AND C AREAS
NOTE: CENTERLINES OF CELLS
SHOWN FOR INFORMATION
PURPOSE ONLY.

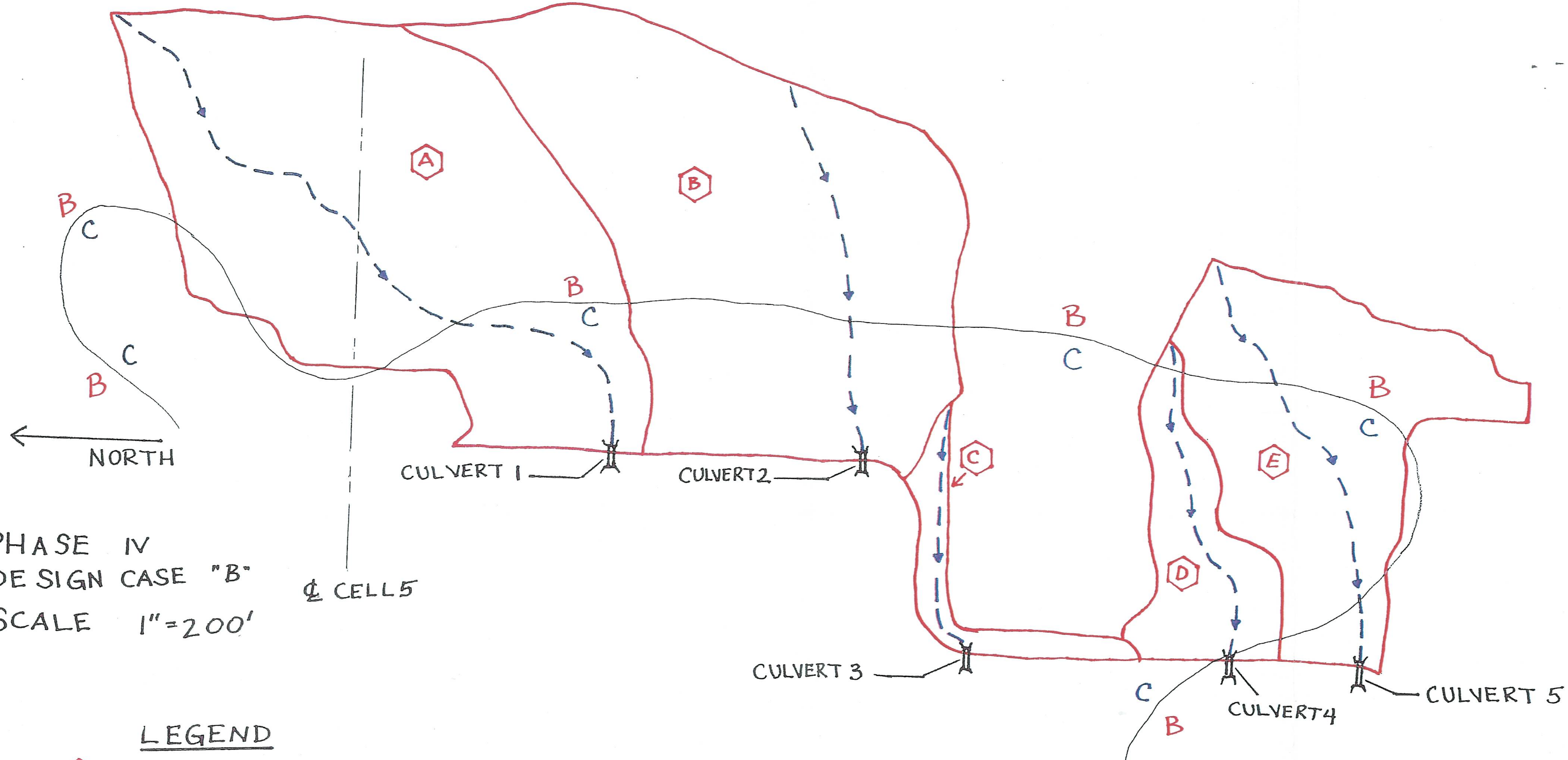


PHASE IV
DESIGN CASE "A"
SCALE 1"=200'

LEGEND

SUBCATCHMENT LABEL
REACH LABEL
REACH ALIGNMENT
FLOW PATH
CENTERLINE OF CELL
DELINEATION OF HSG B AND C AREAS
CULVERT

NOTE: CENTERLINES OF CELLS SHOWN FOR INFORMATION PURPOSE ONLY.



PHASE IV
DESIGN CASE "B"
SCALE 1"=200'

LEGEND

SUBCATCHMENT LABEL
FLOW PATH
CULVERT
CENTERLINE OF CELL
B / C DELINEATION OF HSG B AND C AREAS

NOTE: CENTERLINE OF CELL 5 SHOWN FOR INFORMATION PURPOSE ONLY.

ATTACHMENT A-3

HydroCAD™ TECHNICAL REFERENCE MANUAL

4014

-15/204

HydroCAD Technical Reference

This section contains detailed information on the calculations performed by HydroCAD. It is intended to help HydroCAD users and reviewers fully understand the specific techniques and formulas employed by the program.

Section 21 - Introduction

Understanding HydroCAD

HydroCAD is a collection of techniques for the generation and routing of hydrographs. It also provides many other related calculations, such as time of concentration, weighted curve numbers, pond volumes, stage-discharge curves, etc. This broad range of capabilities allows a large number of studies to be performed entirely within HydroCAD.

HydroCAD is a *hydrograph routing model*. It is designed specifically to handle *time varying flows*, as required for pond design and other volume-sensitive calculations. As such, HydroCAD routes completely through *one node at a time*. Only after determining the outflow hydrograph from a given node does it consider the next node downstream.

Certain calculations, such as channel backwater or pressurized pipe networks, are normally analyzed under *constant flow* conditions. These tasks may be best addressed by steady-state numerical techniques, and not by a hydrograph routing system such as HydroCAD. Some projects may require the use of HydroCAD to model the overall drainage system, combined with a steady-state analysis of specific components. This is an unavoidable consequence of the different methodologies.

Purpose of this section

This Technical Reference describes the exact calculations performed by HydroCAD. It is intended to provide the engineer with insight into these techniques so that their application and limitations can be better understood. This section also provides the information necessary for independent verification of HydroCAD's results.

Where additional information is needed, the reader is urged to consult one of the references listed on page 133. In this manual, specific references are made to the appropriate sources. These references are in the form [3 p.12], meaning reference number 3, page 12.

This Technical Reference does *not* contain operating instructions for HydroCAD, or information on the routing diagram or other operational features. For this information, please see the User's Guide, which begins on page 41.

If you want a more general overview of hydrologic techniques, please see the Introduction to HydroCAD beginning on page 1.

Section 22 - Determining the Time of Concentration

One of the key elements required for any runoff calculation is the *Time of Concentration*, or T_c. The T_c is commonly defined as the time required for runoff to travel from the most hydrologically distant point of the watershed to the point of collection.

The time of concentration is commonly determined by summing the travel time (T_t) for each consecutive *flow segment* along the subcatchment's hydraulic path. This process requires identification of the type of flow occurring in each segment, and application of the appropriate method for calculating the T_t. Although these segments will occur in a given physical order, the order in which they are used in the program has no effect on the total travel time.

HydroCAD provides a variety of techniques for calculating the T_t, plus other procedures (such as the Curve Number method) which are designed to directly determine the overall T_c. These procedures are discussed below. If necessary, the T_c or T_t may also be determined by other procedures and entered into HydroCAD directly.

The determination of the time of concentration is one of the most widely discussed areas of hydrology. The actual method(s) used on any given project depends upon actual site conditions, regulatory requirements, and sound engineering judgement.

Curve Number Method

The Curve Number Method [10 p.15-7] was developed to allow calculation of the overall T_c under a wide range of conditions. The method is designed for areas of 2000 acres or less. The calculation is quite simple, but requires a proper understanding of the input requirements:

$$T_c = \frac{L}{.6} \quad \text{where} \quad L = \frac{l^{.8}\,(S+1)^{.7}}{1900\,Y^{.5}} \quad \text{and} \quad S = \frac{1000}{CN} - 10 \qquad \text{Eq. 7}$$

T_c=Time of concentration [hours]
L=Lag time [hours]
l=Hydraulic length of the watershed [feet]
Y=Average land slope [percent]
S=Potential maximum retention [inches]
CN=Weighted Curve Number (See tables on page 137)

Note the use of the average *land* slope, and not the slope of the hydraulic path. Determining this accurately requires placing a grid over the subcatchment and averaging the slopes for all squares. Although some care is required to determine this value, the Curve Number method has the advantage of using a small number of fairly objective parameters. This provides more consistent results than some other approaches.

000055

4014

TR-55 Sheet Flow Procedure

The TR-55 Sheet Flow procedure [11 p.3-3] is designed for flow over plane surfaces, as usually occurs in the headwaters of a stream. The following equation is used for sheet flow:

$$T_t = \frac{0.007(nL)^{.8}}{P_2^{.5}\, s^{.4}} \qquad \text{Eq. 8}$$

T_t=Travel time [hours]
n=Manning's coefficient for sheet flow (See page 156)
L=Flow length [feet]
P_2=2-year, 24-hour rainfall [inches] (See map on page 146)
s=Land slope (along flow path) [ft/ft]

Determining the actual length of sheet flow is critical to this method. Although the technique was intended for lengths up to 300 feet, some agencies now recommend a maximum of 100 feet. In any case, the length should not extend past the point where there is evidence of concentrated flow on the ground. The length is also critical in that Sheet Flow is often a dominant factor in a subcatchment's total Tc.

Upland Method

The Upland Method [10 p.15-6] is designed for conditions that occur in the headwaters of a watershed, including overland flow, grassed waterways, paved areas, and through small upland gullies. Upland method is applicable to areas of 2000 acres or less. Although commonly published as a chart of velocity vs. slope for various surfaces, upland method is based on the following equation:

$$T_t = \frac{L}{3600\,V} \quad where \quad V = K_v \sqrt{s} \qquad \text{Eq. 9}$$

T_t=Travel time [hours]
L=Flow length [feet]
V=Average velocity [FPS]
K_v=Velocity factor (See table on page 157)
s=Land slope (along flow path) [ft/ft]

Shallow Concentrated Flow

The TR-55 Shallow Concentrated Flow procedure [11 p.3-3] is of the same mathematical form as the Upland Method (above). The essential difference is that it utilizes only two surface types, *paved* and *unpaved*. (See page 157 for the corresponding K_v values for these surfaces.) Due to the similarity between Upland Method and Shallow Concentrated Flow, HydroCAD utilizes a single screen for both methods, and combines the K_v values from both methods into a single table.

Channel Flow

+? 20

The Channel Flow procedure [11 p.3-3] is commonly employed where surveyed cross sections ar[e] available, or anywhere the velocity can be reasonably determined by Manning's equation.

$$T_t = \frac{L}{3600\,V} \quad where \quad V = \frac{1.486\, r^{2/3} s^{1/2}}{n} \quad and \quad r = \frac{a}{P_w} \qquad \text{Eq. 10}$$

T_t=Travel time [hours]
L=Flow length [feet]
V=Average velocity [fps]
n=Manning's coefficient (See table on page 152)
s=Channel slope [ft/ft]
r=Hydraulic radius [feet]
a=Cross sectional flow area [sq-feet]
P_w=Wetted perimeter [feet]

In addition to allowing direct entry of cross sectional area and wetted perimeter, HydroCAD can automatically calculate these parameters for rectangular, vee, trapezoidal, parabolic, and circular channels. See page 128 for details.

Other Tc Procedures

Other T_t procedures can be employed by entering the calculated value directly into HydroCAD. This can be used as the total T_c for a subcatchment, or combined with additional flow segments calculated by other means. One situation that calls for direct T_c entry is modeling the "runoff" on the surface of a pond. This requires the direct entry of a T_c of zero.[39]

[39] When used in combination with a Curve Number of 100, this will produce complete, instantaneous "runoff."

Section 23 - SCS Unit Hydrograph Procedure

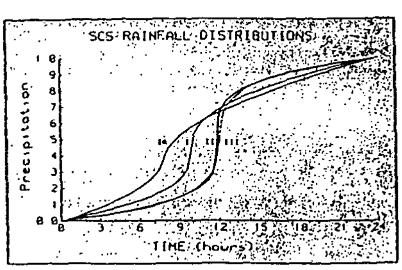
The USDA Soil Conservation Service[40] has developed a number of techniques for analyzing stormwater runoff. One of the most widely used is the SCS Unit Hydrograph procedure (SCSUH). The SCSUH procedure is a principal component of SCS Technical Release 20, commonly known as SCS TR-20 [2].

The SCSUH procedure is the primary runoff technique provided by HydroCAD. Although HydroCAD does not employ any of the actual code from TR-20, it is based on the same SCSUH procedure and will produce essentially the same runoff results.

Due to the computational requirements of TR-20, the SCS produced a simplified, derivative tool in the form of TR-55 [11] Tabular method. The Tabular method is designed to approximate the results that would be obtained from TR-20, but uses various approximations that cause reduced accuracy in many situations. Because of this relationship, TR-20 will always provide equal or greater runoff accuracy, and is therefore preferable for most projects that call for the TR-55 Tabular method. (See page 13 for a detailed comparison of these techniques.)

Synthetic Rainfall Distributions

The SCS unit hydrograph procedure commonly utilizes a *synthetic rainfall distribution*. The SCS has developed several standard distributions that cover the entire U.S. These are commonly expressed as *mass curves*, as shown at right and discussed on page 142. When combined with national rainfall maps (starting on page 146), these distributions eliminate the need for local "IDF" curves as employed by the Rational method. In fact, the rainfall distributions contain the same information in a reduced form.



While the unit hydrograph procedure may initially appear more complex than the Rational method, it is actually easier to apply with a system such as HydroCAD. This is because the synthetic rainfall distributions encompass *all duration events* in a single calculation, while the Rational method requires a separate calculation for each duration.[41]

[Cu]stom rainfall distributions may also be used with the unit hydrograph procedure, but this is [r]arely necessary. It is not uncommon to confuse a local IDF curve with a rainfall distribution. An IDF curve is not only inapplicable to the unit hydrograph procedure, but it usually duplicates the data already contained in one of the rainfall distributions.

[40] Now the Natural Resources Conservation Service or NRCS.

[41] The SCS technique also provides accurate *volume* information for volume-sensitive studies such as ponds. In comparison, the Rational method is intended primarily for determining *peak* flows, and does not provide accurate volume information.

Data Requirements for SCSUH Procedure

The following data is required for the SCS unit hydrograph procedure as employed in both TR-20 and HydroCAD. Some of these items are provided for each individual subcatchment, while others apply to the entire watershed.

> The **curve number** (CN) characterizes the type of soil and ground cover. A high CN (such as 98 for pavement) indicates minimum retention, while a low CN (such as 30 for certain wooded areas) indicates a large retention capability. A detailed table of curve numbers begins on page 137. See [10] for a detailed explanation of CN selection. In the case of a subcatchment composed of more than one CN, HydroCAD calculates a composite CN by summing the products of each CN multiplied by its percentage of the total area.

> The **total storm rainfall** (in inches) is determined for any specific location from U.S. Weather Bureau maps based on the desired return period (2, 5, 10, 25, 50, or 100 years). This is the total precipitation that will occur during the storm. Rainfall maps for the United States begin on page 146, and are taken from [11].

> The **storm type** is selected according to the geographic location and any special project requirements. For each storm, HydroCAD contains a mass curve indicating how the rainfall will be distributed over the total duration of the storm.

HydroCAD provides an extensive library of rainfall distributions, including the SCS 24-hour type I, IA, II, and III. (See page 142 for details.) The SCS rainfall data is in the form of second-order fitted equations, which guarantee a smooth runoff hydrograph free of "steps" or other irregularities.

The actual data for these storms (and others) is contained in the RAINFALL.TXT file which is installed with HydroCAD. The file also contains other rainfall distributions for Florida, Illinois, etc. Other storms may be added by editing this file. See the instructions in the file for details.

> Each storm type includes a default rainfall duration, such as "24-HOUR." Most studies utilize this standard duration. If another duration is required, the duration value may be edited, and HydroCAD rescales the rainfall to match the desired value. For example, a HUFF 6-HOUR storm (used in Illinois) can be automatically rescaled to 9 hours by specifying HUFF 9-HOUR. This is most useful for localized rainfall distributions (such as the Illinois Huff Distributions) that use the same distribution for many durations.

> The **unit hydrograph** is a dimensionless curve that shows the runoff distribution resulting from one inch of precipitation excess occurring uniformly over the watershed during a specified duration [1 p.47]. HydroCAD provides the standard SCS unit hydrograph [2 p.240], plus others, in the UNITHYDR.TXT file. Additional unit hydrographs may be added by editing this file. See the instructions in the file for details.

000056

SCSUH Runoff Generation

The runoff hydrograph is generated by performing a *convolution of the unit hydrograph with the rainfall excess* as described in [1 p.47-53] and [10 p.16-2]. A brief description of the HydroCAD implementation follows:

1) At any time during the storm, the cumulative precipitation (rainfall depth) can be determined from the selected rainfall distribution when multiplied by the total rainfall depth. The cumulative *precipitation excess* (runoff) can then be determined by the SCS runoff equation [11 p.2-1] & [10 p.10-5]:

$$Q = \frac{(P-.2S)^2}{P+.8S} \quad (Q=0 \text{ if } P<.2S) \qquad \text{Eq. 11}$$

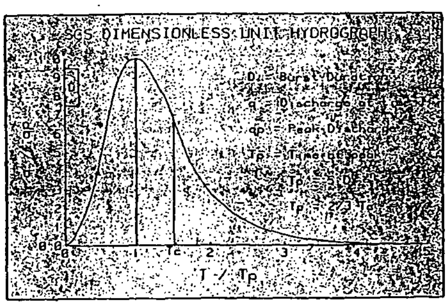
$$\text{where: } S = \frac{1000}{CN} - 10$$

Q=Precipitation excess (runoff) [inches]
P=Cumulative precipitation [inches]
S=Potential maximum retention [inches]
CN=Curve number

2) The storm is divided into a series of rainfall *bursts* of duration D=2/15 Tc. The precipitation excess occurring *during* any interval D at time t can be calculated by:

$$dQ = Q_t - Q_{t-D} \qquad \text{Eq. 12}$$

3) For each burst, the unit hydrograph defines how this volume of runoff will occur over time. The *volume* of the unit hydrograph is given by dQ (above). The *duration* of the unit hydrograph is related to Tc and D as shown. The result is that the *dimensionless* unit hydrograph has been dimensioned.



4) The runoff from the entire storm is determined by summing the hydrographs resulting from each rainfall burst.

* q_p = computed in TR 70 based on a peak rate

SCSUH Runoff Considerations

1) The runoff hydrograph consists of a series of ordinates (CFS flows) at evenly spaced interva "dt." Each ordinate specifies the *average* flow during the interval. As a result, if a narrow pea occurred within one interval, the hydrograph would indicate an average flow that *might* l significantly less than the instantaneous peak. This is likely to occur when T_c is less than 2d so HydroCAD displays an informative warning in these cases.

When you encounter this situation, keep in mind that the instantaneous peak can exceed th average for a time no longer than dt, which is commonly 6 minutes. In practice, such a shor instantaneous peak is usually attenuated to the average value by the storage characteristics c the first reach or pond. However, if a true instantaneous peak is required, the runoff interval (dt may be reduced to approximately one-half the T_c.

2) TR-20 has no inherent limitations on the time of concentration. As T_c approaches 0, the runoi curve approaches the precipitation excess curve, which is the expected limiting case.[42] Similarly for a very large T_c, the entire storm becomes a single rainfall "burst" and the runoff approache the shape of the unit hydrograph.

3) When making comparisons to TR-55, note that the TR-55 tables were produced for a curve number of 75 and require a precipitation excess of at least 1.5 inches. As conditions deviate fror these, an increasing difference of up to 25% can be expected.

4) Runoff hydrographs are generated for a specified *time span*, with a default setting of 10-20 hours. You must ensure that this span is suitable for the purposes of your analysis and the rainfall type being used. If you are primarily concerned with peak flows, you can reduce calculation time by using a shorter time span. However, for ponds and other volume-sensitive studies, make sure the time span begins at or before the earliest runoff, or this early volume won't be included in your calculations. HydroCAD will generate an automatic warning message if the span is not adequate to include the earliest inflow into a pond. Also keep in mind that the volumes displayed by HydroCAD include only the specified time span. By increasing the ending time to 25 hours or so, you'll get a complete picture of the storm. (See page 87 for a further discussion of dt and time span selection.)

5) As a safeguard, HydroCAD performs an automatic check of runoff peaks in relation to the time span. A warning message is displayed if the calculated time of the peak doesn't fall within the middle 90% of the time span. If this warning appears, you should examine the hydrograph and adjust the time span accordingly.[43]

• Although HydroCAD applies a number of tests to check the accuracy of your model, a visual examination of all hydrographs is *highly recommended*. This will help to detect erroneous input data and ensure meaningful results.

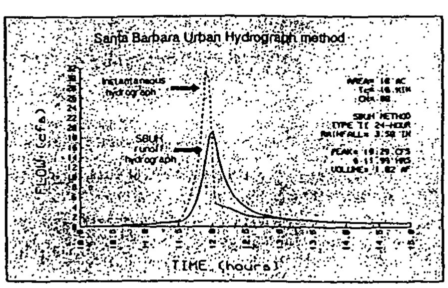
[42] If desired, the HydroCAD runoff procedure is specially designed to permit a Tc of zero. This can be used to model the instantaneous "runoff" from rain falling on the surface of a pond.

[43] This test is for the peak only. If the entire volume is required, you must still determine if the span is sufficient.

Section 24 - Santa Barbara Urban Hydrograph

The Santa Barbara Urban Hydrograph method (SBUH) was developed by the Santa Barbara County (California) Flood Control and Water Conservation District. The SBUH method has many similarities to the SCS Unit Hydrograph procedure discussed in the previous chapter. Both techniques employ the same SCS curve numbers, runoff equation, and rainfall distributions. However, the SBUH method does not utilize a unit hydrograph or the convolution process. Instead, an instantaneous hydrograph is generated and then routed through an imaginary reservoir with a time delay equal to the subcatchment's time of concentration.

This calculation is relatively simple in comparison to the SCSUH procedure, and takes less time to perform. While the availability of the SCSUH procedure might appear to eliminate the need for the SBUH method, some localities prefer the SBUH method for specific situations.



Runoff Calculations

There are two distinct steps involved in generating a runoff hydrograph by the SBUH method:

1) **Compute the instantaneous hydrograph:** The storm is divided into equal time increments (dt). At each increment, the SCS Runoff Equation (see page 107) is used to determine the precipitation excess. The difference between the successive values represents the instantaneous runoff at that point in time. A typical instantaneous hydrograph is represented by the dashed line in the above graph.

2) **Compute the runoff hydrograph:** The runoff hydrograph is obtained by routing the instantaneous hydrograph through an imaginary reservoir with a time delay equal to the time of concentration. The following equation is used to estimate the routed flow at each point in time:

$$Q_t = Q_{t-dt} + w(I_{t-dt} + I_t - 2Q_{t-dt}) \qquad \text{Eq. 13}$$

$$\text{where } w = \frac{dt}{2T_c + dt}$$

Q_t=Runoff at time t [CFS]
I_t=Instantaneous runoff at time t [CFS]
dt=Calculation time increment [minutes]
T_c=Time of concentration [minutes]
w=Routing Coefficient

A typical runoff hydrograph is shown by a solid line in the graph above. Note the delay and reduction in the peak caused by the routing procedure.

Section 25 - Rational Method

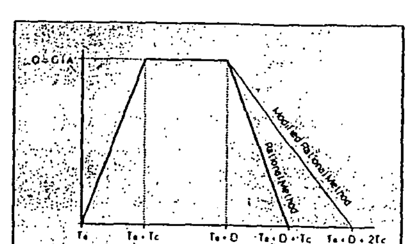
The Rational method or *Modified* Rational method may be used to generate runoff hydrographs. However, since Rational method was developed primarily for predicting peak flow, its use is *not* advised for volume-sensitive routing calculations. The Rational method predicts the peak runoff according to the formula:

$$Q = CIA \qquad \text{Eq. 14}$$

Q=Peak Runoff [CFS]
C=Runoff Coefficient
i=Rainfall intensity [in/hr]
A=Area [acres]

In order to generate a complete hydrograph (as required by HydroCAD), it is assumed that the runoff begins at the start of the storm and increases linearly to the peak value at time T_c. The peak runoff is sustained until the storm duration (D) has elapsed, and then decreases linearly to zero over the interval. When using the *Modified* Rational method, the flow decreases at half the rate, over the interval $2T_c$.

The runoff always begins at the starting time given in the Calculate screen. This may be any value, although 0 hours is traditionally used.



If C is not specified, a warning message is issued and the approximation C=CN/100 is used. This estimate is implemented only for the purpose of performing a *preliminary analysis* and must be verified to produce reliable results.

The user must chose the *critical duration* (and the corresponding intensity) that results in the maximum combined runoff at *each point of study*. Depending on the specific watershed, this may occur at any duration between the shortest and longest T_c. As the study progresses downstream, the critical duration generally increases, but some trial and error is required for an accurate determination. Note that as the duration is changed, all upstream subcatchments are properly recalculated for the new value. This is the correct procedure for applying the Rational method, despite frequent misuse of the method in which these values are held constant.

Since a hydrograph produced by the Rational method does not reflect the total storm runoff or the variation in intensity, it is not recommended for the design and analysis of detention ponds. It is strongly advised that the SCSUH or SBUH runoff methodology be used when pond routing calculations will be performed.

Section 26 - Reach Routing Calculations

A reach is used to perform an *independent hydrograph routing* for an open channel or a pipe flowing under open-channel conditions. A channel or pipe can alternatively be modeled as a flow segment within a subcatchment, where its travel time will contribute to the Tc. The later approach is usually simpler, and may even be necessary in the case of a subcatchment that is draining along the entire length of the reach. However, for a long reach with a significant inflow at one end, a separate reach routing may be called for. This section details the procedures used to perform an independent reach routing.

The Routing Curves

Reach routing requires that the reach first be characterized by two curves: the end-area vs. depth, and the discharge vs. depth (stage-discharge). This information may be determined by one of three procedures:

1) The user may directly specify the end-area and discharge at up to 15 different depths.

2) The user may enter the wetted perimeter (instead of the discharge), and Manning's equation is used to determine the discharge as follows: [3 p.77]

$$V=\frac{1.486\,R^{\frac{2}{3}}S^{\frac{1}{2}}}{n} \qquad \text{Eq. 15}$$

V=Average velocity of flow [feet/sec]
R=Hydraulic radius [feet]
S=Slope of channel bottom [rise/run]
n=Manning's number (See table on page 152)

$$\text{also} \quad R=\frac{A}{P} \quad \text{and} \quad Q=VA \qquad \text{Eq. 16}$$

A=Area of flow [sq-feet]
P=Wetted perimeter [feet]
Q=Flow [CFS]

$$\text{thus} \quad Q=\frac{1.486\left(\frac{A}{P}\right)^{\frac{2}{3}}S^{\frac{1}{2}}A}{n}=1.486\frac{A^{\frac{5}{3}}S^{\frac{1}{2}}}{n\,P^{\frac{2}{3}}} \qquad \text{Eq. 17}$$

3) For certain standard shapes (rectangular, vee, trapezoidal channel, or a circular pipe), the user may provide the appropriate dimensions, and HydroCAD determines the end area and discharge curves. These are determined according to the area and perimeter equations on page 128 and Manning's equation, above.

4014

Reach Routing Limitations

Since the stage-discharge relationship is based solely on Manning's equation, it does *not* consid inlet conditions. It assumes that the Manning's flow, and not the inlet, is the controlling factc Similarly, all automatic pipe sizing is based on Manning's equation alone. If a complete analys is desired for a pipe reach, including entrance losses, the reach should be modeled as a pond wi a culvert outlet.

Reach Routing Calculations

The storage-indication method is the basic reach routing technique provided by HydroCAI and is identified as "STOR-IND" in the program. This standard procedure is well described i [1 p.64-65] and will not be repeated here. Additional operations are performed by HydroCAD a follows:

1) Before routing, any base flow is added to the inflow hydrograph and any inflow loss i subtracted.

2) If a pipe is being resized, its diameter is calculated with Manning's equation based on the pea inflow.

3) The stage-storage curve is obtained by multiplying the end area vs. depth curve by the lengt of the reach.

4) If the range of the storage and discharge curves is exceeded, HydroCAD extrapolates from th last two points on each curve. Since extrapolating from the curves is not the same as extendin the physical sides of the reach, a warning message is issued. To obtain an accurate routing yo must provide storage and discharge data for higher stages.

5) If the peak inflow exceeds the Manning's normal flow capacity of the reach, a warning is issued. Depending on the design criteria, this *may* be an acceptable condition. A reach can handle more than its normal flow capacity when the flow is *not* normal, such as on the rising limb of a storm hydrograph. However, if such flow persists, the reach will fill with water, causing one of the following conditions.

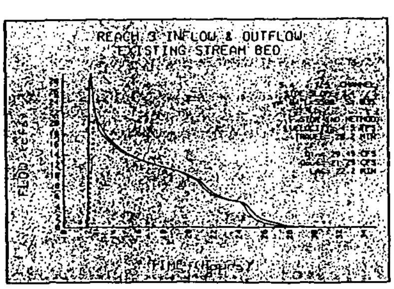
6a) If an open channel fills with water (as defined by the flood elevation), a warning message is issued. The routing continues using the extrapolated curves if required.

6b) If a pipe fills with water, the excess is detained without head so that open-channel conditions can be maintained. A warning message is issued and the detained water is routed when the pipe is no longer full. For an accurate routing, you should model the pipe as a pond with a culvert outlet, as described in the next section.

7) Routing is performed over the time span of the inflow hydrograph(s). The span should include the earliest inflow in order for an accurate routing to be performed. Routing is normally performed using the time interval (dt) of the inflow hydrograph. A finer interval may be specified for each reach, if needed, to improve tracking or eliminate outflow oscillations.

Effects of Reach Routing

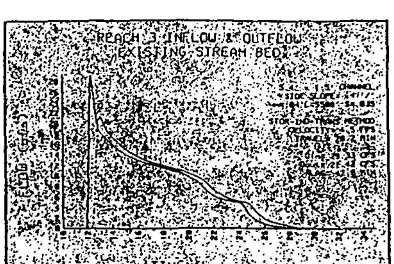
A reach will normally *attenuate and delay* the hydrograph that is routed through it. The extent of this transformation depends on many factors, including the reach dimensions, slope, and Manning's number. Short reaches (up to several hundred feet) often have a minimal effect on the routed hydrograph. For this reason they are frequently modeled as a flow segment within a subcatchment. This is the only option for some methods such as TR-55, which includes no reach routing procedures at all.



On the other hand, for long reaches with large cross-sections, low slopes and/or high Manning's numbers, the routing effect can be significant. The graph at right shows the effects of storage-indication routing through a 5500 foot long channel. Significant attentuation may also occur on shorter reaches if the inflow peak is of short duration.

Allowing for Travel Time

The storage-indication method, as described above, accounts for only the storage effects of the reach. Other techniques must be used to account for the kinematic effects of long reaches. HydroCAD provides the option of adding a time lag or *translation* to the normal storage-indication routing. Setting the reach routing method to "STOR-IND+TRANS" causes the storage-routed hydrograph to be further translated by the travel time. (See page 127 for the determination of travel time.) A close examination of the example at right will reveal that the peak discharge no longer corresponds to a point on the inflow curve, but is translated by the prescribed amount.



000058

Section 27 - Hydraulics Calculations

This section details the hydraulic calculations used within HydroCAD. These equations are used to determine the discharge resulting from a given head applied to each device. They are used primarily in determining the stage-discharge curve(s) for a pond. All equations determine the discharge, Q, in CFS.

Sharp-Crested Rectangular Weir

The basic equation for a sharp-crested weir is derived in [7 p.362]. The discharge coefficient varies slightly based on the crest height and the resulting turbulence. The effective length of the weir is adjusted to allow for end contractions.

$$Q=C\,L_e\,H^{3/2} \quad \text{where} \quad C=3.27+.4\frac{H}{P} \quad \text{and} \quad L_e=L-\frac{nH}{10} \qquad \text{Eq. 18}$$

L=Crest length [feet]
L_e=Effective crest length (reduced for end contractions)
P=Crest height [feet above approach channel]
n=Number of end contractions (0, 1, or 2)
H=Head [feet above invert elevation]

Broad-Crested Rectangular Weir

A broad-crested rectangular weir differs from a sharp-crested weir in that the discharge coefficient may vary significantly with head. (See [6 p.274].) This allows the modeling of a wide range of real-world weirs.

$$Q=C\,L\,H^{3/2} \qquad \text{Eq. 19}$$

C=English discharge coefficient
L=Crest length [feet]
H=Head [feet above invert elevation]

C varies with H depending on the shape of the weir. For the weir under consideration, C must be specified at one or more of the following heads: .5, 1, 1.5, 2, 2.5, 3, 4, 5 feet. For intermediate heads, HydroCAD interpolates linearly between the values given. For heads below or above the given range, HydroCAD uses the first or last coefficient without extrapolation.

Metric discharge coefficients for various weirs are given on page 154, which is reprinted from [6 p.276]. Coefficients are listed only at the heads where a particular weir was studied. When using these coefficients in HydroCAD, you must specify a discharge multiplier of 1.81 to convert to English units.

V-Notch Weir

The basic equation for a v-notch weir is taken from [5 p.5-15].

$$Q = C \tan\frac{\theta}{2} H^{2.5} \qquad \text{Eq. 20}$$

C=English weir coefficient
θ=Notch angle (between two sides, not from vertical)
H=Head [feet above invert elevation]

C may be entered directly, or determined by HydroCAD according to the equation:

$$C = 2.46 \frac{\tan(90-\theta/2)}{[illegible]\,25} \qquad \text{Eq. 21}$$

Trapezoidal Weir

The trapezoidal weir equation is a more general form of the v-notch weir. Setting the crest length to zero yields the previous equation for a v-notch weir. C may be entered directly, or determined by Eq. 20 above.

$$Q = C \tan\frac{\theta}{2}\left[\frac{5}{2} H_1 H^{1.5} - \frac{3}{2} H^{2.5}\right] \quad \text{where } H_1 = H + \frac{L}{2\tan\frac{\theta}{2}} \qquad \text{Eq. 22}$$

C=English weir coefficient
θ=Notch angle (between two sides, not from vertical)
H=Head [feet above invert elevation]
H_1=Head above imaginary apex of notch
L=Horizontal crest length (feet)

Rectangular Orifice/Grate

4014 49/20

The discharge equation for a rectangular orifice under any head condition is derived from the discharge through a thin horizontal strip: [5 p.4-3]

$$dQ = C L \sqrt{2gY}\, dY \qquad \text{Eq. 23}$$

C=Discharge coefficient (Default is .60)
L=Strip length (width of orifice) [feet]
g=Gravitational constant
Y=Head over center of strip [feet]
dY=Height of horizontal strip

Integrating over the height of the orifice yields:

$$Q = \frac{2}{3} C L \sqrt{2g}\left(H^{3/2} - [H-M]^{3/2}\right) \qquad \text{Eq. 24}$$

H=Head above invert elevation [feet]
M=Height of orifice [feet]

When the orifice is partially submerged (H<M) the term [H-M] becomes zero and this reduces to the weir equation:

$$Q = \frac{2}{3} C L \sqrt{2g}\, H^{3/2} = 3.21\, L\, H^{3/2} \qquad \text{Eq. 25}$$

These equations are for a (default) orifice opening in a *vertical plane* (i.e., discharging horizontally). For an orifice in a *horizontal plane*, the above equation is used without adjustment of the head.

A grate consisting of identical rectangular openings can also be modeled with this equation. The orifice dimensions are specified for each opening, and the discharge multiplier is used to specify the number of openings. (This technique may not be appropriate for *vertical* grates with *horizontal openings*, since the openings are at different elevations and therefore not identical.)

Circular Orifice/Grate

The discharge for a circular orifice is derived in [5 p.4-3].

$$Q = C\, a \sqrt{2gh} \qquad \text{Eq. 26}$$

C=Discharge coefficient (Default is .60)
a=Submerged area [sq-feet]
g=Gravitational constant
h=Head above *center* of orifice [feet] given by:

$$h = H - r \ \text{(if fully submerged)} \qquad h = \frac{H}{2} \ \text{(if partially submerged)} \qquad \text{Eq. 27}$$

H=Head above invert [feet]
r=Radius

When partially submerged, the head adjustment closely approximates the weir discharge of an orifice. It also provides continuity between the fully and partially submerged conditions. For critical situations, the resulting discharge curve should be verified by independent means.

These equations are for a (default) orifice opening in a *vertical plane* (i.e., discharging horizontally). For an orifice in a *horizontal plane*, the above equation is used without adjustment of the head.

A horizontal grate consisting of identical circular openings can also be modeled with this equation. The orifice dimensions are specified for each opening, and the discharge multiplier is used to specify the number of openings. (This technique may not be appropriate for *vertical* grates, since the openings are at different elevations and therefore not identical.)

Orifices Under Low-head Conditions

The above orifice equations are generally valid for all openings in a *vertical* plane. Under low-head (partially submerged) conditions, these equations reduce to the appropriate weir equation. For orifice openings in a *horizontal* plane, the equations assume that the head is large in relation to the orifice size. This can lead to overestimating the discharge under low-head conditions. To ensure correct flow under all conditions, discharge can be limited to that predicted by the weir equation:

$$Q = 3.3\, L\, H^{3/2} \qquad \text{Eq. 28}$$

L=Crest length (orifice perimeter) [feet]
H=Head [feet above invert elevation]

000059

This will cause the weir equation to control at low heads, without effecting the high-head discharge predicted by the orifice equation. The result is useful for a range of real-world "orifices," such as the top of a standpipe.

Culvert Flow

When evaluating a culvert, HydroCAD checks multiple flow conditions in order to determine the prevailing control *at each elevation*. This is based on six types of culvert flow identified in [9 p.E-1,7] and characterized as follows (also see [8 p.21-18,19]):

Type	Inlet	Outlet	Slope	Flow Type	Tailwater Dependent?	Type of Control
1a	SUB.	SUB.	ANY	PIPE	YES	OUTLET
1b	SUB.	FREE	MILD	PIPE	NO	OUTLET
1c	SUB.	FREE	ANY	CHANNEL	NO	INLET (orifice)
2a	FREE	TW>Yc	MILD	CHANNEL	YES	OUTLET
2b	FREE	TW<Yc	MILD	CHANNEL	NO	OUTLET
2c	FREE	TW<Yc	STEEP	CHANNEL	NO	INLET (weir)

SUB.=Submerged, TW=Tailwater, Yc=Critical Depth

For type 1b, assuming that the culvert is full along its entire length [9 p.D-11]:

$$V = \sqrt{\frac{H - D + SL}{\frac{K_e + 1}{2g} + \frac{n^2 L}{2.22 R^{4/3}}}} \quad \text{and} \quad Q = A V \qquad \text{Eq. 29}$$

V=Average velocity of flow [ft/sec]
H=Head in [feet above inlet invert elevation]
D=Depth of flow [feet] (=culvert height)
S=Slope [rise/run]
L=Length [feet]
Ke=Entrance energy loss coefficient (See table on page 155)
g=Gravitational constant
n=Manning's number (See table on page 152)
R=Hydraulic radius [feet]
A=Cross-sectional area [sq-feet]

Type 2b discharge is the same as type 1b except that the depth (D) is less than the culvert height. Under these conditions, open channel flow exists and backwater calculations must be performed to precisely determine the depth. To reduce calculation time, the depth is approximated by:

$$D = \frac{3}{4} H \qquad \text{Eq. 30}$$

Rather than directly determining whether type 1b or 2b flow exists, HydroCAD simply uses the lesser of this depth and the culvert height. This also ensures continuity between the two flow conditions, with the cross over occurring when the head is 4/3 of the culvert height.

4014

50/20

Types 1a and 2a are similar to types 1b and 2b, except for the tailwater dependency. This is accommodated by substituting the tailwater depth for D in the above equation.

Types 1c and 2c operate under inlet control, and the discharge is determined with the orifice equations given previously. The orifice discharge coefficient is given by:

$$C_d = \frac{C_c}{1+K_e} \qquad \text{Eq. 31}$$

Cc=Contraction coefficient (default is .90)

Note that for Ke=.5 this yields Cd=.6, which is the default discharge coefficient for a sharp-edged orifice.

The final determination of culvert discharge is made by calculating the type 1a/2a, 1b/2b *and* 1c/2c flows as described above. The least of these values (a, b, and c) is then used as the final discharge for a given head.

NOTE: The approximations used for culvert discharge have generally been found to provide sufficient accuracy for most hydrograph routing purposes. However, it is strongly recommended that the resulting stage-discharge curve be verified using independent culvert data. If a significant discrepancy is found, the desired discharge data should be entered directly as a Special Outlet instead of using the built-in culvert equations.

Special Outlet Device

The special outlet device is designed to handle unusual stage-discharge relationships that can't be reproduced with standard devices. With HydroCAD's ability to model complex series-parallel devices (see page 123), there should be very few situations which actually require a special outlet.

A special outlet consists of user-defined stage-discharge curve. The first discharge value must always be zero CFS and may occur at any desired elevation. Additional discharge values are specified at higher elevations as required to adequately represent the true shape of the desired curve. When choosing the elevations, keep in mind that HydroCAD performs a linear interpolation to determine the discharge at any required intermediate elevations.

Exfiltration Calculations

Since exfiltration is incorporated into a pond's stage-discharge curve, it is classified as an "outlet device." Exfiltration is also distinct from an "inflow loss," in that it continues to occur even when there is no inflow.

To separate exfiltration from other "true" outflows, it is usually directed to a "secondary outflow" to prevent further routing. Although there are few standards as to how to determine exfiltration, HydroCAD provides two basic procedures that can be used to implement a wide range of design methods:

1) A **constant exfiltration rate Q** may be specified in CFS. This value is applied at all stage-discharge increments above the specified invert elevation. Zero exfiltration is used for all stage-discharge increments at or below the invert elevation. A constant exfiltration may also be used to model a pump or other "outlet" that "turns on" at a given elevation.

The invert elevation is commonly set to the bottom of the pond. This yields zero exfiltration when the pond is empty, increasing to the specified rate at the first stage-discharge increment. A higher invert may be specified if lower levels of the pond are impervious and have no exfiltration.

2) An **exfiltration velocity V** may be specified in FPM. This is multiplied by the available exfiltration area at a given elevation to determine the exfiltration rate in CFS.

$$Q_Y = \frac{V}{60}(A_Y - A_{invert}) \qquad \text{Eq. 32}$$

Q_Y=Exfiltration at elevation Y (CFS)
V=Exfiltration velocity (FPM)
A_Y=Exfiltration area at elevation Y (SF)
A_{invert}=Exfiltration area at invert elevation (SF)

The exfiltration area may be defined in two ways: A) if all exfiltration is assumed to be downward (none through the sides of the pond), you may use the pond's *surface area*; B) if exfiltration occurs through all exposed surfaces regardless of slope, you may use the pond's *wetted area*.

In either case, the *available* area is the additional exfiltration area lying above the invert elevation. No exfiltration will occur through the portion of the pond that lies at or below the invert elevation. To allow exfiltration through the bottom of the pond, set the invert elevation to zero.

For shallow ponds, the surface area and wetted area are almost identical, so the surface area method is recommended. Only for drywells and other ponds with significant side-areas is the wetted area method needed. (See page 131 for details on wetted area calculations.)

Using a prec rate

A measured perc rate can be converted to an equivalent exfiltration velocity by the following equation. However, other factors must be considered to determine if this is a reasonable design value for a proposed exfiltration area. (For example, can a large pond be expected to perc at the same rate as a small test pit?)

$$V = \frac{1}{12P} \qquad \text{Eq. 33}$$

V=Exfiltration velocity (FPM)
P=Perc Rate (Minutes per Inch)

Tips for using exfiltration

Setting the invert elevation
The invert elevation is intended to exclude the impervious lower elevations of the pond. Exfiltration will occur only when the water surface exceeds this level. When using the velocity method, exfiltration will apply only to any additional area lying *above* the invert. This distinction is particularly important in the case of flat bottomed ponds, such as drywells. With the invert at this lowest level, any bottom area will be excluded from exfiltration. *If you want to allow exfiltration through a flat bottom by the velocity method*, you must set the exfiltration invert to zero.

Using surface area vs. wetted area
By basing exfiltration on *surface area*, you are stating that all flow will essentially be downward. Only horizontal areas (above the invert) are available for exfiltration. All vertical areas are excluded.

If you wish to include vertical surfaces, such as the sides of a drywell, then you may want to specify *wetted area*. As always, it is your responsibility to ensure that this computation is applicable to your particular situation.

Advanced techniques
While most cases will require just a single exfiltration device, it is also possible to use several exfiltration devices on a single pond. This could be used to model multi-stage exfiltration schemes, such as a drywell that overflows into a perforated pipe.

As with all pond designs, you should view and understand the stage-discharge plot to make sure the pond is exhibiting the behavior you expect. Do not rely solely on a review of the hydrograph where the pond's behavior is intertwined with the complexities of the inflow hydrograph.

000060

Section 28 - Pond Routing Calculations

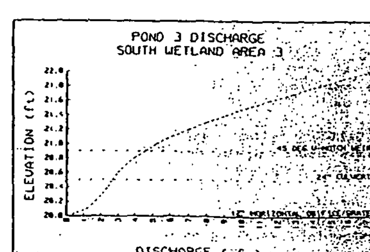
Before routing through any pond, the storage vs. elevation (stage-storage) and the discharge vs. elevation (stage-discharge) must be determined.

Stage-Storage Calculations

The cumulative stage-storage curve may be entered directly, or HydroCAD can sum incremental values to determine the cumulative storage. A third option is to enter surface areas at certain elevations and have HydroCAD calculate the storage. (These calculations are detailed on page 130.) In any case, a minimum of two storage points are required to permit interpolation for intermediate elevations. You should use as many points as necessary to accurately represent the true stage-storage curve for the pond. If the available storage space is less than 100%, or several identical ponds are being modeled, the stage-storage curve is adjusted by these factors.

Stage-Discharge Calculations

The stage-discharge curve is automatically compiled based on the selected outlet devices. Each device is evaluated using the equations in the previous section. Each outlet also has a "discharge multiplier" which may be used for adjusting the normal rating curve or handling several identical devices. A factor may also be applied when modeling several identical ponds in a single calculation.



The individual outlet devices are combined into one or two stage-discharge curves based on the specified *device routing*. In the default configuration, all outlets are routed directly to the primary outflow, as shown in the sample stage-discharge curve at right. They are considered to be independent, *parallel* outlets whose flows are additive. To calculate the composite stage-discharge curve, HydroCAD evaluates up to 101 uniformly spaced elevations that are a multiple of 1/10 foot apart. (HydroCAD automatically chooses the interval and number of steps to cover the range of the stage-storage curve.) The total discharge at each elevation is determined by adding the discharge from each individual device.

If any devices are routed to a *secondary outflow*, two separate stage-discharge curves are compiled using the same basic procedure. Each device is included in the stage-discharge curve to which it is routed. To perform the actual pond routing, a *total discharge curve* is obtained by adding the two curves. When routing is complete, the total outflow hydrograph is split into primary and secondary outflows based on the ratio of the two stage-discharge curves. This produces an automatic split-flow, or "diversion." This is most commonly used when one or more outlets require separate routing, such as an emergency spillway or an exfiltration outflow.

4014

Compound Outlet Devices

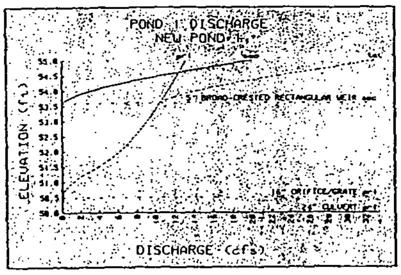
More complex outlets can be modeled by placing standard devices in series. An orifice, for example, could be routed through a culvert. To calculate the discharge at each elevation, HydroCAD evaluates the standard flow through each device, and uses the lower (controlling) flow to build the stage-discharge curve. By making this comparison at each elevation step, different devices may control the outflow at different pond stages.[44]

"ven more complex outlets can be deled by utilizing simultaneous ies/parallel device combinations. A standpipe is a common example of a compound outlet device. This could be modeled by a combination of standard devices as shown in the schematic representation at right.

```
Primary Discharge
└─1=Culvert                    (standpipe outlet)
   ├─2=Orifice/vertical        (side opening(s) in riser)
   └─3=Orifice/horizontal      (top opening of riser)
```

Reading from the bottom up: Device 3 is a horizontal orifice representing the flow into the top of the riser. Device 2 is used to model one or more openings in the side of the riser. Devices 2 and 3 are summed together, and routed through the final outlet culvert, device 1.

This graph shows a typical stage-discharge curve for a pond with a compound outlet. A culvert is positioned with the inlet invert at 50 feet; however, no discharge occurs until the water level reaches an orifice 50.5 feet. (This example might represent an orifice plate used to reduce the flow through an existing culvert.) Above 50.5 feet, both devices are evaluated to determine which will control at each elevation. The resulting curve is labeled "pri" for primary.



This example also includes a broad-crested weir which is directed to the secondary discharge. This might represent an emergency spillway that is being routed separately from the culvert/orifice combination.

When describing compound outlet devices, it is generally easiest to start with the *final* device (such as the culvert shown above) that contributes directly to the primary discharge. Then work *up* from the final device, entering each device that limits flow or contributes to the discharge. The process is then repeated for any secondary discharge.

[44] This procedure uses the standard hydraulic equations as described in the previous section, and does not consider the possible influence of one device on another, except to limit the flow to the lesser of the two. Like all complex calculations, it is important to verify these results by independent means to ensure they are sufficiently accurate for your purposes.

51/204

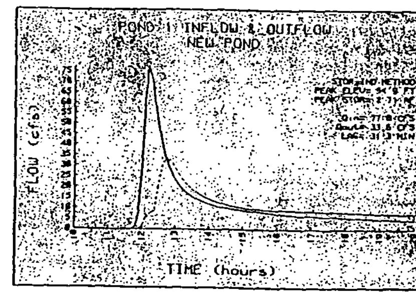
Pond Routing Procedure

After the storage and discharge curves have been determined, the storage-indication metho is used for routing the inflow hydrograph through the pond.[45] This procedure is well describ in [1 p.64-65] and will not be repeated here. Additional operations are performed by HydroCA as follows:

1) Before routing begins, any base flow is added to the inflow hydrograph and any inflow loss i subtracted. If "automatic base flow" is selected, the base flow is set to the pond's discharge at th specified starting elevation. This places the pond in an equilibrium condition (stable wate surface elevation) when routing begins. (To route a pond with no inflow, you must provide a zer flow hydrograph to establish the desired time span and routing interval. This is most easil generated using a subcatchment with a very small area and low curve number.)

2) If a starting elevation is specified, routing begins with the water at this level. If this is abov the lowest outlet device, the pond begins discharging immediately, possibly before any inflow ha occurred. If the starting elevation is below the lowest device, no outflow occurs until this leve is attained. (The outflow volume will also be reduced by the amount of storage below this level.

3) Routing is performed over the time span of the inflow hydrograph(s). The span must include the earliest inflow in order for an accurate routing to be performed. Routing is normally performed using the time interval (dt) of the inflow hydrograph, although a finer interval may be specified for each pond to provide improved tracking. The normal dt is *divided* by the specified finer routing value. Finer routing (usually 2) can also be used to eliminate any oscillations in the pond's outflow.



4) If the range of the storage or discharge curves is exceeded, HydroCAD extrapolates from the last two points on each curve and issues a warning message.

5) If the peak elevation exceeds the specified flood elevation, a warning message is issued and routing will *continue*.

When using the storage-indication method, keep in mind that a zero velocity is assumed in the pond. This is an accurate assumption for most ponds where the storage volume is large in comparison to the inflow. However, if the velocity approaching the outlet device(s) is significant, then this method may *underestimate* the discharge and *overestimate* the peak elevation and storage of the pond.

[45] A significant feature of the storage-indication method is that the stage-discharge curve must be static for the duration of the routing. This means that all outlet characteristics, including any tailwater, cannot change during this period.

Section 29 - Measuring Water Quality

Whereas stormwater management has traditionally been concerned with managing stormwater *quantity*, new regulations in many areas also require the consideration of water *quality*. Since these requirements vary widely, it is difficult for a general stormwater program to provide the precise information requested in every case. Nevertheless, there are a number of calculations performed by HydroCAD that can be used to obtain the information required for most water quality studies.

Detention Time

Many projects must now meet specific requirements for detention time. These requirements are sometimes expressed in terms such as "detain the ten-year storm for 24 hours." Unfortunately, these definitions are sometimes vague and therefore difficult to interpret or implement. Even the intent can be unclear. For example, is a given rule an attempt to address water quality or quantity?

When addressing water *quality*, one of the more useful and objective measures in the *average detention time*. This is a measure of how long water is detained in a pond or other impoundment, and can be used to determine the time available for removal of sediments or the neutralization of runoff contaminants. (See [14 p.257] for a further discussion of detention time.)

The *center of mass* method is one of the most basic methods of calculating detention time. It evaluates the difference in time between the center-of-mass of the inflow and outflow hydrographs. One of the chief advantages of this technique is that it is easily calculated, and can even be estimated graphically. However, the technique does not consider the actual movement of water through the pond, and can fail to give a good measure of detention time in a number of situations.

The *plug flow method* provides a more physically meaningful measure of the average detention time. This technique divides the inflow hydrograph into a number of "plugs" of equal volume, and then calculates the time between each plug entering and leaving the pond. The average time for all plugs is then calculated and used as an overall measure of detention time.

HydroCAD employs the plug flow method to determine the average detention time for each pond.[46] The *theoretical* detention time is calculated by assuming that water initially in the pond is allowed to discharge *before* the first plug from the inflow is allowed to discharge. This "first-in first-out" assumption will yield a maximum detention time, and means that the amount of water initially in the pond *will* effect the calculated result. (Since all water in the pond is displaced before any of the inflow starts to discharge, the detention time is increased by the time required to flush the initial volume.)

·y water retained in the pond, or discharging after the specified time span, is excluded from the ·culation. To obtain an accurate measure of detention time, it is therefore important to use a ·ime span that allows the pond to discharge fully. This can be determined by comparing the *volume* of inflow and outflow. These should be roughly the same (unless the pond was surcharged or water was retained). Also compare the volume of flow included in the plug flow calculation (this is shown to the right of the detention time). For accurate (maximum) results, this should be close to the volume of the outflow hydrograph.

[46] Note that the plug flow method is *not* a routing method. It is used *after* the hydrograph routing has been performed.

Section 30 - Link Calculations

The link is used to introduce an external hydrograph into the HydroCAD routing diagram. It may also be used to apply a flow threshold and/or scale factor as described below. The input hydrograph for a link may be supplied in one of three ways:

An **automatic link** imports a hydrograph directly from a node in another HydroCAD project. The outflow of the specified node is used as the inflow to the link. This capability is commonly used to interconnect portions of a project which have been divided into two or more separate diagrams. (See page 84 details.)

A **manual link** is used to manually enter an arbitrary hydrograph. The hydrograph is defined by its starting time (T_o), interval between points (d_T), and up to 101 ordinates in CFS.

An **import link** is used to read a hydrograph from an ASCII text file. The file contains the same basic information as a manual link, but in a more flexible format. Hydrographs of up to 501 points can be imported. If more ordinates are provided, adjacent points are automatically averaged to reduce the number within this limit. For details on the required file structure, see the sample file LINKTEST.TXT which is installed with HydroCAD.

A link may also specify a flow threshold and/or scale factor. If a threshold is specified, only the portion of the hydrograph above (or below) the threshold is retained. The hydrograph is then multiplied by the specified percentage scale factor to produce the final outflow.

The *time scale* of a link can also be adjusted. This allows an imported hydrograph to be scaled to a different duration, making it possible to utilize dimensionless hydrographs, as used for runoff studies in Ohio. (A link file containing the Ohio dimensionless hydrograph is included in the file OHHYDRO1.TXT.)

The results of a link threshold and/or scaling are readily apparent when you draw the hydrograph. This shows the original "inflow" curve and the scaled "outflow" curve together.

Note that the time span and interval of a manual link or import link are determined by the data supplied to the link, and is independent of the time span and interval used for runoff hydrographs. If a matching time span and/or interval is desired, the link data and runoff settings must be coordinated.

000061

4014

Section 31 - Additional Hydrograph Calculations

This section explains the procedures used to calculate certain values appearing on HydroCAD reports.

The **peak flow** for each hydrograph is calculated using the three highest points on the hydrograph.[47] A parabola is fitted to these points and the apex of the parabola specifies the true peak. This eliminates variations in the peak that would occur if only a single point were considered. This improvement in accuracy is most pronounced with a narrow peak, where the two closest points fall on either side of the peak and may be several percent below the actual peak.

The **peak attenuation** indicates the percentage reduction in peak inflow caused by a routing operation. This is determined by comparing the peak of the inflow and outflow hydrographs as calculated above.

The **time of peak** is determined by the same parabolic fit to the three highest points. The apex of the parabola establishes a time of peak with far greater resolution than the time between points. Like the peak flow, this value is not affected by the placement of the points on the "true" curve.

The **time lag** caused by a reach or pond is the difference between the time of peak obtained from the inflow and outflow hydrographs. (This is distinct from the *travel time*, described below.)

The **hydrograph volume** is determined by integrating the flow over the time span of the hydrograph. Since the volume can include flow only within the given time span, any flow before or after is excluded. Also note that the lag introduced by a pond or reach can cause a discrepancy between the calculated inflow and outflow volume. If necessary, this can be remedied by increasing the calculation time span to include the entire duration of the inflow and outflow hydrographs.

When **adding hydrographs** with the same starting time (To) and time interval (dT), the ordinates at matching times are added directly. If the hydrographs differ as to To or dT, such direct addition is not possible. (This can occur when using a link to introduce an external hydrograph.) In this case, the resulting hydrograph includes the span of both inflows and may have a larger dT if required to cover the new period without exceeding the 501-point limit. Since the inflow ordinates now occur at different times, HydroCAD interpolates between the points of each inflow when performing the summation.

The **peak elevation, peak depth, peak storage, and peak velocity** are the largest actual values attained at discrete times during the routing. Since no interpolation is employed, they may be slightly lower than suggested by the interpolated peak flow. (The value at each routing interval can be tabulated by selecting the DETAILS option during calculations.)

The reach **travel time** is calculated by dividing the length of the reach by the peak velocity. It therefore represents a *minimum* travel time rather than an average. Depending on the selected routing method, the travel time may be used to further translate (delay) the reach outflow.

[47] HydroCAD also checks for flat-topped hydrographs, where curve fitting and extrapolation are not appropriate.

Section 32 - Cross-Sectional Area & Perimeter Equations

The following equations are used to calculate the cross-sectional area and wetted perimeter of common channel geometries.

Rectangular, Vee, or Trapezoidal channel

$$a = Y\left(W + \frac{Y}{2}\left(\frac{1}{S_1} + \frac{1}{S_2}\right)\right)$$

$$P_w = W + Y\left(\sqrt{1+\frac{1}{S_1^2}} + \sqrt{1+\frac{1}{S_2^2}}\right) \qquad \text{Eq 34}$$

a=Cross-sectional area
P_w=Wetted perimeter
Y=Flow depth
W=Bottom width
S_1=Left side slope [rise/run]
S_2=Right side slope [rise/run]

Parabolic Channel

$$a = \frac{2}{3}WY$$

$$P_w = \sqrt{4Y^2 + \frac{W^2}{4}} + \frac{W^2}{8Y}\ln\left(\frac{2Y + \sqrt{4Y^2 + \frac{W^2}{4}}}{\frac{W}{2}}\right)$$

a=Cross-sectional area
P_w=Wetted perimeter
Y=Flow depth
W=Flow width at surface

Circular Pipe (any flow depth)

$$a = r^2\theta - (r - Y)\sqrt{Y(D-Y)}$$

$$P_w = \theta D \quad \text{where } \theta = \cos^{-1}\left(1 - \frac{Y}{r}\right) \qquad \text{Eq 36}$$

a=Cross-sectional area
P_w=Wetted perimeter
D=Diameter
r=Radius
Y=Flow depth
(For multiple pipes, a and P_w are multiplied by the number of pipes)

Section 33 - Pond Volume & Area Calculations

HydroCAD provides several options for determining the stage-storage characteristics of a pond.

1) Direct entry of *cumulative* (total) storage at various elevations.

2) Entry of *incremental* storage, that is, the volume of horizontal sections across the pond. These sections are summed by the program to produce the cumulative storage.

3) Entry of *surface areas* at various elevations, from which HydroCAD determines the incremental (and cumulative) storage at each elevation. The incremental storage may be calculated by prismatic or conic sections as described below.

Prismatic volume determination (Average area method)

This technique assumes that the areas are horizontal planes through a *prism*.[48] The calculation involves taking the average of the area at the top and bottom of the section and multiplying by the thickness. Although this is a commonly used method for calculating volume, it should be noted that it is completely accurate only for prismatic sections.

$$V = h\frac{A_1 + A_2}{2} \qquad \text{Eq 37}$$

V=Volume of section
h=Height of section
A_1=Area of bottom of section
A_2=Area of top of section

000062

[48] A prismatic ...

5 3 / 204

Conic volume determination (Frustum of a cone)

Most real-world ponds are not prismatic. They may be more round than rectangular, and may have more than two sloping sides. For these situations, the volume is more accurately determined by assuming that the areas are horizontal sections of a *cone*. We can use the equation for the volume of the frustum of a cone:

$$V = \frac{1}{3}\pi h\left(R_1^2 + R_2^2 + R_1 R_2\right) \qquad \text{Eq. 38}$$

$$\text{where } R_1 = \sqrt{\frac{A_1}{\pi}} \text{ and } R_2 = \sqrt{\frac{A_2}{\pi}}$$

V=Volume of section
h=Height of section
R_1=Radius of bottom section
R_2=Radius of top section
A_1=Area of bottom of section
A_2=Area of top of section

This equation also yields the correct volume for the frustum of a pyramid. (With A_1 and A_2 taken as the areas of the top and bottom of the frustum.) This is an accurate representation of ponds with four equally sloping sides at right angles.

Wetted area determination

The following equation is used to determine the wetted-area for a section of a cone.[49] This is used as the basis for exfiltration calculations based on wetted-area. The technique requires that storage first be calculated by conic sections, as shown above. The results are accurate for all sections of cylinders and cones, making them suitable for most dry wells and natural ponds.

$$A = \pi\left(R_1 + R_2\right)\sqrt{\left(R_1 - R_2\right)^2 + h^2} \qquad \text{Eq. 39}$$

A=Wetted (side) area of section
h=Height of section
R_1=Radius of bottom section
R_2=Radius of top section

[49] Prismatic sections based on surface area can't be used to calculate wetted-area, since there is no unique solution to their wetted-area equation.

References

The following references contain additional information on the hydrology and hydraulics utilized by HydroCAD. These publications are referred to by number throughout this manual and are listed in approximate order of usage.

[1] McCuen, Richard H. A Guide to Hydrologic Analysis Using SCS Methods, Prentice Hall, 1982. (Out of print. Also see [13], Chapter 8.)

[2] Soil Conservation Service Technical Release Number 20 (TR-20), National Technical Information Service, 1982.

[3] Smith, P.D. Basic Hydraulics, Butterworth Scientific, 1982.

[4] Sharp, J.J. & Sawden, P. Basic Hydrology, Butterworth Scientific, 1984.

[5] King, H.W. & Brater, E.F. Handbook of Hydraulics, McGraw Hill, 1963.

[6] Simon, Andrew Practical Hydraulics, John Wiley & Sons, 1981.

[7] Chow, Ven Te Open Channel Hydraulics, McGraw Hill, 1959.

[8] Merrit, Frederick Standard Handbook for Civil Engineers

[9] Jerome M. Norman et al Culverts - Hydrology & Hydraulics, Lehigh University, 1980.

[10] Soil Conservation Service National Engineering Handbook, Section 4 - Hydrology, 1985.

[11] Soil Conservation Service Technical Release Number 55 (TR-55), 1986.

[12] American Concrete Pipe Association Concrete Pipe Handbook, 1981.

[13] McCuen, Richard H. Hydrologic Analysis and Design, Prentice Hall, 1989.

[14] Barfield and Warner, Applied Hydrology and Sedimentology for Disturbed Areas, Oklahoma Technical Press, 1983.

ATTACHMENT A-4

CulvertMaster® THEORY SECTION

Chapter 5

CulvertMaster Theory

5.1 Analysis of Culvert Systems

When engineers analyze culvert systems, they are usually trying to make one or more of the following basic determinations:

- Determine the size, shape, and number of new or additional culverts required to pass a design discharge.
- Predict the hydraulic capacity of an existing culvert system under some allowable headwater elevation.
- Predict the upstream flood level at an existing culvert system resulting from some check discharge or other discharge magnitude of special interest.
- Develop hydraulic performance curves for a culvert system for assessing hydraulic risk at a crossing or as input to another hydraulic or hydrologic model.

CulvertMaster models complete culvert systems. A roadway cross-drainage culvert system is typically designed to safely carry flood flows from one side of the road to the other. The culvert system consists of the following hydraulic components:

- **Hydrology** – An upstream watershed drainage area discharging design and check storm flows to the culvert system. The magnitude of these discharges is computed using an accepted hydrological method such as Rational, SCS Peak, application of a state Regression Equation, or some other suitable methodology.
- **Culvert Hydraulics** – One or more culvert barrels along with associated headwalls, wingwalls, and other types of end treatments, which convey flow through a roadway embankment. The performance of these culverts is described using culvert hydraulics.
- **Roadway Overtopping** – A roadway embankment may be subject to overtopping flows if the total capacity of the culvert(s) is exceeded. Such overtopping flows are analyzed using weir hydraulics.
- **Tailwater** – A natural stream, improved channel, or other waterway at the discharge or tailwater side of the roadway embankment. The hydraulic response of the downstream discharge areas affects the capacity of the total culvert system. Tailwater is analyzed using uniform flow assumptions or separate floodplain analyses.

5.2 Hydrology

CulvertMaster offers the Rational Method, SCS peak discharge method, and user-defined peak discharge for determination of the design and check storm flows.

5.2.1 Rational Method

The Rational Method solves for peak discharge based on watershed area, rational coefficient, and rainfall intensity for the watershed.

The Rational Method is based on the simple rational equation:

$$Q = CiA \quad \text{or} \quad Q = AiR \qquad (5.1)$$

Where
- Q = Design flow rate (ft^3/s, m^3/s)
- C = Rational coefficient for drainage area
- R = Rational coefficient for drainage area
- i = Rainfall intensity (in/hr, mm/hr)
- A = Drainage area (acres, hectares)

Because of the units that are used in the above equation, CulvertMaster uses a factor to convert the units of intensity and drainage area to units of flow. In English units, this conversion factor is 1.008 ac-in/hr per ft^3/s, and in SI units it is 0.002778 hectare-mm/hr per m^3/s. C, the rational coefficient, is the parameter that is most open to engineering judgment. Engineering references contain tables that will help you estimate C. The following table shows rational coefficients for common land uses. In many cases, an area weighted average of C coefficients is used for the entire drainage area. CulvertMaster will automatically calculate the weighted C for drainage areas.

Table 5-1

Table of Rational Coefficients

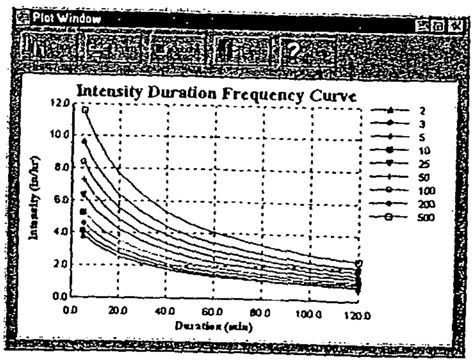
AREA	C Values
Business	
Downtown	0.70-0.95
Neighborhood	0.50-0.70
Residential	
Single Family	0.30-0.50
Multiunit detached	0.40-0.60
Multiunit attached	0.60-0.75
Suburban resident	0.25-0.40
Apartment	0.50-0.70
Residential (1.2 acre lots or more)	0.30-0.45
Industrial	
Light	0.50-0.80
Heavy	0.60-0.90
Parks and Cemeteries	0.10-0.25
Playgrounds	0.20-0.40
Unimproved	0.10-0.30
Pavement	
Asphalt/Concrete	0.70-0.95
Brick	0.70-0.85
Drives and Walks	0.75-0.85
Lawns, Sandy soils	
Flat, 2%	0.05-0.10
Average, 2-7%	0.10-0.15
Steep, 7%	0.15-0.20
Lawns, Heavy Soils	
Flat, 2%	0.13-0.17
Average, 2-7%	0.18-0.22
Steep, 7%	0.25-0.35
Railroad Yard	0.20-0.40
Roofs	0.70-0.95

ensity-Duration-Frequency (IDF) Curves

ainfall intensities for which you will be designing your culverts are determined by regulatory s, which typically analyze historical rainfall information and compile it into IDF Curves based on uency of the storm events. These curves provide a quick reference for determining the intensity of rainfall that will occur at given return periods.

Figure 5.1

Intensity Duration Frequency Curve



IDF curves can be extremely useful when performing hand calculations using the Rational Method. They are less useful when used for computer programs or spreadsheet calculations because you have to look up values and enter them each time you need intensity data. For this reason, it is often easier to use rainfall tables or rainfall equations to calculate IDF curves.

Rainfall Tables

CulvertMaster lets you enter data in a rainfall table and saves the data so you may use it again for other projects. Entering the design intensities for your culvert analysis is simply a matter of looking up data from a graph and entering it into the Rainfall Table. Once the rainfall intensities are gathered, it is helpful if you make a table to enter the data.

Table 5-2

Table of Rainfall Intensities (in/hr)

Duration	Return Periods					
	2-year	5-year	10-year	25-year	50-year	100-year
5 min	7.20	7.86	8.42	9.33	10.01	10.80
10 min	6.02	6.67	7.20	8.04	8.71	9.38
15 min	5.20	5.84	6.35	7.14	7.77	8.40
30 min	3.88	4.42	4.84	5.48	5.98	6.49
60 min	2.50	2.94	3.26	3.74	4.12	4.50

CulvertMaster has provided a tabular format for entering rainfall volumes obtained from the National Weather Services' Hydro-35 technical report. Hydro-35 provides a methodology for estimating rainfall volumes for durations of 5 minutes to 60 minutes for the eastern United States.

Rainfall Equations

Equations can generally be fit to IDF curves with high accuracy. The most common form of these equations is:

$$i = \frac{a}{(b+D)^n} \quad (5.2)$$

Where i = Intensity of rainfall
D = Duration of rainfall
a, b = Equation coefficients
n = Exponent

Other equations supported in CulvertMaster are:

$$i = \frac{a(RP)^m}{(b+D)^n} \quad (5.3)$$

$$i = a + b(\ln D) + c(\ln D)^2 + d(\ln D)^3 \quad (5.4)$$

Where i = Intensity of rainfall
D = Duration of rainfall
RP = Return period
a, b, c, d = Equation coefficients
m, n = Equation exponents

5.2.3 SCS Peak Discharge Method

The SCS Peak Discharge method solves for peak discharge based on watershed area, site specific hydrologic conditions expressed through the curve number (CN), rainfall distribution type, and rainfall depth.

SCS Peak Discharge method is based on the following equations:

$$Q_p = q_u A Q F_p \quad (5.5)$$

Where Q_p = Peak discharge (ft^3/s)
q_u = Unit peak discharge ($cfs/mi^2/in$)
A = Drainage area (mi^2)
Q = Runoff (in)
F_p = Pond and swamp adjustment factor

$$Q = \frac{(P-I_a)^2}{(P-I_a)+S} \quad (5.6)$$

Where Q = Runoff (in)
P = Rainfall (in)
S = Potential maximum retention after runoff begins (in)
I_a = Initial abstraction (in)

$$I_a = 0.2S \quad (5.7)$$

Where $S = \frac{1000}{CN} - 10$

The CulvertMaster implementation of the SCS-graphical peak discharge follows the procedures and methodology set forth in Chapter 4 and Appendix F of the SCS-TR-55 (1986) manual.

Table 5-3

Runoff Curve Numbers for Urban Areas[1]

COVER DESCRIPTION		CURVE NUMBERS FOR HYDROLOGIC SOIL GROUP			
Cover type and hydrologic condition	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.):					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50 to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roof, driveways, etc. (excluding right-of-way)					
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved, open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only)		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	42	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

[1] Average runoff condition, and I_a = 0.2S.

Other runoff curve number tables can be found in Chapter 2 of the SCS-TR-55 (1986) manual.

5.3 Culvert Hydraulics

Obtaining an accurate solution of culvert hydraulics represents a formidable computational task. Culverts often act as a significant constriction to flow and are subject to a range of flow types including both rapidly varied and gradually varied flow.

It is this mix of flow conditions and the highly transitional nature of culvert hydraulics that make the culvert hydraulic solution difficult. For this reason, the accepted approach is to simplify the hydraulics problem and solve the culvert using two different assumptions of flow control:

- **Inlet control assumption** – Computes the upstream energy grade or headwater depth resulting from the constriction effect at the culvert entrance while neglecting the culvert barrel friction and other minor losses.
- **Outlet control assumption** – Computes the upstream headwater depth using conventional hydraulic methodologies that consider the predominant losses due to the culvert barrel friction as well as the minor entrance and exist losses.

CulvertMaster uses the methodologies set forth in *Hydraulic Design Series No. 5, Hydraulic Design of Highway Culverts* (1985) as prepared for the U.S. Federal Highway Administration, and included as Chapter 6 in this manual, to perform these hydraulic calculations automatically under each control assumption. The controlling headwater depth is then the greater of the inlet control headwater or the outlet control headwater.

5.3.1 Inlet Control Hydraulics

When a culvert is operating under inlet control, the hydraulic control section is the culvert entrance itself. The losses due to barrel friction and other minor losses are not as significant as the losses caused by the entrance constriction. This entrance capacity is determined primarily by available opening area, the shape of the opening, and the inlet configuration of the entrance.

Generally, since the control section of a culvert operating under inlet control is at the upstream end of the culvert, barrel flows are in the supercritical flow regime and outlet velocities are determined using frontwater gradually varied flow profiles.

Three types of inlet control hydraulics are in effect over a range of culvert discharges:

- **Unsubmerged** – Occurs in low discharge conditions, and the culvert entrance is performing mainly as a weir. The hydraulics of weir flow are governed by empirical working equations developed as a result of model tests.
- **Submerged** – Occurs in higher discharge ranges when the culvert entrance is fully submerged and the culvert is assumed to be operating as an orifice.
- **Transition** – Occurs in the poorly defined region just above the unsubmerged zone below the culvert entrance crown and the fully submerged zone above the culvert entrance crown.

CulvertMaster computes the inlet control headwater depth using a set of inlet control working equations which are implemented by the design nomographs published in HDS-5 and many other hydraulics manuals and handbooks.

Inlet Control Working Equations

The equations for unsubmerged (weir) and submerged (orifice) flow conditions are presented below. The transition zone is taken by linear interpolation in the region described by Notes 1 and 3. The constant coefficients for use in the working equations, as well as the use of Form (1) and Form (2) unsubmerged equations, vary according to section shape and material.

Inlet Control Design Equations

Unsubmerged[1]:

Form (1) 2:
$$\frac{HW_i}{D} = \frac{H_c}{D} + K\left[\frac{Q}{AD^{0.5}}\right]^M - 0.5S \quad (5.8a)$$

Form (2):
$$\frac{HW_i}{D} = K\left[\frac{Q}{AD^{0.5}}\right]^M \quad (5.8b)$$

Submerged[3]:

$$\frac{HW_i}{D} = c\left[\frac{Q}{AD^{0.5}}\right]^2 + Y - 0.5S \quad (5.9)$$

Where HW_i = Headwater depth above inlet control section invert (ft, m)
D = Interior height of culvert barrel (ft, m)
H_c = Specific head at critical depth ($d_c + V_c^2/2g$) (ft, m)
Q = Discharge (ft^3/s, m^3/s)
A = Full cross-sectional area of culvert barrel (ft^2, m^2)
S = Culvert barrel slope (ft/ft, m/m)
K, M, c, Y = Constants from the following table

Notes:

[1] Unsubmerged equations apply up to $Q/AD^{0.5} = 3.5$
[2] For mitered inlets, use +0.7S instead of –0.5S as the slope correction factor
[3] The submerged equation applies above $Q/AD^{0.5} = 4.0$.

Table 5-4

Constants for Inlet Control Design Equations

				Unsubmerged			Submerged	
Ch No	Shape and Material	Nomograph Scale	Inlet Edge Description	Form	K	M	C	Y
1	Circular Concrete	1 2 3	Square edge w/headwall Groove end w/headwall Groove end projecting	1	0.0098 .0018 .0045	2.0 2.0 2.0	0.0398 .0292 .0317	0.6[illegible] .7[illegible] .6[illegible]
2	Circular CMP	1 2 3	Headwall Mitered to slope Projecting	1	.0078 .0210 .0340	2.0 1.33 1.50	.0379 .0463 .0553	.69 .75 .54
3	Circular	A B	Beveled ring, 45°bevels Beveled ring, 33.7°bevels	1	.0018 .0018	2.50 2.50	.0300 .0243	.74 .83
8	Rectangular Box	1 2 3	30° to 75° wingwall flares 90° to 15° wingwall flares 0° wingwall flares	1	0.26 .061 .061	1.0 .75 .75	.0347 .0400 .0423	.86 .80 .82
9	Rectangular Box	1 2	45° wingwall flare d=.0430 18°-33.7° wwall flare d=.0830	2	.510 .486	.667 .667	.0309 .249	.80 .83

Table 5-4

Continued

				Unsubmerged			Submerged	
Ch No	Shape and Material	Nomo-graph Scale	Inlet Edge Description	Form	K	M	C	Y
10	Rectangular Box	1 2 3	90°headwall w/¾" chamfers 90°headwall w/45° bevels 90°headwall w/33.7° bevels	2	.515 .495 .486	.667 .667 .667	.0375 .0314 .0252	.79 .82 .865
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
12	Rectangular Box	1 2 3	45°non-offset wingwall flares 18.4°non-offset wingwall flares 18.4°non-offset wingwall flares 30°skewed barrel	2	.497 .493 .495	.667 .667 .667	.0339 .0361 .0386	.803 .806 .71
[illegible]	Rectangular Box Top Bevels	1 2 3	45°wingwall flares offset 33.7°wingwall flares offset 18.4°wingwall flares offset	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
16-19	C M Boxes	1 2 3	90°headwall Thick wall projecting Thin wall projecting	1	.0083 .0145 .0340	2.0 1.75 1.5	.0379 .0419 .0496	.69 .64 .57
29	Horizontal Ellipse Concrete	1 2 3	Square edge w/headwall Groove end w/headwall Groove end projecting	1	.0100 .0018 .0045	2.0 2.5 2.0	.0398 .0292 .0317	.67 .74 .69
30	Vertical Ellipse Concrete	1 2 3	Square edge w/headwall Grove end w/headwall Groove end projecting	1	0.0100 .0018 .0095	2.0 2.5 2.0	0.0398 .0292 .0317	0.67 .74 .69
34	Pipe Arch 18" Corner Radius CM	1 2 3	90° headwall Mitered to slope Projecting	1	.0083 .0300 .0340	2.0 1.0 1.5	.0379 .0463 .0496	.69 .75 .57
35	Pipe Arch 18" Corner Radius CM	1 2 3	Projecting No bevels 33.7° bevels	1	.03 .0088 .0030	1.5 2.0 2.0	.0496 .0368 .0269	.57 .68 .77
36	Pipe Arch 31" Corner Radius CM	1 2 3	Projecting No bevels 33.7° bevels	1	.03 .0088 .0030	1.5 2.0 2.0	.0496 .0368 .0269	.57 .68 .77
41-43	Arch CM	1 2 3	90° headwall Mitered to slope Thin wall projecting	1	.0083 .0300 .0340	2.0 1.0 1.5	.0379 .0463 .0496	.69 .75 .57
55	Circular	1 2	Smooth tapered inlet throat Rough tapered inlet throat	2	.534 .519	.555 .64	.0196 .021	.90 .90
56	Elliptical Inlet Face	1 2 3	Tapered inlet-beveled edges Tapered inlet-square edges Tapered inlet-thin edge projecting	2	.536 .5035 .547	.622 .719 .80	.0368 .0478 .0598	.83 .80 .75
57	Rectangular	1	Tapered inlet throat	2	.475	.667	.0179	.97
58	Rectangular Concrete	1 2	Side tapered-less favorable edges side tapered-more favorable edges	2	.56 .56	.667 .667	.0446 .0378	.85 .87
59	Rectangular Concrete	1 2	Slope tapered-less favorable edges Slope tapered-more favorable edges	2	.50 .50	.667 .667	.0446 .0378	65 .71

5.3.2 Outlet Control Hydraulics

Outlet control headwater depths are computed by summing entrance, exit, friction, and other losses along the culvert barrel. The energy basis for solving the outlet control headwater, *HW*, for a culvert is presented graphically in Figure 5.2 and the basic energy equation, Equation 5.10.

4014 57/204

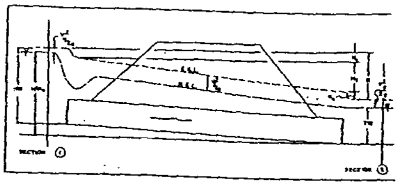


Figure 5.2

Full flow energy and hydraulic grade lines

$$HW_o + \frac{V_u^2}{2g} = TW + \frac{V_d^2}{2g} + H_L$$

Where
- HW_o = Headwater depth above the outlet invert (ft, m)
- V_u = Approach velocity (ft/s, m/s)
- TW = Tailwater depth above the outlet invert (ft,m)
- V_d = Exit velocity (ft/s, m/s)
- H_L = Sum of all losses including entrance (H_e), friction loss (H_f), exit loss (f and other losses
- g = Gravitational acceleration constant (ft/s², m/s²)

HDS-5 presents some simplified procedures and nomographs that can be used to compute outlet co losses, H_L. However, such simplified approaches will result in degenerative solutions in low flow or p full barrels. With the need for performance curves, which quantify culvert response and upstream downstream impacts over a full range of flow magnitudes, it is necessary to take a more rigorous en approach to computing outlet control hydraulics.

Since culverts are frequently hydraulically short, uniform flow depths are not always achiev CulvertMaster incorporates a powerful set of gradually varied flow algorithms, which can correctly anal the following conditions:

- **Partial flow** – Water surface profiles are computed so the actual flow depths at the entrance and exit the culvert during part-full conditions are accurately calculated.
- **Pressure flow** – Backwater is computed during full barrel submergence.
- **Composite pressure and free surface flows** – Both sealing and unsealing conditions are used computations.
- **Composite flow regime profiles** - Mixed supercritical and subcritical flow profiles can be analyzed.
- **Adverse and horizontal culverts** – Backwater profiles can be performed for culverts for whi uniform flows are undefined.

Entrance Minor Loss

The entrance loss H_e is a function of the barrel velocity head just inside the entrance, and is expressed b the equation:

$$H_e = k_e\left(\frac{V^2}{2g}\right) \tag{5.11}$$

Where
- H_e = Entrance loss (ft, m)
- V = Velocity head inside of barrel entrance (ft/s, m/s)
- k_e = Entrance loss coefficient (unitless)
- g = Gravitational acceleration constant (ft/s², m/s²)

The entrance loss coefficient, k_e, is a function of inlet configuration. Values for the coefficients are presented in table 5-5 below.

Table 5-5

Entrance Loss Coefficients: Outlet Control, Full or Partly Full Entrance Head Loss

Type of Structure and Design of Entrance	Coefficient k_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded 3 edges to radius to 1/12 barrel dimension, or beveled edges 3 sides	0.2

Table 5-5

Continued

Type of Structure and Design of Entrance	Coefficient k_e
Box, Reinforced Concrete (continued)	
Wingwalls at 30° to 75° barrel	
Square-edged at crown	0.5
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

* "End Section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance.

Exit Minor Loss

The exit loss is actually an expansion loss, which is a function of the change in velocity head occurring at the discharge location. In culvert hydraulics, the sudden expansion loss is expressed as:

$$H_o = 1.0\left[\frac{V^2}{2g} - \frac{V_d^2}{2g}\right] \tag{5.12}$$

Where
- H_o = Exit minor loss (ft, m)
- V = Velocity just inside the barrel exit (ft/s, m/s)
- V_d = Outfall channel velocity (ft/s, m/s)
- g = Gravitational acceleration constant (ft/s², m/s²)

The discharge velocity can be neglected, in which case CulvertMaster assumes the exit loss is equal to the barrel velocity head.

Friction Loss

CulvertMaster calculates friction losses using gradually varied flow profiles.

Gradually Varied Flow

Gradually varied flow occurs for free surface flow conditions. Starting from a given boundary control depth, usually the tailwater elevation, the depth will change gradually until (if the barrel or conduit is sufficiently long) normal depth is achieved. If the water surface moves above or below the crown of the conduit in sealing or unsealing flow conditions, the program will automatically transition into and out of a pressure analysis mode. Friction losses for the submerged part of the barrel or conduit will be based on full flow friction slope.

CulvertMaster computes both frontwater and backwater profiles. The program automatically determines whether a structure is operating under upstream or downstream control, and then performs the correct gradually varied flow analysis.

Gradually varied flow profiles express the water surface depth curve along the length of the barrel and are developed in an upstream or downstream direction depending on the hydraulic slope of the conduit and the controlling water surface elevations. For structures on a mild slope, which is when the constructed slope is less than critical slope, the depth of flow will increase gradually if the downstream starting elevation is less than normal depth (M2 drawdown curve, Figure 5.3). The depth of flow will decrease gradually if the starting water surface elevation is greater than normal depth (M1 backwater curve, Figure 5.3). In these types of water surface profiles, the flow is operating under subcritical flow conditions.

· a structure operating at a hydraulically steep slope (i.e. constructed slope is greater than critical slope), lually varied flow profiles will be developed in an upstream direction whenever the controlling ,water elevation lies well above critical depth and subcritical flow exists at the pipe exit. Unless the pipe is very short, steep pipes that have tailwater depths above critical depth will experience a hydraulic jump somewhere along their length.

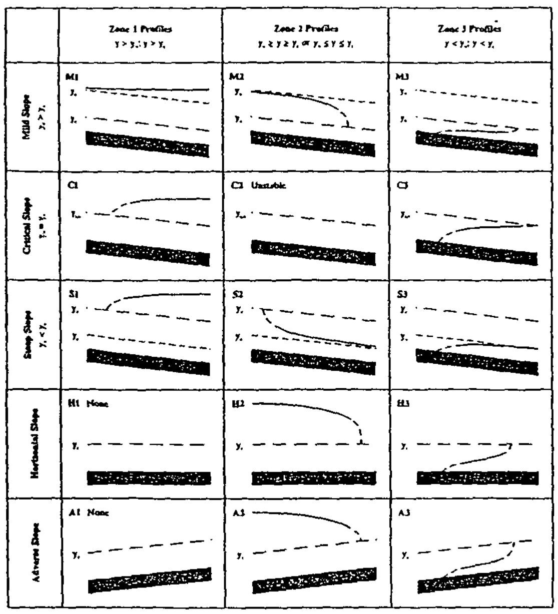


Figure 5.3
Gradually Varied Flow Classifications

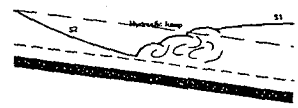
Usually, culverts on a steep slope are either entrance controlled or inlet controlled, and operate under supercritical flow. The gradually varied flow algorithm sets the upstream hydraulic grade equal to the critical depth elevation at the entrance and develops a frontwater curve by computing depth variation in a

4014 58/204

downstream direction. In this type of frontwater analysis, the depth decreases from critical (approaches normal depth at the culvert exit.

CulvertMaster uses two different algorithms for computing the gradually varied flow profile th culvert barrel depending on the hydraulic context of the pipe component as defined by the co slope and the governing tailwater elevations. The two methods are direct step and standard step. computationally equivalent, and the choice of which method to use is based on performance consid

Figure 5.4
Plot of Hydraulic Jump



Backwater Analysis

A backwater analysis for outlet control computations starts at the downstream outfall under free di submerged, or tailwater control, and proceeds in an upstream direction. CulvertMaster will backwater analysis in both free surface flow conditions and pressure flow computations under t conditions.

Frontwater Analysis

CulvertMaster will perform a frontwater analysis in a steep culvert operating under supercritical flo hydraulic control will be at the upstream end of a hydraulically steep culvert, and the gradually vari analysis will proceed in a downstream direction until normal depth is achieved, a hydraulic jump oc the end of the culvert is reached. Even though outlet control rarely occurs in supercritical flow sit the frontwater analysis is still performed for purposes of determining exit velocity.

Direct Method

This is a method for solving gradually varied flow profiles in which a series of explicit solutions f length are made that will result in a specified depth of flow. In the direct step method, the program over a range of depth changes resulting from a backwater or drawdown curve, and computes the i distance that results in the correct amount of headloss to produce the energy balance. This interval d is then summed with all previous intervals to compute the total distance over which the flow prof been computed, and the next interval computation is performed.

The program uses a trial and error procedure to solve for a depth that will yield a final interval, when summed with the previous profile distance, will equal the total length of the conduit.

The basic equation for energy balance is:

$$H_1 = H_2 + h_f + h_e \qquad ($$

Where H_1 = Total head at section 1
H_2 = Total head at section 2
h_f = Friction loss
h_e = Eddy loss

For prismatic channels, the eddy loss, h_e is practically zero and is assumed as zero by the program.

The total head at both ends of the section is defined as:

$$H = Z + \frac{V^2}{2g} \qquad (5.14)$$

Where H = Total head (ft, m)
Z = Water surface elevation (ft, m)
V = Velocity at the end of the section (ft/s, m/s)
g = Gravitational acceleration constant (ft/s², m/s²)

The friction loss, h_f, is the product of step length and uniform flow friction slope, or:

$$S_f \Delta x = \frac{1}{2}(S_1 + S_2)\Delta x \qquad (5.15)$$

Where S_f = Friction slope (ft/ft, m/m)
Δx = Change in horizontal distance in the section (ft, m)
S_1, S_2 = Friction slopes at the end of the section (ft/ft, m/m)

In the direct step solution, the only unknown given two depths at each end of a sub-reach is the parameter Δx. This is solved for each trial depth.

Standard Step Method

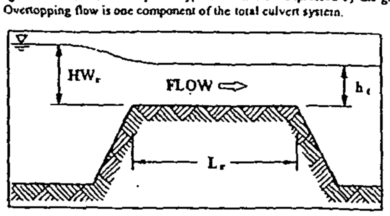
The standard step method is the most popular method of determining the profiles of gradually varied flow. In standard step analysis, the reach is divided into a number of sub-reaches. Computations are performed in steps from one section to the next, where the depth is either incremented or decremented to yield a correct energy balance in the basic equation.

Unlike the direct step method, the standard step method varies the depth at the next section until the friction slope relationship, shown above, yields an average friction slope, S_f, which balances energy between each end of the sub-reach.

5.4 Roadway Overtopping

Whenever the culvert headwater begins to rise above the minimum elevation of the roadway, overtopping will occur. Overtopping flow is modeled as a special type of weir flow expressed by the general broad-crested weir equation. Overtopping flow is one component of the total culvert system.

Figure 5.5
Roadway overtopping



$$Q_o = C_d \times L \times HW_r^{1.5} \qquad (5.1$$

Where Q_o = Overtopping flow rate (ft³/s, m³/s)
C_d = Overtopping discharge coefficient
L = Length of the roadway crest (ft, m)
HW_r = Overtopping depth (ft, m)

The overtopping discharge coefficient is a function of submergence using the equation:

$$C_d = k_t C_r \qquad (5.1$$

along with the figures below. The first two figures are used to derive the base weir coefficient resulti from deep and shallow overtopping, respectively. The submergence correction is determined implicit using the final figure and applied against the base coefficient.

Figure 5.6
Discharge coefficient for $HW_r/L_r > 0.15$

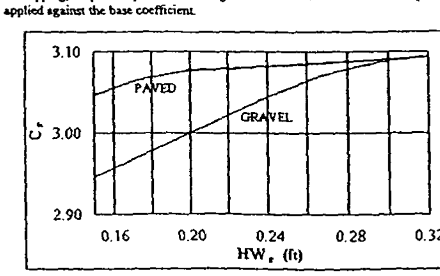


Figure 5.7
Discharge coefficient for $HW_r/L_r \leq 0.15$

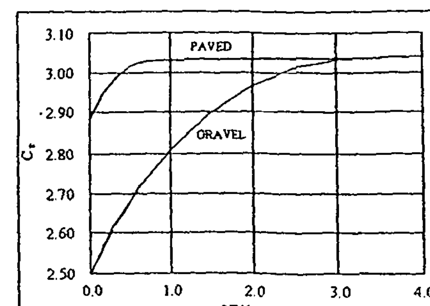
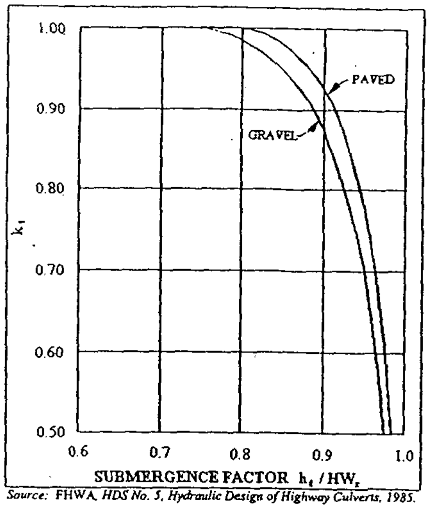


Figure 5.8
Submergence Factor h/HW,



Source: FHWA, *HDS No. 5, Hydraulic Design of Highway Culverts, 1985.*

5.5 Tailwater

The tailwater depth at the culvert discharge channel is an important parameter for computing the response of the culvert. For backwater analyses, the tailwater elevation may be the starting gradually varied flow boundary. CulvertMaster will automatically compute discharge variable tailwater depths using its uniform flow calculation capability.

5.5.1 Uniform Flow

In uniform flow, the discharge, depth, and velocity are constant with distance along the pipe or ditch section. The physical slope of the flow element, friction slope or energy slope, and the water surface slope are all parallel. This means that the hydraulic grade line of flow is parallel to the channel slope and that the depth will be the same at all points along its length. This depth of uniform flow is called the *normal depth.* Uniform flow can occur as free surface flow in a prismatic conduit flowing partially full. A prismatic channel or conduit has a non-varying cross-section and a constant bottom slope.

Uniform flow can be described by the generalized friction equation:

$$V = CR^x S^y \tag{5.18}$$

Where V = Mean velocity (ft/s, m/s)
R = Hydraulic radius (ft, m)

S = Energy slope (ft/ft, m/m)
C = Flow resistance factor (units vary by method)
x = Exponent
y = Exponent.

The roughness, or flow resistance, factor, C, is usually determined by the material lining the flow cha
However, the ultimate value of the C component may be a function of the channel shape, depth
velocity of flow. The hydraulic radius, R, is a strict function of the channel shape. For every geon
shape, R can be readily calculated once a depth is known or assumed. The energy slope, S, is cor
under the uniform flow assumption.

Since velocity is constant under uniform flow conditions, combining the general uniform flow equ
with the continuity equation

$$Q = VA \quad (:$$

results in the equation:

$$Q = ACR^x S^y \quad (:$$

Where Q = Discharge (ft³/s, m³/s)
A = Cross-sectional flow area (ft², m²)
C = Flow resistance factor (units vary by method)
R = Hydraulic radius (ft, m)
S = Energy slope (ft/ft, m/m)
x = Exponent
y = Exponent

[illegible]

Manning's Formula

CulvertMaster implements the Manning's formula for computing uniform flows in tailwater channels,
for computing friction slopes in its gradually varied flow computations in culvert barrels.

The use of Manning's formula is applicable to uniform flow provided the slope of the energy grade li
used and is applied when the flow range is rough. Culvert flow tends to be turbulent when the flow r
is rough and during peak flows. The turbulent regime is comprised of three ranges of flow: smc
transition, and rough. When the flow is rough, Manning's formula is most often used.

Manning's formula is derived from the Chezy Formula, which computes velocity based on a C coeffic
hydraulic radius, and friction slope. Manning's formula relates this C coefficient to a roughness coeffic
and hydraulic radius. The original format incorporates the roughness coefficient:

$$V = CR^{1/2} S^{1/2} \quad (5$$

Where C = Factor of flow resistance
R = Hydraulic radius (ft, m)
S = Energy slope (ft/ft, m/m)

This equation was further modified by others to become the well-known Manning's formula:

U.S. Units:

$$V = \frac{1.49}{n} R^{2/3} S^{1/2} \tag{5.22a}$$

Where V = Section velocity (ft/s)
R = Hydraulic radius (ft)
S = Energy slope
n = Manning's roughness coefficient

Metric:

$$V = \frac{1}{n} R^{2/3} S^{1/2} \tag{5.22b}$$

Where V = Section velocity (m/s)
R = Hydraulic radius (m)
S = Energy slope
n = Manning's roughness coefficient

5.6 Comparing Results with HY-8

CulvertMaster results will generally be similar to HY-8 results. However, in some complex hydraulic cases, HY-8 uses a simplified solution technique that may produce differences. The following is a list of cases where calculation methodology between CulvertMaster and HY-8 differ:

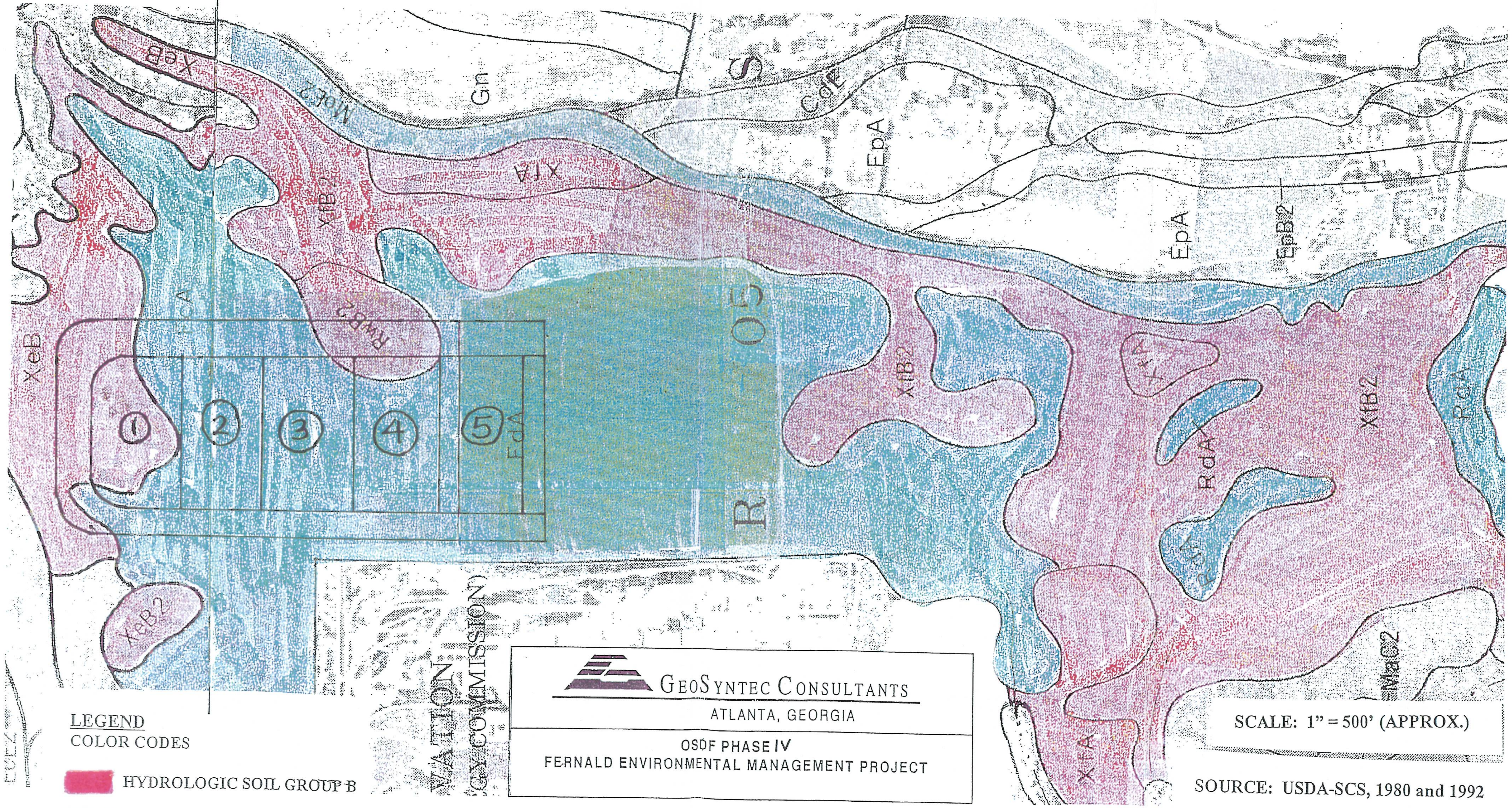
1. CulvertMaster computes inlet control headwater strictly by the inlet control equations presented in HDS-5. HY-8 uses polynomial curve fits for inlet control headwater computation.
2. HY-8 simplifies the computation of the M2 backwater profile for the case where a culvert has unsubmerged tailwater and has discharge in excess of full flow capacity. In this case, HY-8 will assume pressure flow for the full length of the culvert. CulvertMaster will compute the M2 profile backwater curve, and will transition to pressure flow where the profile runs into the top of the pipe.

ATTACHMENT A-5

USDA-SCS SOIL SURVEY MAP

USDA-SCS SOIL SURVEY MAP
BULTER AND HAMILTON COUNTIES , OHIO

N



LEGEND
COLOR CODES

HYDROLOGIC SOIL GROUP B

HYDROLOGIC SOIL GROUP C

GeoSyntec Consultants
ATLANTA, GEORGIA

OSDF PHASE IV
FERNALD ENVIRONMENTAL MANAGEMENT PROJECT

SCALE: 1" = 500' (APPROX.)

SOURCE: USDA-SCS, 1980 and 1992

ATTACHMENT A-6

HYDROLOGIC SOIL DATA

TABLE 18.--SOIL AND WATER FEATURES

["Flooding" and "water table" and terms such as "rare," "brief," "apparent," and "perched" are explained in the text. The symbol < means less than; > means more than. Absence of an entry indicates that the feature is not a concern]

Soil name and map symbol	Hydrologic group	Flooding			High water table			Bedrock		Potential frost action	Risk of corrosion	
		Frequency	Duration	Months	Depth	Kind	Months	Depth	Hardness		Uncoated steel	Concrete
					Ft			In				
ArA, ArB2, ArC2: Ava	C	None	---	---	2.0-4.0	Perched	Mar-Jun	>60	---	High	Moderate	High.
AsB*, AsC*: Ava	C	None	---	---	2.0-4.0	Perched	Mar-Jun	>60	---	High	Moderate	High.
Urban land.												
AvA: Avonburg	D	None	---	---	1.0-3.0	Perched	Jan-Apr	>60	---	High	High	High.
AwA*: Avonburg	D	None	---	---	1.0-3.0	Perched	Jan-Apr	>60	---	High	High	High.
Urban land.												
BoD, BoE, BoF: Bonnell	C	None	---	---	>6.0	---	---	>60	---	Moderate	High	Moderate.
CcC2, CdD, CdE, CdF: Casco	B	None	---	---	>6.0	---	---	>60	---	Low	Low	Low.
CnB2, CnC2: Cincinnati	C	None	---	---	>4.0	Perched	Jan-Apr	>60	---	High	Moderate	High.
DaB: Dana	B	None	---	---	3.0-6.0	Perched	Mar-Apr	>60	---	High	Moderate	Moderate.
EcB2, EcC2, EcD, EcE, EdF: Eden	C	None	---	---	>6.0	---	---	20-40	Soft	High	Moderate	Low.
EeB*, EeC*, EeD*: Eden	C	None	---	---	>6.0	---	---	20-40	Soft	High	Moderate	Low.
Urban land.												
EpA, EpB2, EpC2: Eldean	B	None	---	---	>6.0	---	---	>60	---	Moderate	High	Moderate.
ErA*, ErB*: Eldean	B	None	---	---	>6.0	---	---	>60	---	Moderate	High	Moderate.
Urban land.												
FdA: Fincastle	C	None	---	---	1.0-3.0	Apparent	Jan-Apr	>60	---	High	High	Moderate.

See footnotes at end of table.

TABLE 16.--SOIL AND WATER FEATURES--Continued

Soil name and map symbol	Hydro-logic group	Flooding: Frequency	Flooding: Duration	Flooding: Months	High water table: Depth	High water table: Kind	High water table: Months	Bedrock: Depth	Bedrock: Hardness	Potential frost action	Risk of corrosion: Uncoated steel	Risk of corrosion: Concrete
					Ft			In				
FeA*:												
Fincastle	C	None	---	---	1.0-3.0	Apparent	Jan-Apr	>60	---	High	High	Moderate.
Urban land.												
FoA, FoB2 Fox	B	None	---	---	>6.0	---	---	>60	---	Moderate	Low	Moderate.
FpA*:												
Fox	B	None	---	---	>6.0	---	---	>60	---	Moderate	Low	Moderate.
Urban land.												
Gn Genesee	B	Occasional	Brief	Oct-Jun	>6.0	---	---	>60	---	Moderate	Low	Low.
Go*:												
Genesee	B	Occasional	Brief	Oct-Jun	>6.0	---	---	>60	---	Moderate	Low	Low.
Urban land.												
HeF Hennepin	B	None	---	---	>6.0	---	---	>60	---	Moderate	Low	Low.
HoA Henshaw	C	None	---	---	1.0-2.0	Apparent	Nov-Mar	>60	---	High	High	Moderate.
Hu Huntington	B	Occasional	Brief	Dec-May	4.0-6.0	Apparent	Dec-Apr	>60	---	High	Low	Moderate.
Ju Jules	B	Occasional	Brief	Mar-Jun	>6.0	---	---	>60	---	High	Low	Low.
Lg Lanier	A	Occasional	Brief	Nov-Jun	>6.0	---	---	>60	---	Low	Low	Low.
MaB, MaC2, MaD2, MaE2 Markland	C	None	---	---	3.0-6.0	Perched	Mar-Apr	>60	---	Moderate	High	Moderate.
McA, McB Martinsville	B	None	---	---	>6.0	---	---	>60	---	Moderate	Moderate	Moderate.
MnC2 Miamian	C	None	---	---	>6.0	---	---	>60	---	Moderate	Moderate	Moderate.
MoD2*, MoE2*:												
Miamian	C	None	---	---	>6.0	---	---	>60	---	Moderate	Moderate	Moderate.
Hennepin	B	None	---	---	>6.0	---	---	>60	---	Moderate	Low	Low.
MuC*:												
Miamian	C	None	---	---	>6.0	---	---	>60	---	Moderate	Moderate	Moderate.

See footnotes at end of table.

000074

Source: USDA-SCS-1992.

4014

64/204

TABLE 18.--SOIL AND WATER FEATURES--Continued

Soil name and map symbol	Hydro-logic group	Flooding: Frequency	Flooding: Duration	Flooding: Months	High water table: Depth	High water table: Kind	High water table: Months	Bedrock: Depth	Bedrock: Hardness	Potential frost action	Risk of corrosion: Uncoated steel	Risk of corrosion: Concrete
					Ft			In				
MuC*: Urban land.												
PbB2, PbC2, PbD, PbE--------------- Parke	B	None---------	---	---	>6.0	---	---	>60	---	High-----	Moderate	High.
PcB*, PcC*: Parke------------	B	None---------	---	---	>6.0	---	---	>60	---	High-----	Moderate	High.
Urban land.												
PfC, PfD, PfE----- Pate	C	None---------	---	---	>6.0	---	---	>50	Soft	Moderate	High-----	Moderate.
PhD*: Pate--------------	C	None---------	---	---	>6.0	---	---	>50	Soft	Moderate	High-----	Moderate.
Urban land.												
Pn**-------------- Patton	B/D	None---------	---	---	+.5-2.0	Apparent	Mar-Jun	>60	---	High-----	High-----	Low.
Po*. Pits												
PrA, PrB, PrC2---- Princeton	B	None---------	---	---	>6.0	---	---	>60	---	Moderate	Moderate	Moderate.
RdA--------------- Raub	C	None---------	---	---	1.0-3.0	Apparent	Jan-Apr	>60	---	High-----	High-----	Moderate.
Rn---------------- Ross	B	Rare---------	---	---	4.0-6.0	Apparent	Feb-Apr	>60	---	Moderate	Low------	Low.
RpA, RpB2, RpC2--- Rossmoyne	C	None---------	---	---	1.5-3.0	Perched	Jan-Apr	>60	---	High-----	High-----	High.
RtA*, RtB*, RtC*: Rossmoyne--------	C	None---------	---	---	1.5-3.0	Perched	Jan-Apr	>60	---	High-----	High-----	High.
Urban land.												
RwB2-------------- Russell	B	None---------	---	---	>6.0	---	---	>60	---	High-----	Moderate	Moderate.
RxB*: Russell-----------	B	None---------	---	---	>6.0	---	---	>60	---	High-----	Moderate	Moderate.
Urban land.												
St---------------- Stonelick	B	Frequent----	Very brief	Nov-Jun	>6.0	---	---	>60	---	Moderate	Low------	Low.

See footnotes at end of table.

000075

Source: USDA-SCS, 1992.

4014

05/204

TABLE 18.--SOIL AND WATER FEATURES--Continued

Soil name and map symbol	Hydro-logic group	Flooding			High water table			Bedrock		Potential frost action	Risk of corrosion	
		Frequency	Duration	Months	Depth	Kind	Months	Depth	Hardness		Uncoated steel	Concrete
					Ft			In				
XfA, XfB2---------- Xenia	B	None---------	---	---	2.0-6.0	Apparent	Mar-Apr	>60	---	High-----	High-----	Moderate.

* See description of the map unit for composition and behavior characteristics of the map unit.
** The plus sign preceding the range in depth to the water table means that the range in this soil is from .5 foot above the surface to 2.0 feet below.

Source: USDA-SCS, 1992

0000076

4014

60.204

TABLE 16.--S[illegible] AND WATER FEATURES

[The definitions of "flooding" and "water table" in the text explain terms such as "rare," "brief," "apparent," and "perched." The symbol < means less than; > means more than. Absence of an entry indicates that the feature is not a concern]

Soil name and map symbol	Hydro-logic group	Flooding			High water table			Bedrock		Potential frost action	Risk of corrosion	
		Frequency	Duration	Months	Depth	Kind	Months	Depth	Hardness		Uncoated steel	Concrete
					Ft			In				
AvA----------------- Avonburg	D	None---------	---	---	1.0-3.0	Perched	Jan-Apr	>60	---	High-----	High-----	High.
Bt------------------ Brenton	B	None---------	---	---	1.0-3.0	Apparent	Mar-Jun	>60	---	High-----	High-----	Moderate.
CdD2*, CdE*:												
Casco--------------	B	None---------	---	---	>6.0	---	---	>60	---	Moderate	Low-------	Low.
Rodman-------------	A	None---------	---	---	>6.0	---	---	>60	---	Low-------	Low-------	Low.
CeB----------------- Celina	C	None---------	---	---	1.5-3.0	Perched	Jan-Apr	>60	---	High-----	High-----	Moderate.
CnC2---------------- Cincinnati	C	None---------	---	---	>6.0	---	---	>60	---	High-----	Moderate	High.
CrA----------------- Crosby	C	None---------	---	---	1.0-3.0	Apparent	Jan-Apr	>60	---	High-----	High-----	Moderate.
DaA, DaB------------ Dana	B	None---------	---	---	3.0-6.0	Perched	Mar-Apr	>60	---	High-----	Moderate	Moderate.
DbB----------------- Dana	B	None---------	---	---	3.5-6.0	Perched	Mar-Apr	40-60	Rippable	High-----	Moderate	Moderate.
EcE2, EcF2---------- Eden	C	None---------	---	---	>6.0	---	---	20-40	Rippable	---	Moderate	Low.
Ee------------------ Eel	C	Occasional	Brief-----	Oct-Jun	3.0-6.0	Apparent	Jan-Apr	>60	---	High-----	Moderate	Low.
ElA, ElB2, ElC2, EnA, EnB2--------- Eldean	B	None---------	---	---	>6.0	---	---	>60	---	Moderate	High-----	Moderate.
EuA*, EuB*:												
Eldean-------------	B	None---------	---	---	>6.0	---	---	>60	---	Moderate	High-----	Moderate.
Urban land.												
FcA, FcB------------ Fincastle	C	None---------	---	---	1.0-3.0	Apparent	Jan-Apr	>60	---	High-----	High-----	Moderate.
FdA, FdB------------ Fincastle	C	None---------	---	---	1.0-3.0	Perched	Jan-Apr	48-72	Rippable	High-----	High-----	Moderate.
Gn------------------ Genesee	B	Common------	Brief-----	Oct-Jun	>6.0	---	---	>60	---	Moderate	Low-------	Low.

See footnote at end of table.

0000077

Source: USDA-SCS, 1980.

4014

6/7/204

TABLE 16.--SOIL AND WATER FEATURES--Continued

Soil name and map symbol	Hydrologic group	Flooding: Frequency	Flooding: Duration	Flooding: Months	High water table: Depth	High water table: Kind	High water table: Months	Bedrock: Depth	Bedrock: Hardness	Potential frost action	Risk of corrosion: Uncoated steel	Risk of corrosion: Concrete
					Ft			In				
UnA, UnB: Uniontown	B	None	---	---	2.5-6.0	Apparent	Nov-May	>60	---	High	Low	Moderate.
UpA*: Urban land.												
Eldean	B	None	---	---	>6.0	---	---	>60	---	Moderate	High	Moderate.
UsA*: Urban land.												
Patton	B/D	None	---	---	0-2.0	Apparent	Mar-Jun	>60	---	High	High	Low.
WbA: Warsaw	B	None	---	---	>6.0	---	---	>60	---	Moderate	Low	Moderate.
WeA, WeB: Wea	B	None	---	---	>6.0	---	---	>60	---	Moderate	Moderate	Moderate.
WyB, WyB2, WyC2, WzC3: Wynn	B	None	---	---	>6.0	---	---	20-40	Rippable	Moderate	High	Low.
WuB*, WuC*: Wynn	B	None	---	---	>6.0	---	---	20-40	Rippable	Moderate	High	Low.
Urban land.												
XeA, XeB, XeB2: Xenia	B	None	---	---	2.0-6.0	Apparent	Mar-Apr	>60	---	High	High	Moderate.
XfA, XfB, XfB2: Xenia	B	None	---	---	2.0-4.0	Perched	Mar-Apr	48-72	Rippable	High	High	Moderate.

* See map unit description for the composition and behavior of the map unit.

Source: USDA-SCS, 1980.

ATTACHMENT A-7

TR-55 LAND USE CATEGORIES AND RUNOFF CURVE NUMBERS

BORROW AREA SOILS 3/5

Table 2-2a.—Runoff curve numbers for urban areas[1]

Cover description		Curve numbers for hydrologic soil group—			
Cover type and hydrologic condition	Average percent impervious area[2]	A	B	C	D
Fully developed urban areas (vegetation established)	VEGETATED FINAL COVER SYSTEM		83		
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only)[4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)[5]	UNVEGETATED FINAL COVER SYSTEM AND LINER RUNOUT	77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).			89		

[1]Average runoff condition, and $I_a = 0.2S$.
[2]The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.
[3]CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.
[4]Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.
[5]Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4, based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

Table 2-2c.—Runoff curve numbers for other agricultural lands[1]

Cover description		Curve numbers for hydrologic soil group—			
Cover type	Hydrologic condition	A	B	C	D
Pasture, grassland, or range—continuous forage for grazing.[2]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow—continuous grass, protected from grazing and generally mowed for hay.	—	30	58	71	78
Brush—brush-weed-grass mixture with brush the major element.[3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	[4]30	48	65	73
Woods—grass combination (orchard or tree farm).[5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods.[6]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	[4]30	55	70	77
Farmsteads—buildings, lanes, driveways, and surrounding lots.	—	59	74	82	86

RUNON AREAS EAST OF OSDF

DISTURBED AREA CONSTRUCTION SUPPORT

[1]Average runoff condition, and $I_a = 0.2S$.

[2]*Poor:* <50% ground cover or heavily grazed with no mulch.
Fair: 50 to 75% ground cover and not heavily grazed.
Good: >75% ground cover and lightly or only occasionally grazed.

[3]*Poor:* <50% ground cover.
Fair: 50 to 75% ground cover.
Good: >75% ground cover.

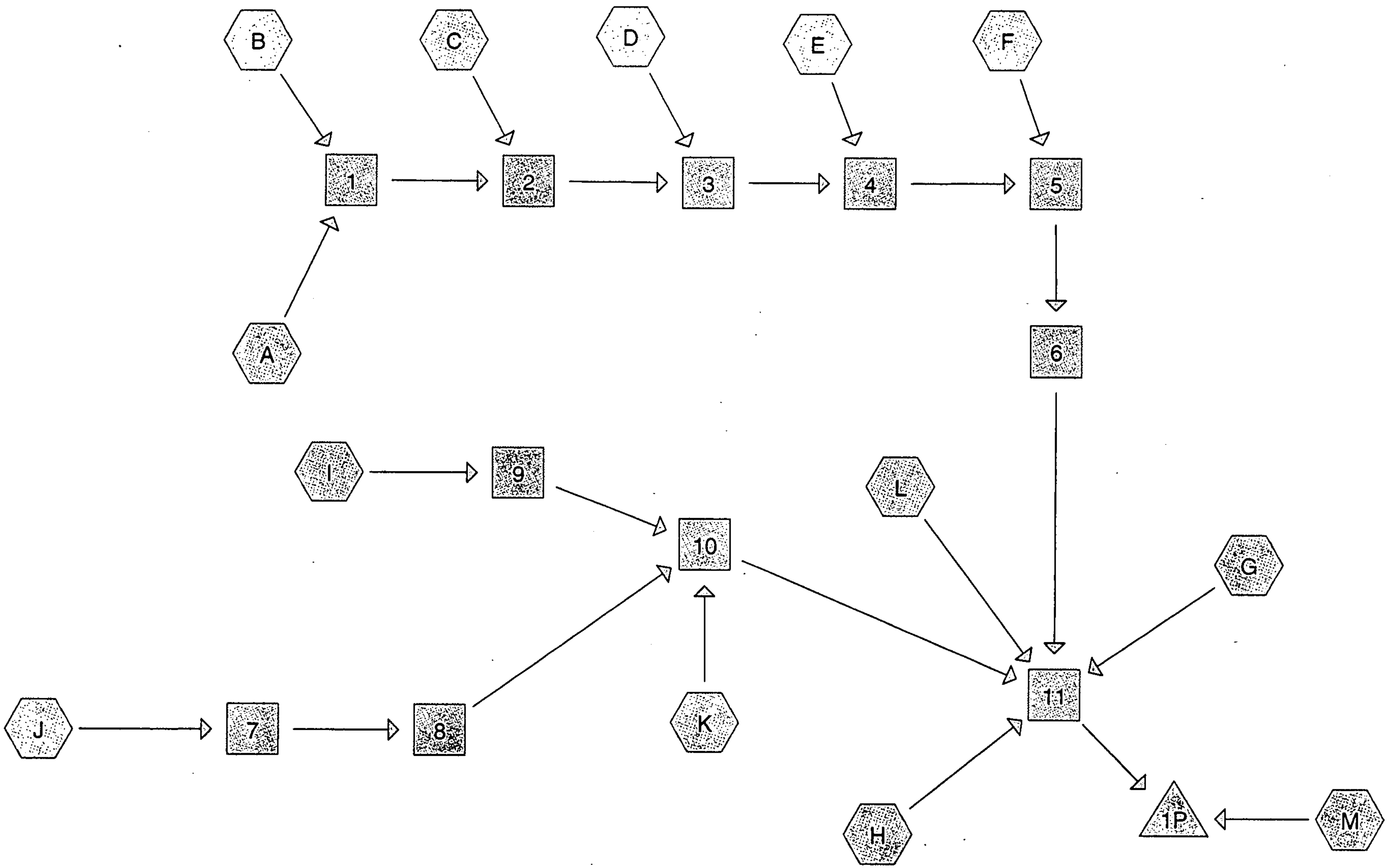
[4]Actual curve number is less than 30; use CN = 30 for runoff computations.

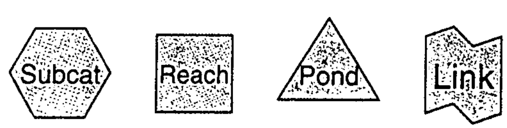
[5]CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture.

[6]*Poor:* Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair: Woods are grazed but not burned, and some forest litter covers the soil.
Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

ATTACHMENT A-8

NODAL NETWORK DIAGRAMS



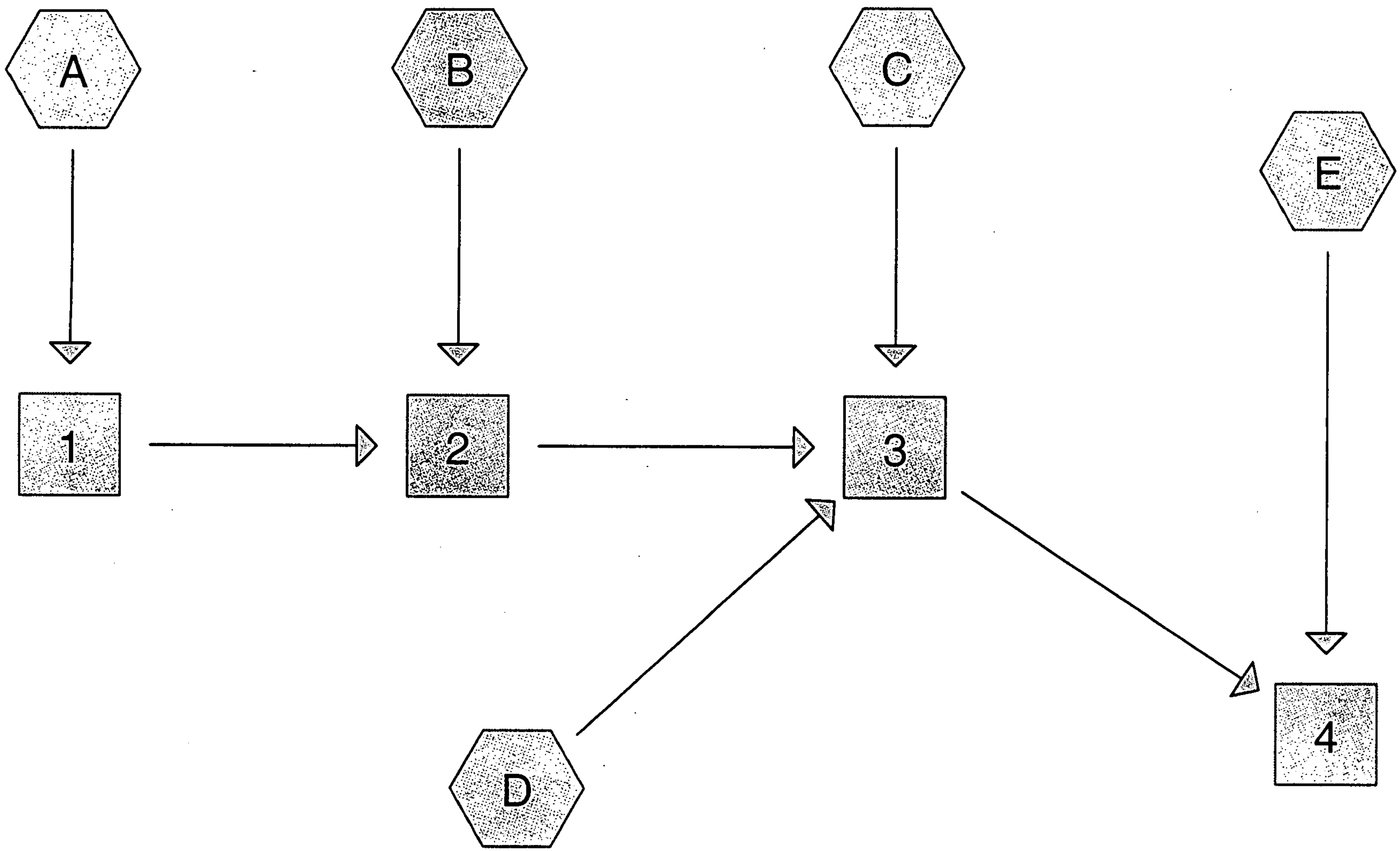


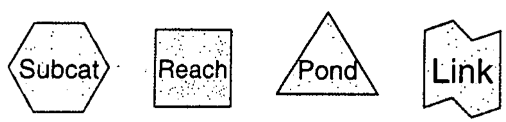
Drainage Diagram for OSDF Design Scenario
Prepared by GeoSyntec Consultants 8/23/2001
HydroCAD® 5.97 s/n 000929 © 1986-2001 Applied Microcomputer Systems

4014

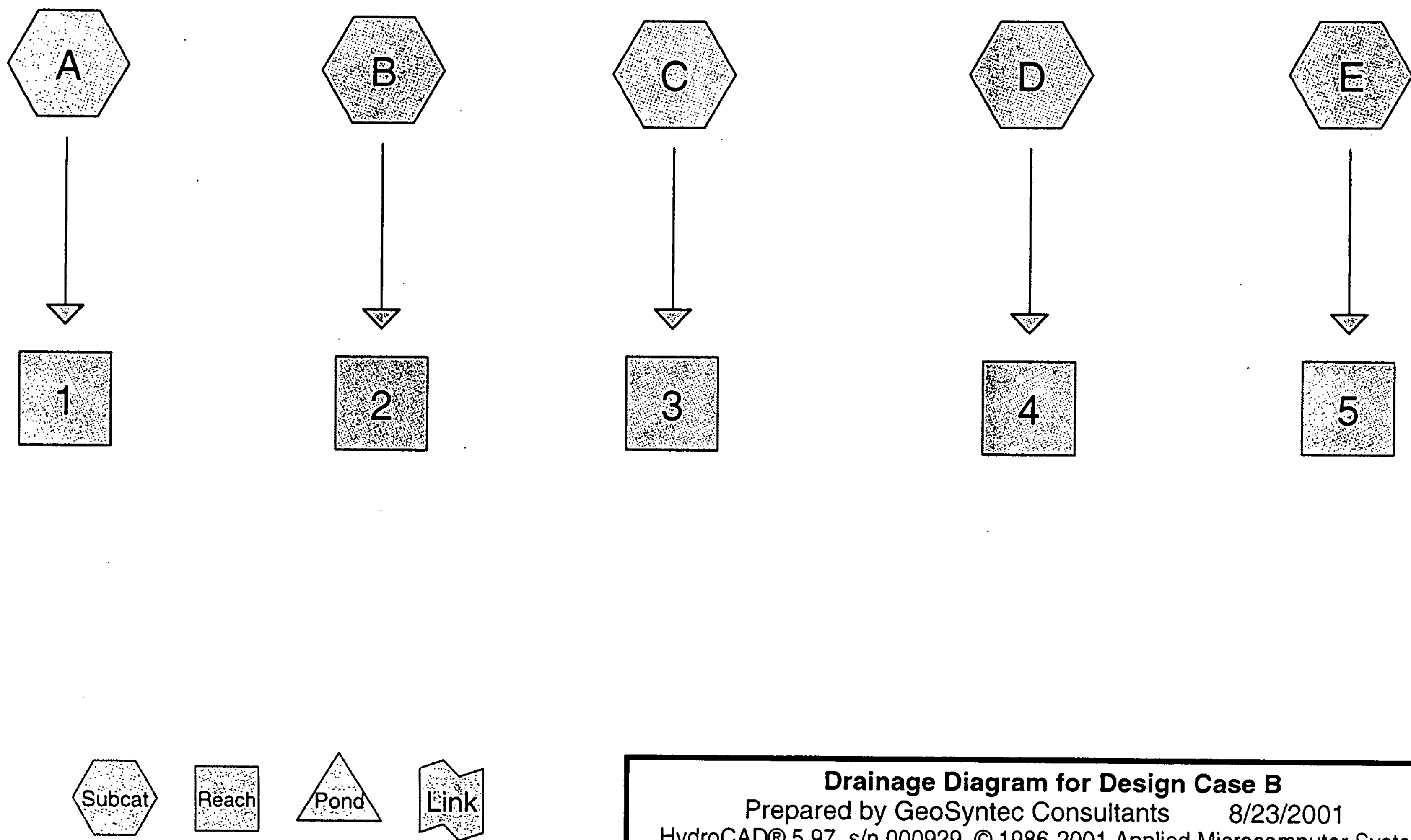
73/204

000083





Drainage Diagram for Design Case A
Prepared by GeoSyntec Consultants 8/23/2001
HydroCAD® 5.97 s/n 000929 © 1986-2001 Applied Microcomputer Systems



Drainage Diagram for Design Case B
Prepared by GeoSyntec Consultants 8/23/2001
HydroCAD® 5.97 s/n 000929 © 1986-2001 Applied Microcomputer Systems

4014

75/204

000085

ATTACHMENT A-9

CMP COVER REQUIREMENTS

CMP COVER REQUIREMENTS

General Guidelines for Minimum Cover Required for Heavy Off-Road Construction Equipment				
Pipe Span, Inches	Minimum Cover (feet) for Indicated Axle Loads (kips)			
	18-50	50-75	75-110	110-150
12-42	2.0	2.5	3.0	3.0
48-72	3.0	3.0	3.5	4.0
78-120	3.0	3.5	4.0	4.0
126-144	3.5	4.0	4.5	4.5

Information reproduced from: [Contech, 1999]

PHASE IV SURFACE-WATER MANAGEMENT SYSTEM DESIGN

DATA VERIFICATION

INTRODUCTION

As described in the Procedures Section, the purpose of this calculation package is to design the OSDF surface-water management (SWM) structures to be constructed as part of the Phase IV development of the OSDF. In addition, the adequacy of existing SWM structures to convey the 25-year, 24-hour storm event is assessed. Required modifications or additions to existing structures are incorporated into the Construction Drawings. This section presents the selection of parameters used to perform analyses in the Calculation Section of this calculation package.

HYDROLOGIC AND BASIN ROUTING ANALYSES [HydroCad™, 2001]

Subcatchment Runoff

For the OSDF Design Scenario, Design Case "A", and Design Case "B", the relationships of subcatchments and reaches are shown in the nodal network diagrams presented in Attachment A-8.

Rainfall Distribution

A SCS Type II Rainfall Distribution is selected for the Fernald site location. Attachment B-1 shows the location of the OSDF on a rainfall distribution map of the United States [SCS, 1986].

Rainfall Depth

Attachment B-2 [Parsons, 1995] provides rainfall depths for design storms of 24-hour duration for the Fernald site. The rainfall depths for use in these analyses are listed below.

Return Period (years)	Rainfall Depth (inches)
2	2.6
10	4.1
25	4.7
100	5.6

Runoff curve numbers

Subcatchment characteristics including total area and data for calculation of weighted CN are tabulated in Attachment B-3. Data includes the percentage of subcatchment area for combinations of HSG, CN, and land use for each subcatchment.

Subcatchment Time of Concentration

Subcatchment characteristics for calculation of time of concentration are tabulated in Attachment B-4. Parameters include those for sheet, shallow concentrated, and channel flow.

Reaches

Channels

For the purpose of hydrologic modeling, the Manning's roughness coefficient for all channels, both grass and riprap (permanent channels along OSDF perimeter) lined, is selected as 0.030. Similarly, representative average geometric characteristics (i.e., sideslopes, width, longitudinal slope) were selected. Data for additional channel parameters are tabulated in Attachment B-5.

Culverts

The Manning's roughness coefficient for CMP culverts is selected as 0.024.

Sedimentation Basin

Planimetered areas within contour elevation lines (for calculation of the stage-storage relationship) are presented in Attachment B-7. Coefficients for principal and emergency spillways are selected as follows and input into the HydroCAD™ models.

Principal Spillway Riser Pipes

- Discharge coefficient for orifice flow, C = 0.60

Principal Spillway Outlet Pipes

- Manning's roughness coefficient, n = 0.024 (for CMP)
- Entrance energy loss coefficient, Ke = 0.7
- Contraction coefficient, Cc = 0.9

OSDF Basin 1 Emergency Spillway

- Type, shape = broad crested weir, rectangular (approximation)
- Weir coefficient (English units), C = 3.0 [Stahre and Urbonas, 1990]

Additional data concerning the size, shapes, and elevations of the principal and emergency spillways are tabulated in Attachment B-8.

HYDRAULIC ANALYSES

Channels

Data for channel segments are obtained from the Construction Drawings and are presented in Attachment B-5. The Manning's roughness coefficient for all channels (grass-lined) is selected as 0.030 [Chow, 1959].

Culverts

Input data for culverts includes: (i) physical characteristics; (ii) CulvertMaster® modeling characteristics; and (iii) profile. Physical characteristics and profile data were obtained from the following sources: (i) as-built drawings; (ii) inspection by GeoSyntec and Fluor Fernald personnel; and (iii) existing maps showing topography and features. Available thickness of cover for each culvert is obtained from the Construction Drawings and presented along with input data for hydraulic analyses in Attachment B-6.

REFERENCES

Chow, V.T. *Open Channel Hydraulics*, McGraw-Hill, Inc., New York, 1959, 680 p.

HydroCAD. *HydroCAD™: Stormwater Modeling System, Version 5.96.* Applied Microcomputer Systems, Chocorua, New Hampshire, 2001.

Parsons. *2000-Year Peak Water and Probable Maximum Peak Water, Site-Wide Peak Water Plain Determination*, Parsons Engineering, Revision A, Aug 1995.

Stahre, P. and Urbonas, B. *Stormwater Detention*, Prentice Hall, Englewood Cliffs, New Jersey, 1990, 338 p.

U.S. Department of Agriculture (USDA-SCS). *Urban Hydrology for Small Watersheds, Technical Release 55.* 2nd ed., Soil Conservation Service Washington, D.C., 1986.

ATTACHMENT B-1

SCS RAINFALL DISTRIBUTION TYPE

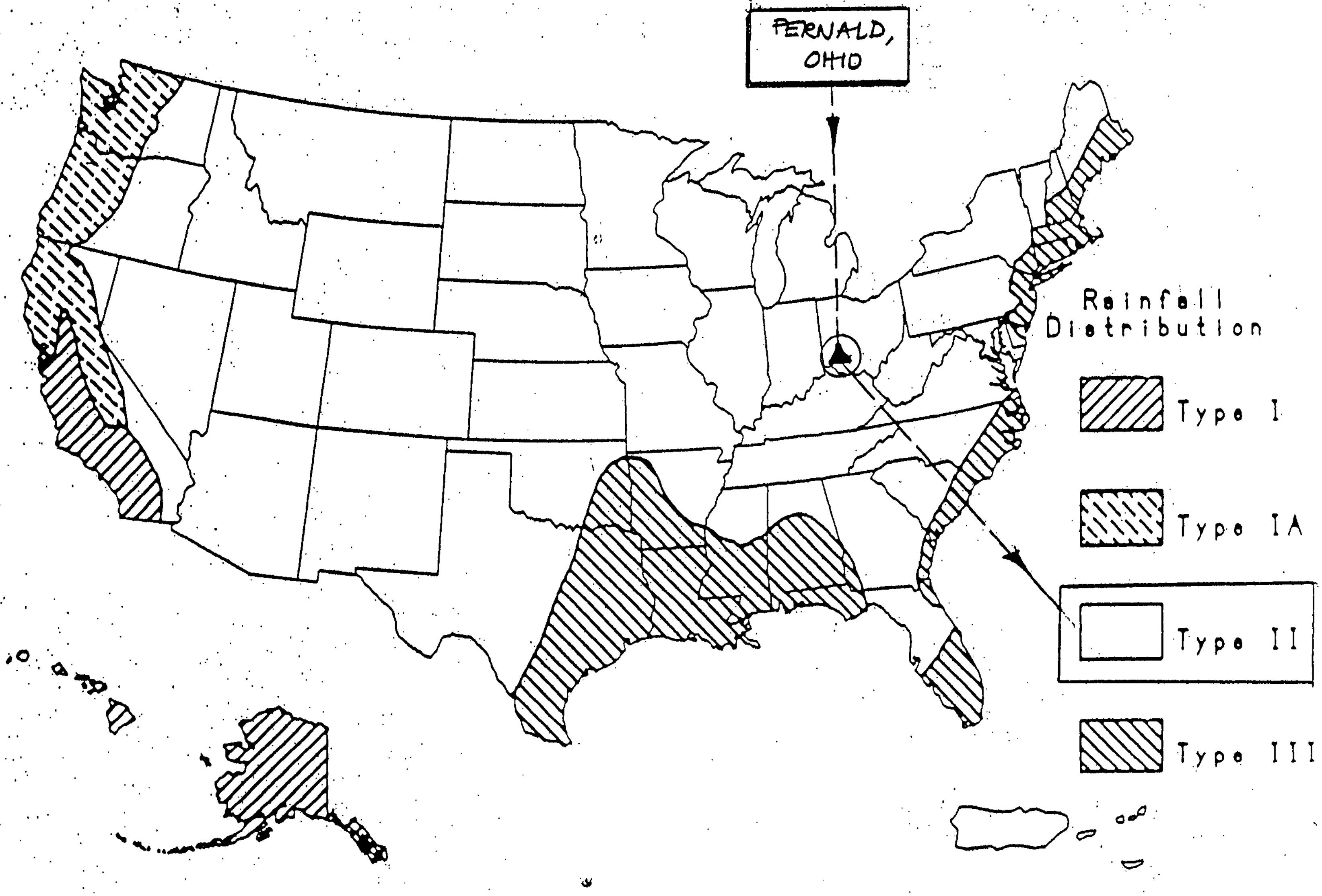


Figure B-2.—Approximate geographic boundaries for SCS rainfall distributions.

33/204

ATTACHMENT B-2

RAINFALL DEPTHS

TABLE 1 - RAINFALL DEPTH FOR A GIVEN DURATION
(INCHES)

DURATION (YEARS)	TIME							
	HYDRO-35			TP-40				
	5 MIN	15 MIN	60 MIN	2 HOURS	3 HOURS	6 HOURS	12 HOURS	24 HOURS
2	0.387	0.739	1.223	1.408	1.54	1.848	2.2	2.552
5	0.49	0.979	1.69	2.016	2.112	2.496	2.88	3.072
10	0.564	1.139	1.99	2.277	2.475	3.069	3.465	4.059
25	0.65	1.34	2.38	2.6	3.0	3.4	4.0	4.7
50	0.72	1.49	2.67	3.0	3.25	3.9	4.2	5.2
* 100	0.78	1.64	2.95	3.3	3.6	4.1	5.0	5.6
* 500	0.93	2.1	4.3	5.2	5.9	7.3	8.5	9.4
2000	1.2	2.6	5.8	7.2	8.2	10.2	12.0	13.0
10,000	1.4	3.4	8.2	10.5	12.0	15.5	18.0	19.2
100,000	1.9	5.0	13.3	17.7	20.8	27.4	31.8	33.4
	DESIGN OF SMALL DAMS					HMR-51		

NOTE: Rainfall Points for 2, 5, and 10 year rainfall events were adjusted per TP-40. Values for the 500, 2000, and 10,000 Year Events were interpolated from Figure 3 (Appendix E)

* These Values are used on PH records for HEC-1.

Reference: Parsons, "2,000-Year Flood and Probable Maximum Flood, Sitewide Flood Plan Determination", August 1995.

ATTACHMENT B-3

SUBCATCHMENT AREA AND WEIGHTED CURVE NUMBER DATA

SUBCATCHMENT AREA AND HYDROCAD™ INPUT PARAMETERS FOR THE CALCULATION OF WEIGHTED CN

OSDF Design Scenario

Subcatchment Label	Area (acres)	Percent of Total Area %	HSG	Land Use Category	CN	Weighted CN
A	2.05	100%	B/C	Vegetated Final Cover System	83	Intentionally left blank. Will be addressed in the calculation section.
B	1.10	100%	B/C	Vegetated Final Cover System	83	
C	4.56	100%	B/C	Unvegetated Final Cover System	89	
D	2.59	100%	B/C	Unvegetated Final Cover System	89	
E	1.30	100%	B/C	Unvegetated Final Cover System	89	
F	1.44	100%	B/C	Unvegetated Final Cover System	89	
G	1.75	100%	C	Disturbed Area - Construction Support	82	
H	1.78	100%	C	Disturbed Area - Construction Support	82	
I	2.16	78% 22%	B/C C	Liner Runout Disturbed Area - Construction Support	89 82	
J	0.49	100%	C	Disturbed Area - Construction Support	82	
K	0.17	100%	C	Disturbed Area - Construction Support	82	
L	0.14	100%	C	Disturbed Area - Construction Support	82	
M	1.62	100%	N/A	Sedimentation Basin	98	

N/A - Not Applicable

33,204

SUBCATCHMENT AREA AND HYDROCAD™ INPUT PARAMETERS FOR THE CALCULATION OF WEIGHTED CN

Design Case "A"

Subcatchment Label	Area (acres)	Percent of Total Area %	HSG	Land Use Category	CN	Weighted CN
A	0.63	100%	C	Runon Area East of OSDF	79	Intentionally left blank. Will be addressed in the calculation section.
B	4.00	50% 50%	B C	Runon Area East of OSDF Runon Area East of OSDF	69 79	
C	8.86	60% 40%	B C	Runon Area East of OSDF Runon Area East of OSDF	69 79	
D	0.77	100%	B/C	Unvegetated Final Cover System	89	
E	9.12	70% 30%	B C	Runon Area East of OSDF Runon Area East of OSDF	69 79	

SUBCATCHMENT AREA AND HYDROCAD™ INPUT PARAMETERS FOR THE CALCULATION OF WEIGHTED CN

Design Case "B"

Subcatchment Label	Area (acres)	Percent of Total Area %	HSG	Land Use Category	CN	Weighted CN
A	11.09	80%	B	Runon Area East of OSDF	69	Intentionally left blank. Will be addressed in the calculation section.
		20%	C	Runon Area East of OSDF	79	
B	10.61	70%	B	Runon Area East of OSDF	69	
		30%	C	Runon Area East of OSDF	79	
C	0.81	100%	C	Runon Area East of OSDF	79	
D	1.90	10%	B	Runon Area East of OSDF	69	
		90%	C	Runon Area East of OSDF	79	
E	5.61	50%	B	Runon Area East of OSDF	69	
		50%	C	Runon Area East of OSDF	79	

ATTACHMENT B-4

DATA FOR CALCULATION OF TIME OF CONCENTRATION

HYDROCAD™ INPUT PARAMETERS FOR THE CALCULATION OF TIME OF CONCENTRATION
OSDF DESIGN SCENARIO

2-year, 24-hr Design Rainfall Depth, $P_{2\text{-}24}$ = 2.60 inches

SUBCATCHMENT LABEL AND DESCRIPTION		SHEET FLOW 1				SHEET FLOW 2				SHEET FLOW 3				SHALLOW CONCENTRATED FLOW			CHANNEL FLOW 1					
No.	Description	Flow Length (ft)	Surface Description	Manning's n	Land Slope (ft/ft)	Flow Length (ft)	Surface Description	Manning's n	Land Slope (ft/ft)	Flow Length (ft)	Surface Description	Manning's n	Land Slope (ft/ft)	Flow Length (ft)	Surface Description	Land Slope (ft/ft)	Flow Length (ft)	Bottom Width (ft)	Flow Depth (ft)	Sideslopes (ft/ft)	Manning's n	Longitudinal Slope (ft/ft)
A	cell one vegetated cover	90	grass: short	0.150	0.0500	60	grass: short	0.150	0.1000	150	grass: short	0.150	0.1700	70	UNPAVED	0.1700	-	-	-	-	-	-
B	cell one vegetated cover	50	grass: short	0.150	0.1800	-	-	-	-	-	-	-	-	-	-	-	450	0	1.02	6.0, 3.0	0.030	0.0045
C	unvegetated cell two cover	90	smooth	0.011	0.0500	60	smooth	0.011	0.1000	150	smooth	0.011	0.1700	140	UNPAVED	0.1700	380	0	1.02	6.0, 4.0	0.030	0.0045
D	cell three	55	smooth	0.011	0.1540	160	smooth	0.011	0.0687	-	-	-	-	-	-	-	260	0	1.29	5.0, 3.0	0.030	0.0045
E	cell four	135	smooth	0.011	0.1148	-	-	-	-	-	-	-	-	-	-	-	320	0	1.59	4.0, 3.0	0.030	0.0050
F	cell five	130	smooth	0.011	0.0769	-	-	-	-	-	-	-	-	-	-	-	360	0	1.65	4.0, 3.0	0.030	0.0050
G	development area	120	grass: short	0.150	0.0100	100	grass: short	0.150	0.0500	20	grass: short	0.150	0.2000	-	-	-	-	-	-	-	-	-
H	development area	30	grass: short	0.150	0.0100	85	grass: short	0.150	0.0500	45	grass: short	0.150	0.2000	-	-	-	-	-	-	-	-	-
I	liner runout	70	smooth	0.011	0.1428	70	smooth	0.011	0.0142	-	-	-	-	-	-	-	580	0	2.00	3.0, 3.0	0.030	0.0100
J	development area	45	grass: short	0.150	0.5000	115	grass: short	0.150	0.0100	-	-	-	-	-	-	-	70	0	0.32	3.0, 3.0	0.030	0.0160
K	development area	110	grass: short	0.150	0.0100	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
L	development area	130	grass: short	0.150	0.0100	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
M	sedimentation basin	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

HYDROCAD™ INPUT PARAMETERS FOR THE CALCULATION OF TIME OF CONCENTRATION
DESIGN CASE "A"

2-year, 24-hr Design Rainfall Depth, $P_{2\text{-}24}$ = 2.60 inches

SUBCATCHMENT LABEL AND DESCRIPTION		SHEET FLOW 1				SHALLOW CONCENTRATED FLOW			CHANNEL FLOW 1					
No.	Description	Flow Length (ft)	Surface Description	Manning's n	Land Slope (ft/ft)	Flow Length (ft)	Surface Description	Land Slope (ft/ft)	Flow Length (ft)	Bottom Width (ft)	Flow Depth (ft)	Sideslopes (ft/ft)	Manning's n	Longitudinal Slope (ft/ft)
A	Runon East of OSDF	80	grass: short	0.150	0.0800	-	-	-	350	0	1.50	3.0, 5.0	0.030	0.0040
B	Runon East of OSDF	300	grass: short	0.150	0.0070	100	UNPAVED	0.0200	600	0	2.00	3.0, 5.0	0.030	0.0070
C	Runon East of OSDF	300	grass: short	0.150	0.0120	520	UNPAVED	0.0120	330	0	2.00	5.0, 4.0	0.030	0.0110
D	Unvegetated Final Cover	20	smooth	0.011	0.2000	-	-	-	750	0	2.00	3.0, 3.0	0.030	0.0100
E	Runon East of OSDF	300	grass: short	0.150	0.0150	1110	UNPAVED	0.0150	-	-	-	-	-	-

93204

4014

HYDROCAD™ INPUT PARAMETERS FOR THE CALCULATION OF TIME OF CONCENTRATION
DESIGN CASE "B"

2-year, 24-hr Design Rainfall Depth, $P_{2\text{-}24}$ = 2.60 inches

SUBCATCHMENT LABEL AND DESCRIPTION		SHEET FLOW 1				SHALLOW CONCENTRATED FLOW		
No.	Description	Flow Length (ft)	Surface Description	Manning's n	Land Slope (ft/ft)	Flow Length (ft)	Surface Description	Land Slope (ft/ft)
A	Runon East of OSDF	300	grass: short	0.150	0.0150	990	UNPAVED	0.0150
B	Runon East of OSDF	300	grass: short	0.150	0.0180	390	UNPAVED	0.0180
C	Runon East of OSDF	300	grass: short	0.150	0.0200	160	UNPAVED	0.0200
D	Runon East of OSDF	300	grass: short	0.150	0.0230	320	UNPAVED	0.0230
E	Runon East of OSDF	300	grass: short	0.150	0.0400	510	UNPAVED	0.0180

ATTACHMENT B-5

CHANNEL INPUT DATA

SUMMARY OF CHANNEL INPUT DATA

Channel Identification			Channel Characteristics							Hydrologic Calculations		Hydraulic Calculations							
Channel Name[1]	Status	Design Scenario	Section Shape	Available Flow Depth (ft)	Longitudinal Slope[2] (%)	Manning n	Bottom Width B (ft)	Side Slope M_1:1	Side Slope M_2:1	HydroCAD Node [3]	HydroCAD Q (cfs)	Area of Flow A (sq ft)	Perimeter P (ft)	Hydraulic Radius, R (ft)	Peak Flow Depth Y (ft)	Estimated Q (cfs)	Channel Freeboard (ft)	Peak Flow Velocity (fps)	Lining Type
G6	new	OSDF	vee	3	1.39%	0.030	0	3	3	6									
I	new	OSDF	vee	2	1.00%	0.030	0	3	3	N/A									
J	new	OSDF	vee	2	1.60%	0.030	0	3	3	N/A	Intentionally left blank. Will be addressed in the calculation section.								
8	new	OSDF	vee	3	1.60%	0.030	0	3	3	8									
9	new	OSDF	vee	4	1.47%	0.030	0	3	3	9									
10	new	OSDF	vee	3	1.47%	0.030	0	3	3	10									
1A	existing	DC A	vee	1.6	0.77%	0.030	0	6	4	1									
1B	existing	DC A	vee	2.3	0.67%	0.030	0	5	3	1									
1C	existing	DC A	vee	2	0.45%	0.030	0	4	2	1									
1D	existing	DC A	vee	2.8	0.81%	0.030	0	4	2	1									
2A	existing	DC A	vee	4	0.67%	0.030	0	6	3	2	Intentionally left blank. Will be addressed in the calculation section.								
2B	existing	DC A	vee	3	1.00%	0.030	0	4	5	2									
2C	existing	DC A	vee	4	1.50%	0.030	0	4	5	2									
3	new	DC A	vee	3	2.11%	0.030	0	3	3	3									
4	new	DC A	vee	2.5	0.52%	0.030	0	3	3	4									

1. Channels are named after the corresponding subcatchment or reach.
2. Longitudinal slopes taken from the Drawings.
3. N/A indicates that there is not specific HydroCAD node associated with the particular channel.

ATTACHMENT B-6

CULVERT LOCATIONS AND INPUT DATA

- 4014

SUMMARY OF CULVERT DESIGN PARAMETERS

CULVERT IDENTIFICATION			PHYSICAL CHARACTERISTICS OF CULVERT						CULVERT PROFILE				CULVERTMASTER® MODELING CHARACTERISTICS			HYDRAULIC CAPACITY					STRUCTURAL CAPACITY				OUTLET PROTECTION				
Culvert Name	Status (1)	Design Scenario	Material - Type	Entrance Loss Coefficient (Ke)	Manning's n	Entrance Configuration (2)	Number of Culverts - Diameter	Approximate Length (ft)	Inlet Invert Elevation (ft MSL)	Outlet Invert Elevation (ft MSL)	Slope ft/ft	Overtopping Elevation (ft MSL)	Entrance Configuration (2)	Entrance Loss Coefficient (Ke)	Number of Culverts - Diameter (3)	HydroCad Node for Peak Flow Rate (cfs)	Calculated Freeboard (ft)	Tailwater Elevation (ft MSL)	Calculated Headwater Depth - Inlet Control (ft MSL)	Calculated Headwater Depth - Outlet Control (ft MSL)	Available Cover (ft)	Design Traffic Type	Minimum Required Cover (ft)	Structurally Stable	Outlet Velocity (ft/s)	Riprap Length at Inlet (ft)	Riprap Length at Outlet (ft)	d_{50} (in)	Thickness (in)
1	new	OSDF	CMP	0.9	0.024	Projecting	1 - 36 inch	96.1	594.00	592.66	0.014	597.85	Projecting	0.9	1 - 36 inch	5													
2	new (4)	OSDF	CMP	0.9	0.024	Projecting	2 - 42 inch	90.0	583.14	581.89	0.014	587.00	Projecting	0.9	2 - 42 inch	11	Intentionally left blank. Will be addressed in the calculation section.												
3	new	OSDF	CMP	0.9	0.024	Projecting	1 - 12 inch	74.0	586.50	585.32	0.016	588.00	Projecting	0.9	1 - 12 inch	7													
4	new	OSDF	CMP	0.9	0.024	Projecting	1 - 36 inch	95.5	589.22	587.89	0.014	593.50	Projecting	0.9	1 - 36 inch	N/A													
5	new	OSDF	CMP	0.9	0.024	Projecting	1 - 42 inch	55.0	584.39	583.63	0.014	590.00	Projecting	0.9	1 - 42 inch	N/A													
1	new	DC A	CMP	0.9	0.024	Projecting	1 - 36 inch	70.0	600.00	598.50	0.021	605	Projecting	0.9	1 - 36 inch	N/A	Intentionally left blank. Will be addressed in the calculation section.												
1	new	DC B	CMP	0.9	0.024	Projecting	1 - 24 inch	50.0	598.00	597.50	0.010	602.00	Projecting	0.9	1 - 24 inch	1													
2	new	DC B	CMP	0.9	0.024	Projecting	1 - 24 inch	50.0	596.00	595.50	0.010	600.00	Projecting	0.9	1 - 24 inch	2	Intentionally left blank. Will be addressed in the calculation section.												
3	new	DC B	CMP	0.9	0.024	Projecting	1 - 12 inch	50.0	590.50	590.00	0.010	593.50	Projecting	0.9	1 - 12 inch	3													
4	new	DC B	CMP	0.9	0.024	Projecting	1 - 18 inch	50.0	592.50	592.00	0.010	595.75	Projecting	0.9	1 - 18 inch	4													
5	new	DC B	CMP	0.9	0.024	Projecting	1 - 24 inch	50.0	599.00	598.50	0.010	603.00	Projecting	0.9	1 - 24 inch	5													

- OSDF = OSDF Design Scenario
- DC A = Design Case "A"
- DC B = Design Case "B"
- CMP = Corrugated Metal Pipe(s)
- N/A = Not Applicable
- d_{50} = Average particle diameter

Notes
(1) New indicates a new culvert to be installed.
(2) Entrance configuration assumed.
(3) Dimensions and entrance configurations used in the Culvertmaster software package were selected to match existing culvert characteristics as closely as possible.
(4) Culvert 2 is not installed as a part of the Phase IV work.

ATTACHMENT B-7

BASIN ELEVATION CONTOURS DATA

STAGE-STORAGE INPUT DATA FOR SEDIMENTATION BASIN

OSDF BASIN 1	
Elevation (ft MSL)	Planimetered Area (acres)
581.0	0.148
582.0	0.972
583.0	1.124
584.0	1.217
585.0	1.318
586.0	1.407
587.0	1.494
588.0	1.563
-	-

ATTACHMENT B-8

BASIN OUTLET STRUCTURE DATA

OSDF BASIN 1 ROUTING INPUT DATA

ITEM		OSDF BASIN 1		
		DESCRIPTION	PARAMETER VALUE IN UNITS SHOWN	
PRINCIPAL SPILLWAY	Riser Pipe	Twin 48" CMP	ELEVATION (ft MSL)	585.75
PRINCIPAL SPILLWAY	Outlet Pipe	Twin 36" CMP	ELEVATION (ft MSL)	580.0
EMERGENCY SPILLWAY		30' wide trapezoidal channel	ELEVATION (ft MSL)	586.5
EMBANKMENT		40' wide top width	ELEVATION (ft MSL)	588.0
AVAILABLE STORAGE VOLUME (TO RISER INLET)		To primary spillway riser pipe inlet	VOLUME (acre-ft)	5.0
SUBCATCHMENT		Drainage area	AREA (acres)	21.15

000111

4014

PHASE IV SURFACE-WATER MANAGEMENT SYSTEM DESIGN

CALCULATIONS

INTRODUCTION

As described in the Procedures Section, the purpose of this calculation package is to design the OSDF surface-water management (SWM) structures to be constructed as part of the Phase IV development of the OSDF. In addition, the adequacy of existing SWM structures is assessed. Required modifications or additions to existing structures are incorporated into the Construction Drawings and this calculation package. This section presents calculations based on procedures and data presented in the Procedures and Data Verification Sections of this calculation package.

HYDROLOGIC AND BASIN ROUTING ANALYSES

Hydrologic analyses are completed for the OSDF Design Scenario, Design Case "A", and Design Case "B", and a nodal network is prepared. For the OSDF Design Scenario nodal network, HydroCAD™ runs are performed for the 10-year, 25-year, and 100-year, 24-hour storm events. For the Design Case "A" nodal network, a HydroCAD™ run is performed for the 25-year, 24-hour storm event. For the Design Case "B" nodal network, a HydroCAD™ run is performed for the 25-year, 24-hour storm event. In total, 5 runs are performed. HydroCAD™ output reports for these runs are presented in Attachments C-1A, C-1B, and C-1C for the OSDF Design Scenario , Design Case "A", and Design Case "B", respectively.

Subcatchment Runoff

Runoff curve numbers

Weighted runoff curve numbers are calculated using a spreadsheet. Results are presented in Attachment C-2. An example calculation is provided in the same attachment. The calculated weighted runoff curve numbers are input directly to HydroCAD™.

HYDRAULIC ANALYSES

Calculation parameters obtained from HydroCAD™ for use in design include the following.

- For channels: 25-year, 24-hour peak flow rates (cfs).
- For culverts: 25-year, 24-hour peak flow rates (cfs), and 25-year, 24-hour peak water elevation (ft MSL) at the basin into which a culvert discharges.
- For the OSDF Basin 1: 25-year and 100-year, 24-hour peak water elevations (ft MSL), and 10-year, 24-hour runoff volumes (acre-ft).

Channels

Hydraulic Capacity

For each new channel, peak flow depths are calculated using a computer spreadsheet. The peak flow depth is subtracted from the minimum available depth (from Construction Drawings) to obtain the minimum available freeboard. A computer spreadsheet is presented in Attachment C-3 that includes minimum available freeboard and example calculations for the computations performed in the spreadsheets. For new channels, the minimum available freeboard is equal or greater than 0.5 ft.

Lining

For each new channel, peak flow velocity is calculated using a computer spreadsheet. Peak flow velocities and channel linings are presented for each new channel, in Attachment C-3. All new channels are grass-lined channels and have maximum flow velocities less than 5 fps.

Culverts

Hydraulic Capacity

For each culvert, peak flow rates are obtained from HydroCAD™ output. Based on peak flow rates and culvert input data, CulvertMaster® is used to calculate headwater elevations for inlet and outlet control conditions. CulvertMaster® output summary sheets are presented in

Attachment C-4A. Calculated headwater elevations and overtopping elevations are tabulated and presented in Attachment C-4B.

For all new culverts, the overtopping elevation exceeds the maximum calculated headwater elevation by at least 0.5 feet.

Outlet Protection

For each new culvert, recommendations for outlet protection, based on guidelines described in the Procedure Section of this calculation package, are presented in Attachment C-4B.

Structural Stability

For each new CMP culvert, minimum required and available cover are tabulated and presented in Attachment C-4B. For all four new CMP culverts, the available cover exceeds the minimum required. Calculations are provided in Attachment C-4B.

Sedimentation Basin

OSDF Basin 1

The required storage volume based on the 10-year, 24-hour storm event is obtained from HydroCAD™ output. In addition, the required storage based on total upstream drainage area is calculated (an example calculation is provided in Attachment C-5). The available storage volume exceeds both of the required storage volumes. Analysis results are tabulated and presented in Attachment C-5.

The peak water elevations for the 25-year and 100-year storm events are obtained from HydroCAD™ output. The peak water elevation for the 25-year storm event is below the elevation of the emergency spillway, and the peak water elevation for the 100-year storm allows for more than 1 ft of freeboard to the minimum embankment crest elevation. Analysis results are tabulated and presented in Attachment C-5.

ATTACHMENT C

HydroCAD^{TM} OUTPUT REPORTS

ATTACHMENT C-1A

HydroCAD™ OUTPUT REPORTS

OSDF DESIGN SCENARIO

OSDF DESIGN SCENARIO
10-YEAR, 24-HOUR STORM EVENT

Pond 1P: sedimentation basin 1

Inflow	=	40.69 cfs @ 12.02 hrs,	Volume=	4.834 af
Outflow	=	0.00 cfs @ 0.00 hrs,	Volume=	0.000 af, Atten= 100%, Lag= 0.0 min
Primary	=	0.00 cfs @ 0.00 hrs,	Volume=	0.000 af

Routing by Stor-Ind method, Time Span= 0.00-24.00 hrs, dt= 0.10 hrs

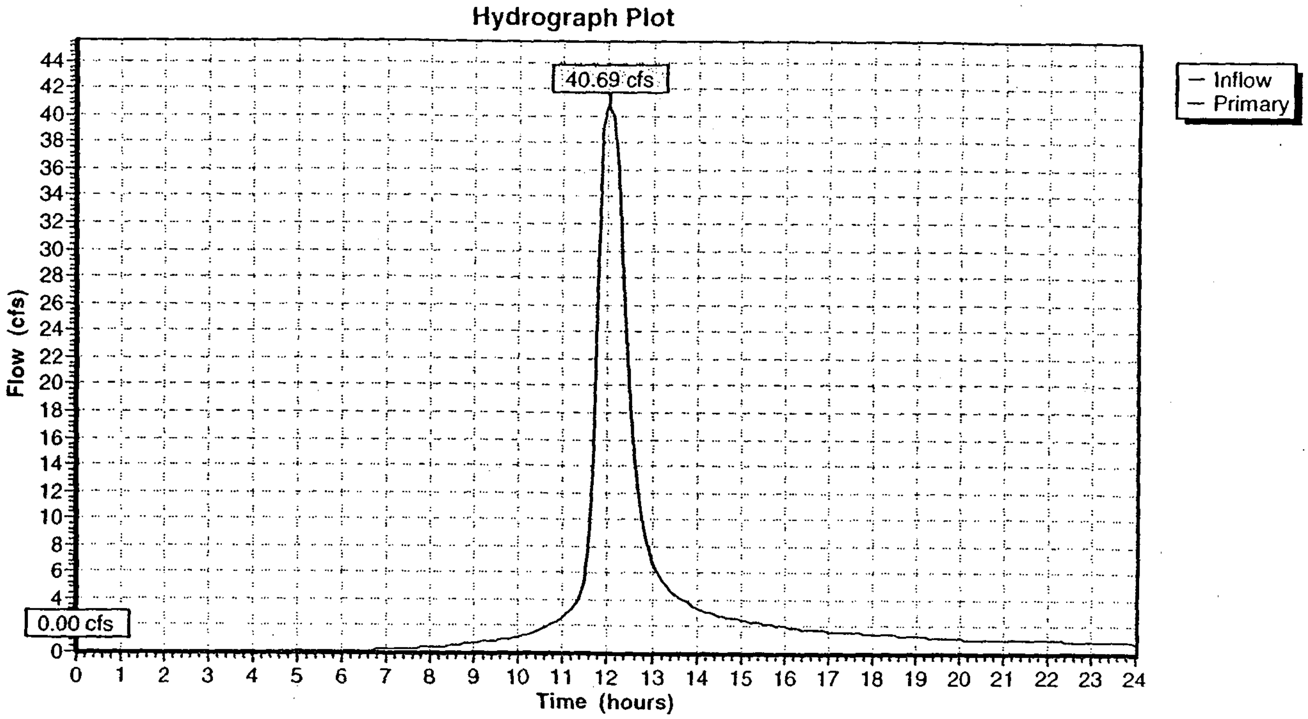
Peak Elev= 585.62' Storage= 4.831 af
Plug-Flow detention time= (not calculated)
Storage and wetted areas determined by Conic sections

Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)	Wet.Area (acres)
581.00	0.148	0.000	0.000	0.148
582.00	0.972	0.500	0.500	0.972
583.00	1.124	1.047	1.547	1.125
584.00	1.217	1.170	2.717	1.220
585.00	1.318	1.267	3.984	1.323
586.00	1.407	1.362	5.346	1.414
587.00	1.494	1.450	6.797	1.503
588.00	1.563	1.528	8.325	1.575

Primary OutFlow (Free Discharge)
- 2=Culvert
  - 1=Orifice/Grate
- 3=Broad-Crested Rectangular Weir

#	Routing	Invert	Outlet Devices
1	Device 2	585.75'	**48.0" Horiz. Orifice/Grate X 2.00** Limited to weir flow C= 0.600
2	Primary	580.00'	**36.0" x 61.0' long Culvert X 2.00** Ke= 0.700 Outlet Invert= 578.86' S= 0.0187 '/' n= 0.024 Cc= 0.900
3	Primary	586.50'	**30.0' long Broad-Crested Rectangular Weir** Head (feet) 0.50 1.00 1.50 2.00 2.50 3.00 4.00 5.00 Coef. (English) 3.00 3.00 3.00 3.00 3.00 3.00 3.00 3.00

Pond 1P: sedimentation basin 1



OSDF DESIGN SCENARIO
25-YEAR, 24-HOUR STORM EVENT

Time span=0.00-24.00 hrs, dt=0.10 hrs, 241 points
Runoff by SCS TR-20 method, UH=SCS, Type II 24-hr Rainfall=4.70"
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment A: vegetated cover of cell one
Tc=17.3 min CN=83 Area=2.049 ac Runoff= 6.85 cfs 0.495 af

Subcatchment B: vegetated cover of cell one
Tc=6.2 min CN=83 Area=1.096 ac Runoff= 5.05 cfs 0.265 af

Subcatchment C: unvegetated cover of cell 2
Tc=5.6 min CN=89 Area=4.562 ac Runoff= 25.22 cfs 1.325 af

Subcatchment D: unvegetated final cover system
Tc=3.4 min CN=89 Area=2.590 ac Runoff= 15.18 cfs 0.752 af

Subcatchment E: unvegetated final cover system
Tc=2.6 min CN=89 Area=1.295 ac Runoff= 7.55 cfs 0.376 af

Subcatchment F: unvegetated final cover system
Tc=3.0 min CN=89 Area=1.437 ac Runoff= 8.42 cfs 0.417 af

Subcatchment G: construction support area
Tc=25.3 min CN=82 Area=1.751 ac Runoff= 4.70 cfs 0.408 af

Subcatchment H: construction support area
Tc=14.4 min CN=82 Area=1.780 ac Runoff= 6.37 cfs 0.416 af

Subcatchment I: liner runout/construction support area
Tc=3.7 min CN=87 Area=2.164 ac Runoff= 12.11 cfs 0.592 af

Subcatchment J: construction support area
Tc=18.3 min CN=82 Area=0.491 ac Runoff= 1.55 cfs 0.115 af

Subcatchment K: construction support area
Tc=15.5 min CN=82 Area=0.171 ac Runoff= 0.59 cfs 0.040 af

Subcatchment L: construction support area
Tc=17.7 min CN=82 Area=0.142 ac Runoff= 0.45 cfs 0.033 af

Subcatchment M: direct runon into pond
Tc=1.0 min CN=98 Area=1.623 ac Runoff= 9.98 cfs 0.604 af

Reach 1: east channel Inflow= 10.08 cfs 0.760 af
Length= 400.0' Max Vel= 2.0 fps Capacity= 754.76 cfs Outflow= 9.31 cfs 0.757 af

Reach 2: east channel Inflow= 30.65 cfs 2.082 af
Length= 575.0' Max Vel= 2.8 fps Capacity= 1,206.56 cfs Outflow= 25.80 cfs 2.075 af

Reach 3: east channel Inflow= 33.88 cfs 2.828 af
Length= 375.0' Max Vel= 3.1 fps Capacity= 1,565.76 cfs Outflow= 31.60 cfs 2.823 af

Reach 4: east channel Inflow= 33.67 cfs 3.199 af
Length= 475.0' Max Vel= 3.1 fps Capacity= 1,565.32 cfs Outflow= 32.55 cfs 3.192 af

Reach 5: CMP culvert Inflow= 34.14 cfs 3.609 af
Length= 96.1' Max Vel= 5.4 fps Capacity= 541.77 cfs Outflow= 34.26 cfs 3.608 af

Reach 6: south east channel Inflow= 34.26 cfs 3.608 af
Length= 640.0' Max Vel= 4.7 fps Capacity= 199.53 cfs Outflow= 33.64 cfs 3.601 af

Reach 7: CMP culvert Inflow= 1.55 cfs 0.115 af
Length= 74.0' Max Vel= 2.2 fps Capacity= 579.36 cfs Outflow= 1.53 cfs 0.115 af

Reach 8: channel Inflow= 1.53 cfs 0.115 af
Length= 90.0' Max Vel= 2.3 fps Capacity= 214.02 cfs Outflow= 1.51 cfs 0.115 af

Reach 9: channel Inflow= 12.11 cfs 0.592 af
Length= 60.0' Max Vel= 3.7 fps Capacity= 441.30 cfs Outflow= 11.80 cfs 0.592 af

Reach 10: channel Inflow= 12.84 cfs 0.747 af
Length= 60.0' Max Vel= 3.8 fps Capacity= 204.91 cfs Outflow= 12.51 cfs 0.747 af

Reach 11: CMP culvert Inflow= 49.39 cfs 5.205 af
Length= 90.0' Max Vel= 5.1 fps Capacity= 432.56 cfs Outflow= 49.32 cfs 5.204 af

Pond 1P: sedimentation basin 1 Peak Storage= 5.100 af Inflow= 50.18 cfs 5.808 af
Primary= 1.59 cfs 0.736 af Outflow= 1.59 cfs 0.736 af

Subcatchment A; vegetated cover of cell one

Runoff = 6.85 cfs @ 12.04 hrs, Volume= 0.495 af

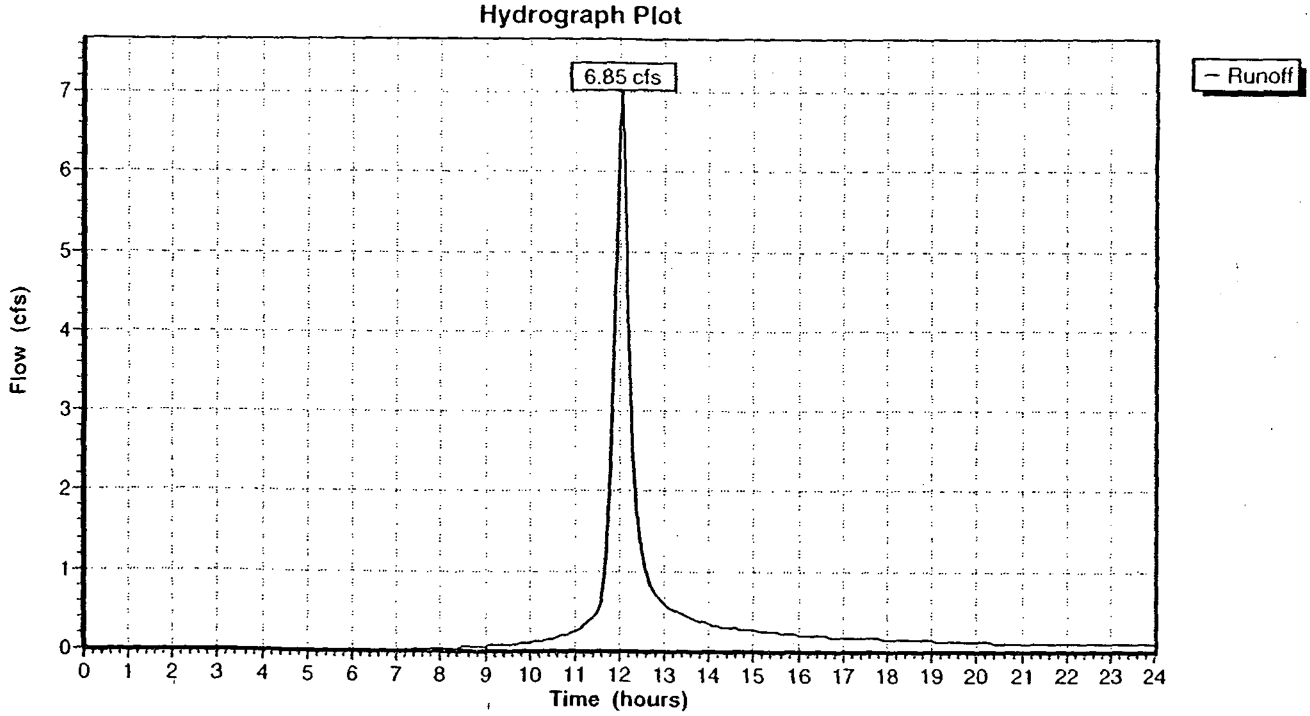
Runoff by SCS TR-20 method, UH=SCS, Time Span= 0.00-24.00 hrs, dt= 0.10 hrs
Type II 24-hr Rainfall=4.70"

Area (ac)	CN	Description
2.049	83	

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.9	90	0.0500	0.2		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.60"
3.8	60	0.1000	0.3		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.60"
6.4	150	0.1700	0.4		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.60"
0.2	70	0.1700	6.6		**Shallow Concentrated Flow,** Unpaved Kv= 16.1 fps
17.3	370	Total			

Subcatchment A: vegetated cover of cell one

Hydrograph Plot



Subcatchment B: vegetated cover of cell one

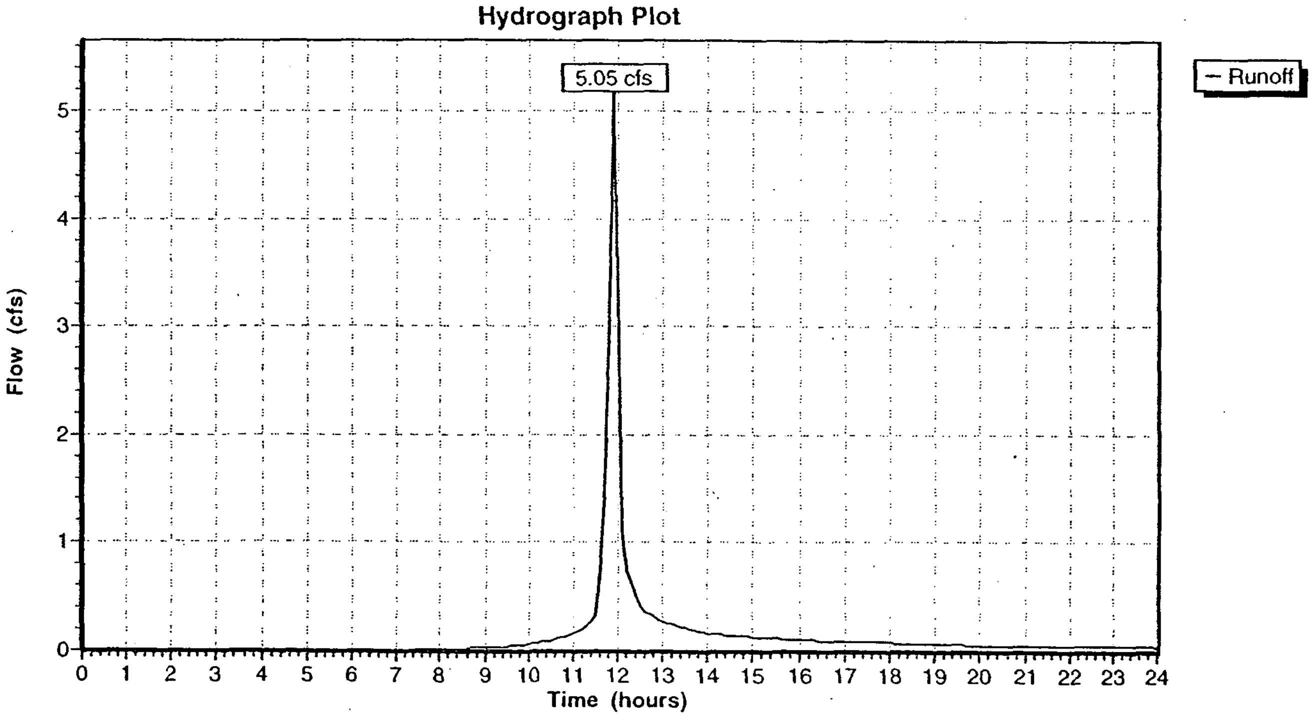
Runoff = 5.05 cfs @ 11.91 hrs, Volume= 0.265 af

Runoff by SCS TR-20 method, UH=SCS, Time Span= 0.00-24.00 hrs, dt= 0.10 hrs
Type II 24-hr Rainfall=4.70"

Area (ac)	CN	Description
1.096	83	

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
2.6	50	0.1800	0.3		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.60"
3.6	450	0.0045	2.1	9.75	**Trap/Vee/Rect Channel Flow,** Bot.W=0.00' D=1.02' Z= 6.0 & 3.0 '/' n= 0.030
6.2	500	Total			

Subcatchment B: vegetated cover of cell one



Subcatchment C: unvegetated cover of cell 2

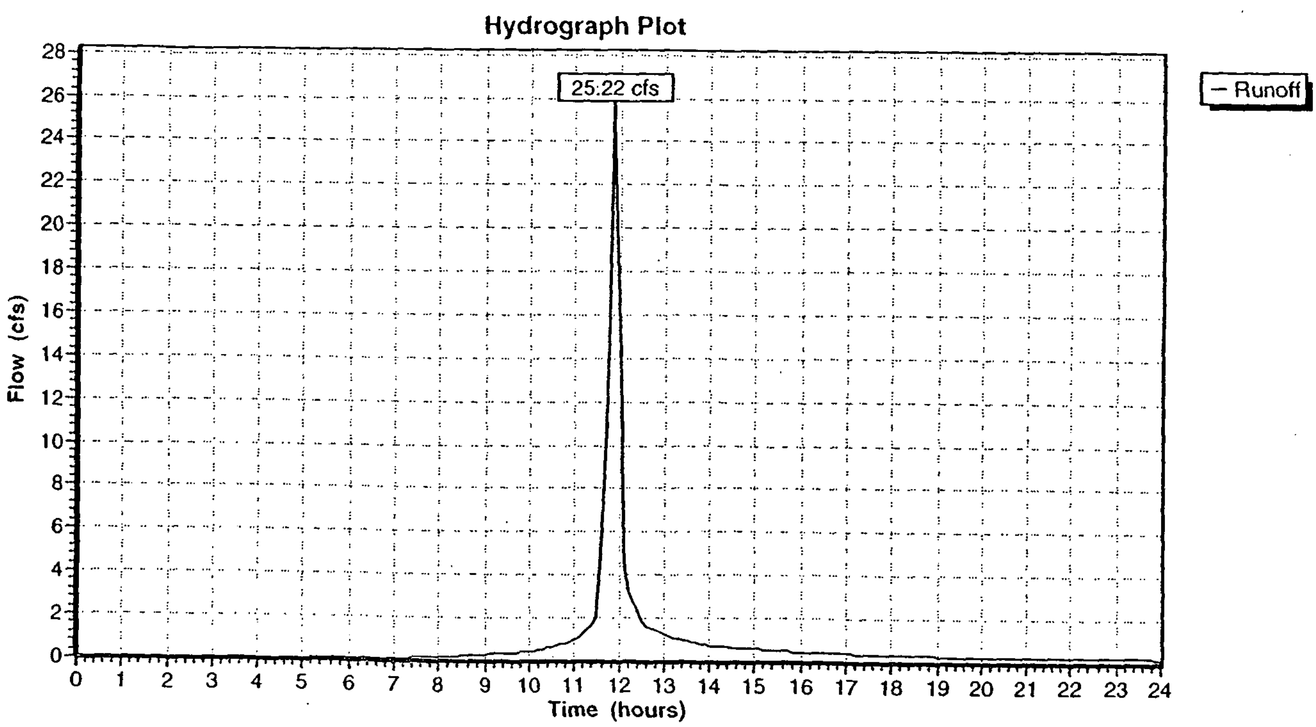
Runoff = 25.22 cfs @ 11.90 hrs, Volume= 1.325 af

Runoff by SCS TR-20 method, UH=SCS, Time Span= 0.00-24.00 hrs, dt= 0.10 hrs
Type II 24-hr Rainfall=4.70"

Area (ac)	CN	Description
4.562	89	

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
0.9	90	0.0500	1.8		**Sheet Flow,** Smooth surfaces n= 0.011 P2= 2.60"
0.5	60	0.1000	2.1		**Sheet Flow,** Smooth surfaces n= 0.011 P2= 2.60"
0.8	150	0.1700	3.2		**Sheet Flow,** Smooth surfaces n= 0.011 P2= 2.60"
0.4	140	0.1700	6.6		**Shallow Concentrated Flow,** Unpaved Kv= 16.1 fps
3.0	380	0.0045	2.1	10.88	**Trap/Vee/Rect Channel Flow,** Bot.W=0.00' D=1.02' Z= 6.0 & 4.0 '/' n= 0.030
5.6	820	Total			

Subcatchment C: unvegetated cover of cell 2



Subcatchment D: unvegetated final cover system

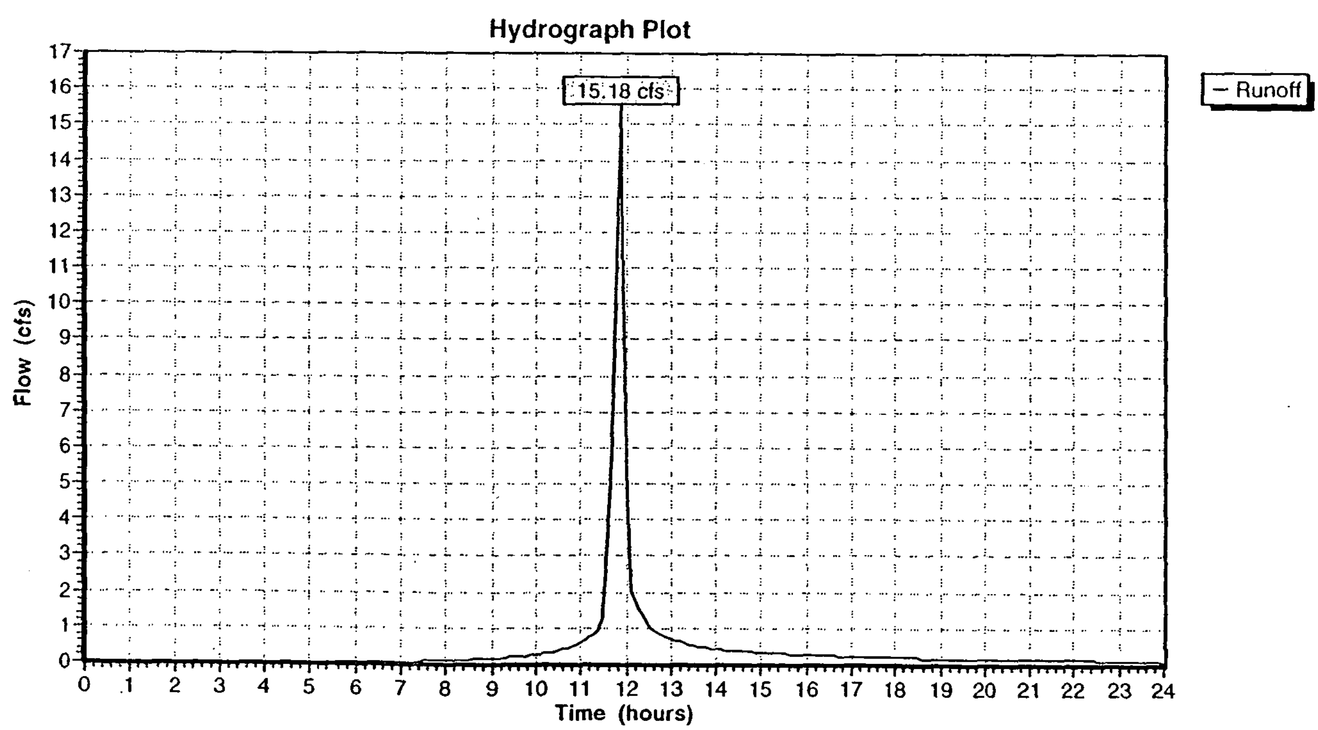
Runoff = 15.18 cfs @ 11.88 hrs, Volume= 0.752 af

Runoff by SCS TR-20 method, UH=SCS, Time Span= 0.00-24.00 hrs, dt= 0.10 hrs
Type II 24-hr Rainfall=4.70"

Area (ac)	CN	Description
2.590	89	

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
0.4	55	0.1540	2.5		**Sheet Flow,** Smooth surfaces n= 0.011 P2= 2.60"
1.2	160	0.0687	2.2		**Sheet Flow,** Smooth surfaces n= 0.011 P2= 2.60"
1.8	260	0.0045	2.4	16.16	**Trap/Vee/Rect Channel Flow,** Bot.W=0.00' D=1.29' Z= 5.0 & 3.0 '/' n= 0.030
3.4	475	Total			

Subcatchment D: unvegetated final cover system



Subcatchment E: unvegetated final cover system

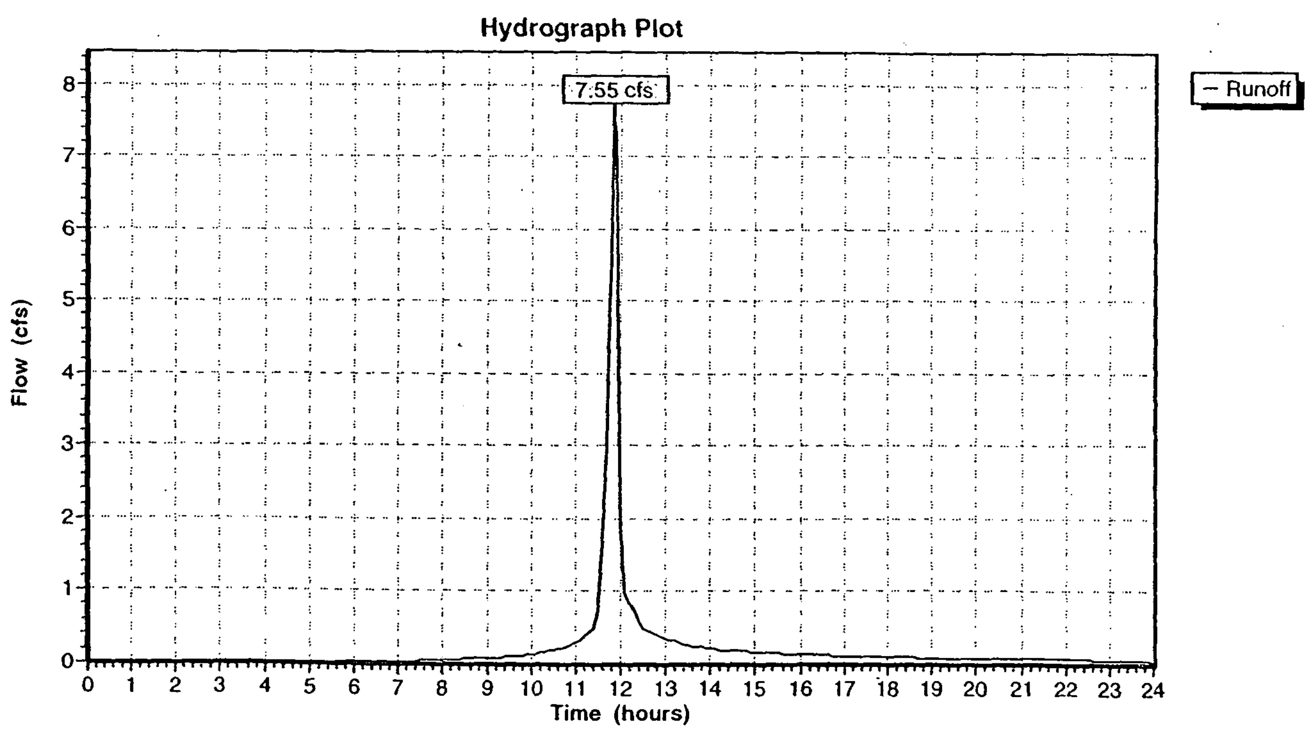
Runoff = 7.55 cfs @ 11.87 hrs, Volume= 0.376 af

Runoff by SCS TR-20 method, UH=SCS, Time Span= 0.00-24.00 hrs, dt= 0.10 hrs
Type II 24-hr Rainfall=4.70"

Area (ac)	CN	Description
1.295	89	

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
0.8	135	0.1148	2.6		**Sheet Flow,** Smooth surfaces n= 0.011 P2= 2.60"
1.8	320	0.0050	2.9	25.90	**Trap/Vee/Rect Channel Flow,** Bot.W=0.00' D=1.59' Z= 4.0 & 3.0 '/' n= 0.030
2.6	455	Total			

Subcatchment E: unvegetated final cover system



Subcatchment F: unvegetated final cover system

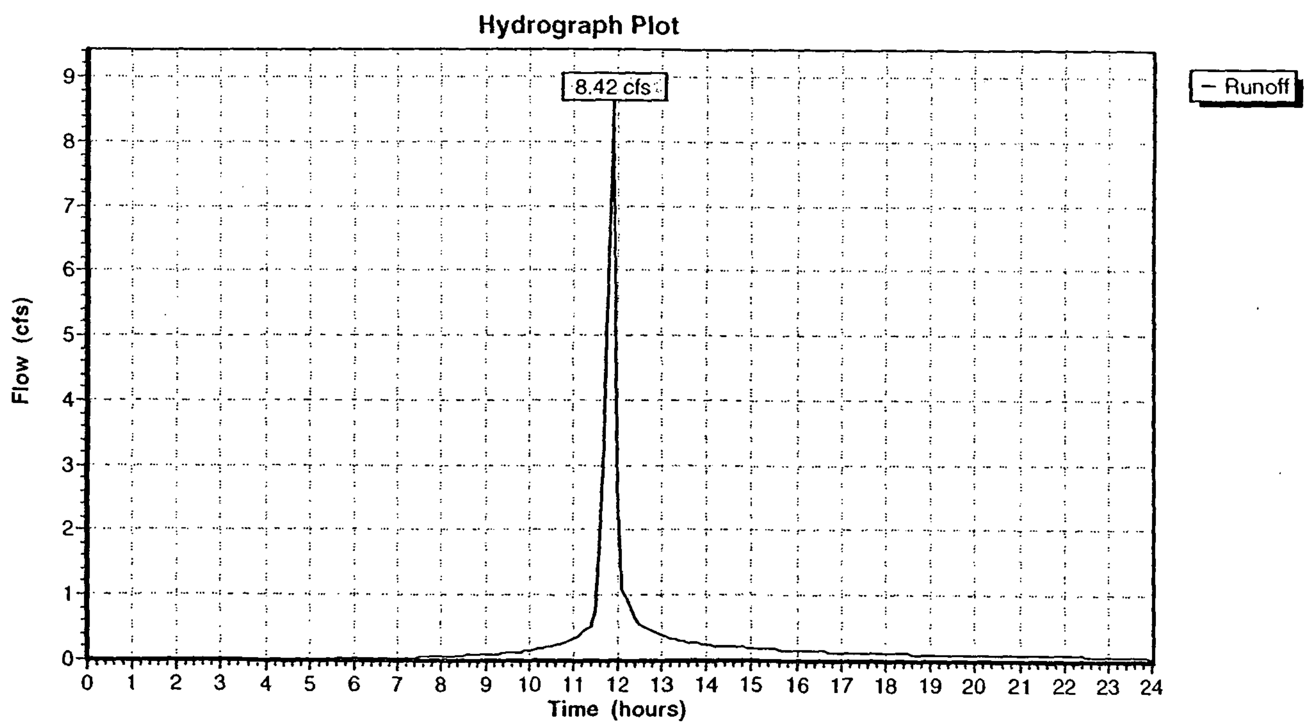
Runoff = 8.42 cfs @ 11.88 hrs, Volume= 0.417 af

Runoff by SCS TR-20 method, UH=SCS, Time Span= 0.00-24.00 hrs, dt= 0.10 hrs
Type II 24-hr Rainfall=4.70"

Area (ac)	CN	Description
1.437	89	

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
1.0	130	0.0769	2.2		**Sheet Flow,** Smooth surfaces n= 0.011 P2= 2.60"
2.0	360	0.0050	3.0	28.59	**Trap/Vee/Rect Channel Flow,** Bot.W=0.00' D=1.65' Z= 4.0 & 3.0 '/' n= 0.030
3.0	490	Total			

Subcatchment F: unvegetated final cover system



Subcatchment G: construction support area

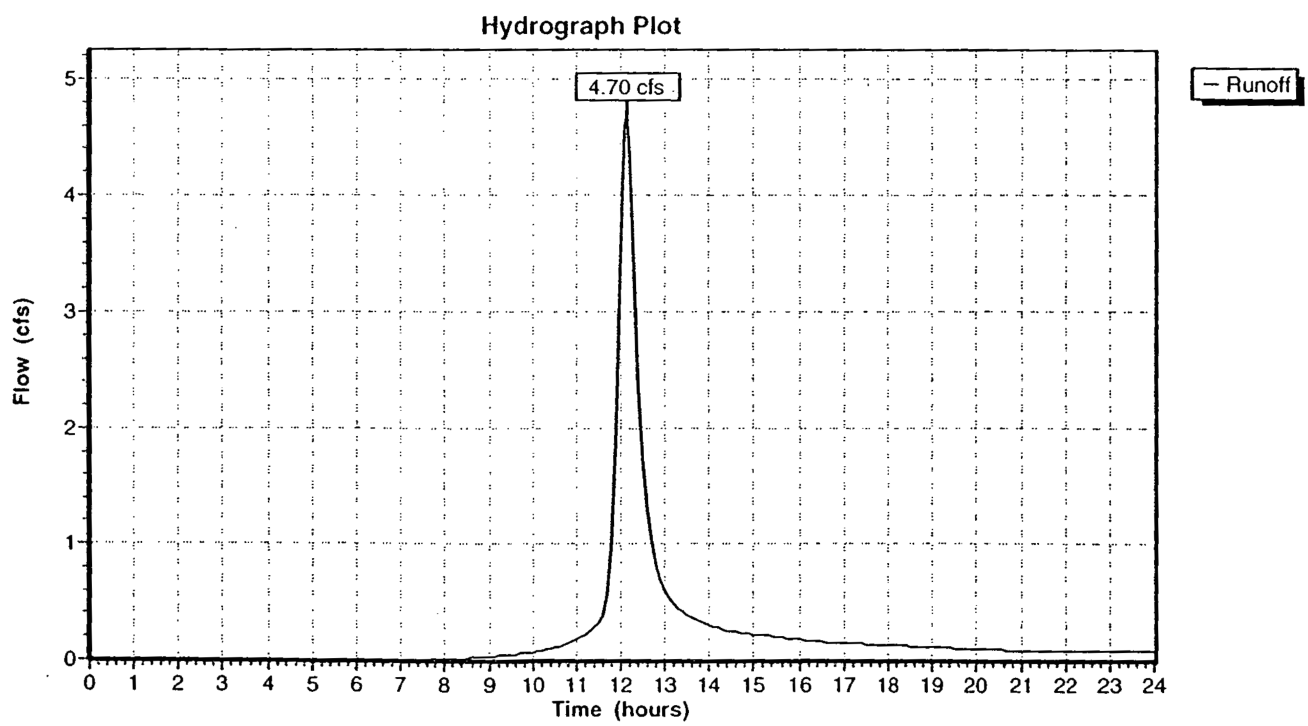
Runoff = 4.70 cfs @ 12.14 hrs, Volume= 0.408 af

Runoff by SCS TR-20 method, UH=SCS, Time Span= 0.00-24.00 hrs, dt= 0.10 hrs
Type II 24-hr Rainfall=4.70"

Area (ac)	CN	Description
1.751	82	

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
16.6	120	0.0100	0.1		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.60"
7.5	100	0.0500	0.2		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.60"
1.2	20	0.2000	0.3		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.60"
25.3	240	Total			

Subcatchment G: construction support area



Subcatchment H: construction support area

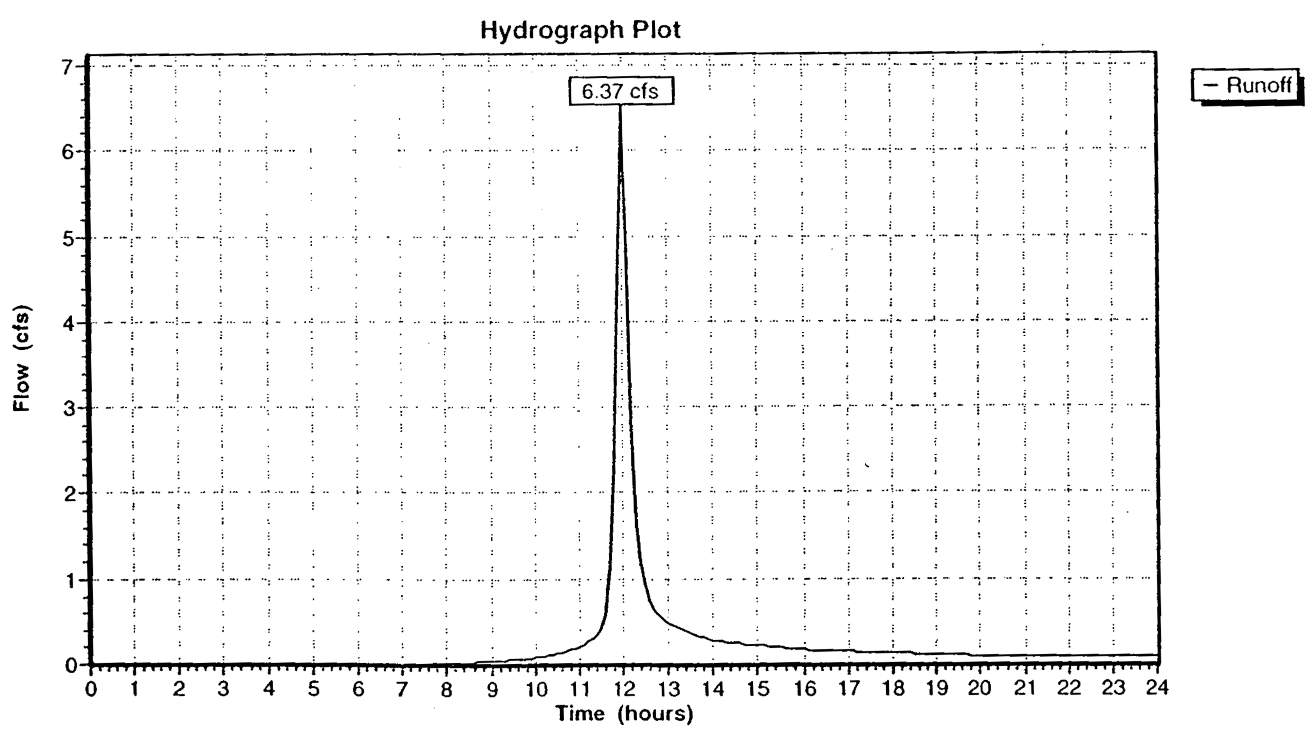
Runoff = 6.37 cfs @ 12.01 hrs, Volume= 0.416 af

Runoff by SCS TR-20 method, UH=SCS, Time Span= 0.00-24.00 hrs, dt= 0.10 hrs
Type II 24-hr Rainfall=4.70"

Area (ac)	CN	Description
1.780	82	

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.5	30	0.0100	0.1		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.60"
6.6	85	0.0500	0.2		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.60"
2.3	45	0.2000	0.3		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.60"
14.4	160	Total			

Subcatchment H: construction support area



Subcatchment I: liner runout/construction support area

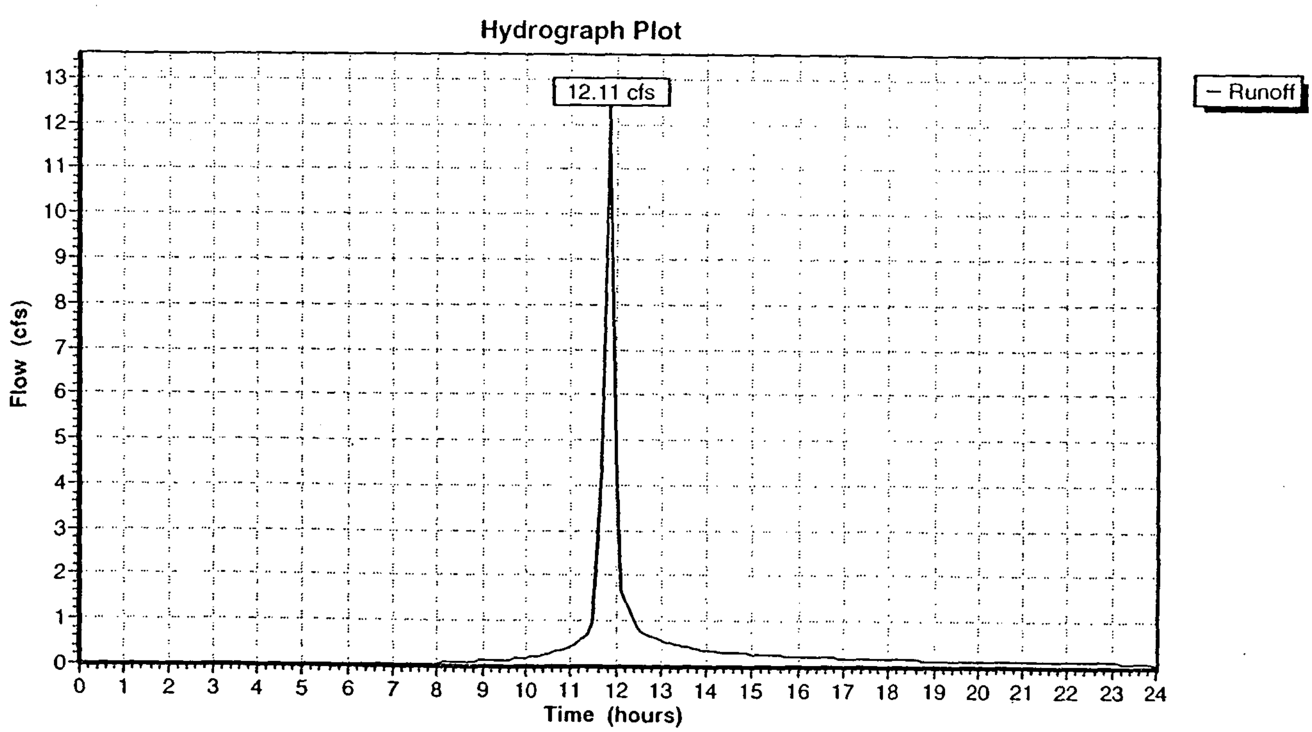
Runoff = 12.11 cfs @ 11.88 hrs, Volume= 0.592 af

Runoff by SCS TR-20 method, UH=SCS, Time Span= 0.00-24.00 hrs, dt= 0.10 hrs
Type II 24-hr Rainfall=4.70"

Area (ac)	CN	Description
2.164	87	

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
0.5	70	0.1428	2.5		**Sheet Flow,** Smooth surfaces n= 0.011 P2= 2.60"
1.2	70	0.0142	1.0		**Sheet Flow,** Smooth surfaces n= 0.011 P2= 2.60"
2.0	580	0.0100	4.8	57.39	**Trap/Vee/Rect Channel Flow,** Bot.W=0.00' D=2.00' Z= 3.0 '/' n= 0.030
3.7	720	Total			

Subcatchment I: liner runout/construction support area



Subcatchment J: construction support area

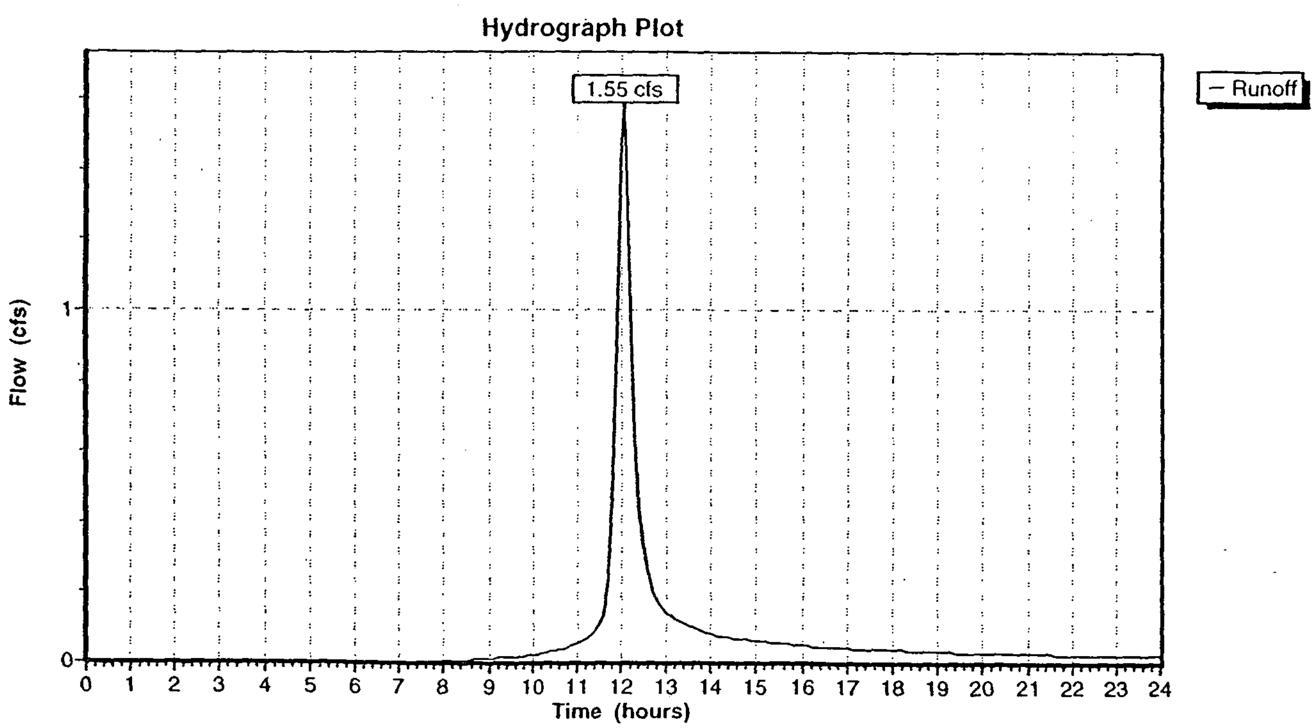
Runoff = 1.55 cfs @ 12.06 hrs, Volume= 0.115 af

Runoff by SCS TR-20 method, UH=SCS, Time Span= 0.00-24.00 hrs, dt= 0.10 hrs
Type II 24-hr Rainfall=4.70"

Area (ac)	CN	Description
0.491	82	

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
1.6	45	0.5000	0.5		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.60"
16.0	115	0.0100	0.1		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.60"
0.7	70	0.0160	1.8	0.55	**Trap/Vee/Rect Channel Flow,** Bot.W=0.00' D=0.32' Z= 3.0 '/' n= 0.030
18.3	230	Total			

Subcatchment J: construction support area



Subcatchment K: construction support area

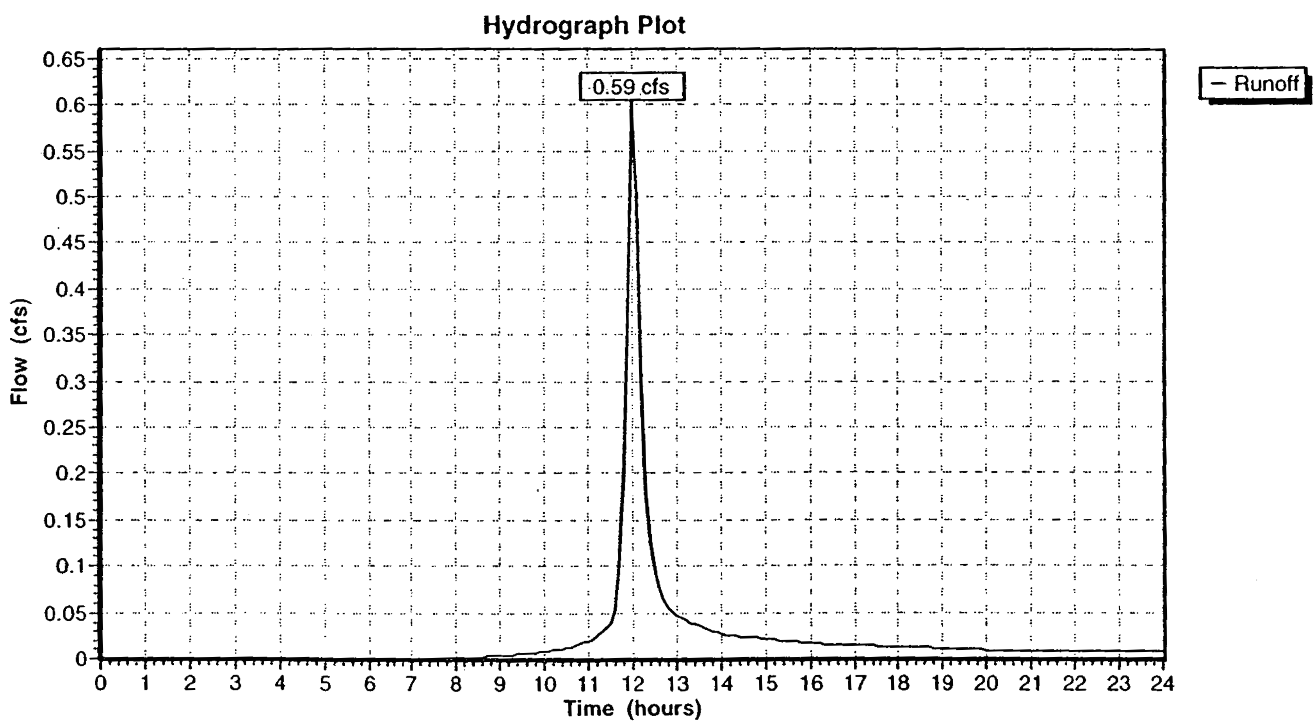
Runoff = 0.59 cfs @ 12.02 hrs, Volume= 0.040 af

Runoff by SCS TR-20 method, UH=SCS, Time Span= 0.00-24.00 hrs, dt= 0.10 hrs
Type II 24-hr Rainfall=4.70"

Area (ac)	CN	Description
0.171	82	

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
15.5	110	0.0100	0.1		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.60"

Subcatchment K: construction support area



Subcatchment L: construction support area

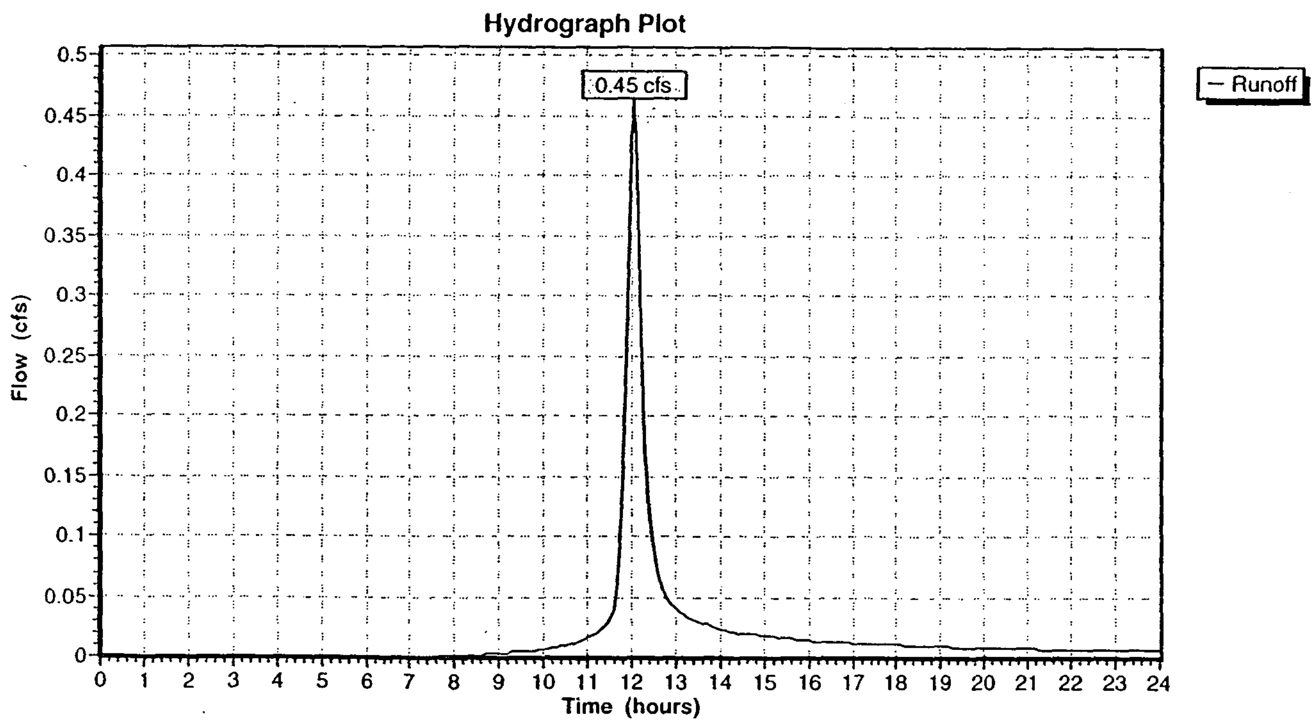
Runoff = 0.45 cfs @ 12.05 hrs, Volume= 0.033 af

Runoff by SCS TR-20 method, UH=SCS, Time Span= 0.00-24.00 hrs, dt= 0.10 hrs
Type II 24-hr Rainfall=4.70"

Area (ac)	CN	Description
0.142	82	

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
17.7	130	0.0100	0.1		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.60"

Subcatchment L: construction support area



Subcatchment M: direct runon into pond

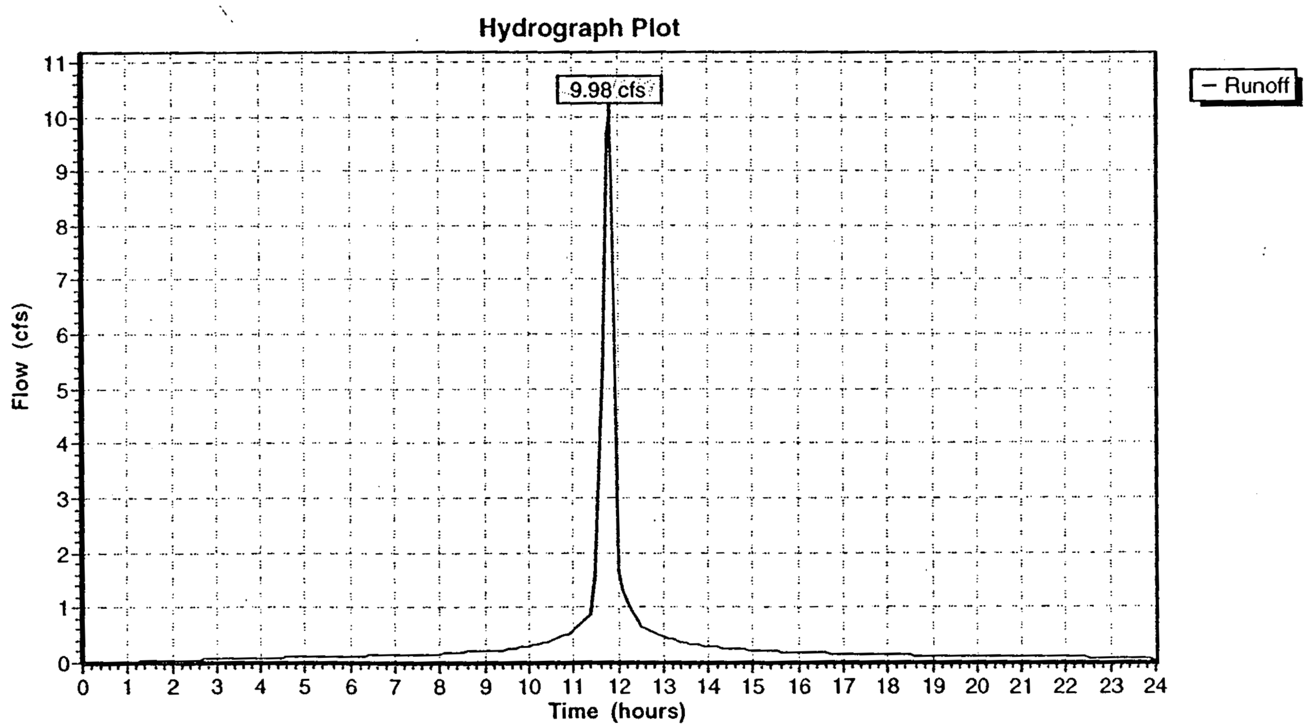
Runoff = 9.98 cfs @ 11.83 hrs, Volume= 0.604 af

Runoff by SCS TR-20 method, UH=SCS, Time Span= 0.00-24.00 hrs, dt= 0.10 hrs
Type II 24-hr Rainfall=4.70"

Area (ac)	CN	Description
1.623	98	

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
1.0					**Direct Entry, direct runon into pond**

Subcatchment M: direct runon into pond



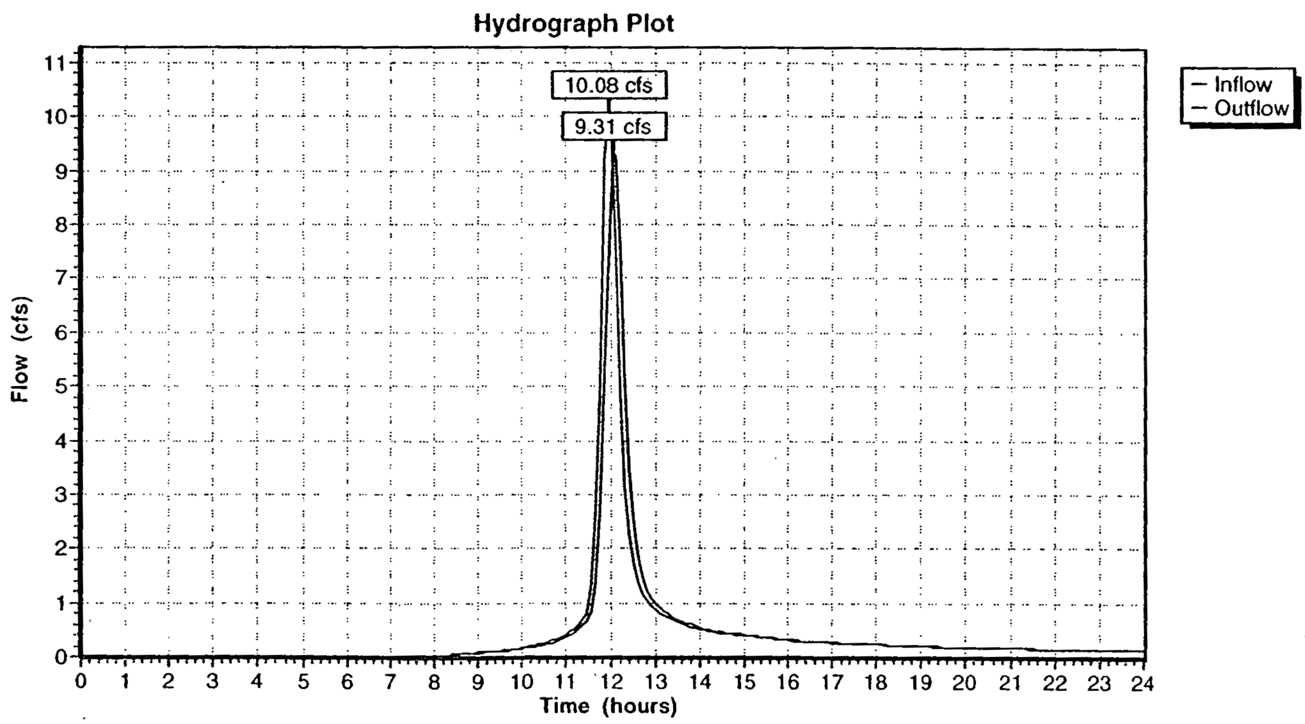
Reach 1: east channel

Inflow = 10.08 cfs @ 11.97 hrs, Volume= 0.760 af
Outflow = 9.31 cfs @ 12.08 hrs, Volume= 0.757 af, Atten= 8%, Lag= 6.6 min

Routing by Stor-Ind+Trans method, Time Span= 0.00-24.00 hrs, dt= 0.10 hrs
Max. Velocity= 2.0 fps, Min. Travel Time= 3.3 min
Avg. Velocity = 0.8 fps, Avg. Travel Time= 8.1 min

Peak Depth= 0.98'
Capacity at bank full= 754.76 cfs
0.00' x 5.00' deep channel, n= 0.030 Length= 400.0' Slope= 0.0045 '/'
Side Slope Z-value= 6.0 4.0 '/'

Reach 1: east channel



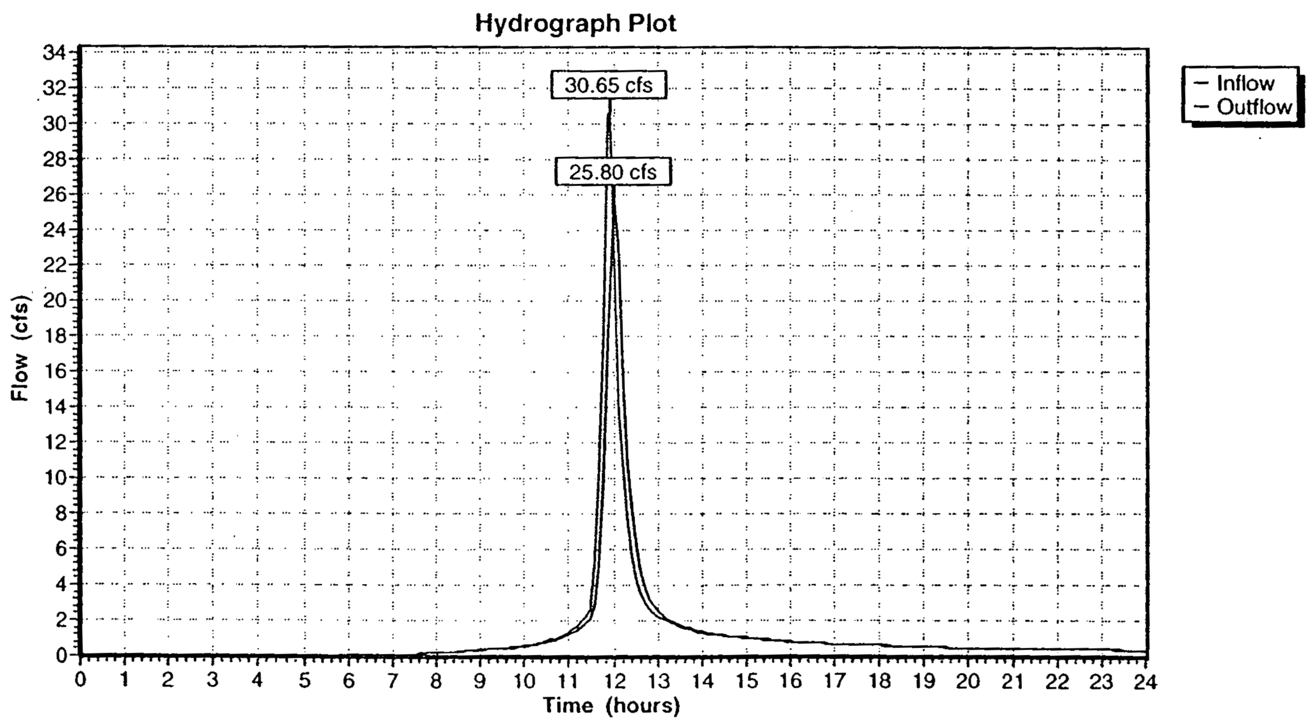
Reach 2: east channel

Inflow	=	30.65 cfs @	11.91 hrs, Volume=	2.082 af
Outflow	=	25.80 cfs @	12.03 hrs, Volume=	2.075 af, Atten= 16%, Lag= 6.8 min

Routing by Stor-Ind+Trans method, Time Span= 0.00-24.00 hrs, dt= 0.10 hrs
Max. Velocity= 2.8 fps, Min. Travel Time= 3.5 min
Avg. Velocity = 1.1 fps, Avg. Travel Time= 9.0 min

Peak Depth= 1.57'
Capacity at bank full= 1,206.56 cfs
0.00' x 6.50' deep channel, n= 0.030 Length= 575.0' Slope= 0.0045 '/'
Side Slope Z-value= 5.0 3.0 '/'

Reach 2: east channel



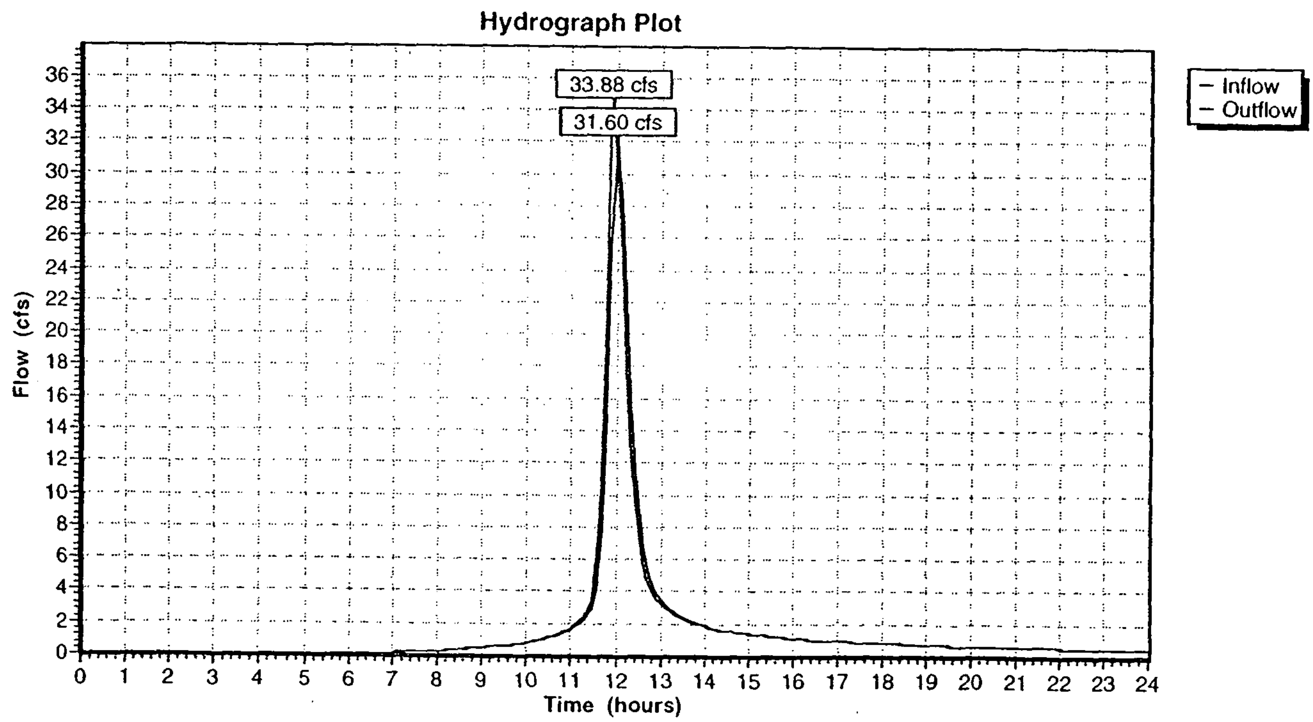
Reach 3: east channel

Inflow	=	33.88 cfs @	11.93 hrs, Volume=	2.828 af
Outflow	=	31.60 cfs @	12.02 hrs, Volume=	2.823 af, Atten= 7%, Lag= 5.0 min

Routing by Stor-Ind+Trans method, Time Span= 0.00-24.00 hrs, dt= 0.10 hrs
Max. Velocity= 3.1 fps, Min. Travel Time= 2.0 min
Avg. Velocity = 1.2 fps, Avg. Travel Time= 5.0 min

Peak Depth= 1.73'
Capacity at bank full= 1,565.76 cfs
0.00' x 7.40' deep channel, n= 0.030 Length= 375.0' Slope= 0.0050 '/'
Side Slope Z-value= 4.0 3.0 '/'

Reach 3: east channel



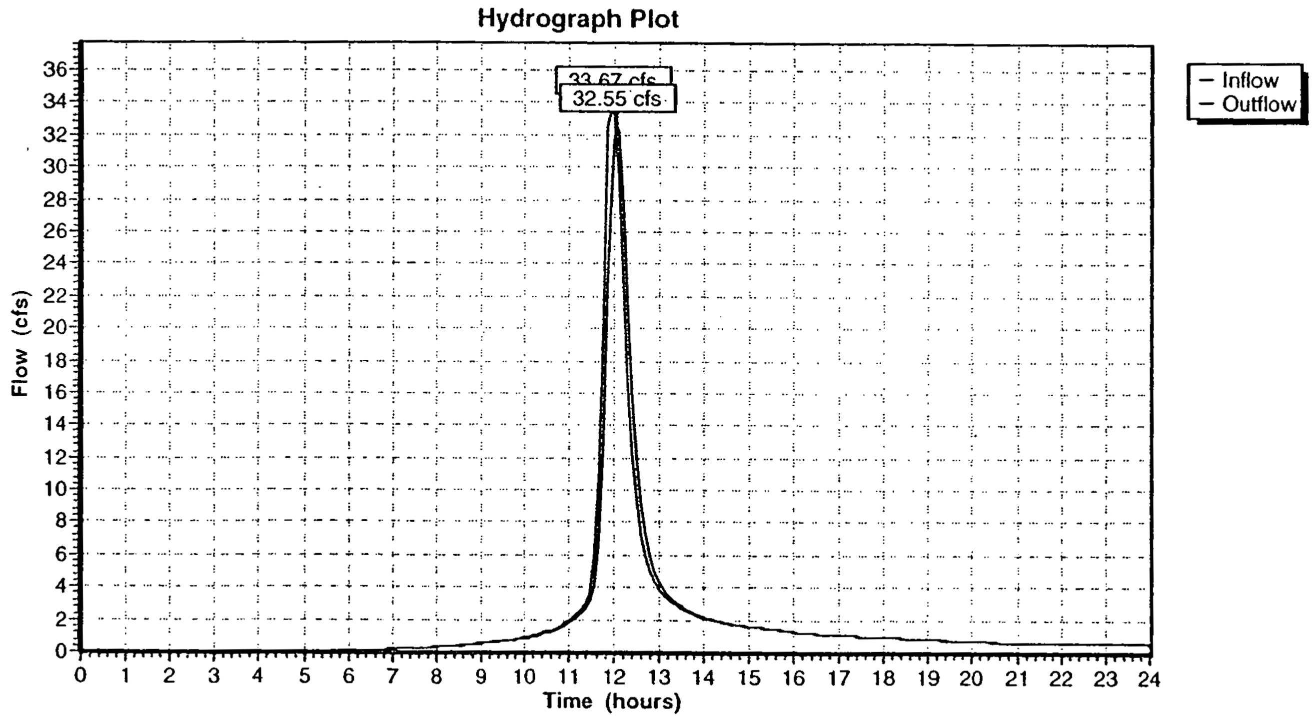
Reach 4: east channel

Inflow	=	33.67 cfs @	11.97 hrs, Volume=	3.199 af
Outflow	=	32.55 cfs @	12.06 hrs, Volume=	3.192 af, Atten= 3%, Lag= 5.7 min

Routing by Stor-Ind+Trans method, Time Span= 0.00-24.00 hrs, dt= 0.10 hrs
Max. Velocity= 3.1 fps, Min. Travel Time= 2.5 min
Avg. Velocity = 1.3 fps, Avg. Travel Time= 6.2 min

Peak Depth= 1.76'
Capacity at bank full= 1,565.32 cfs
0.00' x 7.40' deep channel, n= 0.030 Length= 475.0' Slope= 0.0050 '/'
Side Slope Z-value= 4.0 3.0 '/'

Reach 4: east channel



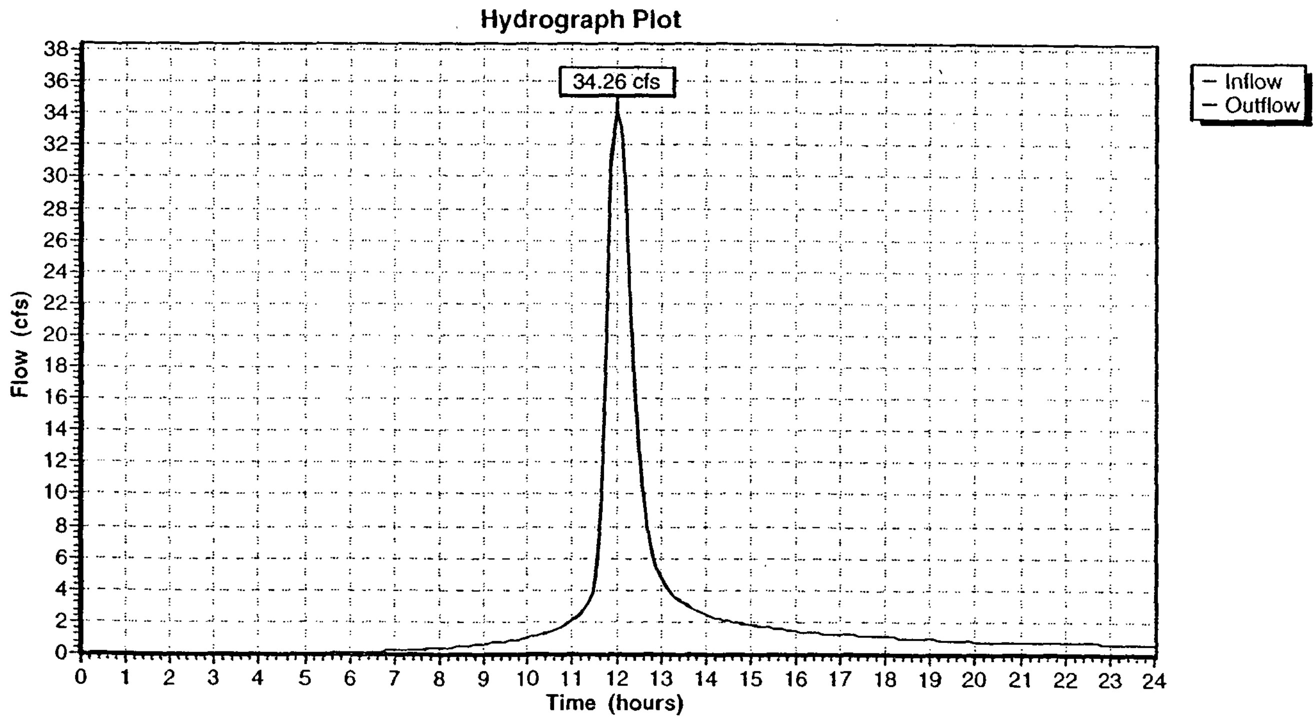
Reach 5: CMP culvert

Inflow	=	34.14 cfs @	12.03 hrs,	Volume=	3.609 af
Outflow	=	34.26 cfs @	12.03 hrs,	Volume=	3.608 af, Atten= 0%, Lag= 0.2 min

Routing by Stor-Ind+Trans method, Time Span= 0.00-24.00 hrs, dt= 0.10 hrs
Max. Velocity= 5.4 fps, Min. Travel Time= 0.3 min
Avg. Velocity = 1.9 fps, Avg. Travel Time= 0.8 min

Peak Depth= 1.02'
Capacity at bank full= 541.77 cfs
A factor of 2.00 has been applied to the supplied storage and discharge data
72.0" Diameter Pipe n= 0.024 Length= 96.1' Slope= 0.0139 '/'

Reach 5: CMP culvert



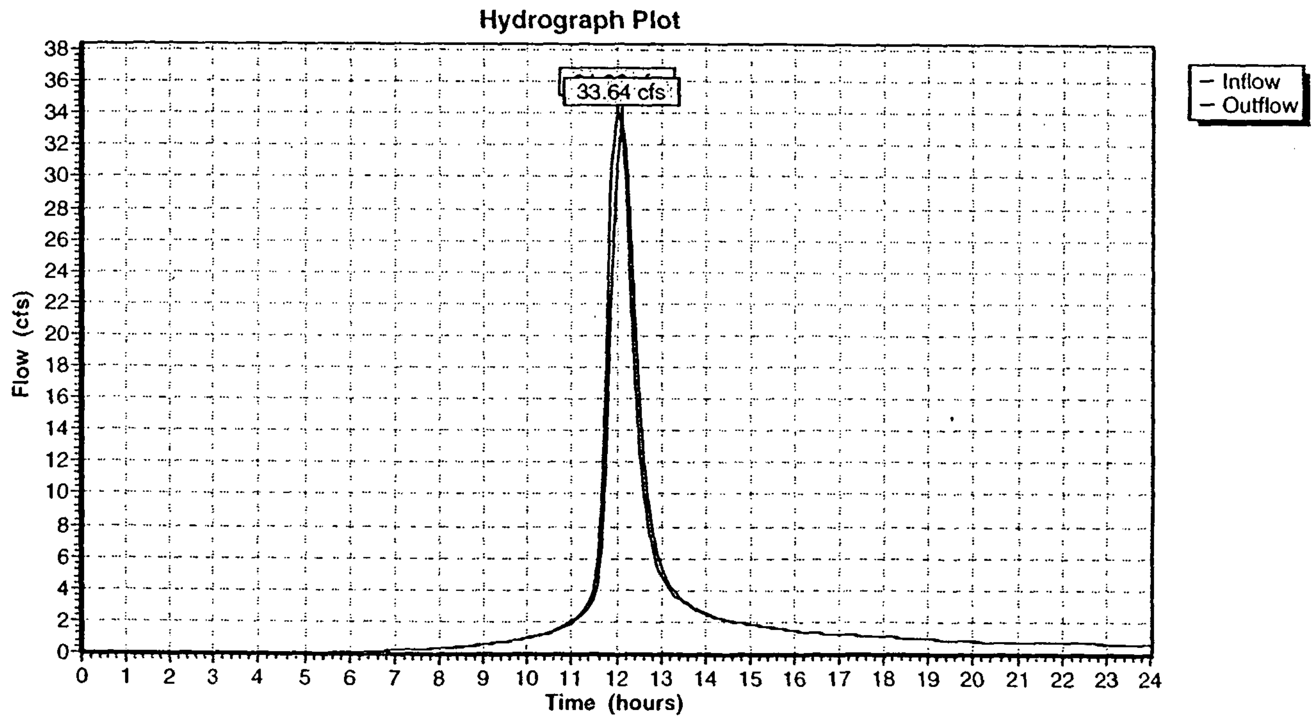
Reach 6: south east channel

Inflow	=	34.26 cfs @	12.03 hrs, Volume=	3.608 af
Outflow	=	33.64 cfs @	12.11 hrs, Volume=	3.601 af, Atten= 2%, Lag= 4.6 min

Routing by Stor-Ind+Trans method, Time Span= 0.00-24.00 hrs, dt= 0.10 hrs
Max. Velocity= 4.7 fps, Min. Travel Time= 2.3 min
Avg. Velocity = 2.0 fps, Avg. Travel Time= 5.4 min

Peak Depth= 1.56'
Capacity at bank full= 199.53 cfs
0.00' x 3.00' deep channel, n= 0.030 Length= 640.0' Slope= 0.0139 '/'
Side Slope Z-value= 3.0 '/'

Reach 6: south east channel



Reach 7: CMP culvert

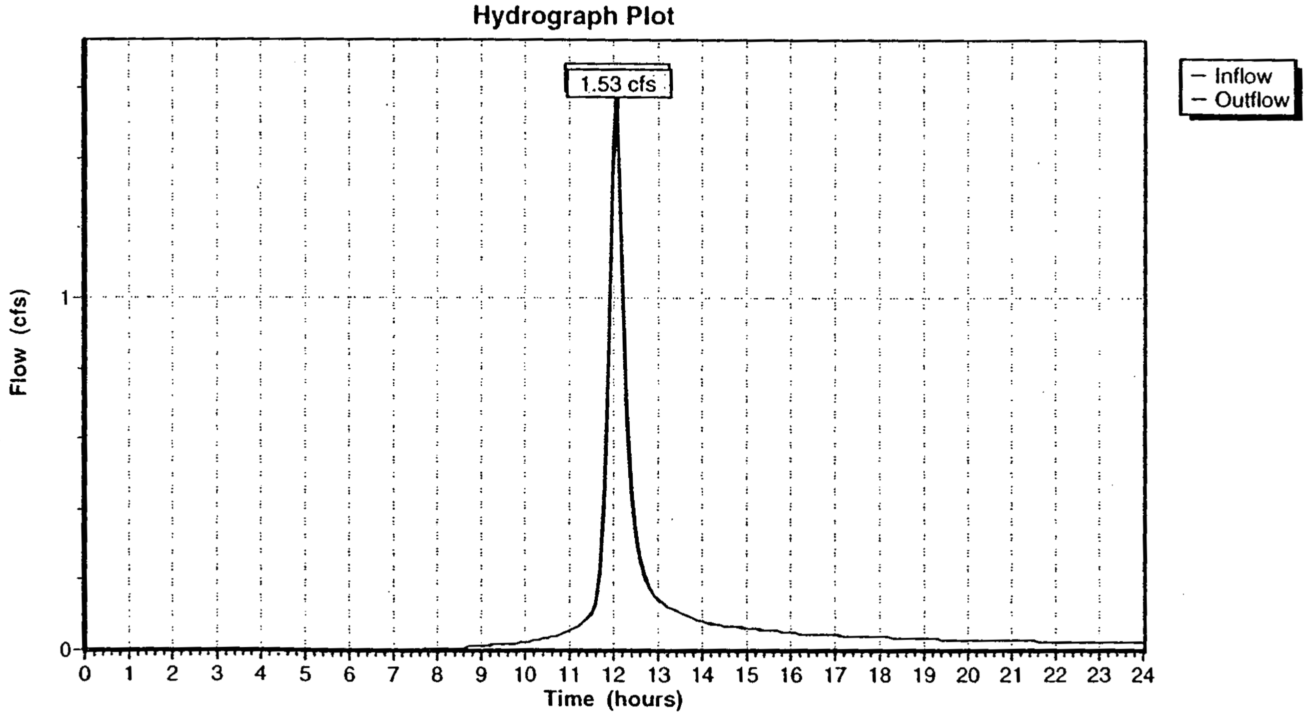
Inflow	=	1.55 cfs @	12.06 hrs, Volume=	0.115 af	
Outflow	=	1.53 cfs @	12.08 hrs, Volume=	0.115 af,	Atten= 1%, Lag= 1.1 min

Routing by Stor-Ind+Trans method, Time Span= 0.00-24.00 hrs, dt= 0.10 hrs
Max. Velocity= 2.2 fps, Min. Travel Time= 0.6 min
Avg. Velocity = 1.0 fps, Avg. Travel Time= 1.3 min

Peak Depth= 0.23'
Capacity at bank full= 579.36 cfs
A factor of 2.00 has been applied to the supplied storage and discharge data
72.0" Diameter Pipe n= 0.024 Length= 74.0' Slope= 0.0159 '/'

Reach 7: CMP culvert

Hydrograph Plot



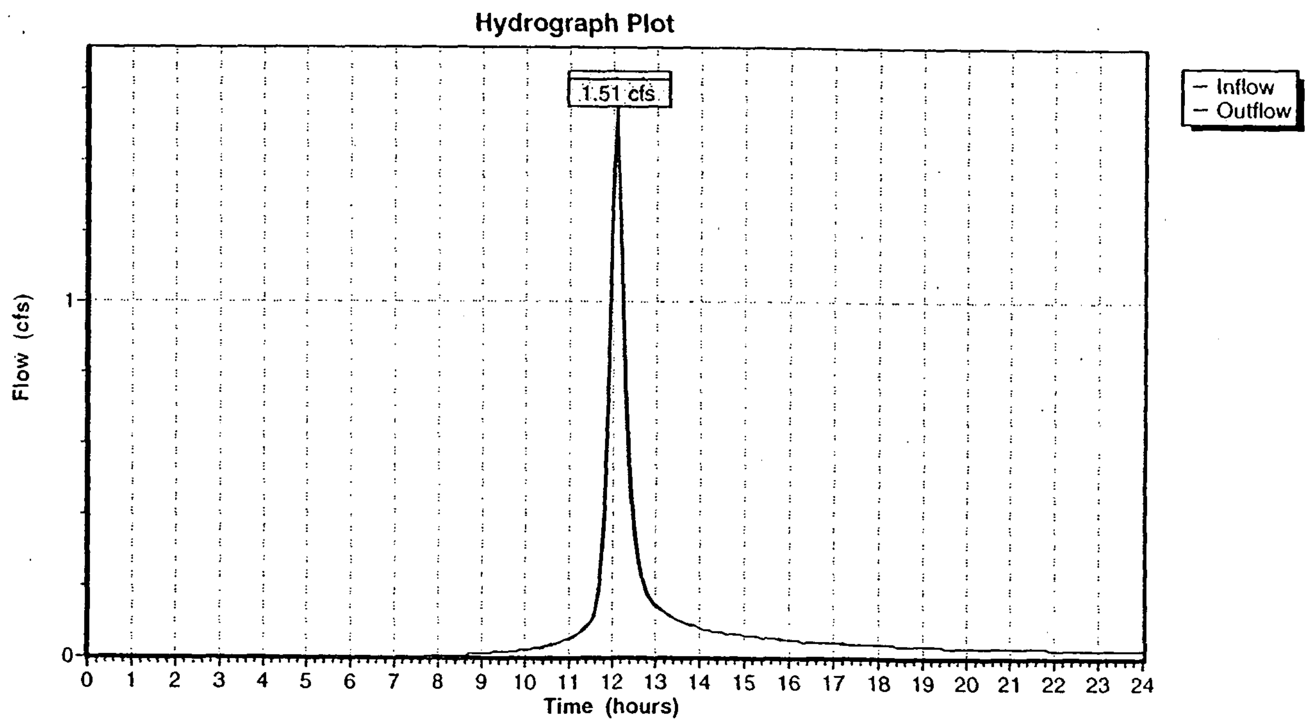
Reach 8: channel

Inflow	=	1.53 cfs @	12.08 hrs,	Volume=	0.115 af
Outflow	=	1.51 cfs @	12.10 hrs,	Volume=	0.115 af, Atten= 2%, Lag= 1.1 min

Routing by Stor-Ind+Trans method, Time Span= 0.00-24.00 hrs, dt= 0.10 hrs
Max. Velocity= 2.3 fps, Min. Travel Time= 0.7 min
Avg. Velocity = 0.9 fps, Avg. Travel Time= 1.6 min

Peak Depth= 0.47'
Capacity at bank full= 214.02 cfs
0.00' x 3.00' deep channel, n= 0.030 Length= 90.0' Slope= 0.0160 '/'
Side Slope Z-value= 3.0 '/'

Reach 8: channel



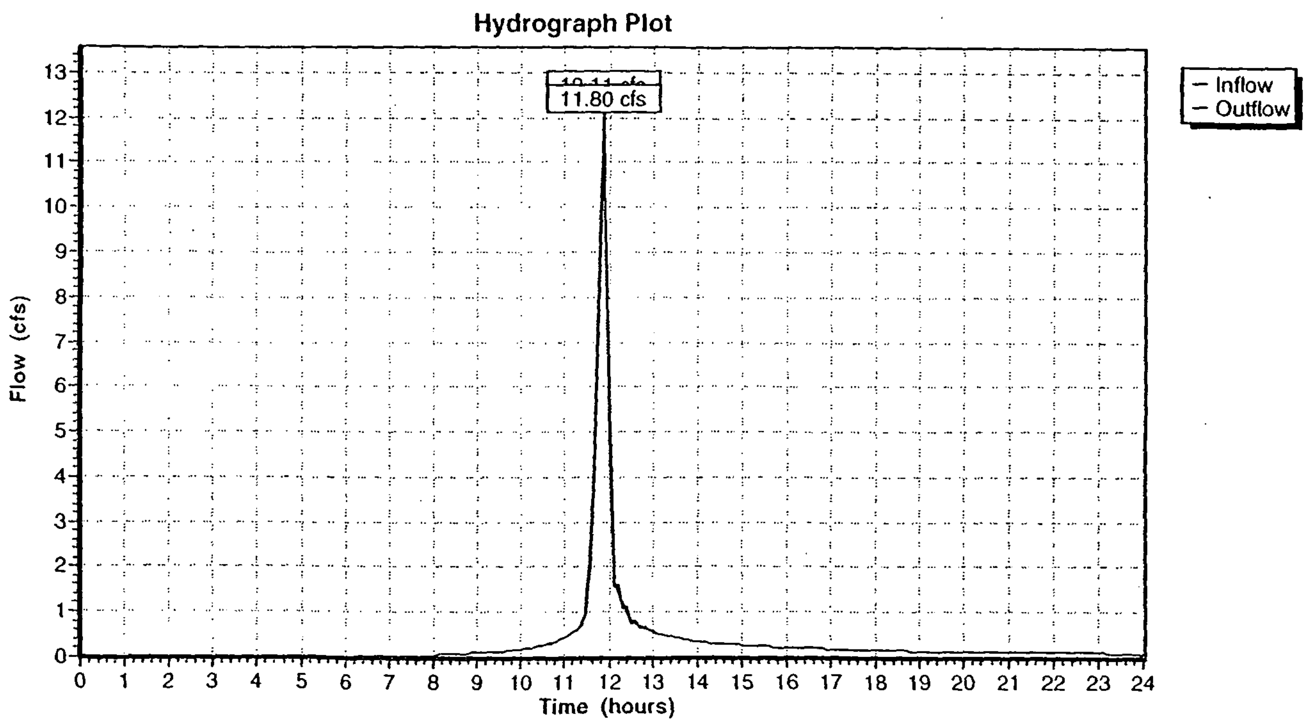
Reach 9: channel

Inflow	=	12.11 cfs @	11.88 hrs,	Volume=	0.592 af
Outflow	=	11.80 cfs @	11.89 hrs,	Volume=	0.592 af, Atten= 3%, Lag= 0.3 min

Routing by Stor-Ind+Trans method, Time Span= 0.00-24.00 hrs, dt= 0.10 hrs
Max. Velocity= 3.7 fps, Min. Travel Time= 0.3 min
Avg. Velocity = 1.3 fps, Avg. Travel Time= 0.8 min

Peak Depth= 1.04'
Capacity at bank full= 441.30 cfs
0.00' x 4.00' deep channel, n= 0.030 Length= 60.0' Slope= 0.0147 '/'
Side Slope Z-value= 3.0 '/'

Reach 9: channel



Reach 10: channel

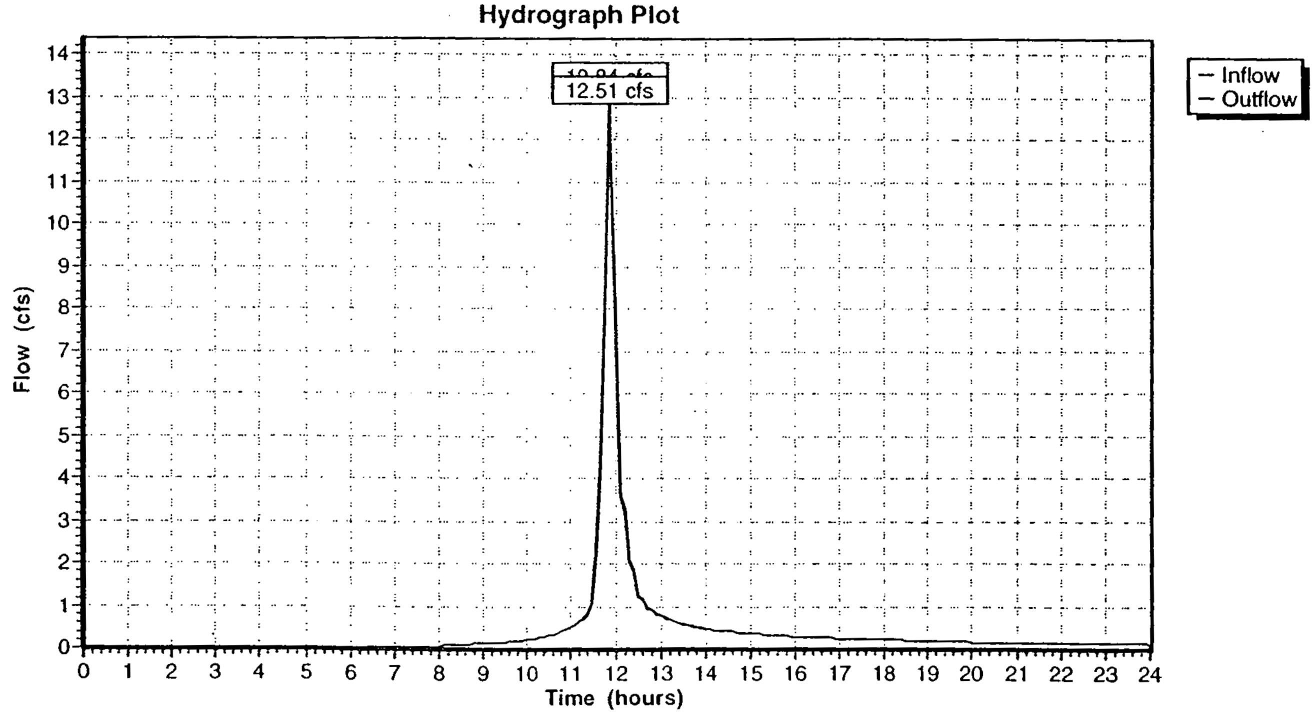
Inflow = 12.84 cfs @ 11.89 hrs, Volume= 0.747 af
Outflow = 12.51 cfs @ 11.90 hrs, Volume= 0.747 af, Atten= 3%, Lag= 0.3 min

Routing by Stor-Ind+Trans method, Time Span= 0.00-24.00 hrs, dt= 0.10 hrs
Max. Velocity= 3.8 fps, Min. Travel Time= 0.3 min
Avg. Velocity = 1.4 fps, Avg. Travel Time= 0.7 min

Peak Depth= 1.06'
Capacity at bank full= 204.91 cfs
0.00' x 3.00' deep channel, n= 0.030 Length= 60.0' Slope= 0.0147 '/'
Side Slope Z-value= 3.0 '/'

Reach 10: channel

Hydrograph Plot



Reach 11: CMP culvert

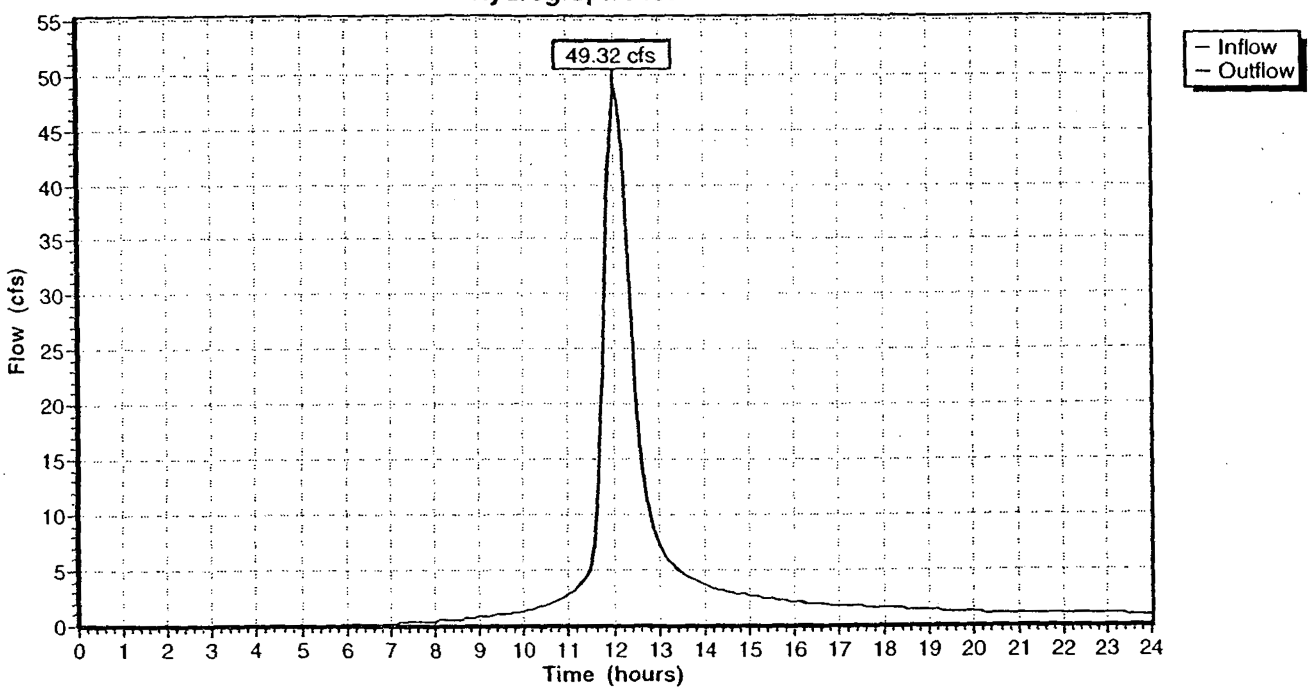
Inflow = 49.39 cfs @ 12.03 hrs, Volume= 5.205 af
Outflow = 49.32 cfs @ 12.04 hrs, Volume= 5.204 af, Atten= 0%, Lag= 0.3 min

Routing by Stor-Ind+Trans method, Time Span= 0.00-24.00 hrs, dt= 0.10 hrs
Max. Velocity= 5.1 fps, Min. Travel Time= 0.3 min
Avg. Velocity = 1.8 fps, Avg. Travel Time= 0.8 min

Peak Depth= 1.37'
Capacity at bank full= 432.56 cfs
A factor of 2.00 has been applied to the supplied storage and discharge data
72.0" Diameter Pipe n= 0.030 Length= 90.0' Slope= 0.0139 '/'

Reach 11: CMP culvert

Hydrograph Plot



Pond 1P: sedimentation basin 1

Inflow	=	50.18 cfs @	12.01 hrs, Volume=	5.808 af	
Outflow	=	1.59 cfs @	18.70 hrs, Volume=	0.736 af,	Atten= 97%, Lag= 401.5 min
Primary	=	1.59 cfs @	18.70 hrs, Volume=	0.736 af	

Routing by Stor-Ind method, Time Span= 0.00-24.00 hrs, dt= 0.10 hrs

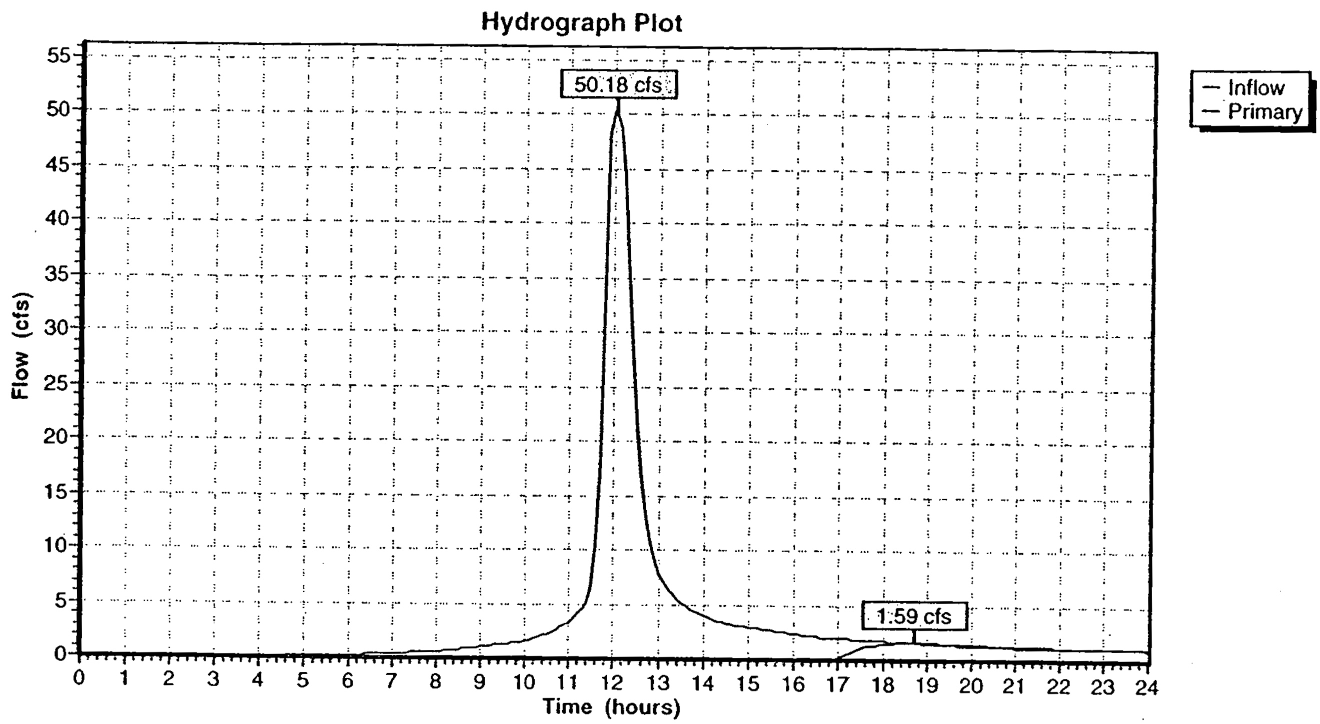
Peak Elev= 585.82' Storage= 5.100 af
Plug-Flow detention time= 626.6 min calculated for 0.736 af (13% of inflow)
Storage and wetted areas determined by Conic sections

Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)	Wet.Area (acres)
581.00	0.148	0.000	0.000	0.148
582.00	0.972	0.500	0.500	0.972
583.00	1.124	1.047	1.547	1.125
584.00	1.217	1.170	2.717	1.220
585.00	1.318	1.267	3.984	1.323
586.00	1.407	1.362	5.346	1.414
587.00	1.494	1.450	6.797	1.503
588.00	1.563	1.528	8.325	1.575

Primary OutFlow (Free Discharge)
- 2=Culvert
  - 1=Orifice/Grate
- 3=Broad-Crested Rectangular Weir

#	Routing	Invert	Outlet Devices
1	Device 2	585.75'	**48.0" Horiz. Orifice/Grate X 2.00** Limited to weir flow C= 0.600
2	Primary	580.00'	**36.0" x 61.0' long Culvert X 2.00** Ke= 0.700
			Outlet Invert= 578.86' S= 0.0187 '/' n= 0.024 Cc= 0.900
3	Primary	586.50'	**30.0' long Broad-Crested Rectangular Weir**
			Head (feet) 0.50 1.00 1.50 2.00 2.50 3.00 4.00 5.00
			Coef. (English) 3.00 3.00 3.00 3.00 3.00 3.00 3.00 3.00

Pond 1P: sedimentation basin 1



OSDF DESIGN SCENARIO
100-YEAR, 24-HOUR STORM EVENT

Pond 1P: sedimentation basin 1

Inflow	=	64.79 cfs @	12.00 hrs,	Volume=	7.294 af
Outflow	=	5.21 cfs @	13.85 hrs,	Volume=	2.210 af, Atten= 92%, Lag= 111.1 min
Primary	=	5.21 cfs @	13.85 hrs,	Volume=	2.210 af

Routing by Stor-Ind method, Time Span= 0.00-24.00 hrs, dt= 0.10 hrs

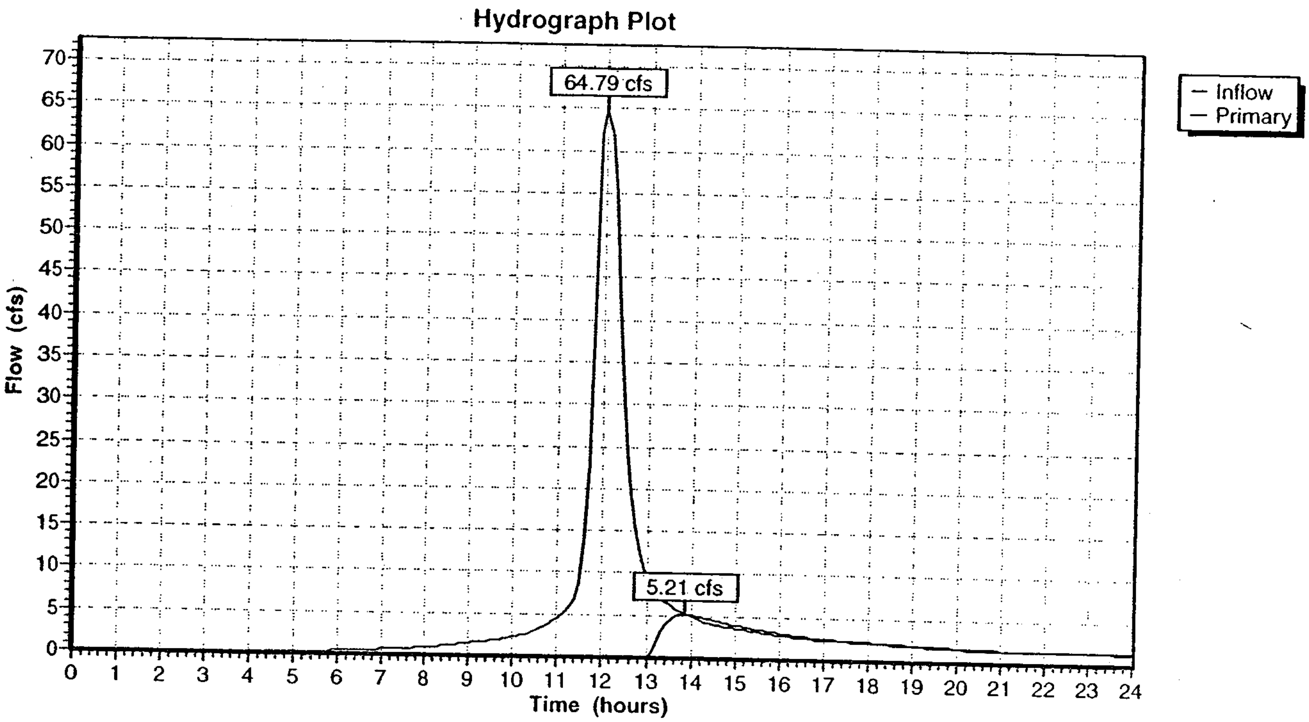
Peak Elev= 585.91' Storage= 5.221 af
Plug-Flow detention time= 378.0 min calculated for 2.201 af (30% of inflow)
Storage and wetted areas determined by Conic sections

Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)	Wet.Area (acres)
581.00	0.148	0.000	0.000	0.148
582.00	0.972	0.500	0.500	0.972
583.00	1.124	1.047	1.547	1.125
584.00	1.217	1.170	2.717	1.220
585.00	1.318	1.267	3.984	1.323
586.00	1.407	1.362	5.346	1.414
587.00	1.494	1.450	6.797	1.503
588.00	1.563	1.528	8.325	1.575

Primary OutFlow (Free Discharge)
—2=Culvert
  —1=Orifice/Grate
—3=Broad-Crested Rectangular Weir

#	Routing	Invert	Outlet Devices
1	Device 2	585.75'	**48.0" Horiz. Orifice/Grate X 2.00** Limited to weir flow C= 0.600
2	Primary	580.00'	**36.0" x 61.0' long Culvert X 2.00** Ke= 0.700 Outlet Invert= 578.86' S= 0.0187 '/' n= 0.024 Cc= 0.900
3	Primary	586.50'	**30.0' long Broad-Crested Rectangular Weir** Head (feet) 0.50 1.00 1.50 2.00 2.50 3.00 4.00 5.00 Coef. (English) 3.00 3.00 3.00 3.00 3.00 3.00 3.00 3.00

Pond 1P: sedimentation basin 1



ATTACHMENT C-1B

HydroCAD™ OUTPUT REPORTS

DESIGN CASE "A"

DESIGN CASE "A"
25-YEAR, 24-HOUR STORM EVENT

Time span=0.00-24.00 hrs, dt=0.01 hrs, 2401 points
Runoff by SCS TR-20 method, UH=SCS, Type II 24-hr Rainfall=4.70"
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment A: modified from Phase III, subcatchment 50
Tc=7.5 min CN=79 Area=0.631 ac Runoff= 2.70 cfs 0.134 af

Subcatchment B: modified from Phase III, subcatchment 1
Tc=43.0 min CN=74 Area=4.001 ac Runoff= 5.66 cfs 0.700 af

Subcatchment C: modified from Phase III, subcatchment 2
Tc=38.1 min CN=73 Area=8.860 ac Runoff= 13.07 cfs 1.495 af

Subcatchment D: runoff developed from ditch west of road
Tc=2.7 min CN=89 Area=0.768 ac Runoff= 5.04 cfs 0.223 af

Subcatchment E: runon area (1/2 of subarea 3 from phase III)
Tc=38.7 min CN=72 Area=9.124 ac Runoff= 12.76 cfs 1.480 af

Reach 1: Phase III, reach 1 Inflow= 2.70 cfs 0.134 af
Length= 880.0' Max Vel= 1.7 fps Capacity= 64.90 cfs Outflow= 1.97 cfs 0.132 af

Reach 2: Phase III, reach 2 Inflow= 6.67 cfs 0.833 af
Length= 530.0' Max Vel= 2.6 fps Capacity= 271.26 cfs Outflow= 6.61 cfs 0.830 af

Reach 3: South East Channel- Section One Inflow= 19.91 cfs 2.547 af
Length= 250.0' Max Vel= 4.9 fps Capacity= 245.90 cfs Outflow= 19.90 cfs 2.545 af

Reach 4: South East Channel- Section Two Inflow= 32.57 cfs 4.025 af
Length= 250.0' Max Vel= 3.2 fps Capacity= 75.03 cfs Outflow= 32.51 cfs 4.019 af

Subcatchment A: modified from Phase III, subcatchment 50

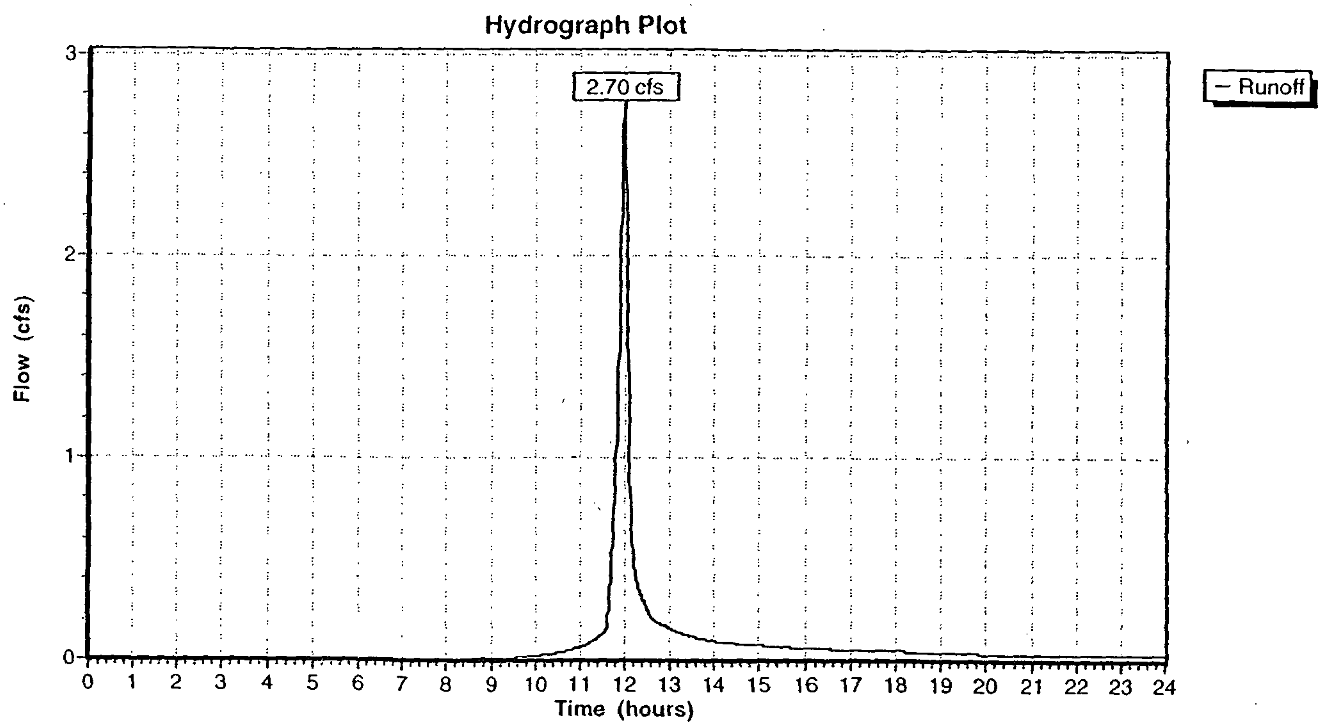
Runoff = 2.70 cfs @ 11.99 hrs, Volume= 0.134 af

Runoff by SCS TR-20 method, UH=SCS, Time Span= 0.00-24.00 hrs, dt= 0.01 hrs
Type II 24-hr Rainfall=4.70"

Area (ac)	CN	Description
0.631	79	

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.2	80	0.0800	0.3		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.60"
2.3	350	0.0040	2.5	22.78	**Trap/Vee/Rect Channel Flow,** Bot.W=0.00' D=1.50' Z= 3.0 & 5.0 '/' n= 0.030
7.5	430	Total			

Subcatchment A: modified from Phase III, subcatchment 50



Subcatchment B: modified from Phase III, subcatchment 1

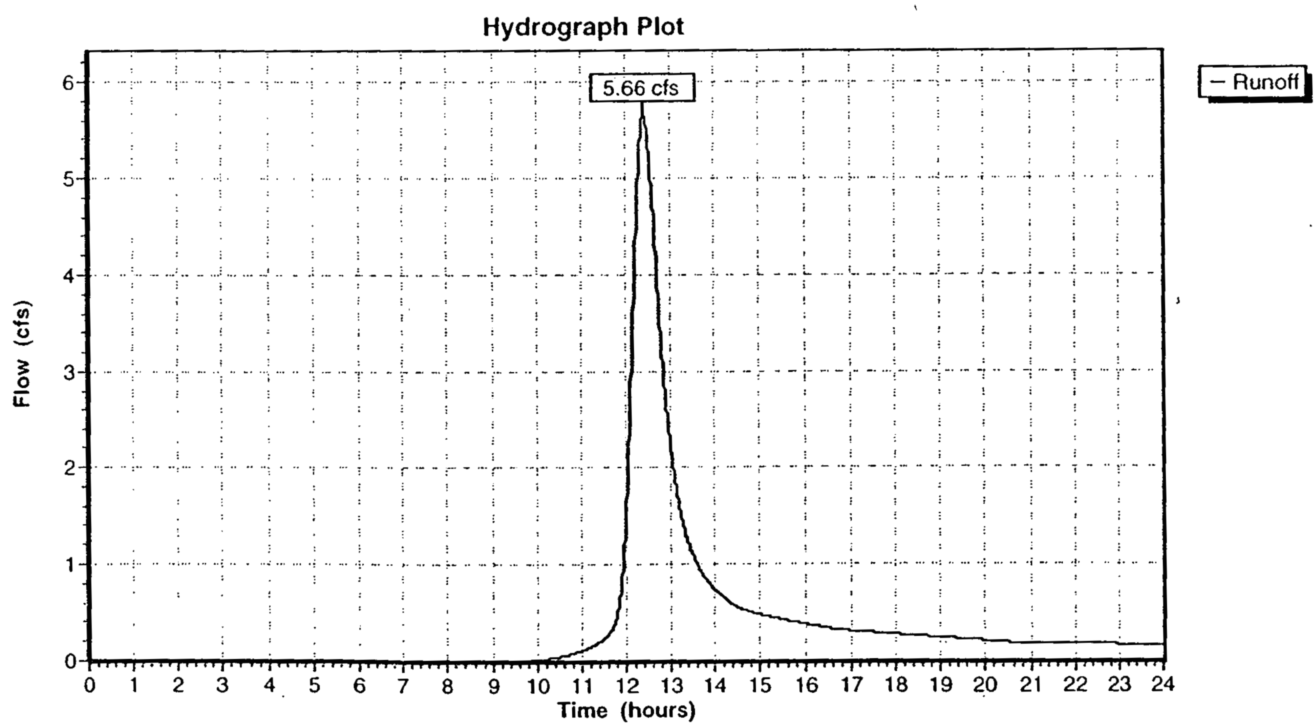
Runoff = 5.66 cfs @ 12.42 hrs, Volume= 0.700 af

Runoff by SCS TR-20 method, UH=SCS, Time Span= 0.00-24.00 hrs, dt= 0.01 hrs
Type II 24-hr Rainfall=4.70"

Area (ac)	CN	Description
4.001	74	

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
39.8	300	0.0070	0.1		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.60"
0.7	100	0.0200	2.3		**Shallow Concentrated Flow,** Unpaved Kv= 16.1 fps
2.5	600	0.0070	4.1	64.90	**Trap/Vee/Rect Channel Flow,** Bot.W=0.00' D=2.00' Z= 3.0 & 5.0 '/' n= 0.030
43.0	1,000	Total			

Subcatchment B: modified from Phase III, subcatchment 1



Subcatchment C: modified from Phase III, subcatchment 2

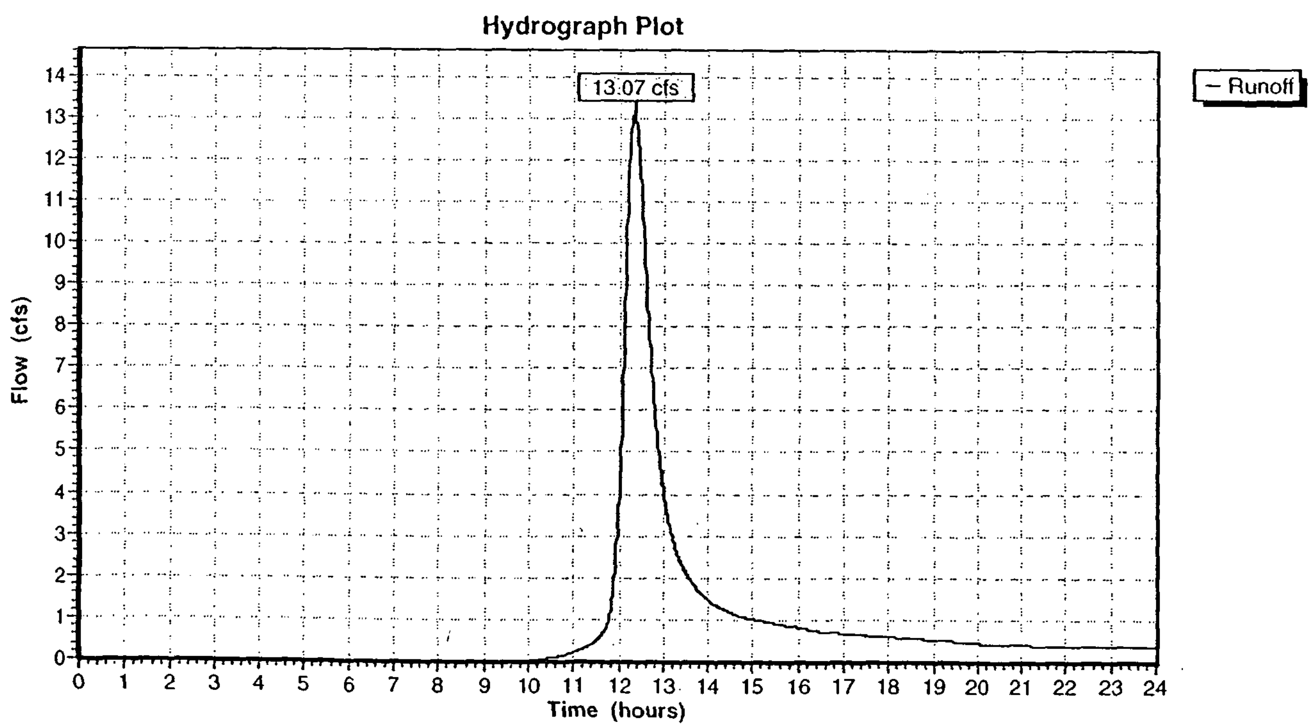
Runoff = 13.07 cfs @ 12.35 hrs, Volume= 1.495 af

Runoff by SCS TR-20 method, UH=SCS, Time Span= 0.00-24.00 hrs, dt= 0.01 hrs
Type II 24-hr Rainfall=4.70"

Area (ac)	CN	Description
8.860	73	

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
32.1	300	0.0120	0.2		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.60"
4.9	520	0.0120	1.8		**Shallow Concentrated Flow,** Unpaved Kv= 16.1 fps
1.1	330	0.0110	5.1	92.00	**Trap/Vee/Rect Channel Flow,** Bot.W=0.00' D=2.00' Z= 5.0 & 4.0 '/' n= 0.030
38.1	1,150	Total			

Subcatchment C: modified from Phase III, subcatchment 2



Subcatchment D: runoff developed from ditch west of road

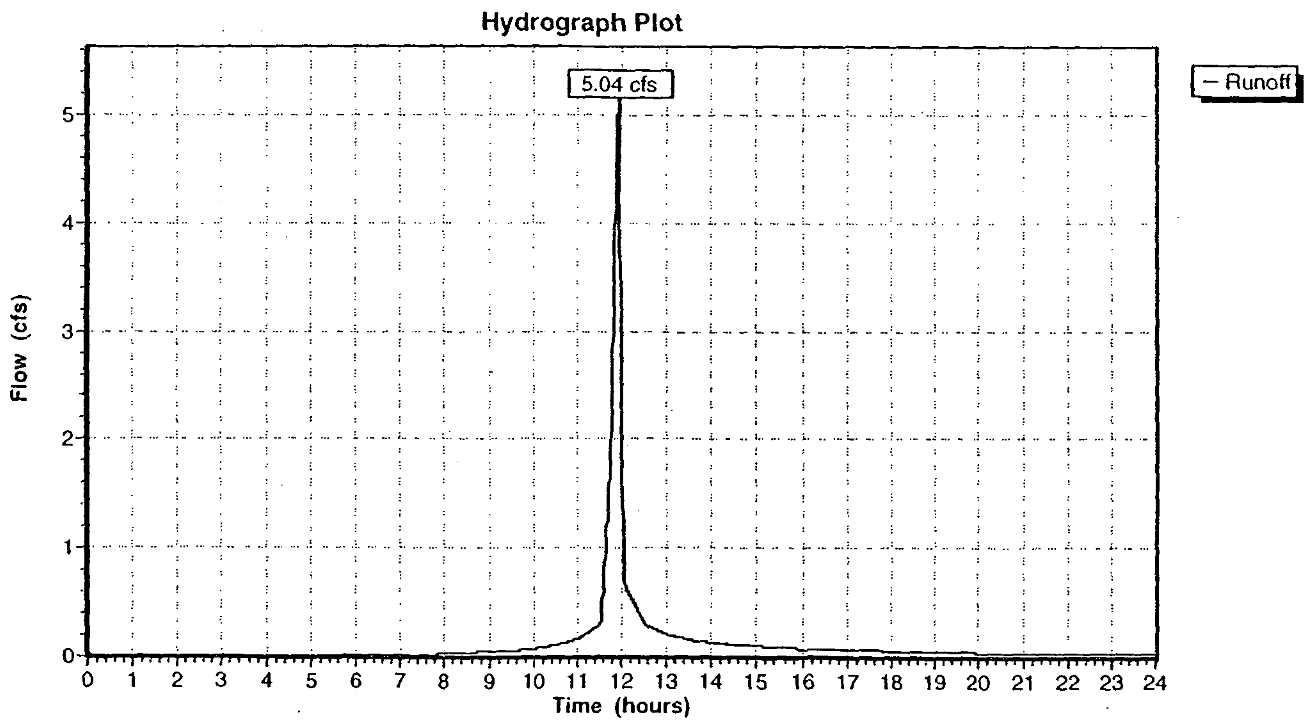
Runoff = 5.04 cfs @ 11.93 hrs, Volume= 0.223 af

Runoff by SCS TR-20 method, UH=SCS, Time Span= 0.00-24.00 hrs, dt= 0.01 hrs
Type II 24-hr Rainfall=4.70"

Area (ac)	CN	Description
0.768	89	

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
0.1	20	0.2000	2.3		**Sheet Flow,** Smooth surfaces n= 0.011 P2= 2.60"
2.6	750	0.0100	4.8	57.39	**Trap/Vee/Rect Channel Flow,** Bot.W=0.00' D=2.00' Z= 3.0 '/' n= 0.030
2.7	770	Total			

Subcatchment D: runoff developed from ditch west of road



Subcatchment E: runon area (1/2 of subarea 3 from phase III)

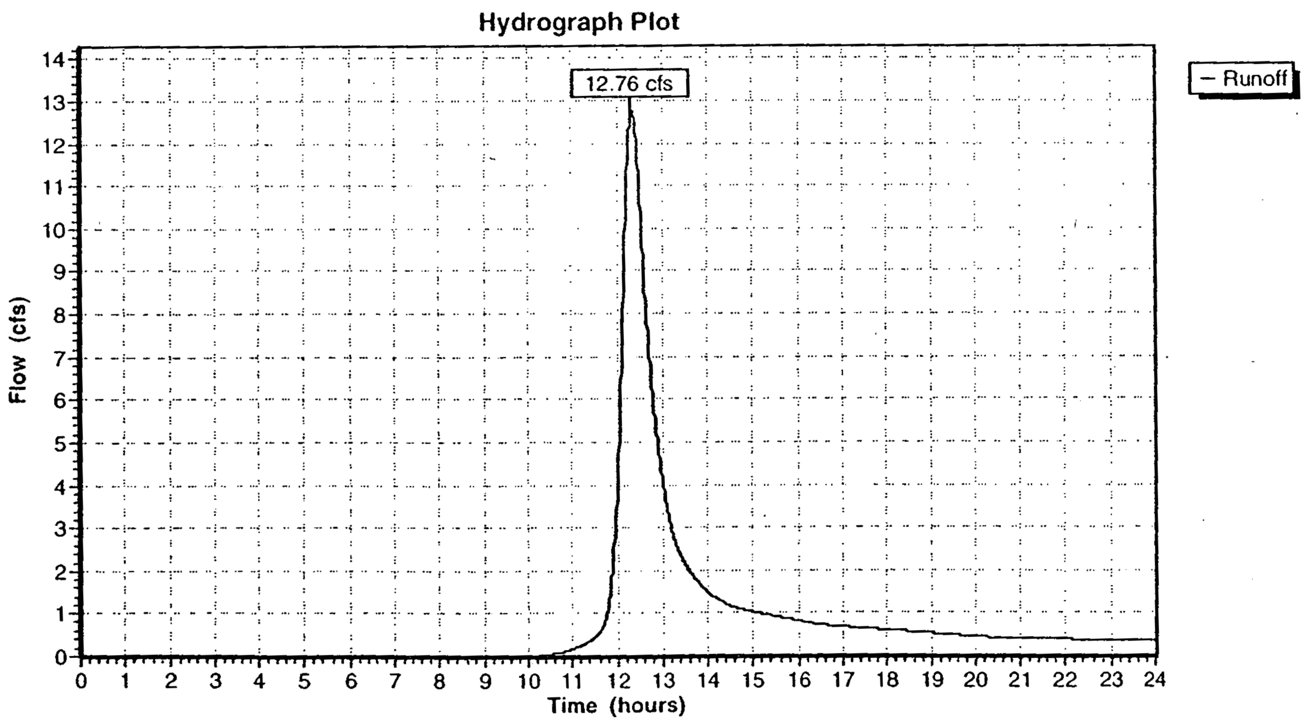
Runoff = 12.76 cfs @ 12.34 hrs, Volume= 1.480 af

Runoff by SCS TR-20 method, UH=SCS, Time Span= 0.00-24.00 hrs, dt= 0.01 hrs
Type II 24-hr Rainfall=4.70"

Area (ac)	CN	Description
9.124	72	

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
29.4	300	0.0150	0.2		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.60"
9.3	1,100	0.0150	2.0		**Shallow Concentrated Flow,** Unpaved Kv= 16.1 fps
38.7	1,400	Total			

Subcatchment E: runon area (1/2 of subarea 3 from phase III)



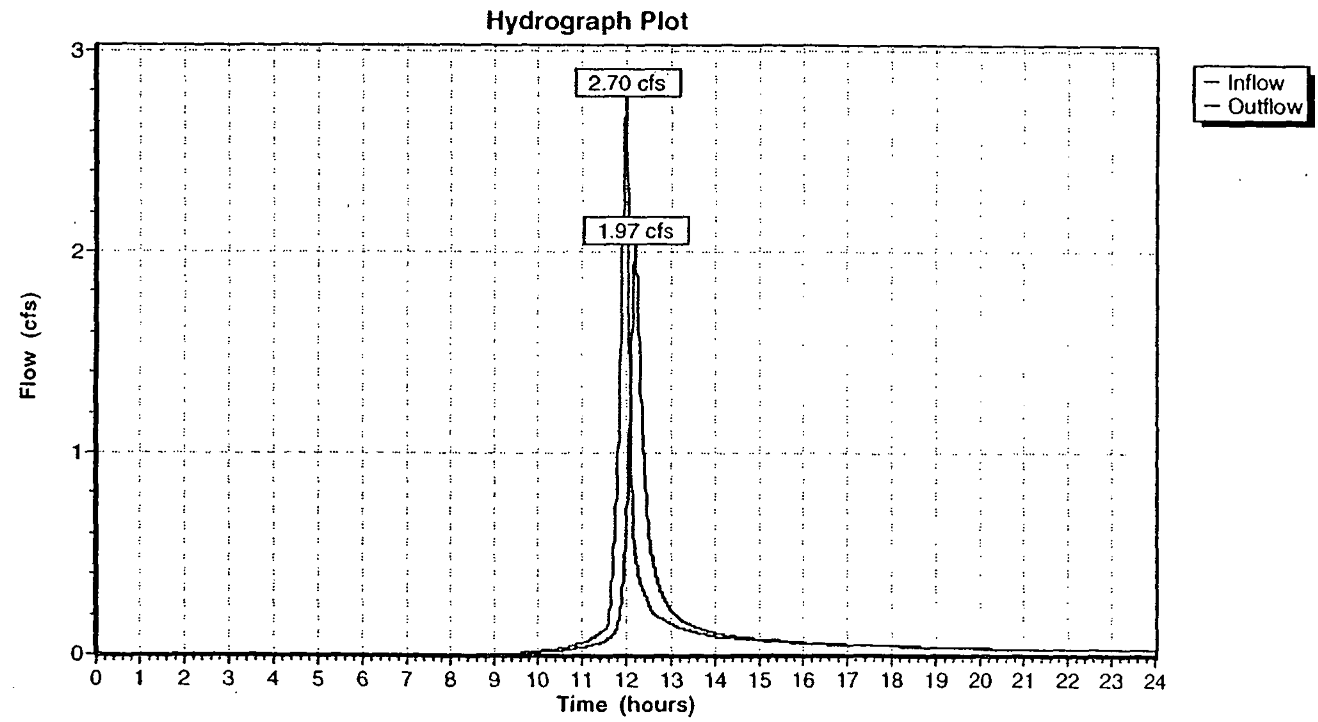
Reach 1: Phase III, reach 1

Inflow	=	2.70 cfs @	11.99 hrs,	Volume=	0.134 af
Outflow	=	1.97 cfs @	12.19 hrs,	Volume=	0.132 af, Atten= 27%, Lag= 12.5 min

Routing by Stor-Ind+Trans method, Time Span= 0.00-24.00 hrs, dt= 0.01 hrs
Max. Velocity= 1.7 fps, Min. Travel Time= 8.7 min
Avg. Velocity = 0.7 fps, Avg. Travel Time= 21.6 min

Peak Depth= 0.54'
Capacity at bank full= 64.90 cfs
0.00' x 2.00' deep channel, n= 0.030 Length= 880.0' Slope= 0.0070 '/'
Side Slope Z-value= 3.0 5.0 '/'

Reach 1: Phase III, reach 1



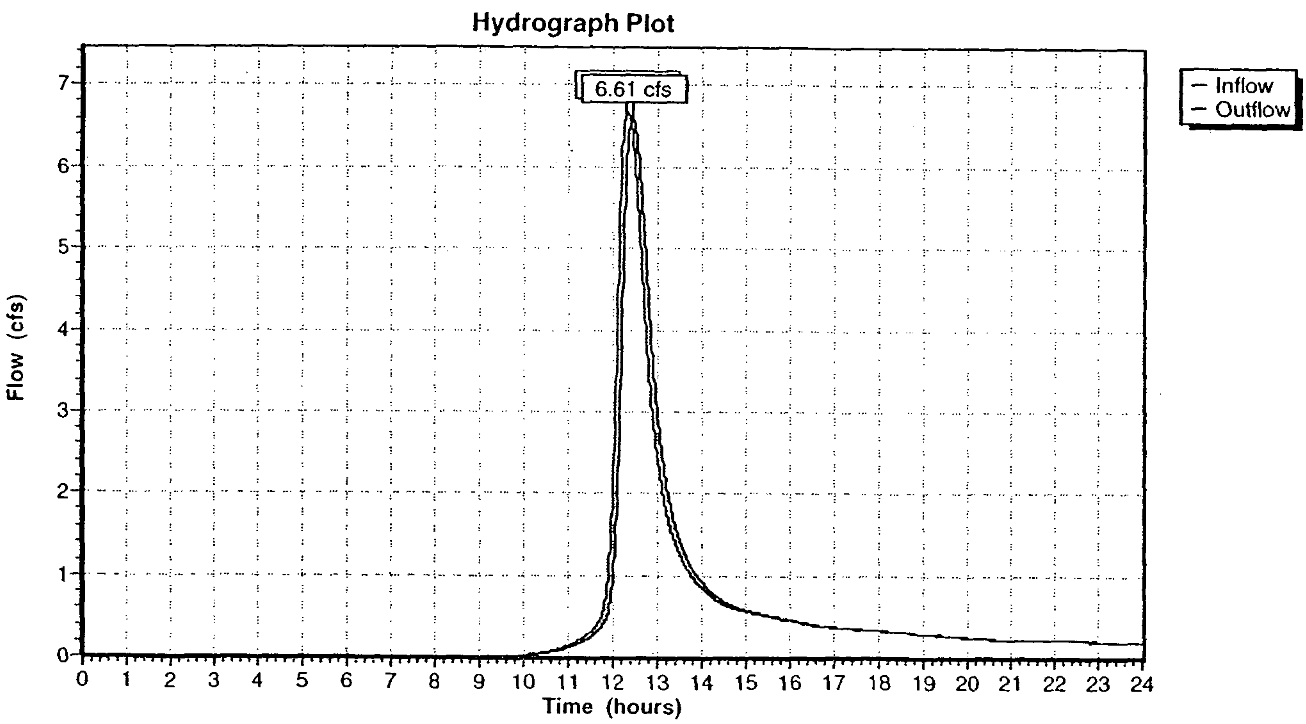
Reach 2: Phase III, reach 2

Inflow	=	6.67 cfs @	12.32 hrs,	Volume=	0.833 af
Outflow	=	6.61 cfs @	12.44 hrs,	Volume=	0.830 af, Atten= 1%, Lag= 6.8 min

Routing by Stor-Ind+Trans method, Time Span= 0.00-24.00 hrs, dt= 0.01 hrs
Max. Velocity= 2.6 fps, Min. Travel Time= 3.3 min
Avg. Velocity = 1.2 fps, Avg. Travel Time= 7.2 min

Peak Depth= 0.75'
Capacity at bank full= 271.26 cfs
0.00' x 3.00' deep channel, n= 0.030 Length= 530.0' Slope= 0.0110 '/'
Side Slope Z-value= 5.0 4.0 '/'

Reach 2: Phase III, reach 2



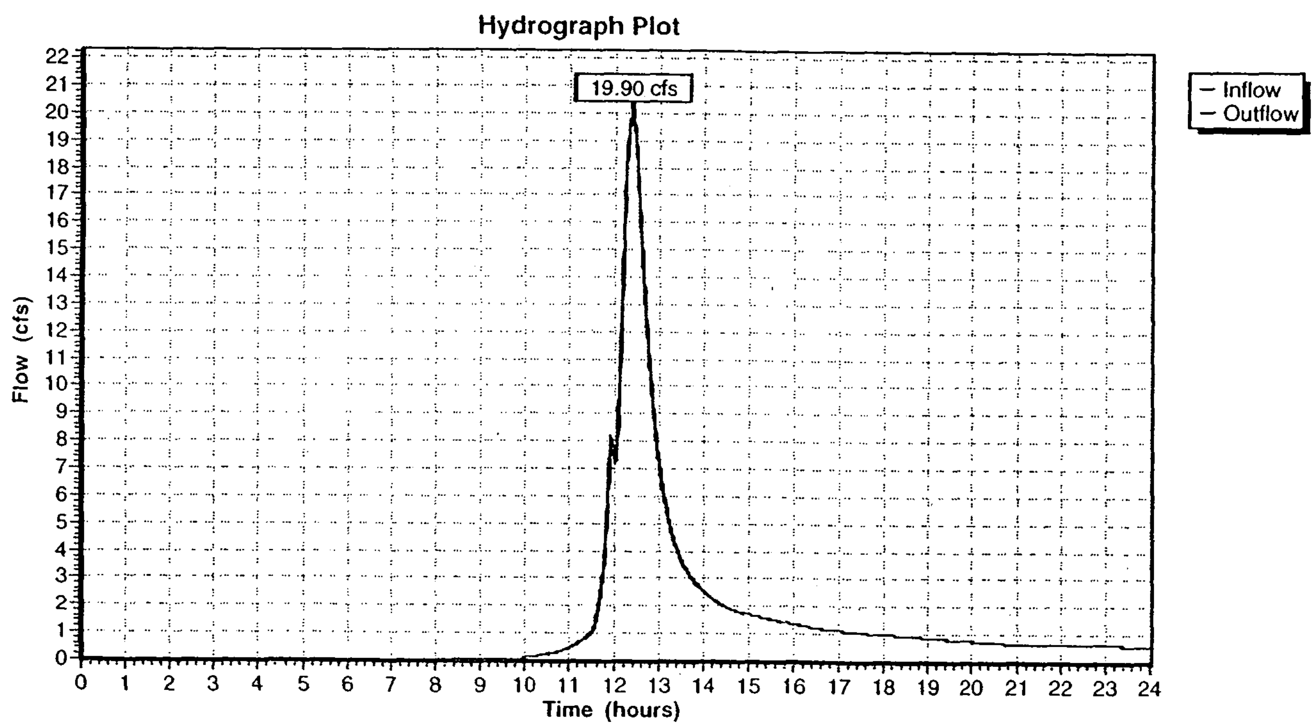
Reach 3: South East Channel- Section One

Inflow	=	19.91 cfs @	12.37 hrs, Volume=	2.547 af	
Outflow	=	19.90 cfs @	12.40 hrs, Volume=	2.545 af,	Atten= 0%, Lag= 1.7 min

Routing by Stor-Ind+Trans method, Time Span= 0.00-24.00 hrs, dt= 0.01 hrs
Max. Velocity= 4.9 fps, Min. Travel Time= 0.9 min
Avg. Velocity = 2.0 fps, Avg. Travel Time= 2.1 min

Peak Depth= 1.17'
Capacity at bank full= 245.90 cfs
0.00' x 3.00' deep channel, n= 0.030 Length= 250.0' Slope= 0.0211 '/'
Side Slope Z-value= 3.0 '/'

Reach 3: South East Channel- Section One



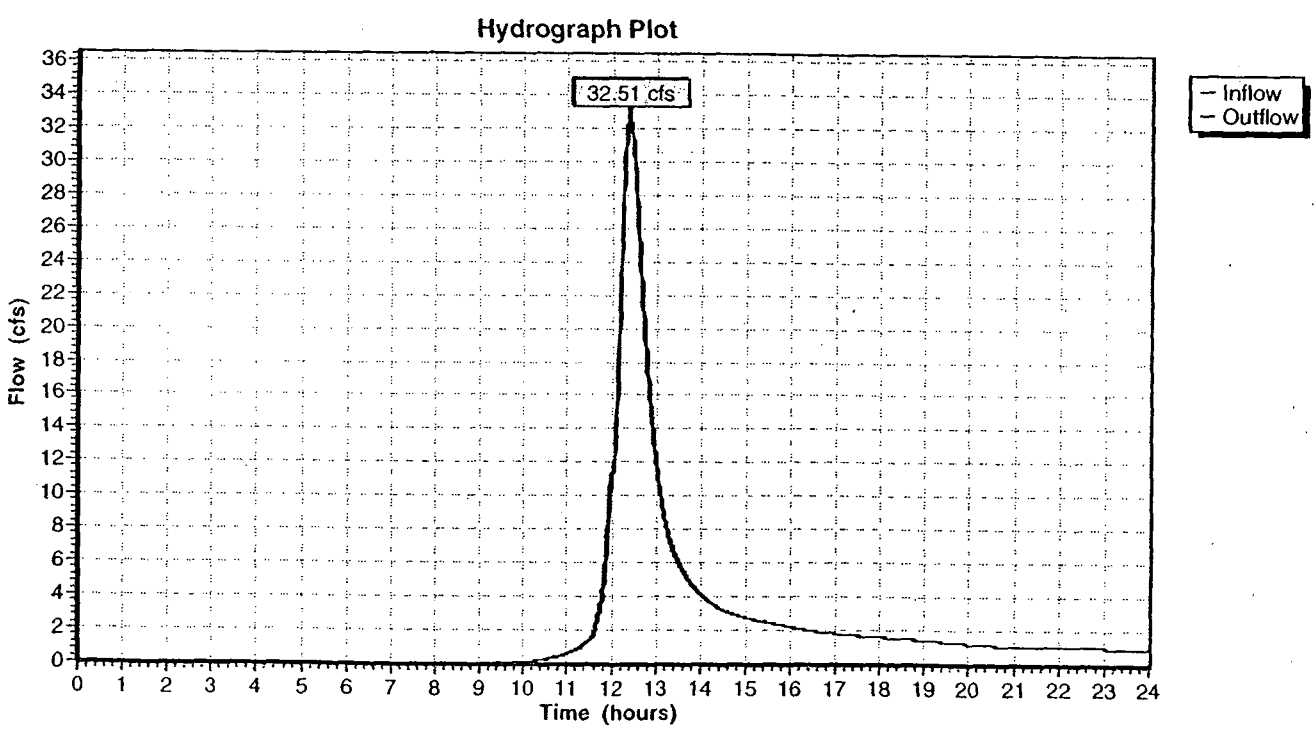
Reach 4: South East Channel- Section Two

Inflow	=	32.57 cfs @	12.38 hrs, Volume=	4.025 af	
Outflow	=	32.51 cfs @	12.42 hrs, Volume=	4.019 af,	Atten= 0%, Lag= 2.4 min

Routing by Stor-Ind+Trans method, Time Span= 0.00-24.00 hrs, dt= 0.01 hrs
Max. Velocity= 3.2 fps, Min. Travel Time= 1.3 min
Avg. Velocity = 1.3 fps, Avg. Travel Time= 3.2 min

Peak Depth= 1.83'
Capacity at bank full= 75.03 cfs
0.00' x 2.50' deep channel, n= 0.030 Length= 250.0' Slope= 0.0052 '/'
Side Slope Z-value= 3.0 '/'

Reach 4: South East Channel- Section Two



ATTACHMENT C-1C

HydroCAD™ OUTPUT REPORTS

DESIGN CASE "B"

DESIGN CASE "B"
25-YEAR, 24-HOUR STORM EVENT

Time span=5.00-20.00 hrs, dt=0.05 hrs, 301 points
Runoff by SCS TR-20 method, UH=SCS, Type II 24-hr Rainfall=4.70"
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment A: runon area east of road
Tc=37.8 min CN=71 Area=11.090 ac Runoff= 15.01 cfs 1.573 af

Subcatchment B: runon area east of road
Tc=30.3 min CN=72 Area=10.610 ac Runoff= 17.54 cfs 1.574 af

Subcatchment C: runon area east of road
Tc=27.4 min CN=79 Area=0.810 ac Runoff= 1.89 cfs 0.158 af

Subcatchment D: runon area east of road
Tc=27.0 min CN=78 Area=1.900 ac Runoff= 4.32 cfs 0.356 af

Subcatchment E: runon area east of road
Tc=23.7 min CN=74 Area=5.610 ac Runoff= 11.81 cfs 0.905 af

Reach 1: culvert 1 Inflow= 15.01 cfs 1.573 af
Length= 50.0' Max Vel= 3.7 fps Capacity= 458.80 cfs Outflow= 15.00 cfs 1.572 af

Reach 2: culvert 2 Inflow= 17.54 cfs 1.574 af
Length= 50.0' Max Vel= 3.9 fps Capacity= 458.80 cfs Outflow= 17.51 cfs 1.574 af

Reach 3: culvert 3 Inflow= 1.89 cfs 0.158 af
Length= 50.0' Max Vel= 2.0 fps Capacity= 458.80 cfs Outflow= 1.88 cfs 0.157 af

Reach 4: culvert 4 Inflow= 4.32 cfs 0.356 af
Length= 50.0' Max Vel= 2.6 fps Capacity= 458.80 cfs Outflow= 4.30 cfs 0.356 af

Reach 5: culvert5 Inflow= 11.81 cfs 0.905 af
Length= 50.0' Max Vel= 3.5 fps Capacity= 458.80 cfs Outflow= 11.78 cfs 0.905 af

Subcatchment A: runon area east of road

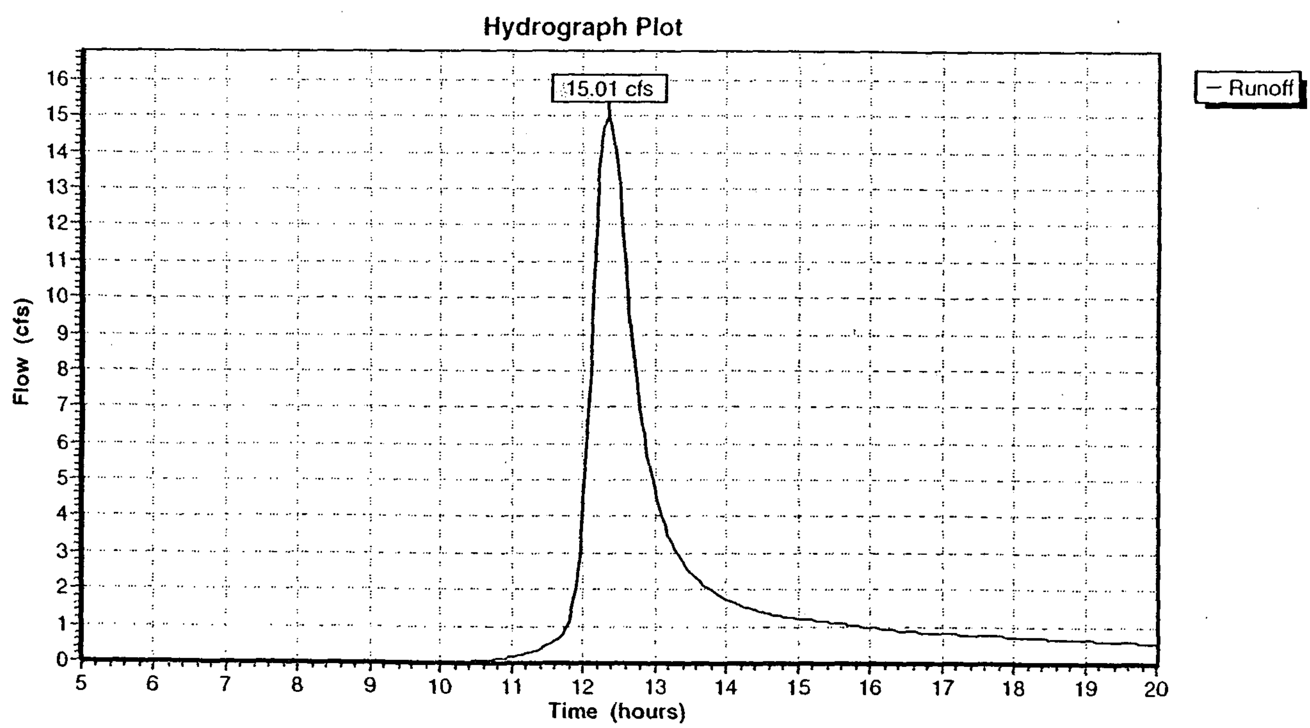
Runoff = 15.01 cfs @ 12.36 hrs, Volume= 1.573 af

Runoff by SCS TR-20 method, UH=SCS, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
Type II 24-hr Rainfall=4.70"

Area (ac)	CN	Description
8.810	69	
2.280	79	
11.090	71	Weighted Average

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
29.4	300	0.0150	0.2		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.60"
8.4	990	0.0150	2.0		**Shallow Concentrated Flow,** Unpaved Kv= 16.1 fps
37.8	1,290	Total			

Subcatchment A: runon area east of road



Subcatchment B: runon area east of road

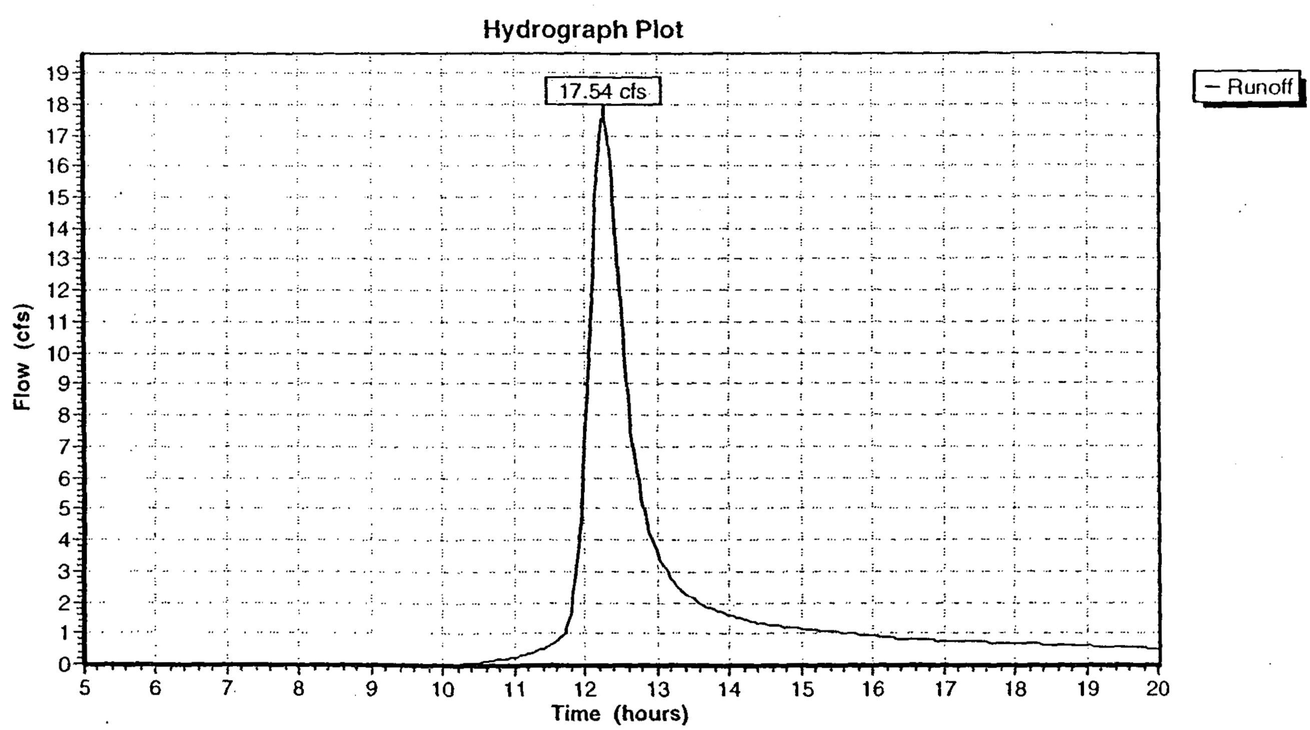
Runoff = 17.54 cfs @ 12.26 hrs, Volume= 1.574 af

Runoff by SCS TR-20 method, UH=SCS, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
Type II 24-hr Rainfall=4.70"

Area (ac)	CN	Description
7.270	69	
3.340	79	
10.610	72	Weighted Average

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
27.3	300	0.0180	0.2		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.60"
3.0	390	0.0180	2.2		**Shallow Concentrated Flow,** Unpaved Kv= 16.1 fps
30.3	690	Total			

Subcatchment B: runon area east of road



Subcatchment C: runon area east of road

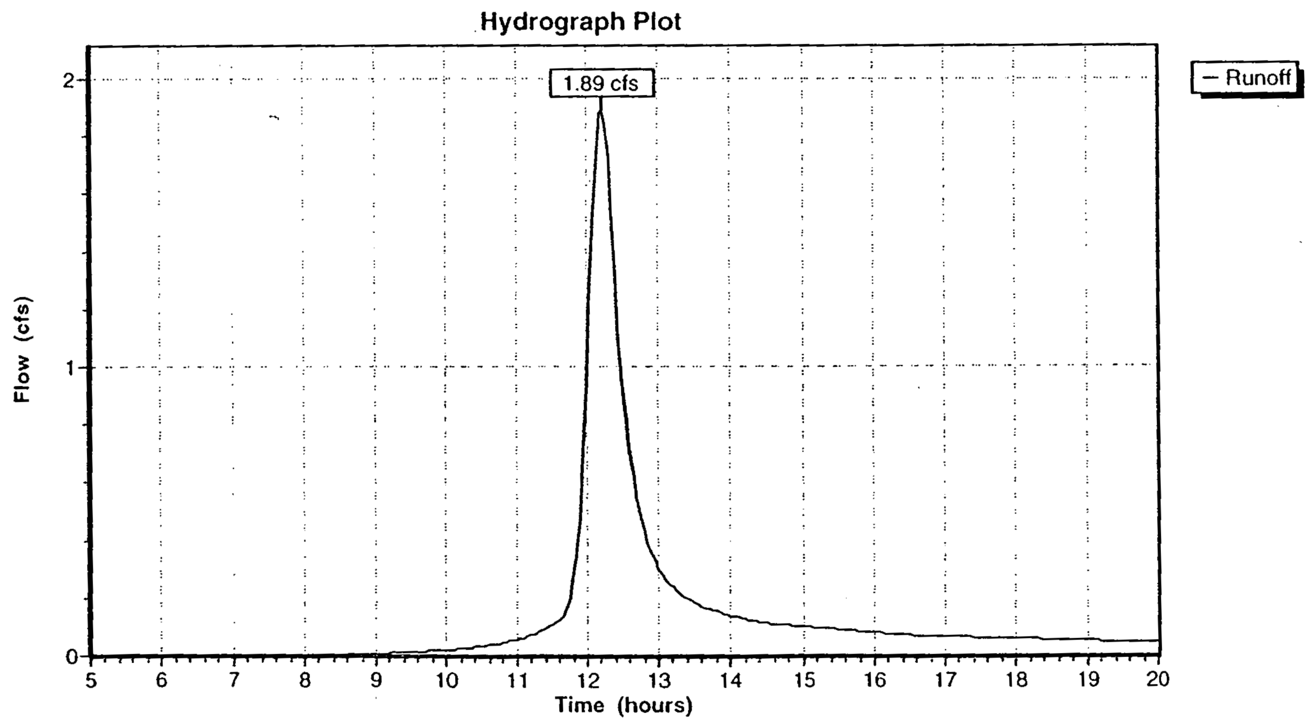
Runoff = 1.89 cfs @ 12.21 hrs, Volume= 0.158 af

Runoff by SCS TR-20 method, UH=SCS, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
Type II 24-hr Rainfall=4.70"

Area (ac)	CN	Description
0.810	79	

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
26.2	300	0.0200	0.2		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.60"
1.2	160	0.0200	2.3		**Shallow Concentrated Flow,** Unpaved Kv= 16.1 fps
27.4	460	Total			

Subcatchment C: runon area east of road



4014

Subcatchment D: runon area east of road

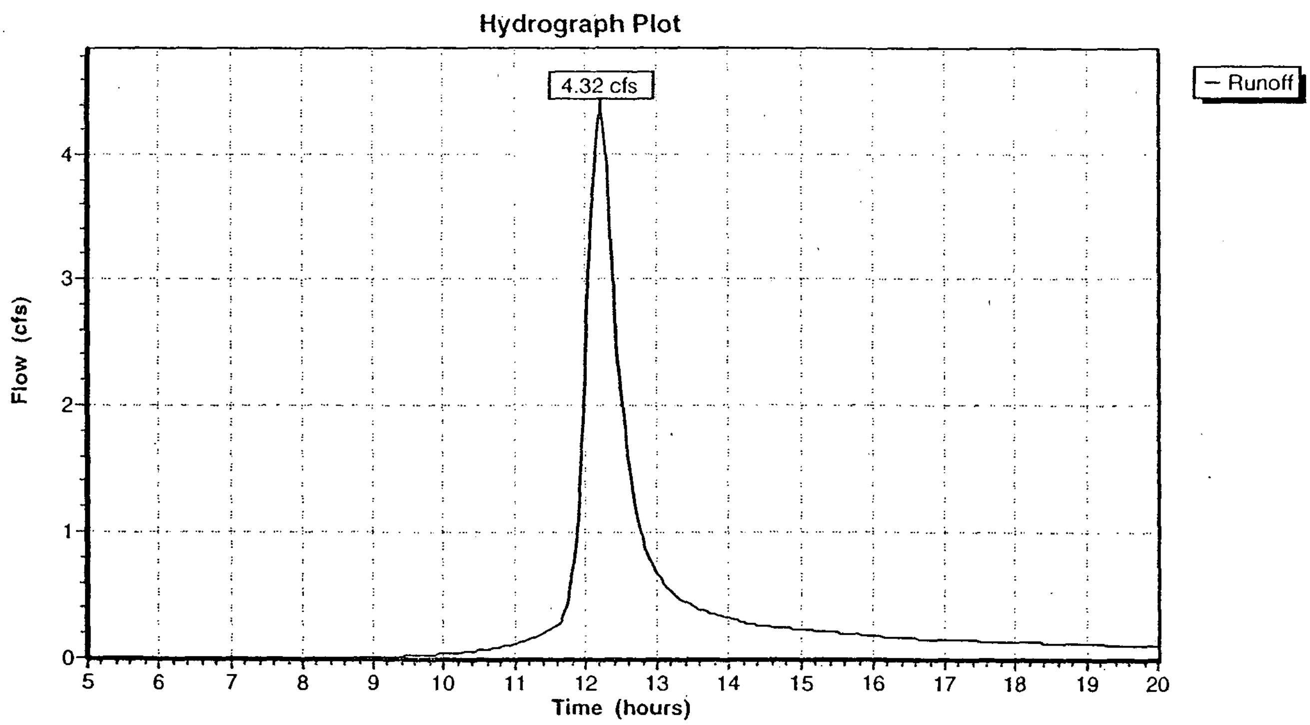
Runoff = 4.32 cfs @ 12.21 hrs, Volume= 0.356 af

Runoff by SCS TR-20 method, UH=SCS, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
Type II 24-hr Rainfall=4.70"

Area (ac)	CN	Description
0.100	69	
1.800	79	
1.900	78	Weighted Average

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
24.8	300	0.0230	0.2		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.60"
2.2	320	0.0230	2.4		**Shallow Concentrated Flow,** Unpaved Kv= 16.1 fps
27.0	620	Total			

Subcatchment D: runon area east of road



4014

Subcatchment E: runon area east of road

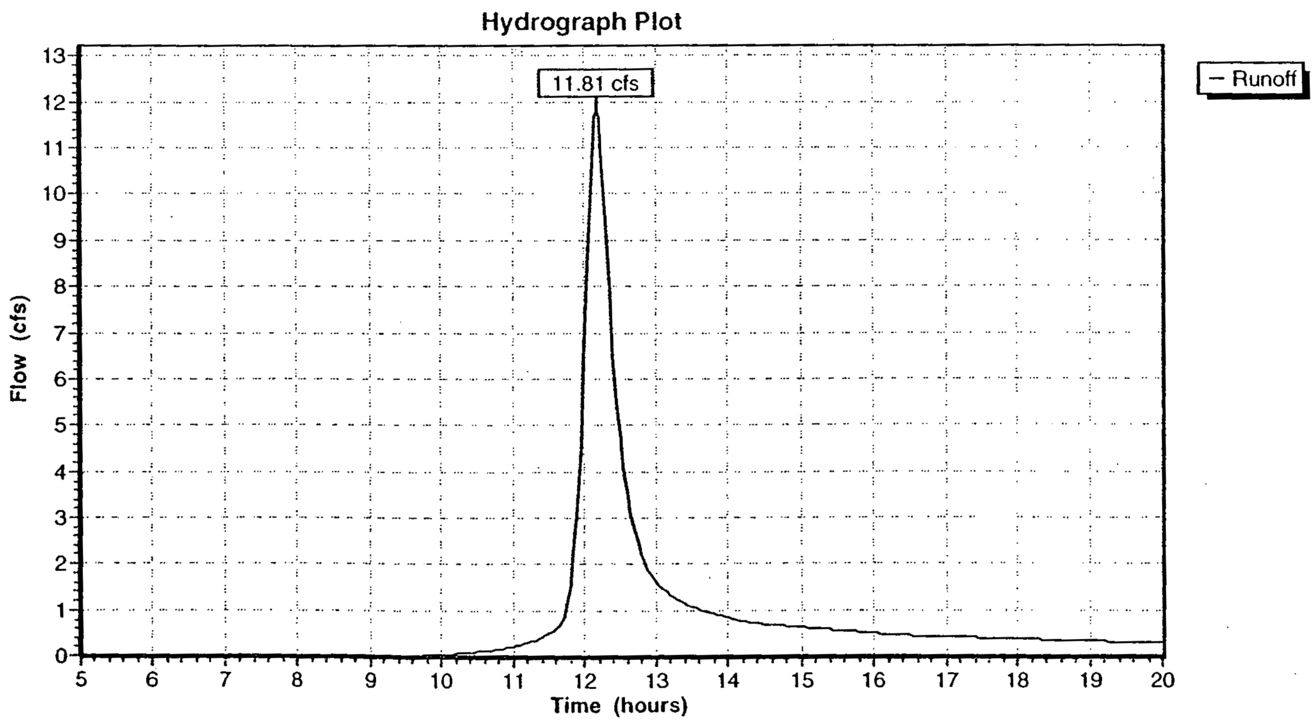
Runoff = 11.81 cfs @ 12.18 hrs, Volume= 0.905 af

Runoff by SCS TR-20 method, UH=SCS, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
Type II 24-hr Rainfall=4.70"

Area (ac)	CN	Description
2.780	69	
2.830	79	
5.610	74	Weighted Average

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
19.8	300	0.0400	0.3		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.60"
3.9	510	0.0180	2.2		**Shallow Concentrated Flow,** Unpaved Kv= 16.1 fps
23.7	810	Total			

Subcatchment E: runon area east of road



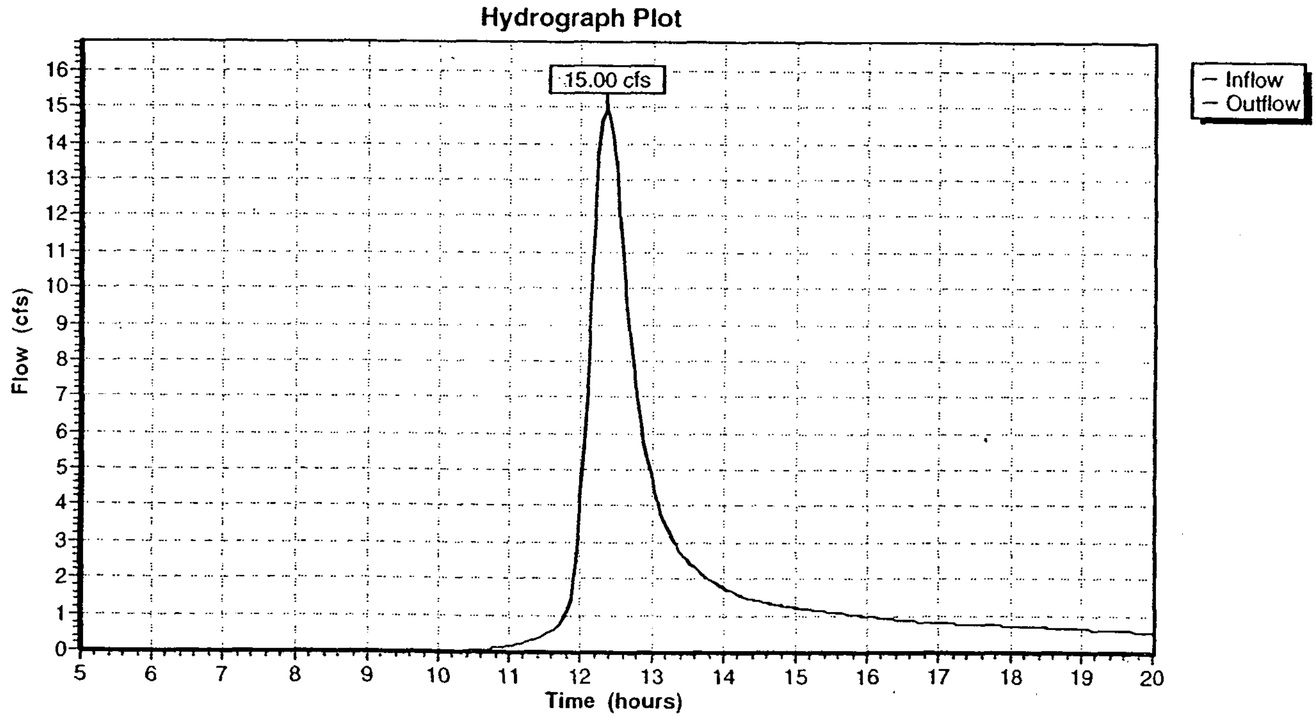
Reach 1: culvert 1

Inflow	=	15.01 cfs @	12.36 hrs, Volume=	1.573 af	
Outflow	=	15.00 cfs @	12.36 hrs, Volume=	1.572 af,	Atten= 0%, Lag= 0.4 min

Routing by Stor-Ind+Trans method, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
Max. Velocity= 3.7 fps, Min. Travel Time= 0.2 min
Avg. Velocity = 1.7 fps, Avg. Travel Time= 0.5 min

Peak Depth= 0.74'
Capacity at bank full= 458.80 cfs
A factor of 2.00 has been applied to the supplied storage and discharge data
72.0" Diameter Pipe n= 0.024 Length= 50.0' Slope= 0.0100 '/'

Reach 1: culvert 1



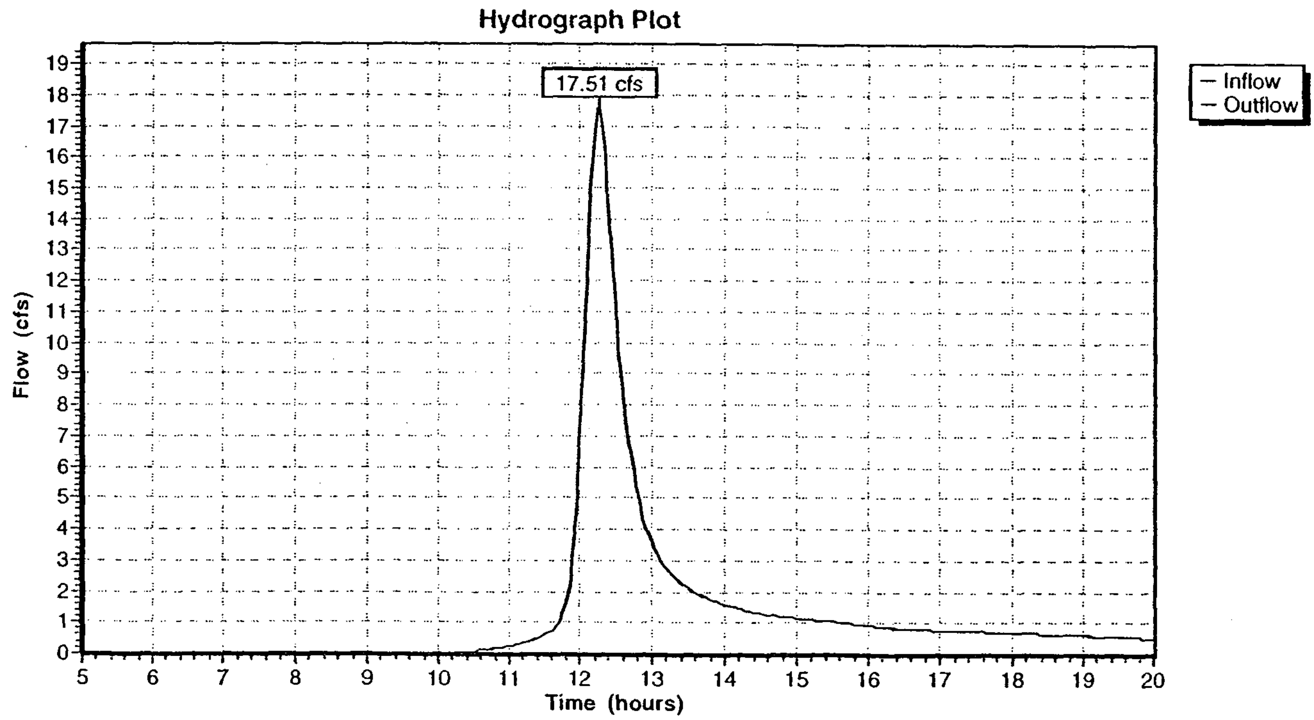
Reach 2: culvert 2

Inflow	=	17.54 cfs @	12.26 hrs, Volume=	1.574 af
Outflow	=	17.51 cfs @	12.26 hrs, Volume=	1.574 af, Atten= 0%, Lag= 0.4 min

Routing by Stor-Ind+Trans method, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
Max. Velocity= 3.9 fps, Min. Travel Time= 0.2 min
Avg. Velocity = 1.7 fps, Avg. Travel Time= 0.5 min

Peak Depth= 0.80'
Capacity at bank full= 458.80 cfs
A factor of 2.00 has been applied to the supplied storage and discharge data
72.0" Diameter Pipe n= 0.024 Length= 50.0' Slope= 0.0100 '/'

Reach 2: culvert 2



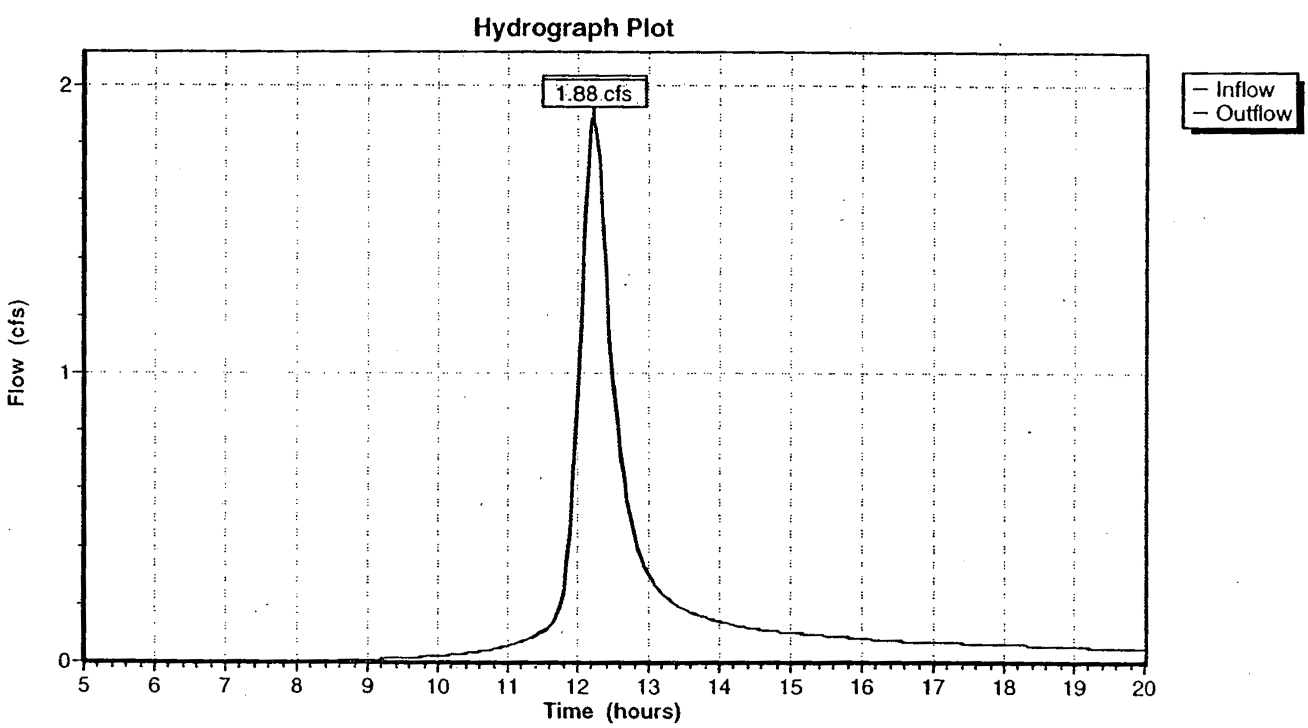
Reach 3: culvert 3

Inflow	=	1.89 cfs @	12.21 hrs, Volume=	0.158 af
Outflow	=	1.88 cfs @	12.23 hrs, Volume=	0.157 af, Atten= 1%, Lag= 0.7 min

Routing by Stor-Ind+Trans method, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
Max. Velocity= 2.0 fps, Min. Travel Time= 0.4 min
Avg. Velocity = 0.9 fps, Avg. Travel Time= 1.0 min

Peak Depth= 0.28'
Capacity at bank full= 458.80 cfs
A factor of 2.00 has been applied to the supplied storage and discharge data
72.0" Diameter Pipe n= 0.024 Length= 50.0' Slope= 0.0100 '/'

Reach 3: culvert 3



Reach 4: culvert 4

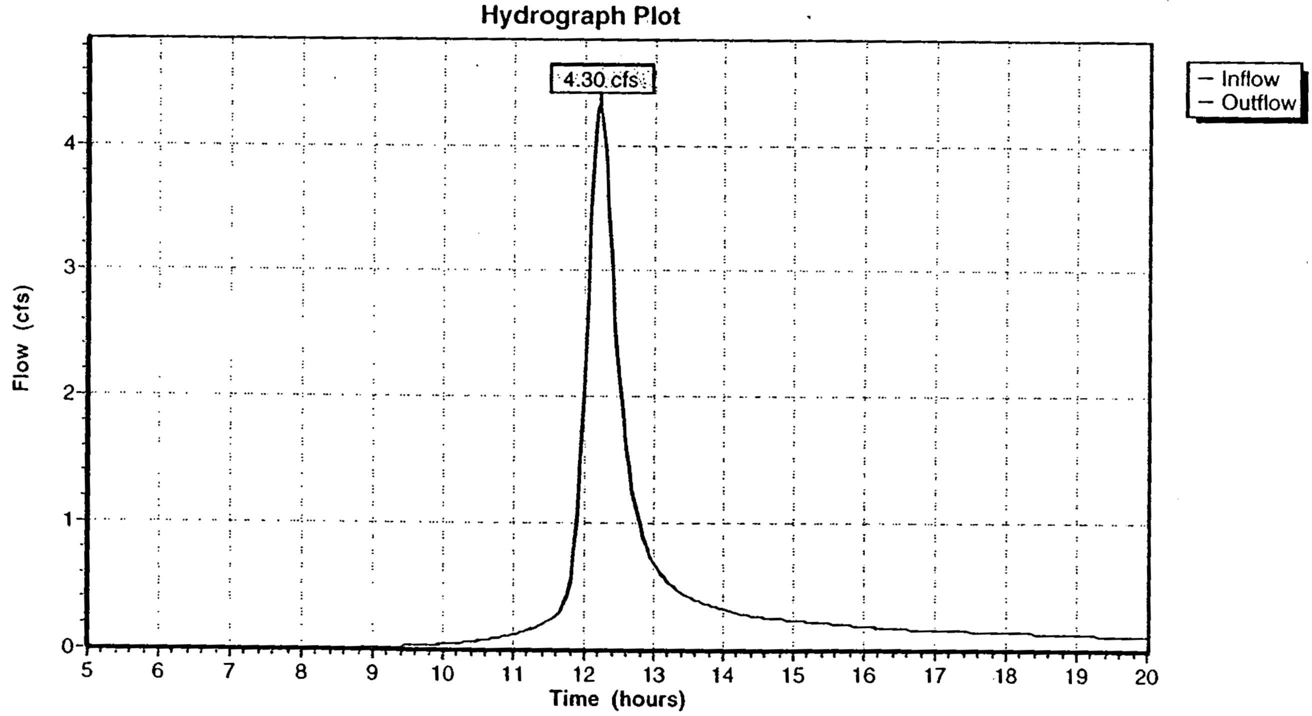
Inflow	=	4.32 cfs @	12.21 hrs,	Volume=	0.356 af
Outflow	=	4.30 cfs @	12.22 hrs,	Volume=	0.356 af, Atten= 0%, Lag= 0.6 min

Routing by Stor-Ind+Trans method, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
Max. Velocity= 2.6 fps, Min. Travel Time= 0.3 min
Avg. Velocity = 1.1 fps, Avg. Travel Time= 0.8 min

Peak Depth= 0.41'
Capacity at bank full= 458.80 cfs
A factor of 2.00 has been applied to the supplied storage and discharge data
72.0" Diameter Pipe n= 0.024 Length= 50.0' Slope= 0.0100 '/'

Reach 4: culvert 4

Hydrograph Plot



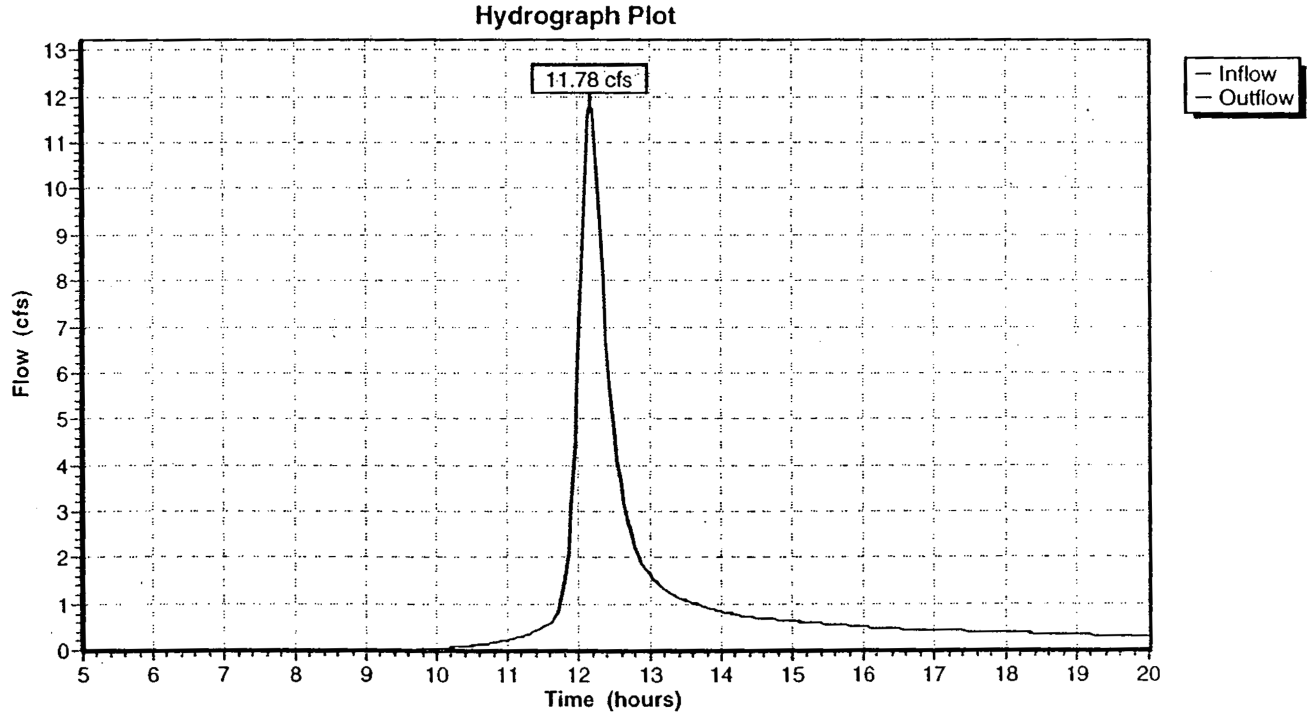
Reach 5: culvert5

Inflow	=	11.81 cfs @	12.18 hrs,	Volume=	0.905 af
Outflow	=	11.78 cfs @	12.18 hrs,	Volume=	0.905 af, Atten= 0%, Lag= 0.5 min

Routing by Stor-Ind+Trans method, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
Max. Velocity= 3.5 fps, Min. Travel Time= 0.2 min
Avg. Velocity = 1.4 fps, Avg. Travel Time= 0.6 min

Peak Depth= 0.66'
Capacity at bank full= 458.80 cfs
A factor of 2.00 has been applied to the supplied storage and discharge data
72.0" Diameter Pipe n= 0.024 Length= 50.0' Slope= 0.0100 '/'

Reach 5: culvert5



ATTACHMENT C-2

WEIGHTED CURVE NUMBER CALCULATIONS

1-03-204

RESULTS FOR THE CALCULATION OF WEIGHTED CN
OSDF Design Scenario

Subcatchment Label	Area (acres)	Percent of Total Area %	HSG	Land Use Category	CN	Weighted CN
A	2.05	100%	B/C	Vegetated Final Cover System	83	83
B	1.10	100%	B/C	Vegetated Final Cover System	83	83
C	4.56	100%	B/C	Unvegetated Final Cover System	89	89
D	2.59	100%	B/C	Unvegetated Final Cover System	89	89
E	1.30	100%	B/C	Unvegetated Final Cover System	89	89
F	1.44	100%	B/C	Unvegetated Final Cover System	89	89
G	1.75	100%	C	Disturbed Area - Construction Support	82	82
H	1.78	100%	C	Disturbed Area - Construction Support	82	82
I	2.16	78% 22%	B/C C	Liner Runout Disturbed Area - Construction Support	89 82	87
J	0.49	100%	C	Disturbed Area - Construction Support	82	82
K	0.17	100%	C	Disturbed Area - Construction Support	82	82
L	0.14	100%	C	Disturbed Area - Construction Support	82	82
M	1.62	100%	N/A	Sedimentation Basin	98	98

N/A - Not Applicable

27. 204

RESULTS FOR THE CALCULATION OF WEIGHTED CN
Design Case "A"

Subcatchment Label	Area (acres)	Percent of Total Area %	HSG	Land Use Category	CN	Weighted CN
A	0.63	100%	C	Runon Area East of OSDF	79	79
B	4.00	50% 50%	B C	Runon Area East of OSDF Runon Area East of OSDF	69 79	74
C	8.86	60% 40%	B C	Runon Area East of OSDF Runon Area East of OSDF	69 79	73
D	0.77	100%	B/C	Unvegetated Final Cover System	89	89
E	9.12	70% 30%	B C	Runon Area East of OSDF Runon Area East of OSDF	69 79	72

RESULTS FOR THE CALCULATION OF WEIGHTED CN
Design Case "B"

Subcatchment Label	Area (acres)	Percent of Total Area %	HSG	Land Use Category	CN	Weighted CN
A	11.09	80% 20%	B C	Runon Area East of OSDF Runon Area East of OSDF	69 79	71
B	10.61	70% 30%	B C	Runon Area East of OSDF Runon Area East of OSDF	69 79	72
C	0.81	100%	C	Runon Area East of OSDF	79	79
D	1.90	10% 90%	B C	Runon Area East of OSDF Runon Area East of OSDF	69 79	78
E	5.61	50% 50%	B C	Runon Area East of OSDF Runon Area East of OSDF	69 79	74

GEOSYNTEC CONSULTANTS Page ____ of ____

Written by: Dana Mehlman (DBM) Date: 8/23/2001 Reviewed by: ____ Date: ____

Client: Fluor Fernald, Inc. Project: OSDF PHASE IV Project/Proposal No.:GQ1342 Task No.:16

EXAMPLE CALCULATION FOR WEIGHTED CURVE NUMBER OF SUBCATCHMENT I

PURPOSE

The computation below illustrates the method for calculating a composite curve number (CN) for a subcatchment area comprising more than one CN. This CN value is used as one of the input parameters in the computer program "HydroCAD" for computing runoff.

CALCULATION PROCEDURE

The composite CN for a subcatchment area comprising more than one CN is calculated by summing the products of each CN multiplied by its percentage of the total area.

DATA VERIFICATION

The table below lists the CN's for subcatchment area I grouped according to Hydrological Soil Group (HSG), the contributing percentage of the total area for subcatchment area I, and land use.

HSG	Percent of Area %	Land Use Description	CN
B/C	78	Liner Runout	89
C	22	Disturbed Area-Construction Support Zone	82

CALCULATIONS

Weighted CN = (89)(0.78)+(82)(0.22)

= 87

ATTACHMENT C-3

ANALYSIS RESULTS FOR CHANNELS

SUMMARY OF CHANNEL ANALYSIS RESULTS

Channel Identification			Channel Characteristics							Hydrologic Calculations		Hydraulic Calculations							
Channel Name[1]	Status	Design Scenario	Section Shape	Available Flow Depth (ft)	Longitudinal Slope[2] (%)	Manning n	Bottom Width B (ft)	Side Slope M_1:1	Side Slope M_2:1	HydroCAD Node[3]	HydroCAD Q[4] (cfs)	Area of Flow A (sq ft)	Perimeter P (ft)	Hydraulic Radius, R (ft)	Peak Flow Depth[5] Y (ft)	Estimated Q[5] (cfs)	Channel Freeboard[7] (ft)	Peak Flow Velocity (fps)	Lining Type[6]
G6	new	OSDF	vee	3	1.39%	0.030	0	3	3	6	34.26	7.21	9.80	0.74	1.55	34.13	1.45	4.7	grass
1	new	OSDF	vee	2	1.00%	0.030	0	3	3	N/A	12.11 [8]	3.70	7.02	0.53	1.11	11.97	0.89	3.5	grass
1	new	OSDF	vee	2	1.60%	0.030	0	3	3	N/A	1.53 [9]	0.66	2.97	0.22	0.47	1.53	1.53	2.4	grass
8	new	OSDF	vee	3	1.60%	0.030	0	3	3	8	1.53	0.66	2.97	0.22	0.47	1.53	2.53	2.3	grass
9	new	OSDF	vee	4	1.47%	0.030	0	3	3	9	12.11	3.18	6.51	0.49	1.03	11.89	2.97	3.8	grass
10	new	OSDF	vee	3	1.47%	0.030	0	3	3	10	12.84	3.43	6.77	0.51	1.07	13.16	1.93	3.8	grass
1A	existing	DC A	vee	1.6	0.77%	0.030	0	6	4	1	2.70	1.51	5.61	0.27	0.55	2.75	1.05	1.8	grass
1B	existing	DC A	vee	2.3	0.67%	0.030	0	5	3	1	2.70	1.49	5.04	0.30	0.61	2.68	1.69	1.8	grass
1C	existing	DC A	vee	2	0.45%	0.030	0	4	2	1	2.70	1.60	4.64	0.34	0.73	2.62	1.27	1.6	grass
1D	existing	DC A	vee	2.8	0.81%	0.030	0	4	2	1	2.70	1.35	4.26	0.32	0.67	2.79	2.13	2.1	grass
2A	existing	DC A	vee	4	0.67%	0.030	0	6	3	2	6.67	3.10	7.67	0.40	0.83	6.89	3.17	2.2	grass
2B	existing	DC A	vee	3	1.00%	0.030	0	4	5	2	6.67	2.67	7.10	0.38	0.77	6.90	2.23	2.6	grass
2C	existing	DC A	vee	4	1.50%	0.030	0	4	5	2	6.67	2.27	6.55	0.35	0.71	6.80	3.29	3.0	grass
3	new	DC A	vee	3	2.11%	0.030	0	3	3	3	19.91	4.11	7.40	0.55	1.17	20.00	1.83	4.9	grass
4	new	DC A	vee	2.5	0.52%	0.030	0	3	3	4	32.57	10.05	11.57	0.87	1.83	32.74	0.67	3.3	grass

1. Channels are named after the corresponding subcatchment or reach.
2. Longitudinal slopes taken from the Drawings.
3. N/A indicates that there is not specific HydroCAD node associated with the particular channel.
4. Peak flow rates calculated by HydroCAD for each reach. See attachment C-1.
5. Calculated flow rates using an iterative procedure and compared with flow from HydroCAD.
6. Maximum permissible velocity for grass lined channels is 5 fps.
7. Calculated as the difference between minimum available flow depth and peak flow depth.
8. HydroCAD Q assumed to equal the inflow to reach 9.
9. HydroCAD Q assumed to equal the inflow to reach 8.

4014

000183

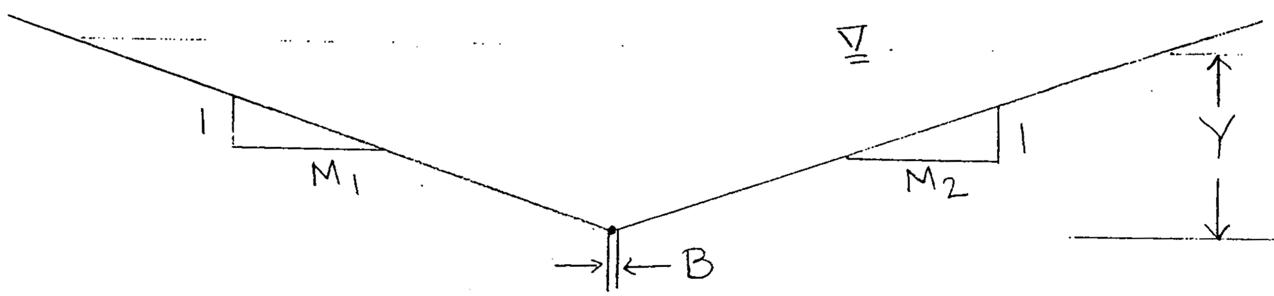
Written by: DBM Date: 01/08/15 Reviewed by: ______ Date: __/__/__

Client: Fernald Project: OSDF - Phase IV Project/Proposal No.: [illegible] Task No.: 16

EXAMPLE CALCULATION FOR ANALYSIS OF CHANNEL G6 - OSDF DESIGN SCENARIO

PEAK DISCHARGE (25 YR - 24 HOUR STORM EVENT)	Q_{MAX}	= 34.26 cfs
BASE WIDTH	B	= 0 ft
LEFT SIDE SLOPE	M_1	= 3H : 1V
RIGHT SIDE SLOPE	M_2	= 3H : 1V
MANNING'S ROUGHNESS COEFFICIENT	n	= 0.030
LONGITUDINAL CHANNEL SLOPE	S_0	= 0.0137 ft/ft

CHANNEL GEOMETRY (NOT TO SCALE)



- ASSUME DEPTH OF FLOW = 1.55 ft = Y

AREA OF FLOW

$$A = \frac{1}{2} Y (2B + (M_1 Y + M_2 Y))$$

$$= YB + M_1 \frac{Y^2}{2} + M_2 \frac{Y^2}{2}$$

$$= 1.55(0) + 3\left(\frac{1.55^2}{2}\right) + 3\left(\frac{1.55^2}{2}\right)$$

$$= 7.2075 \text{ ft}^2$$

$$\approx \underline{\underline{7.21 \text{ ft}^2}}$$

Written by: DBM Date: 01/03/15 Reviewed by: ________ Date: __/__/__

Client: Fernald Project: OSDF Phase IV Project/Proposal No.: GQ1342 Task No.: 16

WETTED PERIMETER

$$P = B + \sqrt{Y^2 + (M_1 Y)^2} + \sqrt{Y^2 + (M_2 Y)^2}$$
$$= 0 + \sqrt{1.55^2 + (3*1.55)^2} + \sqrt{1.55^2 + (3*1.55)^2}$$
$$= 0 + 4.901 + 4.901$$
$$= \underline{9.80 \text{ ft}}$$

HYDRAULIC RADIUS

$$R = A/P$$
$$= 7.21/9.80$$
$$= \underline{0.74 \text{ ft}}$$

AVERAGE VELOCITY

$$V = Q/A = \frac{1}{A}\left(\frac{1.49}{n} * A * R^{2/3} * S_o^{1/2}\right) = \frac{1.49}{n} * R^{2/3} * S_o^{1/2}$$
$$= \frac{1.49}{0.030} * (.74)^{2/3} * (0.0137)^{1/2}$$
$$= 4.737$$
$$= \underline{4.74 \text{ ft/s}}$$

DISCHARGE (VERIFICATION)

$$Q_{MAX} = A * V$$
$$= 7.21 * 4.74$$
$$= \underline{34.17 \text{ cfs}}$$

ATTACHMENT C-4A

Culvertmaster® OUTPUT REPORTS FOR CULVERTS

177/

Culvert Designer/Analyzer Report
culvert 1-OSDF

Peak Discharge Method: User-Specified			
Design Discharge	34.14 cfs	Check Discharge	0.00 cfs

Grades Model: Inverts			
Invert Upstream	594.00 ft	Invert Downstream	592.66 ft
Length	96.10 ft	Slope	0.014 ft/ft
Drop	1.34 ft		

Headwater Model: Maximum Allowable HW	
Headwater Elevation	597.85 ft

Tailwater Conditions: Constant Tailwater	
Tailwater Elevation	594.22 ft

Name	Description	Discharge	HW Elev.	Velocity
x Trial-1	1-36 inch Circular	34.14 cfs	597.36 ft	

000187

173/...

Culvert Designer/Analyzer Report
culvert 1-OSDF

Design:Trial-1

Solve For: Headwater Elevation

Culvert Summary			
Allowable HW Elevation	597.85 ft	Storm Event	Design
Computed Headwater Elevi	597.36 ft	Discharge	34.14 cfs
Headwater Depth/Height	1.12	Tailwater Elevation	594.22 ft
Inlet Control HW Elev.	597.17 ft	Control Type	Outlet Control
Outlet Control HW Elev.	597.36 ft		

Grades			
Upstream Invert	594.00 ft	Downstream Invert	592.66 ft
Length	96.10 ft	Constructed Slope	0.014 ft/ft

Hydraulic Profile			
Profile	M2	Depth, Downstream	1.90 ft
Slope Type	Mild	Normal Depth	2.03 ft
Flow Regime	Subcritical	Critical Depth	1.90 ft
Velocity Downstream	7.24 ft/s	Critical Slope	0.017 ft/ft

Section			
Section Shape	Circular	Mannings Coefficient	0.024
Section Material	CMP	Span	3.00 ft
Section Size	36 inch	Rise	3.00 ft
Number Sections	1		

Outlet Control Properties			
Outlet Control HW Elev.	597.36 ft	Upstream Velocity Head	0.70 ft
Ke	0.90	Entrance Loss	0.63 ft

Inlet Control Properties			
Inlet Control HW Elev.	597.17 ft	Flow Control	Unsubmerged
Inlet Type	Projecting	Area Full	7.1 ft^2
K	0.03400	HDS 5 Chart	2
M	1.50000	HDS 5 Scale	3
C	0.05530	Equation Form	1
Y	0.54000		

000188

4014

Culvert Designer/Analyzer Report
culvert 2-OSDF

179/204

Peak Discharge Method: User-Specified			
Design Discharge	49.39 cfs	Check Discharge	0.00 cfs

Grades Model: Inverts			
Invert Upstream	583.14 ft	Invert Downstream	581.89 ft
Length	90.00 ft	Slope	0.014 ft/ft
Drop	1.25 ft		

Headwater Model: Maximum Allowable HW	
Headwater Elevation	587.00 ft

Tailwater Conditions: Constant Tailwater	
Tailwater Elevation	585.82 ft

Name	Description	Discharge	HW Elev.	Velocity
x Trial-1	2-42 inch Circular	49.39 cfs	586.23 ft	

000189

GeoSyntec Consultants
37 Brookside Road Waterbury, CT 06708 USA +1-203-755-1666
Project Engineer: Dana Mehlman

4014

/30/204

Culvert Designer/Analyzer Report
culvert 2-OSDF

Design:Trial-1

Solve For: Headwater Elevation

Culvert Summary			
Allowable HW Elevation	587.00 ft	Storm Event	Design
Computed Headwater Elev:	586.23 ft	Discharge	49.39 cfs
Headwater Depth/Height	0.88	Tailwater Elevation	585.82 ft
Inlet Control HW Elev.	585.82 ft	Control Type	Outlet Control
Outlet Control HW Elev.	586.23 ft		

Grades			
Upstream Invert	583.14 ft	Downstream Invert	581.89 ft
Length	90.00 ft	Constructed Slope	0.014 ft/ft

Hydraulic Profile			
Profile	CompositePressureProfileS1	Depth, Downstream	3.93 ft
Slope Type	N/A	Normal Depth	1.51 ft
Flow Regime	Subcritical	Critical Depth	1.53 ft
Velocity Downstream	2.57 ft/s	Critical Slope	0.013 ft/ft

Section			
Section Shape	Circular	Mannings Coefficient	0.024
Section Material	CMP	Span	3.50 ft
Section Size	42 inch	Rise	3.50 ft
Number Sections	2		

Outlet Control Properties			
Outlet Control HW Elev.	586.23 ft	Upstream Velocity Head	0.14 ft
Ke	0.90	Entrance Loss	0.12 ft

Inlet Control Properties			
Inlet Control HW Elev.	585.82 ft	Flow Control	Unsubmerged
Inlet Type	Projecting	Area Full	19.2 ft^2
K	0.03400	HDS 5 Chart	2
M	1.50000	HDS 5 Scale	3
C	0.05530	Equation Form	1
Y	0.54000		

000190

:81, 204

Culvert Designer/Analyzer Report
culvert 3-OSDF

Peak Discharge Method: User-Specified			
Design Discharge	1.55 cfs	Check Discharge	0.00 cfs

Grades Model: Inverts			
Invert Upstream	586.50 ft	Invert Downstream	585.32 ft
Length	73.75 ft	Slope	0.016 ft/ft
Drop	1.18 ft		

Headwater Model: Maximum Allowable HW	
Headwater Elevation	588.00 ft

Tailwater Conditions: Constant Tailwater	
Tailwater Elevation	585.79 ft

Name	Description	Discharge	HW Elev.	Velocity
x Trial-1	1-12 inch Circular	1.55 cfs	587.40 ft	

Culvert Designer/Analyzer Report
culvert 3-OSDF

Design:Trial-1

Solve For: Section Size

Culvert Summary

Allowable HW Elevation	588.00 ft	Storm Event	Design
Computed Headwater Elev:	587.40 ft	Discharge	1.55 cfs
Headwater Depth/Height	0.90	Tailwater Elevation	585.79 ft
Inlet Control HW Elev.	587.33 ft	Control Type	Outlet Control
Outlet Control HW Elev.	587.40 ft		

Grades

Upstream Invert	586.50 ft	Downstream Invert	585.32 ft
Length	73.75 ft	Constructed Slope	0.016 ft/ft

Hydraulic Profile

Profile	M2	Depth, Downstream	0.53 ft
Slope Type	Mild	Normal Depth	0.58 ft
Flow Regime	Subcritical	Critical Depth	0.53 ft
Velocity Downstream	3.68 ft/s	Critical Slope	0.021 ft/ft

Section

Section Shape	Circular	Mannings Coefficient	0.024
Section Material	CMP	Span	1.00 ft
Section Size	12 inch	Rise	1.00 ft
Number Sections	1		

Outlet Control Properties

Outlet Control HW Elev.	587.40 ft	Upstream Velocity Head	0.17 ft
Ke	0.90	Entrance Loss	0.15 ft

Inlet Control Properties

Inlet Control HW Elev.	587.33 ft	Flow Control	N/A
Inlet Type	Projecting	Area Full	0.8 ft^2
K	0.03400	HDS 5 Chart	2
M	1.50000	HDS 5 Scale	3
C	0.05530	Equation Form	1
Y	0.54000		

000192

152, 204

Culvert Designer/Analyzer Report
culvert 4-OSDF

Peak Discharge Method: User-Specified			
Design Discharge	33.64 cfs	Check Discharge	0.00 cfs

Grades Model: Inverts			
Invert Upstream	589.22 ft	Invert Downstream	587.89 ft
Length	95.50 ft	Slope	0.014 ft/ft
Drop	1.33 ft		

Headwater Model: Maximum Allowable HW	
Headwater Elevation	593.50 ft

Tailwater Conditions: Constant Tailwater	
Tailwater Elevation	589.45 ft

Name	Description	Discharge	HW Elev.	Velocity
x Trial-1	1-36 inch Circular	33.64 cfs	592.55 ft	

000193

/34/204

Culvert Designer/Analyzer Report
culvert 4-OSDF

Design: Trial-1

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	593.50	ft	Storm Event	Design	
Computed Headwater Elev:	592.55	ft	Discharge	33.64	cfs
Headwater Depth/Height	1.11		Tailwater Elevation	589.45	ft
Inlet Control HW Elev.	592.35	ft	Control Type	Outlet Control	
Outlet Control HW Elev.	592.55	ft			

Grades					
Upstream Invert	589.22	ft	Downstream Invert	587.89	ft
Length	95.50	ft	Constructed Slope	0.014	ft/ft

Hydraulic Profile					
Profile	M2		Depth, Downstream	1.88	ft
Slope Type	Mild		Normal Depth	2.01	ft
Flow Regime	Subcritical		Critical Depth	1.88	ft
Velocity Downstream	7.20	ft/s	Critical Slope	0.017	ft/ft

Section					
Section Shape	Circular		Mannings Coefficient	0.024	
Section Material	CMP		Span	3.00	ft
Section Size	36 inch		Rise	3.00	ft
Number Sections	1				

Outlet Control Properties					
Outlet Control HW Elev.	592.55	ft	Upstream Velocity Head	0.69	ft
Ke	0.90		Entrance Loss	0.62	ft

Inlet Control Properties					
Inlet Control HW Elev.	592.35	ft	Flow Control	Unsubmerged	
Inlet Type	Projecting		Area Full	7.1	ft^2
K	0.03400		HDS 5 Chart	2	
M	1.50000		HDS 5 Scale	3	
C	0.05530		Equation Form	1	
Y	0.54000				

000194

4014

Culvert Designer/Analyzer Report
culvert 5 - OSDF

135/204

Peak Discharge Method: User-Specified

Design Discharge	33.64 cfs	Check Discharge	0.00 cfs

Grades Model: Inverts

Invert Upstream	584.39 ft	Invert Downstream	583.63 ft
Length	55.00 ft	Slope	0.013900 ft/ft
Drop	0.76 ft		

Headwater Model: Maximum Allowable HW

Headwater Elevation	590.00 ft

Tailwater Conditions: Constant Tailwater

Tailwater Elevation	585.82 ft

Name	Description	Discharge	HW Elev.	Velocity
x Trial-1	1-42 inch Circular	33.64 cfs	587.54 ft	

000195

Culvert Designer/Analyzer Report
culvert 5 - OSDF

186; 204

Design: Trial-1

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	590.00	ft	Storm Event	Design	
Computed Headwater Elevi	587.54	ft	Discharge	33.64	cfs
Headwater Depth/Height	0.90		Tailwater Elevation	585.82	ft
Inlet Control HW Elev.	587.18	ft	Control Type	Outlet Control	
Outlet Control HW Elev.	587.54	ft			

Grades					
Upstream Invert	584.39	ft	Downstream Invert	583.63	ft
Length	55.00	ft	Constructed Slope	0.013900	ft/ft

Hydraulic Profile					
Profile	M1		Depth, Downstream	2.19	ft
Slope Type	Mild		Normal Depth	1.80	ft
Flow Regime	Subcritical		Critical Depth	1.80	ft
Velocity Downstream	5.30	ft/s	Critical Slope	0.013947	ft/ft

Section					
Section Shape	Circular		Mannings Coefficient	0.024	
Section Material	CMP		Span	3.50	ft
Section Size	42 inch		Rise	3.50	ft
Number Sections	1				

Outlet Control Properties					
Outlet Control HW Elev.	587.54	ft	Upstream Velocity Head	0.71	ft
Ke	0.90		Entrance Loss	0.64	ft

Inlet Control Properties					
Inlet Control HW Elev.	587.18	ft	Flow Control	Unsubmerged	
Inlet Type	Projecting		Area Full	9.6	ft^2
K	0.03400		HDS 5 Chart	2	
M	1.50000		HDS 5 Scale	3	
C	0.05530		Equation Form	1	
Y	0.54000				

4014

Culvert Designer/Analyzer Report
culvert 1 - DC A

187/204

Peak Discharge Method: User-Specified					
Design Discharge	19.50	cfs	Check Discharge	0.00	cfs

Grades Model: Inverts					
Invert Upstream	600.00	ft	Invert Downstream	598.50	ft
Length	70.00	ft	Slope	0.021429	ft/ft
Drop	1.50	ft			

Headwater Model: Maximum Allowable HW		
Headwater Elevation	605.00	ft

Tailwater Conditions: Constant Tailwater		
Tailwater Elevation	599.50	ft

Name	Description	Discharge	HW Elev.	Velocity
x Trial-1	1-36 inch Circular	19.50 cfs	602.46 ft	

000197

183.204

Culvert Designer/Analyzer Report
culvert 1 - DC A

Design:Trial-1

Solve For: Headwater Elevation

Culvert Summary			
Allowable HW Elevation	605.00 ft	Storm Event	Design
Computed Headwater Elev:	602.46 ft	Discharge	19.50 cfs
Headwater Depth/Height	0.82	Tailwater Elevation	599.50 ft
Inlet Control HW Elev.	602.14 ft	Control Type	Entrance Control
Outlet Control HW Elev.	602.46 ft		

Grades			
Upstream Invert	600.00 ft	Downstream Invert	598.50 ft
Length	70.00 ft	Constructed Slope	0.021429 ft/ft

Hydraulic Profile			
Profile	S2	Depth, Downstream	1.26 ft
Slope Type	Steep	Normal Depth	1.26 ft
Flow Regime	Supercritical	Critical Depth	1.42 ft
Velocity Downstream	6.91 ft/s	Critical Slope	0.014191 ft/ft

Section			
Section Shape	Circular	Mannings Coefficient	0.024
Section Material	CMP	Span	3.00 ft
Section Size	36 inch	Rise	3.00 ft
Number Sections	1		

Outlet Control Properties			
Outlet Control HW Elev.	602.46 ft	Upstream Velocity Head	0.55 ft
Ke	0.90	Entrance Loss	0.49 ft

Inlet Control Properties			
Inlet Control HW Elev.	602.14 ft	Flow Control	Unsubmerged
Inlet Type	Projecting	Area Full	7.1 ft^2
K	0.03400	HDS 5 Chart	2
M	1.50000	HDS 5 Scale	3
C	0.05530	Equation Form	1
Y	0.54000		

000198

Culvert Designer/Analyzer Report
culvert 1 - DC B

39/204

Peak Discharge Method: User-Specified					
Design Discharge	15.01	cfs	Check Discharge	0.00	cfs

Grades Model: Inverts					
Invert Upstream	598.00	ft	Invert Downstream	597.50	ft
Length	50.00	ft	Slope	0.010000	ft/ft
Drop	0.50	ft			

Headwater Model: Maximum Allowable HW		
Headwater Elevation	602.00	ft

Tailwater Conditions: Constant Tailwater		
Tailwater Elevation	598.50	ft

Name	Description	Discharge	HW Elev.	Velocity
x Trial-1	1-24 inch Circular	15.01 cfs	600.58 ft	

000199

-4014

Culvert Designer/Analyzer Report
culvert 1 - DC B

190/204

Design:Trial-1

Solve For: Headwater Elevation

Culvert Summary			
Allowable HW Elevation	602.00 ft	Storm Event	Design
Computed Headwater Elev:	600.58 ft	Discharge	15.01 cfs
Headwater Depth/Height	1.29	Tailwater Elevation	598.50 ft
Inlet Control HW Elev.	600.45 ft	Control Type	Outlet Control
Outlet Control HW Elev.	600.58 ft		

Grades			
Upstream Invert	598.00 ft	Downstream Invert	597.50 ft
Length	50.00 ft	Constructed Slope	0.010000 ft/ft

Hydraulic Profile			
Profile	M2	Depth, Downstream	1.40 ft
Slope Type	Mild	Normal Depth	N/A ft
Flow Regime	Subcritical	Critical Depth	1.40 ft
Velocity Downstream	6.41 ft/s	Critical Slope	0.021545 ft/ft

Section			
Section Shape	Circular	Mannings Coefficient	0.024
Section Material	CMP	Span	2.00 ft
Section Size	24 inch	Rise	2.00 ft
Number Sections	1		

Outlet Control Properties			
Outlet Control HW Elev.	600.58 ft	Upstream Velocity Head	0.37 ft
Ke	0.90	Entrance Loss	0.34 ft

Inlet Control Properties			
Inlet Control HW Elev.	600.45 ft	Flow Control	Unsubmerged
Inlet Type	Projecting	Area Full	3.1 ft²
K	0.03400	HDS 5 Chart	2
M	1.50000	HDS 5 Scale	3
C	0.05530	Equation Form	1
Y	0.54000		

000200

4014

Culvert Designer/Analyzer Report
culvert 2 - DC B

191/204

Peak Discharge Method: User-Specified

Design Discharge	17.54 cfs	Check Discharge	0.00 cfs

Grades Model: Inverts

Invert Upstream	596.00 ft	Invert Downstream	595.50 ft
Length	50.00 ft	Slope	0.010000 ft/ft
Drop	0.50 ft		

Headwater Model: Maximum Allowable HW

Headwater Elevation	600.00 ft

Tailwater Conditions: Constant Tailwater

Tailwater Elevation	596.50 ft

	Name	Description	Discharge	HW Elev.	Velocity
x	Trial-1	1-24 inch Circular	17.54 cfs	599.17 ft	

000201

192/204

Culvert Designer/Analyzer Report
culvert 2 - DC B

Design:Trial-1

Solve For: Section Size

Culvert Summary

Allowable HW Elevation	600.00	ft	Storm Event	Design	
Computed Headwater Elevz	599.17	ft	Discharge	17.54	cfs
Headwater Depth/Height	1.58		Tailwater Elevation	596.50	ft
Inlet Control HW Elev.	598.81	ft	Control Type	Outlet Control	
Outlet Control HW Elev.	599.17	ft			

Grades

Upstream Invert	596.00	ft	Downstream Invert	595.50	ft
Length	50.00	ft	Constructed Slope	0.010000	ft/ft

Hydraulic Profile

Profile	CompositeM2PressureProfile		Depth, Downstream	1.51	ft
Slope Type	Mild		Normal Depth	N/A	ft
Flow Regime	Subcritical		Critical Depth	1.51	ft
Velocity Downstream	6.90	ft/s	Critical Slope	0.024287	ft/ft

Section

Section Shape	Circular	Mannings Coefficient	0.024	
Section Material	CMP	Span	2.00	ft
Section Size	24 inch	Rise	2.00	ft
Number Sections	1			

Outlet Control Properties

Outlet Control HW Elev.	599.17	ft	Upstream Velocity Head	0.48	ft
Ke	0.90		Entrance Loss	0.44	ft

Inlet Control Properties

Inlet Control HW Elev.	598.81	ft	Flow Control	Transition	
Inlet Type	Projecting		Area Full	3.1	ft^2
K	0.03400		HDS 5 Chart	2	
M	1.50000		HDS 5 Scale	3	
C	0.05530		Equation Form	1	
Y	0.54000				

000202

193/204

Culvert Designer/Analyzer Report
culvert 3 - DC B

Peak Discharge Method: User-Specified

Design Discharge	1.89 cfs	Check Discharge	0.00 cfs

Grades Model: Inverts

Invert Upstream	590.50 ft	Invert Downstream	590.00 ft
Length	50.00 ft	Slope	0.010000 ft/ft
Drop	0.50 ft		

Headwater Model: Maximum Allowable HW

Headwater Elevation	593.50 ft

Tailwater Conditions: Constant Tailwater

Tailwater Elevation	591.00 ft

Name	Description	Discharge	HW Elev.	Velocity
x Trial-1	1-12 inch Circular	1.89 cfs	591.62 ft	

194/204

Culvert Designer/Analyzer Report
culvert 3 - DC B

Design:Trial-1

Solve For: Section Size

Culvert Summary

Allowable HW Elevation	593.50 ft	Storm Event	Design
Computed Headwater Elev:	591.62 ft	Discharge	1.89 cfs
Headwater Depth/Height	1.12	Tailwater Elevation	591.00 ft
Inlet Control HW Elev.	591.45 ft	Control Type	Outlet Control
Outlet Control HW Elev.	591.62 ft		

Grades

Upstream Invert	590.50 ft	Downstream Invert	590.00 ft
Length	50.00 ft	Constructed Slope	0.010000 ft/ft

Hydraulic Profile

Profile	M1	Depth, Downstream	1.00 ft
Slope Type	Mild	Normal Depth	0.80 ft
Flow Regime	Subcritical	Critical Depth	0.59 ft
Velocity Downstream	2.41 ft/s	Critical Slope	0.022876 ft/ft

Section

Section Shape	Circular	Mannings Coefficient	0.024
Section Material	CMP	Span	1.00 ft
Section Size	12 inch	Rise	1.00 ft
Number Sections	1		

Outlet Control Properties

Outlet Control HW Elev.	591.62 ft	Upstream Velocity Head	0.10 ft
Ke	0.90	Entrance Loss	0.09 ft

Inlet Control Properties

Inlet Control HW Elev.	591.45 ft	Flow Control	Unsubmerged
Inlet Type	Projecting	Area Full	0.8 ft²
K	0.03400	HDS 5 Chart	2
M	1.50000	HDS 5 Scale	3
C	0.05530	Equation Form	1
Y	0.54000		

-4014 195/204

Culvert Designer/Analyzer Report
culvert 4 - DC B

Peak Discharge Method: User-Specified			
Design Discharge	4.32 cfs	Check Discharge	0.00 cfs

Grades Model: Inverts			
Invert Upstream	592.50 ft	Invert Downstream	592.00 ft
Length	50.00 ft	Slope	0.010000 ft/ft
Drop	0.50 ft		

Headwater Model: Maximum Allowable HW	
Headwater Elevation	596.00 ft

Tailwater Conditions: Constant Tailwater	
Tailwater Elevation	593.00 ft

	Name	Description	Discharge	HW Elev.	Velocity
x	Trial-3	1-18 inch Circular	4.32 cfs	593.85 ft	

000205

Culvert Designer/Analyzer Report
culvert 4 - DC B

Design:Trial-3

Solve For: Headwater Elevation

Culvert Summary			
Allowable HW Elevation	596.00 ft	Storm Event	Design
Computed Headwater Elev:	593.85 ft	Discharge	4.32 cfs
Headwater Depth/Height	0.90	Tailwater Elevation	593.00 ft
Inlet Control HW Elev.	593.75 ft	Control Type	Outlet Control
Outlet Control HW Elev.	593.85 ft		

Grades			
Upstream Invert	592.50 ft	Downstream Invert	592.00 ft
Length	50.00 ft	Constructed Slope	0.010000 ft/ft

Hydraulic Profile			
Profile	M1	Depth, Downstream	1.00 ft
Slope Type	Mild	Normal Depth	0.98 ft
Flow Regime	Subcritical	Critical Depth	0.80 ft
Velocity Downstream	3.45 ft/s	Critical Slope	0.018816 ft/ft

Section			
Section Shape	Circular	Mannings Coefficient	0.024
Section Material	CMP	Span	1.50 ft
Section Size	18 inch	Rise	1.50 ft
Number Sections	1		

Outlet Control Properties			
Outlet Control HW Elev.	593.85 ft	Upstream Velocity Head	0.19 ft
Ke	0.90	Entrance Loss	0.17 ft

Inlet Control Properties			
Inlet Control HW Elev.	593.75 ft	Flow Control	Unsubmerged
Inlet Type	Projecting	Area Full	1.8 ft²
K	0.03400	HDS 5 Chart	2
M	1.50000	HDS 5 Scale	3
C	0.05530	Equation Form	1
Y	0.54000		

000206

197 204

Culvert Designer/Analyzer Report
culvert 5 - DC B

Peak Discharge Method: User-Specified			
Design Discharge	11.81 cfs	Check Discharge	0.00 cfs

Grades Model: Inverts			
Invert Upstream	599.00 ft	Invert Downstream	598.50 ft
Length	50.00 ft	Slope	0.010000 ft/ft
Drop	0.50 ft		

Headwater Model: Maximum Allowable HW	
Headwater Elevation	603.00 ft

Tailwater Conditions: Constant Tailwater	
Tailwater Elevation	599.50 ft

Name	Description	Discharge	HW Elev.	Velocity
x Trial-1	1-24 inch Circular	11.81 cfs	601.15 ft	

Culvert Designer/Analyzer Report
culvert 5 - DC B

Design:Trial-1

Solve For: Headwater Elevation

Culvert Summary			
Allowable HW Elevation	603.00 ft	Storm Event	Design
Computed Headwater Elev:	601.15 ft	Discharge	11.81 cfs
Headwater Depth/Height	1.07	Tailwater Elevation	599.50 ft
Inlet Control HW Elev.	601.04 ft	Control Type	Outlet Control
Outlet Control HW Elev.	601.15 ft		

Grades			
Upstream Invert	599.00 ft	Downstream Invert	598.50 ft
Length	50.00 ft	Constructed Slope	0.010000 ft/ft

Hydraulic Profile			
Profile	M2	Depth, Downstream	1.23 ft
Slope Type	Mild	Normal Depth	1.58 ft
Flow Regime	Subcritical	Critical Depth	1.23 ft
Velocity Downstream	5.80 ft/s	Critical Slope	0.018886 ft/ft

Section			
Section Shape	Circular	Mannings Coefficient	0.024
Section Material	CMP	Span	2.00 ft
Section Size	24 inch	Rise	2.00 ft
Number Sections	1		

Outlet Control Properties			
Outlet Control HW Elev.	601.15 ft	Upstream Velocity Head	0.32 ft
Ke	0.90	Entrance Loss	0.29 ft

Inlet Control Properties			
Inlet Control HW Elev.	601.04 ft	Flow Control	Unsubmerged
Inlet Type	Projecting	Area Full	3.1 ft²
K	0.03400	HDS 5 Chart	2
M	1.50000	HDS 5 Scale	3
C	0.05530	Equation Form	1
Y	0.54000		

000208

ATTACHMENT C-4B

ANALYSIS RESULTS FOR CULVERTS

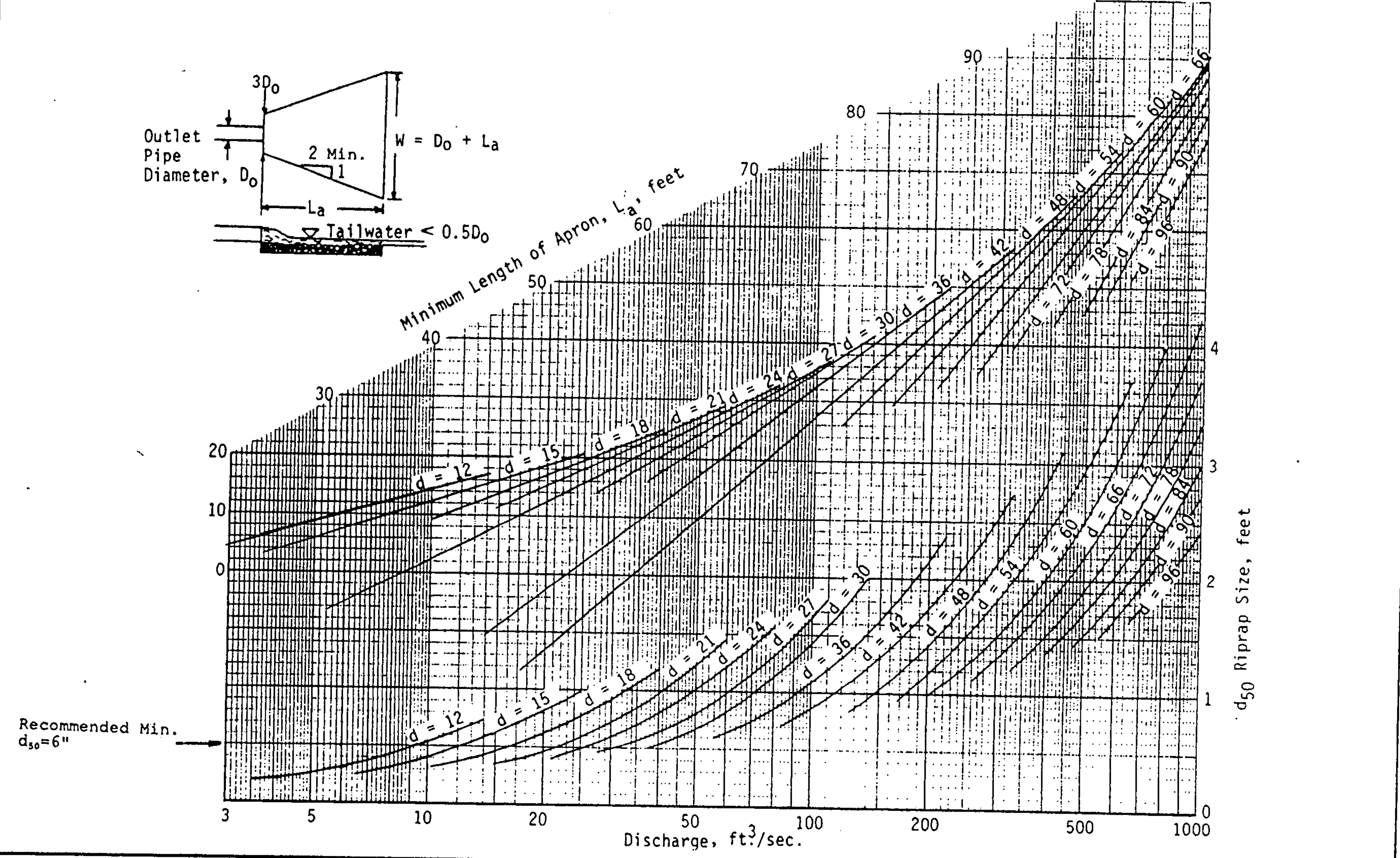
SUMMARY OF CULVERT ANALYSES RESULTS

CULVERT IDENTIFICATION			PHYSICAL CHARACTERISTICS OF CULVERT						CULVERT PROFILE				CULVERTMASTER® MODELING CHARACTERISTICS			HYDRAULIC CAPACITY					STRUCTURAL CAPACITY				OUTLET PROTECTION				
Culvert Name	Status [1]	Design Scenario	Material - Type	Entrance Loss Coefficient (Ke)	Manning's n	Entrance Configuration [2]	Number of Culverts - Diameter	Approximate Length (ft)	Inlet Invert Elevation (ft MSL)	Outlet Invert Elevation (ft MSL)	Slope ft/ft	Overtopping Elevation (ft MSL)	Entrance Configuration [3]	Entrance Loss Coefficient (Ke)	Number of Culverts - Diameter [3]	HydroCad Node / Peak Flow Rate (cfs)	Calculated Freeboard (ft)	Tailwater Elevation (ft MSL)	Calculated Headwater Depth - Inlet Control [5] (ft MSL)	Calculated Headwater Depth - Outlet Control [5] (ft MSL)	Available Cover (ft)	Design Traffic Type	Minimum Required Cover (ft)	Structurally Stable	Outlet Velocity (ft/s)	Riprap Length at Inlet (ft)	Riprap Length at Outlet (ft)	d_{50} (in)	Thickness (in)
1	new	OSDF	CMP	0.9	0.024	Projecting	1 - 36 inch	96.1	594.00	592.66	0.014	597.85	Projecting	0.9	1 - 36 inch	5 / 34.14	0.5	594.22	597.17	597.36	3.00 [6]	Off-Highway	3	yes	7.2	6	12	6	12
2	new [4]	OSDF	CMP	0.9	0.024	Projecting	2 - 42 inch	90.0	583.14	581.89	0.014	587.00	Projecting	0.9	2 - 42 inch	11 / 49.39	0.8	585.82	585.82	586.23	3.16	Off-Highway	3	yes	2.6	7	14	6	12
3	new	OSDF	CMP	0.9	0.024	Projecting	1 - 12 inch	74.0	586.50	585.32	0.016	588.00	Projecting	0.9	1 - 12 inch	7 / 1.55	0.6	585.79	587.33	587.40	5.00	Off-Highway	3	yes	3.7	0	0	0	0
4	new	OSDF	CMP	0.9	0.024	Projecting	1 - 36 inch	95.5	589.22	587.89	0.014	593.50	Projecting	0.9	1 - 36 inch	N/A / 33.64 [7]	1.0	589.45	592.35	592.55	3.06	Off-Highway	3	yes	7.2	6	12	6	12
5	new	OSDF	CMP	0.9	0.024	Projecting	1 - 42 inch	55.0	584.39	583.63	0.014	590.00	Projecting	0.9	1 - 42 inch	N/A / 33.64 [8]	2.5	585.82	587.18	587.54	4.61	Off-Highway	3	yes	6.1	6	12	6	12
1	new	DC A	CMP	0.9	0.024	Projecting	1 - 36 inch	70.0	600.00	598.50	0.021	605.00	Projecting	0.9	1 - 36 inch	N/A / 19.50 [9]	2.5	599.50	602.14	602.46	2.00	On - Highway	2	yes	6.9	6	12	6	12
1	new	DC B	CMP	0.9	0.024	Projecting	1 - 24 inch	50.0	598.00	597.50	0.010	602.00	Projecting	0.9	1 - 24 inch	1 / 15.01	1.4	598.50	600.45	600.58	2.00	On - Highway	2	yes	6.4	4	8	6	12
2	new	DC B	CMP	0.9	0.024	Projecting	1 - 24 inch	50.0	596.00	595.50	0.010	600.00	Projecting	0.9	1 - 24 inch	2 / 17.54	0.8	596.50	598.81	599.17	2.00	On - Highway	2	yes	6.9	4	8	6	12
3	new	DC B	CMP	0.9	0.024	Projecting	1 - 12 inch	50.0	590.50	590.00	0.010	593.50	Projecting	0.9	1 - 12 inch	3 / 1.89	1.9	591.00	591.45	591.62	2.00	On - Highway	2	yes	2.4	0	0	0	0
4	new	DC B	CMP	0.9	0.024	Projecting	1 - 18 inch [10]	50.0	592.50	592.00	0.010	596.00	Projecting	0.9	1 - 18 inch [10]	4 / 4.32	2.1	593.00	593.75	593.85	2.00	On - Highway	2	yes	3.5	3	6	6	12
5	new	DC B	CMP	0.9	0.024	Projecting	1 - 24 inch	50.0	599.00	598.50	0.010	603.00	Projecting	0.9	1- 24 inch	5 / 11.81	1.9	599.50	601.04	601.15	2.00	On - Highway	2	yes	5.8	4	8	6	12

- OSDF = OSDF Design Scenario
- DC A = Design Case "A"
- DC B = Design Case "B"
- CMP = Corrugated Metal Pipe(s)
- N/A = Not Applicable
- d_{50} = Average particle diameter

Notes
(1) New indicates a new culvert to be installed.
(2) Entrance configuration assumed.
(3) Dimensions and entrance configurations used in the Culvertmaster software package were selected to match existing culvert characteristics as closely as possible.
(4) Culvert 2 is not installed as a part of the Phase IV work.
(5) Headwater depths calculated using Culvertmaster software package. Summary output presented for each culvert in Attachment C-4A.
(6) Grading is provided in the Construction Drawings
(7) The peak flow rate used for culvert 4 is the outflow from reach 6 in the OSDF Design Scenario.
(8) The peak flow rate used for culvert 5 is the outflow from reach 6 in the OSDF Design Scenario.
(9) The peak flow rate used for culvert 1 is the inflow into reach 3 not including the inflow from subcatchment D in the Design Case "A".
(10) This culvert is shown as 12" on the CFC Drawings. DCN will be submitted to change this diameter to 18".

DESIGN OF OUTLET PROTECTION FROM A ROUND PIPE FLOWING FULL
MINIMUM TAILWATER CONDITION (T_w < 0.5 DIAMETER)



Source: USDA-SCS

GEOSYNTEC CONSULTANTS

Written by: Dana Mehlman Date: 01/08/24 Reviewed by: ______ Date: __/__/__

Client: Fernald Project: OSDF Phase IV Project/Proposal No.: GQ1342 Task No.: 16

Design of Outlet Protection

Culvert ID	Diameter (inch)	Discharge (cfs)	Length Apron at Outlet (ft)
OSDF 1	36	34.14	9
OSDF 2	42	49.39	10
OSDF 3	12	1.55	N/A
OSDF 4	36	33.64	9
OSDF 5	36	33.64	9
DCA 1	36	19.50	0
DCB 1	24	15.01	8
DCB 2	24	17.54	10
DCB 3	12	1.89	N/A
DCB 4	15	4.32	4
DCB 5	24	11.81	4

N/A indicates that the discharge is not significant enough to warrant outlet protection

ATTACHMENT C-5

ANALYSIS RESULTS FOR BASIN OUTLET STRUCTURES

ITEM		OSDF BASIN 1 DESCRIPTION	OSDF BASIN 1 PARAMETER VALUE IN UNITS SHOWN	
PRINCIPAL SPILLWAY	Riser Pipe	Twin 48" CMP	ELEVATION (ft MSL)	585.75
	Outlet Pipe	Twin 36" CMP		850.0
EMERGENCY SPILLWAY		30' wide trapezoidal channel		586.5
EMBANKMENT		10' wide average width		588.0
AVAILABLE STORAGE VOLUME (TO RISER INLET)		To primary spillway riser pipe inlet	VOLUME (acre-ft)	5.0
SUBCATCHMENT		Total Upstream Drainage area	AREA (acres)	21.15

OSDF BASIN 1 ROUTING RESULTS

DESIGN PARAMETER	Value
10-YEAR 24-HOUR RUNOFF VOLUME (acre-ft)	4.834
25-YEAR 24-HOUR PEAK WATER ELEVATION (ft MSL)	585.82
100-YEAR 24-HOUR PEAK WATER ELEVATION (ft MSL)	585.91
REQUIRED STORAGE VOLUME BASED ON DRAINAGE AREA (acre-ft)	2.644

Notes

1. Minimum Sedimentation Basin volume of 5.440 acre-ft exceeds runoff volume of 4.528 acre-ft and disturbed area-based volume of 2.1 acre-ft.
2. Flow does not enter emergency spillway for the 25-year, 24-hour storm event.
3. A freeboard greater than 1 ft is maintained for the 100-yr, 24-hour storm event.
4. Calculation for volume based on drainage area.
   (0.125 acre-ft/acre-year) x (21.15 acres of upstream drainage area) x (1 year/cleanout) = 2.64 acre-ft/cleanout

4014

000214