2983

REVISED FINAL DESIGN
CALCULATION PACKAGE

ON-SITE DISPOSAL FACILITY

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

prepared by

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VOLUME V of V
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REVISED FINAL DESIGN PACKAGE
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Fernald Environmental Management Project

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COMPUTATION COVER SHEET

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SUBJECT OF COMPUTATIONS PHASE III SURFACE-WATER MANAGEMENT SYSTEM DESIGN

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GEOSYNTEC CONSULTANTS

Written by: DGP Date: 00 / 05 / 09 Reviewed by: Vander Linde Date: 00 / 05 / 15

Client: Fluor Fernald, Inc. Project: OSDF-Phase III Proj./Proposal No.: GQ1001 Task No.: 04

ADDENDUM TO SECTION 12.3

(Added to Revision 1A Package)

Surface-water management system design calculations presented in this Section were prepared to support construction of (i) Cell 1 final cover system; (ii) Cell 4 liner system; (iii) access Control Facility south of OSDF Sedimentation Basin 1; (iv) Enhanced Permanent Leachate Transmission System (EPLTS); and (v) Borrow Area Subareas 1 and 2A. For the purposes of this Section, construction of these features will be referred to as OSDF-Phase III (Phase III).

PHASE III SURFACE-WATER MANAGEMENT SYSTEM DESIGN

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EXECUTIVE SUMMARY

PURPOSE OF ANALYSES

The purpose of this calculation package is to design the OSDF and Borrow Area surface-water management (SWM) structures to be constructed as part of the Phase III development of the OSDF. These SWM structures are 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] 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 sedimentation basins (basins), the total drainage area is divided into two parts and modeled as the OSDF and the Borrow Area Design Scenarios. The OSDF Design Scenario is a composite of "worst case" conditions (i.e., maximum runoff and peak flow rates) for the period of Phase III construction. The Borrow Area Design Scenario represents the "worst case" condition for development and interim restoration of the borrow area Subareas 1 and 2A. An additional design scenario, Design Case A represents a channel and culvert constructed as part of the SWM system for Cell 4. The channel and culvert are not incorporated into one of the above mentioned design scenarios.

For Phase III 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 North, East, and West perimeters of the OSDF was performed as part of the OSDF Final Design Package [GeoSyntec, 2000b]. No analyses are performed as part of this Calculation Package to assess the adequacy of these channels.

For existing SWM structures (other than channels along the North, East, and West perimeters of the OSDF), analyses are performed to assess the adequacy of channels, culverts, and sedimentation basins (basins). Channels and culverts are considered to be adequate if they have sufficient hydraulic capacity to convey peak flow rates from the 25-year, 24-hour storm event. The OSDF Basin 1 is considered adequate if it complies with the performance criteria for the 10-, 25-, and 100-year, 24-hour storm events, outlined in the DCP for construction, filling, and closure of the OSDF. The Borrow Area Basin is considered adequate if it complies with the

performance criterion for the 25-year, 24-hour storm event, outlined in the DCP for development and interim restoration of the Borrow Area.

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 is 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 III (i.e., new) channels and culverts are designed in accordance with the requirements of the SWMEC Plan [GeoSyntec, 2000c] 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.
- For the one RCP culvert, a Class IV Wall B reinforced concrete pipe is selected to provide adequate structural stability.

Existing channels, culverts, and basins (with modifications shown in the Construction Drawings) have adequate capacity to convey applicable design storm events, in particular as follows:

Existing Channels

- Calculated freeboard for all channels equal or exceed the minimum required of 0.5 ft.

Existing Culverts

- Calculated headwater elevation at the upstream end of culverts allow for 0.5 ft or greater freeboard, with two exceptions. For one of these cases, the calculated freeboard is 0.4 ft. For the remaining case, the road surface under which the culvert crosses is expected to overtop. This road is subject only to site traffic (i.e., no public access), is not used for access to the OSDF, and overtopping of the road will result in no impact to the OSDF structure. It is recommended that no modifications be made to these culverts.

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.

Borrow Area Sedimentation Basin

- The available minimum storage volume exceeds the calculated volume of 1,800 cf per acre (total upstream drainage area multiplied by the basin cleanout frequency).
- The calculated peak water elevation for the 25-year, 24-hour storm event allows more than 1 ft of freeboard from the minimum embankment crest elevation.

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

PHASE III SURFACE-WATER MANAGEMENT SYSTEM DESIGN 2983

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PHASE III SURFACE-WATER MANAGEMENT SYSTEM DESIGN

PROCEDURES

INTRODUCTION AND PURPOSE

The major elements of the OSDF Phase III project include construction of the Cell 4 liner system and Cell 1 final cover system, excavation, and interim restoration of the borrow area (Subareas 1 and 2A). These and other construction elements are listed and their relationship to the design scenarios shown in Table A-1. The layout of the SWM system is shown on Construction Drawings.

The purpose of this Calculation Package is to design the OSDF and Borrow Area surface-water management (SWM) structures to be constructed as part of the Phase III development of the OSDF. 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. Analyses for channels along the North, East, and West perimeters of the OSDF was performed as part of the OSDF Final Design Package [GeoSyntec, 2000b]. No analyses are performed as part of this Calculation Package to assess the adequacy of these channels.

DESIGN CRITERIA

The OSDF Phase III 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]. Applicable sections from the DCP [GeoSyntec, 2000] include Section 2.8, *Surface-Water Management* which contains requirements for the OSDF runon/runoff structures and Section 2.10.2.4, *Surface-Water Management for Borrow Area* which contains requirements for the borrow area structures. These sections of the DCP are included as Attachment A-1.

The design of Phase III structures includes analyses for 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).

The assessment of existing structures includes analyses for channels, culverts, and basins. For existing SWM structures, analyses are performed to assess the adequacy of channels, culverts, and sedimentation basins (basins). Channels and culverts are considered to be adequate if they have sufficient hydraulic capacity to convey peak flow rates from the 25-year, 24-hour storm event. The OSDF Basin 1 is considered adequate if it complies with the performance criteria for the 10-, 25-, and 100-year, 24-hour storm events, outlined in the DCP for construction, filling and closure of the OSDF. The Borrow Area Basin is considered adequate if it complies with the performance criterion for the 25-year, 24-hour storm event, outlined in the DCP for development and interim restoration of the Borrow Area.

ANALYSIS METHODOLOGIES

The design of Phase III SWM structures and assessment of hydraulic capacity for existing structures is performed using hydrologic and basin routing analyses in which the total drainage area is divided into two parts, modeled in this calculation package as the OSDF and the Borrow Area Design Scenarios. The OSDF Design Scenario is a composite of "worst case" conditions (i.e., maximum runoff and peak flow rates) for the period of Phase III construction. The Borrow Area Design Scenario represents the "worst case" condition for development and interim restoration of Subareas 1 and 2A. In addition, Design Case A represents a channel and culvert constructed as part of the SWM system for Cell 4. This channel and culvert are not incorporated into one of the above mentioned design scenarios. These scenarios and the design case are further described in Table A-1, and below.

OSDF Design Scenario

The OSDF Design Scenario incorporates existing conditions, including the construction of the enhanced permanent leachate transmission system (EPLTS) project (planned for year 2000), the Cell 1 final cover system, the Impacted Access Road, and the Relocated Access Control

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Facility. For this design scenario the Cell 1 final cover system is unvegetated and Cells 2 and 3 are receiving impacted material (i.e., runoff from within the perimeter berm of cells receiving impacted material does not enter the Phase III SWM system). The OSDF Basin 1 is expanded to include the former Impacted Sedimentation Basin to the north. A schematic layout of the SWM system represented in the OSDF Design Scenario is provided in Attachment A-2.

Borrow Area Design Scenario

The Borrow Area Design Scenario incorporates existing conditions, with Subarea 1 fully excavated. A schematic layout of the SWM system represented in the Borrow Area Design Scenario is provided in Attachment A-2.

Design Case A

Design Case A incorporates a channel and culvert which is part of the Phase III SWM system for the Cell 4 liner system. This channel and culvert are not represented in the design scenarios. A schematic layout of Design Case A 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™, 1998]. 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 sediment basins. 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 is performed using methodologies presented in USDOT [1985], as coded into the computer program CulvertMaster® [Haestad Methods, 1998]. Built-in subroutines allow the user to perform calculations for inlet and outlet control. The CulvertMaster® User's Guide [Haestad Methods, 1998] 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 Attachments C-1A, C-1B, and C-1C 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.

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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

A HSG classification of D is selected for final cover and liner system areas.

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 III SWM system is presented in the following table. This list is developed based on characterization of the ground cover and land use for existing runon area conditions and anticipated runoff area conditions during Phase III construction. Land use categories are

associated with TR-55 categories of land use and ground cover (See Attachment A-7 from [SCS, 1986]) 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	D	94
Vegetated borrow area and runon areas to borrow area	Open space in good condition	B	61
		C	74
Unvegetated borrow	Newly graded areas	B	86
		C	91
Stockpiles (CN based on weighted average of B and C soils in borrow area Subareas 1 and 2A)	Newly graded areas	B/C	87
Disturbed area used for construction support activities	Farmsteads – buildings, lanes, driveways, and surrounding lots	B	74
		C	82
Sedimentation basin (SB) or direct runoff to SB	Impervious surfaces	-	98
Runon areas east of OSDF and north of borrow area	Pasture in fair condition	B	69
		C	79

A delineation of selected land use categories for each design scenario and Design Case A is presented in Attachment A-2. 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 borrow area, stockpiles, and liner system runout	Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Vegetated area, construction support area, runon area east of OSDF, and runon area	Grass: short grass prairie	0.15

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north of the borrow area		

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. Where upstream storage is present, in selected cases, this storage is accounted for by modeling the culvert and upstream storage area as a pond. 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 widely accepted methodologies described in a later section.

Sedimentation Basins

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Surface-water runoff is routed through individual basins for the scenarios noted below.

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

Routing through the basins 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. The borrow area basin does not have an emergency spillway and none is modeled. 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.

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HYDRAULIC ANALYSES

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

Channels

The hydraulic capacity of each new and existing channel is evaluated for conveyance of peak flow rates from the 25-year, 24-hour design storm event. In addition for new channels, analyses are performed to evaluate lining stability. For the purpose of 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), 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.

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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 and existing culvert is evaluated for conveyance of peak flow rates from the 25-year, 24-hour design storm event. New culverts include corrugated metal pipe (CMP) and reinforced concrete pipe (RCP) types. Existing culverts include CMP, RCP, asphalted corrugated metal pipe (CMPA), high density polyethylene (HDPE), and polyvinyl chloride (PVC) types.

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 (overtopping 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), and (vii) overtopping elevation. CulvertMaster® does not include all diameters and inlet configurations, therefore characteristics are selected within the CulvertMaster® program to best approximate those of the existing culvert. For the case of diameter, the closest diameter was used but no more than two inches larger than the existing diameter. For the case of inlet configuration, an inlet configuration with an entrance loss coefficient equal to that estimated for the actual inlet is selected. Tailwater elevations for

culverts discharging directly into basins are taken as the peak water level in the basin for the 25-year, 24-hour storm event, otherwise tailwater elevation is assumed equal to the outlet invert elevation.

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 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 but outlet velocities 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.

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.

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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.

For the one RCP culvert, structural stability is evaluated using the indirect method. This method is presented in ACPA [1992] and summarized by the following steps.

Step 1: Calculation of earth load, W_E (lb/lf)

$$W_E = \gamma_s H B_c$$

Where, γ_s is the unit weight of overlying soil (pcf), H is the height of cover soil above the crown of the pipe (ft), and B_c is the outer diameter of the culvert (ft).

Step 2: Calculation of live load acting on culvert, W_L (lb/lf)

Length and width of tire footprint

$$A_t = 0.01P$$

$$l_F = \sqrt{\frac{A_t}{2.5}}$$

$$w_F = 2.5 l_F$$

Where, A_t is the tire footprint area (in^2), P is the wheel load (axle load of 95 kips/2) (lb), l_F is the length of footprint (ft), and w_F is the width of footprint (ft).

Pressure from live load acting at horizontal plane at level of the culvert crown, w (psf) = 2983

$$w = P\left(\frac{1 + I.F.}{A_{LL}}\right)$$

$$A_{LL} = L_{LL}W_{LL}$$

$$L_{LL} = w_F + 1.75H$$

$$W_{LL} = l_F + 1.75H$$

Where, I.F. is the impact factor (dimensionless), A_{LL} is the distributed live load area (ft^2), L_{LL} is the length of distributed live load area parallel to longitudinal axis of culvert, W_{LL} is the width of distributed live load area transverse to longitudinal axis of culvert.

Total live load acting on culvert, W_T (lb)

$$W_T = wL_{LL}S$$

$$S = \min(B_c, w_{LL})$$

Live load acting on culvert, W_L (lb/lf)

$$W_L = \frac{W_T}{L_e}$$

$$L_e = L_{LL} + 1.75\frac{(3B_c)}{4}$$

Where L_e is the effective supporting length of the pipe (ft).

Step 3: Selection of Bedding Factor, B_F

B_F is selected as 1.9 for a Class B installation (ACPA, 1998) which is consistent with the installation shown on Construction Drawings.

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Step 4: Selection of Culvert Class

$$D_{0.01} = \left(\frac{W_L + W_E}{B_f D} \right) F.S.$$

Where, $D_{0.01}$ is the D-Load strength (lb/lf), D is the culvert inner diameter (ft) and F.S. is the factor of safety for use of 0.01-in crack criterion (1.0). Based on the calculated $D_{0.01}$, a Class IV reinforced concrete pipe is selected using ASTM C 76, Reinforced Concrete Culvert, Storm Drain, and Sewer Pipe.

Sedimentation Basins

Existing basins are 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.

Borrow Area Sedimentation Basin (See DCP Section 2.10.2.4 in Attachment A-1)

- The minimum available storage volume exceeds the calculated volume of 1,800 cf per acre (total upstream drainage area multiplied by the basin cleanout frequency).
- The principal spillway should discharge runoff from the 25-year, 24-hour design storm event, allowing for 1 ft of freeboard from the minimum embankment crest elevation.

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TABLE A-1

OSDF PHASE III CONSTRUCTION ELEMENTS, DESIGN SCENARIOS, AND DESIGN CASE

Analysis	Feature	Status	Considered in Design of Structures	Comments
OSDF DS	Cell 1, 2, 3 liner system	Existing	Yes	
OSDF DS	OSDF Basin 1	Existing	Yes	
OSDF DS	Channels and culverts	Existing	Yes	
OSDF DS	EPLTS Project	New	Yes	Not part of Phase III project but constructed in same period.
OSDF DS	Cell 1 final cover system (unvegetated)	New	Yes	
OSDF DS	Impacted Access Road	New	Yes	Not part of Phase III project but constructed in same period.
OSDF DS	Relocated Access Control Facility	New	Yes	Not part of Phase III project but constructed in same period.
No	Subarea 1 partially excavated	Existing	No	
BA DS	Subarea 1 diversions	Existing	Yes	
BA DS	Borrow area basin	Existing	Yes	
BA DS	Channels and culverts	Existing	Yes	
BA DS	Runon areas East of OSDF	Existing	Yes	
BA DS	Subarea 1 fully excavated	New	Yes	
No	Subarea 2A excavation	New	No	Reduces runoff to SWM system, thus not included in Borrow Area Design Scenario.
No	Subarea 1 and 2A interim restoration	New	No	
BA DS	Channels and Culverts	New	Yes	
DC A	Cell 4 liner system	New	Yes	Reduces runoff to SWM system, thus not included in OSDF Design Scenario.
DC A	Cell 4 channel and culvert	New	Yes	

OSDF DS = OSDF Design Scenario, BA DS = Borrow Area Design Scenario, DC A = Design Case A.

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ATTACHMENT A-1

DCP SECTIONS 2.8 AND 2.10.2.4

DCP SECTION 2.8
SURFACE-WATER MANAGEMENT

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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];
    - $D_{50} = 62.4 \text{ pcf} \times d \times S/4$

      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]:
    - $D_{max} = 1.5 \times D_{50}$
    - $D_{15} = 0.5 \text{ to } 0.75 \times D_{50}$

      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™ 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.

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DCP SECTION 2.10.2.4
SURFACE-WATER MANAGEMENT FOR BORROW AREA

American Association of State Highway and Transportation Officials (AASHTO), "*AASHTO Guide for Design of Pavement Structures*," Washington, D.C., 1992.

Portland Cement Association (PCA), "*Thickness Design for Concrete Highway and Street Pavement*", 1984.

State of Ohio, Department of Transportation (ODOT), "*Location and Design Manual, Volume I, Roadway Design*", Revision, October 1992, Columbus, OH, 1992.

State of Ohio, Department of Transportation (ODOT), "*Construction Materials Specifications*," Columbus, OH, January 1993.

Yang, H.H., "*Pavement Analysis and Design*", Prentice-Hall, Inc., New York, NY, 1993.

Yoder, E. and Witczack, M., "*Principles of Pavement Design*", John Wiley and Sons, Inc., New York, NY, 1975.

2.10 Borrow Area

2.10.1 General Design Criteria

The function of the borrow area is to provide soil for construction of the liner and final cover systems, and for other earthwork, related to construction, filling, and closure of the OSDF. This consists of soil borrow for construction of the compacted clay liner and cap, vegetative soil layer, compacted fill, and other earthwork associated with the project. The topsoil layer in the borrow area will be removed and stockpiled ~~in the borrow area~~ for later use unless it is determined to be impacted material, in which case it should be disposed of in the OSDF in accordance with the Impacted Material Placement Plan (IMP Plan). The borrow area for OSDF construction is located in the "East Field" area of the FEMP property. This borrow area will be developed over a period of time from 7 to 25 years (depending on the actual schedule for OSDF construction, filling, and closure), with a 10-year design life used for planning purposes. After the borrow area is restored, it should function with minimal maintenance.

The borrow area should be developed in a manner that results in efficient utilization of the available soils, results in efficient utilization of the natural topography of the borrow area to effectively control runon and runoff, provides soils with the engineering properties required for the OSDF earthwork components being constructed, and minimizes the costs associated with the development and restoration of the area (design considerations). In addition to these requirements, the design of the borrow area should consider the requirements described below (design considerations).

- The borrow area should be laid out in a manner that provides easy ingress and egress.
- The area should be developed in a manner that requires minimal additional earthwork after borrow activities are complete to achieve the required restoration grades.
- The borrow area should be developed incrementally as soil is needed for construction of the OSDF.
- The size of disturbed borrow area should be minimized to the extent possible.
- Interim restoration should be performed progressively as ~~work~~ excavation in each borrow area subarea is completed. ~~in spent portions of the borrow area.~~
- Erosion and sediment controls should be implemented in the borrow area prior to excavation, ~~through~~ during borrow activities, and in conjunction with restoration activities.

2.10.2 Specific Criteria

2.10.2.1 Location of Borrow Area

A. Design Criteria

- The borrow area should be located in an area having material quantities and types sufficient for OSDF construction, filling, and closure (design consideration).

- The borrow area should be located within the FEMP property and should not encroach on the transmission line, oil pipeline, or other easements (design consideration).

- The borrow area should not extend vertically into the gray till except as noted in the approved *Borrow Area Strategy Report* (Fluor Fernald, Inc. 2000). The reasons for this are: (i) the efficiency of the borrow area operation will be improved by limiting the depth of excavation; and (ii) by limiting the depth of excavation, the restored borrow grades will be flatter and restoration of the area will be more cost effective. The borrow area should be constructed with sufficient final slope ~~(e.g., at least 1.0 percent)~~ to ~~promote drainage~~ minimize erosion and support final restoration (design considerations).

- Within the above constraints, the borrow area should be as deep as possible to minimize the size of the area (design consideration).

B. Calculations

No formal calculation is required for the location of the borrow area. A review of past geotechnical investigations and the results of ongoing laboratory testing of borrow area soil samples collected in November 1995 should be conducted to assess the engineering properties of the material and to predict the suitability of the materials for use as compacted clay liner and cap materials, as well as other earthwork components of the OSDF, as listed in Section 2.10.1.

2.10.2.2 Layout of Borrow Area

A. Design Criteria

- The borrow area should be laid out to facilitate surface-water management throughout the life of the borrow area (design consideration).
- The borrow area should be graded to promote ~~stormwater~~ surface-water runoff and prevent ponding of surface water (design consideration).
- The borrow area should be laid out to avoid interferences with roads, utility corridors, easements, etc. (design consideration).
- A soil contingency volume (equal to at least 15 percent of the required soil bank/unbulked volume based on the design estimate) should be provided in the borrow area for possible use in (design consideration):
  - construction of an OSDF contingency cell if disposal volumes are larger than anticipated;
  - backfilling to subgrade elevations beneath the footprint of the OSDF in the event that additional excavation of impacted material is necessary;
  - constructing portions of the contouring layer of the cover system and the non-granular portion of the protective layer of the liner system with "~~clean~~non-impacted" soil ~~in the event that suitable impacted soil is not available for this purpose~~; and
  - providing material for "seasonal" closure between construction seasons in the event that suitable impacted soil is not available for this purpose.
- Intermediate borrow area slopes ~~that will not be vegetated~~ should be cut at a safe angle of repose for the material (design consideration).

- Permanent restored slopes within the borrow area should be no steeper than 5H:1V (design consideration).

- As may be required, design will comply with OSHA requirements, 29 CFR 1926 Subpart P-Excavations.

B. Calculations

A review of previous geotechnical field and laboratory test data, including brown till in-situ densities and moisture contents, standard Proctor compaction test results, and permeability test results should be made. The results of ~~currently-on-going~~previous laboratory testing of borrow area soil should also be reviewed. The calculations described below should be undertaken in support of the design of the borrow area.

- The bank/unbulked volume of borrow soil required to build the earthwork components of the OSDF described in 2.10.1 should be calculated. The calculation should be conducted using the required compacted earthwork volume of on-site borrow material. This required compacted earthwork volume should be corrected to account for shrinkage/bulking of the borrow soil, the possible need for contingency volume, and the removal of the topsoil layer for stockpiling and later use as the topsoil layer in the OSDF final cover system.

- The bank/unbulked volume of available borrow soil, as calculated using computed-aided design (CAD) techniques, should be verified by hand calculation.

- The in-situ moisture content of the soil at the borrow area should be compared to the anticipated compaction moisture contents for the various earthwork components of the OSDF liner and final cover systems to assess the construction water demand and/or amount of soil drying required to moisture condition the borrow soils.

2.10.2.3 Development of Borrow Area

A. Design Criteria

- The borrow area should be developed in a manner that produces consistent materials for construction of the OSDF and effectively utilizes the existing topography to control stormwater runoff and runon (design consideration).

- Requirements for processing of the borrow area soils should be identified during design. If possible, such processing should be performed in the borrow area or any temporary stockpile area. Such processing could include moisture conditioning, blending, screening, or admixture modification (design consideration).

- Prior to the start of construction in the borrow area, the following activities should be performed (design considerations):

  - establishment of temporary ~~stormwater~~ surface-water management and erosion and sediment controls for the borrow area, including sediment basins;

  - establishment of access controls for the borrow area (consistent with the requirements of Section 2.9.2.6 of the DCP); and

  - establishment of soil processing facilities (if any are needed).

- Prior to the start of borrow area development, topsoil in the area should be removed and stockpiled for later use ~~in~~ for closure of the OSDF unless it is determined to be impacted material, in which case ~~if~~ it should be disposed in the OSDF in accordance with the IMP Plan (design consideration).

B. Calculations

No formal calculations are required for the development of the borrow area.

2.10.2.4 Surface-Water Management for Borrow Area

A. Design Criteria

- Temporary and permanent surface-water control structures for the borrow area with the exception of sediment basins, should be designed for the 25-year, 24-hour storm event (design consideration). Depending on the size of the contributory area, sediment basins should be designed for either the 10-year, 24-hour storm event, or the 25-year, 24-hour storm event [~~ODNR-DSWC~~ODNR, 1996]. This criterion is discussed further later in this section of the DCP.

- ~~Stormwater~~ Surface-water runoff from ~~borrow area~~ the FEMP watersheds 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).

- ~~Stormwater~~ Surface-water runoff should be managed so that, after restoration of the borrow area, the effects of erosion of the borrow area ground surface are minimal. Long-term erosion of the ground surface in the borrow area must not impact the OSDF (design considerations).

- Temporary runon control structures should be implemented to minimize runon from entering work areas. Such runon should be diverted around work areas using diversion dikes or channels as appropriate (design consideration).

- Temporary channels for stormwater runoff should be designed to meet the criteria presented in Section 2.8.3. of the DCP (design consideration).

- Permanent channels should be designed to meet the criteria presented in Section 2.8.4 of the DCP (design consideration).

- Culverts should be designed to meet the criteria presented in Section 2.8.4 of the DCP (design consideration).

- Riprap should be designed to meet the criteria presented in Section 2.8.4 of the DCP (design consideration).

- Sediment basins should meet the following criteria [ODNR ~~DSWC~~, 1996] (design considerations).

  - The minimum capacity of the sediment basin to the elevation of the crest of the pipe spillway should be 1,800 cubic ft for each acre (~~155 cubic meter for each hectare~~) within the drainage area that will be disturbed by construction during the designed life of the sediment basin.

  - The capacity of the pipe spillway should be sufficient to pass the runoff from the 2-year, 24-hour storm event. For the FEMP property, this event has a rainfall intensity of 2.5 in. ~~(63 mm)~~ [Parsons, 1995a].

  - The combination of the principal and emergency spillways should be capable of safely discharging the flow from the 10-year, 24-hour storm event if the drainage area to the sediment basin is less than or equal to 20 acres ~~(8 ha)~~, or the 25-year, 24-hour storm event if the drainage area to the sediment basin is greater than 20 acres ~~(8 ha)~~.

  - Consideration should be given to using the permanent Fernald facility main entrance road as a containment dike for the stormwater runoff in lieu of an emergency spillway.

  - If an emergency spillway is implemented, a minimum freeboard of 1 ft ~~(0.3 m) should be provided~~ measured from the peak water elevation in the emergency spillway to the top of the embankment, should be provided.

- Temporary erosion control measures should include those items described in Section 2.8.3 in the DCP (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 route the design storm through the borrow area for existing conditions and for construction conditions (i.e., during borrow activities), and to size sediment basins. 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 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 should be evaluated using the procedures described in USDA-SCS Technical Release 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,1987,1988].

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.

2.~~9~~10.2.5 Restoration

A. Design Criteria

The borrow area should be restored in accordance with the approved ~~FEMP site-wide restoration plan~~ *Borrow Area Strategy Report* (Fluor Fernald, Inc., 2000) and *Borrow Area Management and Restoration Plan* (GeoSyntec, 2000) ~~(which is currently being developed)~~ to a condition that requires minimal maintenance (functional requirement). The borrow area should be restored to a condition that is consistent with the geomorphological character of the area surrounding the FEMP (design consideration). The following criteria should be followed in preparing the restoration plan for the borrow area.

- Final grades should be selected that minimize long-term erosion in the area. The final grades should not result in erosion that is greater than the calculated erosion rate for stabilized slopes in the vicinity of the FEMP that have similar grades and similar soil types (design consideration).

- ~~Stormwater~~ Surface-water runoff from the restored borrow area should be routed as indicated in Section 2.10.2.4~~3~~ (design consideration).

- The restored borrow area should be revegetated with plant species that minimize erosion, that can be successfully established ~~on~~ given the design final grades and soil types, and that are self-propagating in the expected conditions of the restored borrow area (design consideration).

- The design of the restoration plan should provide for progressive restoration of the borrow area, with commencement of restoration in an area as soon as borrow activities have been completed. To the extent possible, activities should not be performed in portions of the borrow area that have been restored (design consideration).

B. Calculations

No formal calculations are required for restoration of the borrow area. Procedures and materials for establishing permanent vegetation in the borrow area should be consistent with those procedures and materials selected for the OSDF final cover system (see Section 2.6 of this DCP). Additional references are as follows;

2.10.3 References

FEMP property data and information required to design the borrow area 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 the borrow area are given in Section 2.8.6 of the DCP. Additional references are as follows:

Fluor Fernald, Inc. "*Borrow Area Strategy Report*", Revision O, Fernald, OH, April 2000.

GeoSyntec Consultants, Inc., "*Borrow Area Management and Restoration Plan*", Revision 1C, Atlanta, GA, April-2000.

2.11 Impacted Material Management

2.11.1 General Design Criteria

This section of the DCP addresses impacted material management activities within the OSDF battery limit. Other activities, primarily associated with remediation of the operable units, and use of an OMTA or other temporary staging areas outside of the battery limit, are addressed in other design packages being prepared as part of the integrated FEMP remediation.

Impacted material management activities must be conducted in a manner that is protective of the OSDF liner system, leachate management system, and final cover system, that prevents the uncontrolled release of impacted material to the environment, and that is safe and cost effective. In this section of the DCP, the criteria that should be

ATTACHMENT A-2

LAYOUT OF DESIGN SCENARIO AND DESIGN CASE SWM SYSTEMS

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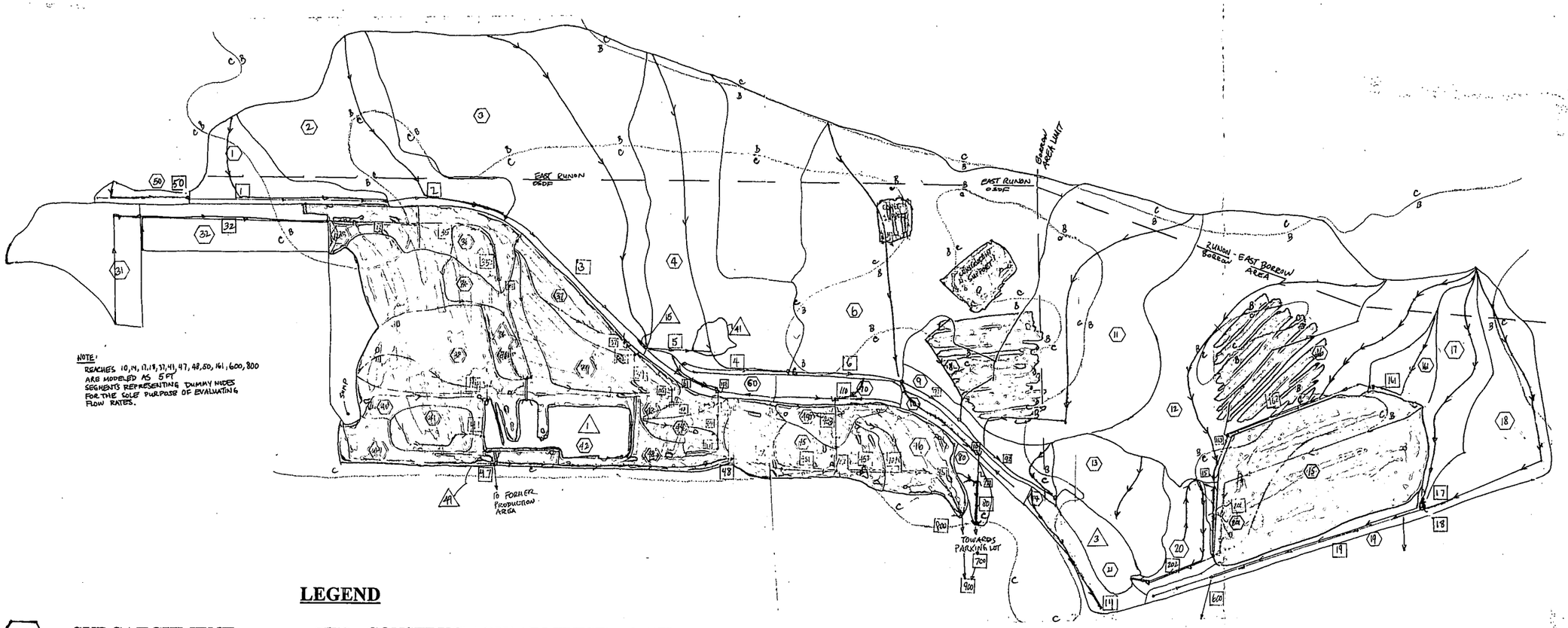
OSDF DESIGN AND BORROW AREA DESIGN SCENARIOS

N



LEGEND

- SUBCATCHMENT LABEL
- REACH LABEL
- POND LABEL
- REACH ALIGNMENT
- FLOW PATH
- CULVERT
- CONSTRUCTION/ ADMINISTRATIVE SUPPORT ZONES
- UNVEGETATED ZONES IN BORROW AREA
- STOCKPILE
- DELINEATION OF HSG B & C AREAS

SCALE: 1" = 500' (APPROX.)

GEOSYNTEC CONSULTANTS
ATLANTA, GEORGIA

OSDF PHASE III
FERNALD ENVIRONMENTAL MANAGEMENT PROJECT
DRAINAGE AREA SCHEMATIC

PROJECT NO. GQ1001-03	DESIGN BY: DGP/JGT	
DOCUMENT NO. F0030032	CHECK BY: RDE	

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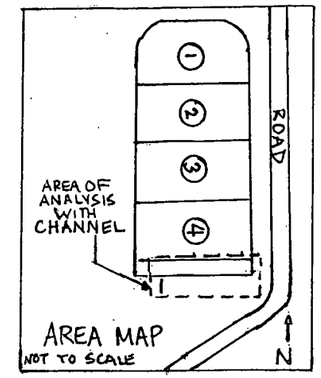
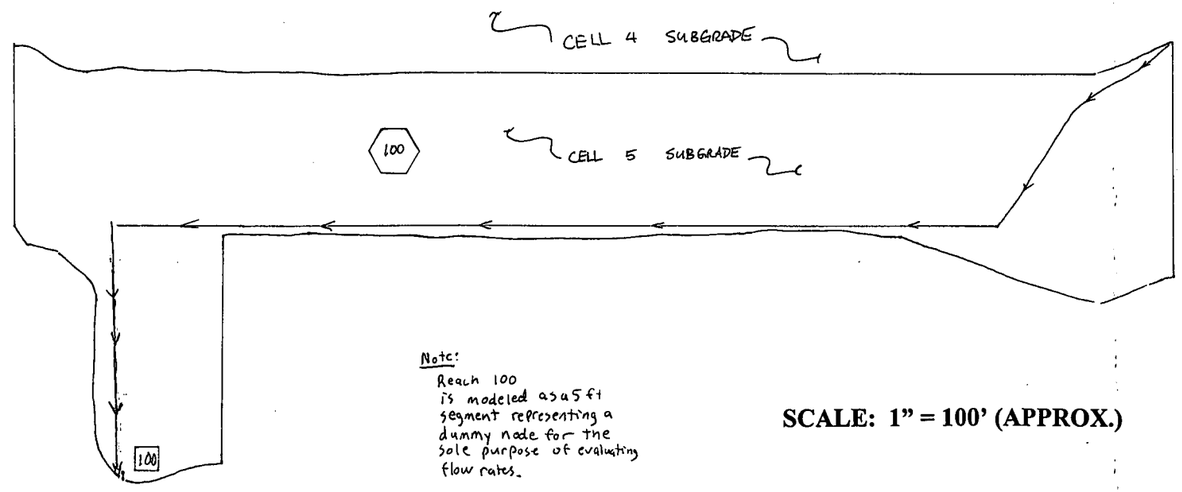
DESIGN CASE A





Note:
Reach 100 is modeled as a 5 ft segment representing a dummy node for the sole purpose of evaluating flow rates.

SCALE: 1" = 100' (APPROX.)

N

EXISTING REROUTED NORTH ENTRANCE ROAD

LEGEND

- SUBCATCHMENT LABEL
- REACH LABEL
- REACH ALIGNMENT
- FLOW PATH

000070

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OSDF PHASE III
FERNALD ENVIRONMENTAL MANAGEMENT PROJECT
DRAINAGE AREA SCHEMATIC

PROJECT NO. GQ1001-03	DESIGN BY: DGP/JGT
DOCUMENT NO. F0030032	CHECK BY: RDE

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ATTACHMENT A-3

HydroCAD™ TECHNICAL REFERENCE MANUAL

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ROE 5/5/00
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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.

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Source: HydroCAD™, 1998.

Section 21 - Introduction

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Understanding HydroCAD

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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.

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Section 22 - Determining the Time of Concentration

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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.

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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:

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

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.

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Channel Flow

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

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

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_c 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]

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

Section 23 - SCS Unit Hydrograph Procedure

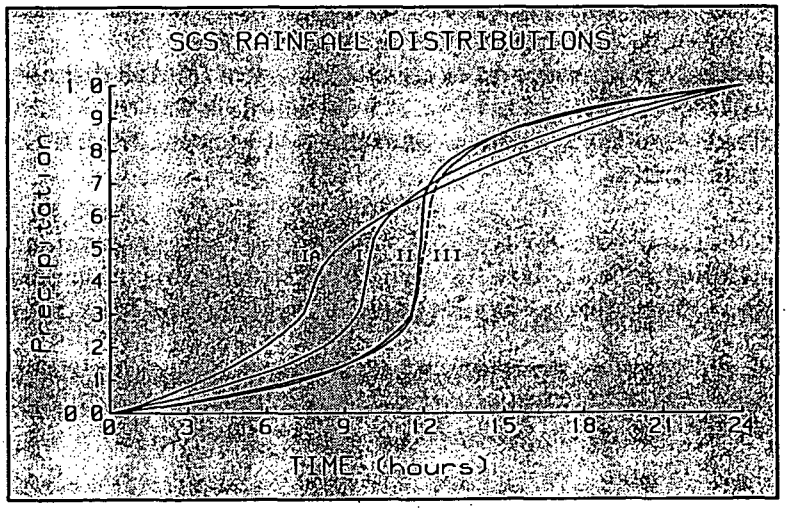
The USDA Soil Conservation Service[40] has developed a number of techniques for analyzing stormwater runoff. One of the most widely used is the SCS Unit Hydrograph procedure (SCSUH). The SCSUH procedure is a principal component of SCS Technical Release 20, commonly known as **SCS TR-20** [2].

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]

Custom rainfall distributions may also be used with the unit hydrograph procedure, but this is rarely 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.

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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.

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SCSUH Runoff Generation

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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]:

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

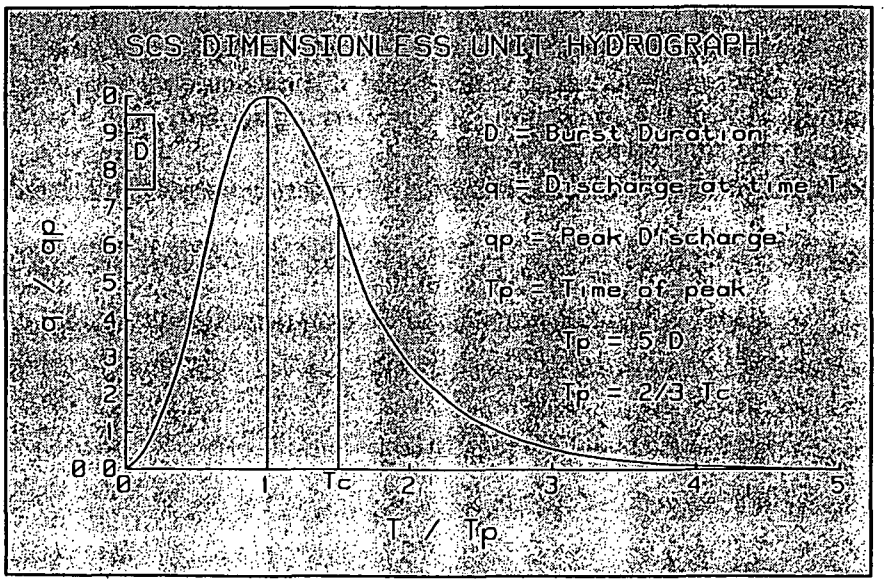
$$\text{where} \quad S = \frac{1000}{CN} - 10$$

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:

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

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.

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SCSUH Runoff Considerations

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

When you encounter this situation, keep in mind that the instantaneous peak can exceed the average for a time no longer than dt, which is commonly 6 minutes. In practice, such a short instantaneous peak is usually attenuated to the average value by the storage characteristics of 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 runoff 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 approaches 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 from 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.

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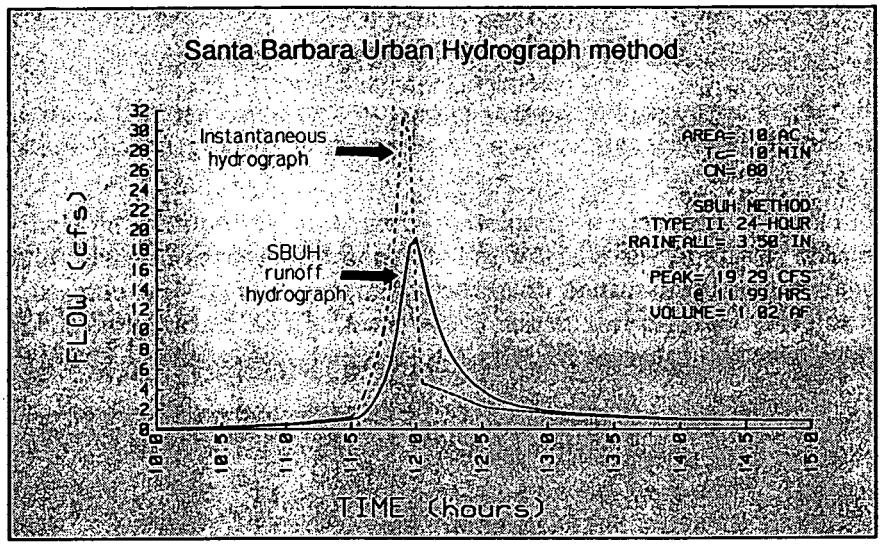
Section 24 - Santa Barbara Urban Hydrograph

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.

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Section 25 - Rational Method

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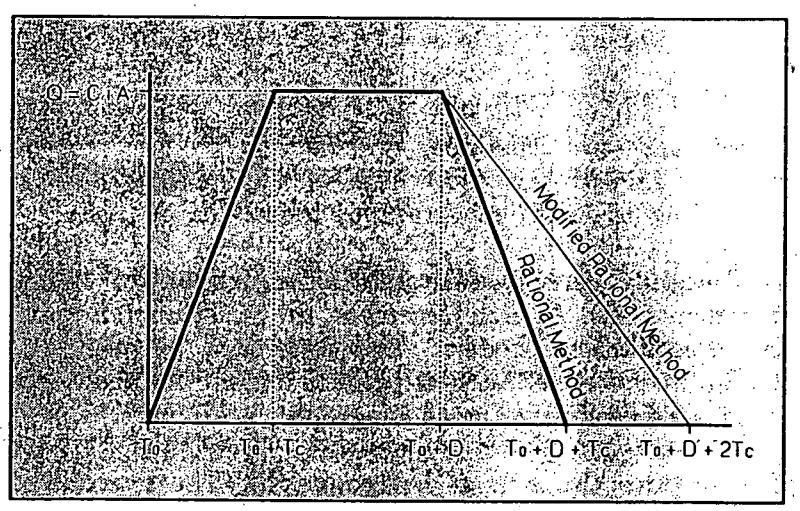
The Rational method or *Modified* Rational method may be used to generate runoff hydrographs. However, since Rational method was developed primarily for predicting peak flow, its use is *not* advised for volume-sensitive routing calculations. The Rational method predicts the peak runoff according to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

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Section 26 - Reach Routing Calculations

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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]

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

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.

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Reach Routing Limitations

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

Reach Routing Calculations

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

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

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

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

4) If the range of the storage and discharge curves is exceeded, HydroCAD extrapolates from the last two points on each curve. Since extrapolating from the curves is not the same as extending the physical sides of the reach, a warning message is issued. To obtain an accurate routing you 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.

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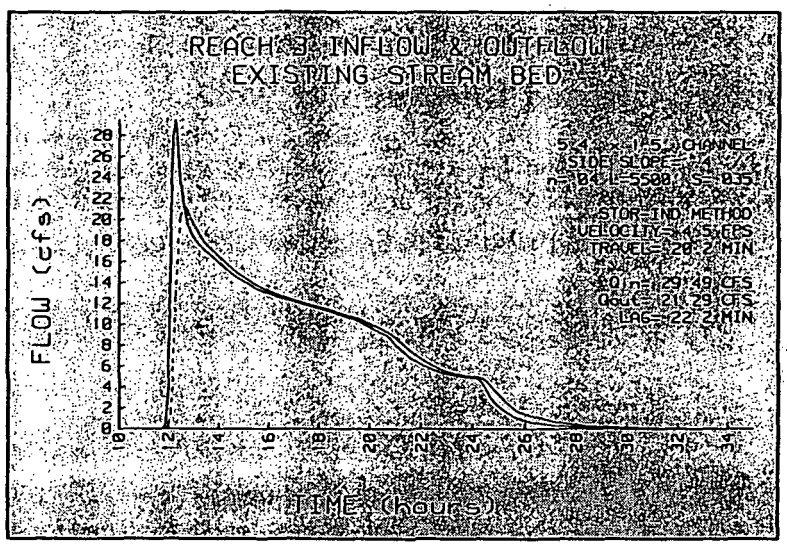
Effects of Reach Routing

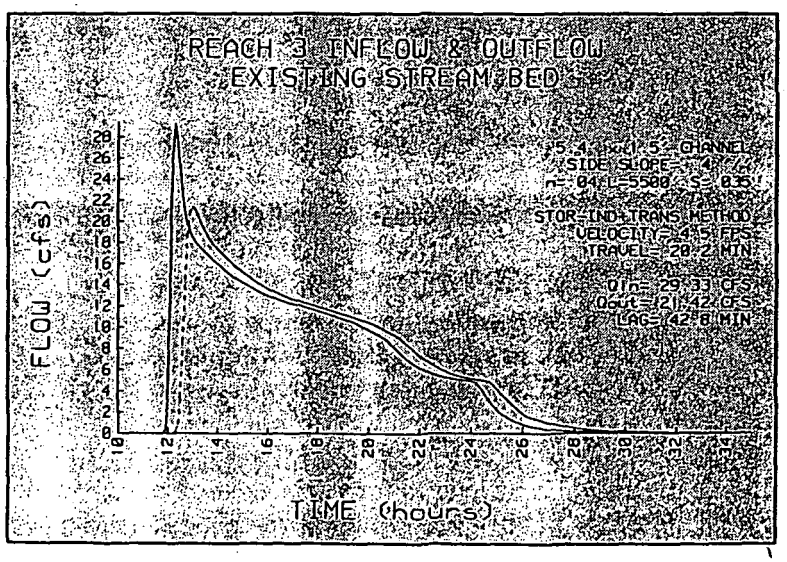
A reach will normally *attenuate and delay* the hydrograph that is routed through it. The extent of this transformation depends on many factors, including the reach dimensions, slope, and Manning's number. Short reaches (up to several hundred feet) often have a minimal effect on the routed hydrograph. 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.



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Section 27 - Hydraulics Calculations

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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.

$$Q = C\,L_e\,H^{3/2} \quad \text{where}$$

$$C = 3.27 + 4\frac{H}{P} \quad \text{and} \quad L_e = L - \frac{nH}{10} \qquad \text{Eq. 18}$$

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 <u>without</u> 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.

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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:

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

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.

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

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)

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Rectangular Orifice/Grate

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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.)

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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:

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

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]

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.

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Culvert Flow

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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]:

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

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.

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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:

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

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.

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Exfiltration Calculations

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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.)

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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.

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Section 28 - Pond Routing Calculations

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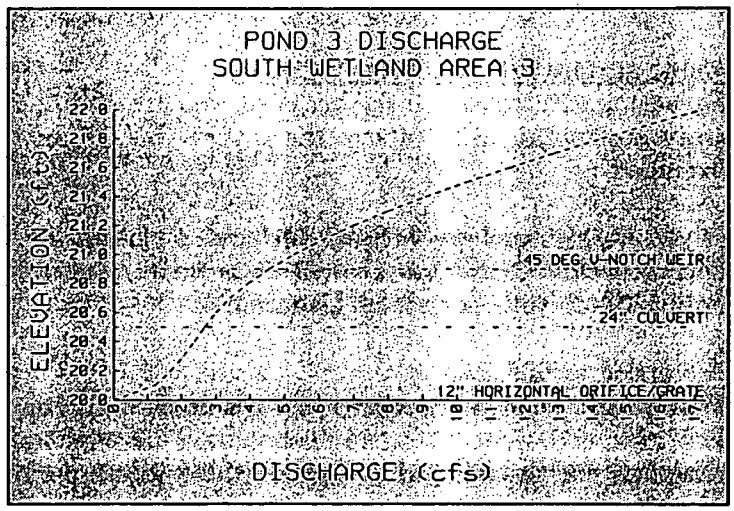
Before routing through any pond, the storage vs. elevation (stage-storage) and the discharge vs. elevation (stage-discharge) must be determined.

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.

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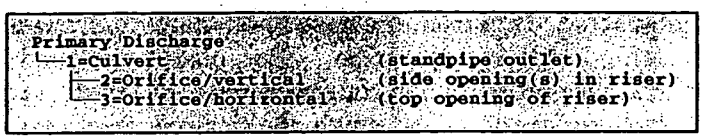
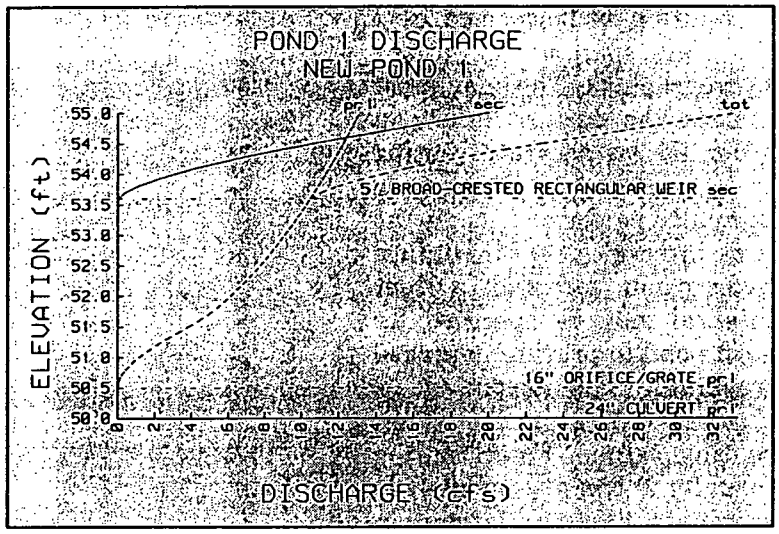
Compound Outlet Devices

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]



Even more complex outlets can be modeled by utilizing simultaneous series/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.

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.

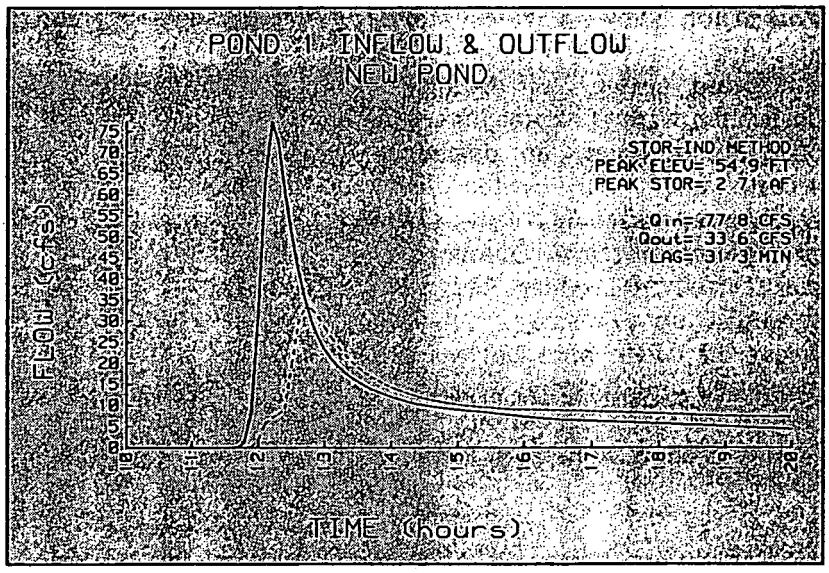
Pond Routing Procedure

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

1) Before routing begins, any base flow is added to the inflow hydrograph and any inflow loss is subtracted. If "automatic base flow" is selected, the base flow is set to the pond's discharge at the specified starting elevation. This places the pond in an equilibrium condition (stable water surface elevation) when routing begins. (To route a pond with no inflow, you must provide a zero-flow hydrograph to establish the desired time span and routing interval. This is most easily 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 above the lowest outlet device, the pond begins discharging immediately, possibly before any inflow has occurred. If the starting elevation is below the lowest device, no outflow occurs until this level 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

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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.)

Any water retained in the pond, or discharging after the specified time span, is excluded from the calculation. To obtain an accurate measure of detention time, it is therefore important to use a time 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.

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Section 30 - Link Calculations

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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_0), 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.

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Section 31 - Additional Hydrograph Calculations

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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.

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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.

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Rectangular, Vee, or Trapezoidal channel

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

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

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

$$a = \frac{2}{3} Y W$$

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

Eq. 35

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

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Circular Pipe (any flow depth)

$$a = \theta r^2 - (Y - r)\sqrt{Y(D - Y)} \qquad \text{Eq. 36}$$

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

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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)

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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

[48] A prismatic section must have four sides, with no more than two sloping sides, which must be opposite.

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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.

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

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.

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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.

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[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.

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[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.

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ATTACHMENT A-4

CulvertMaster® THEORY SECTION

Notes

Section 3: Theory

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; or,
- Develop hydraulic performance curves for a culvert system for assessing hydraulic risk at a crossing or as input to another hydraulic or hydrologic model.

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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/or other types of end treatments which act to 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.

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Hydrology

CulvertMaster offers the rational method, SCS peak discharge method, and user defined peak discharge for input of the design and check storm flows.

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:

$$Q = CiA \quad \text{or} \quad Q = AiR$$

where Q is design flow rate (cfs)

C, R is rational coefficient for drainage area

i is rainfall intensity (inches/hour)

A is drainage area (acres).

The conversion factor (1.008 ac-in/hr per cfs) which makes the Rational equation unit consistent is used by CulvertMaster. This factor converts the units of intensity in inches/hour and drainage area in acres to flow units of cubic feet per second. C, the rational coefficient, is the parameter that is most open to engineering judgment. Engineering references (refer to "References" at the end of this section) contain tables that will help you estimate C. On the next page is a table showing rational coefficients for common land uses. In many cases, an area weighted average of C coefficients is used as the C for the entire drainage area. CulvertMaster will calculate the weighted C for drainage areas.

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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

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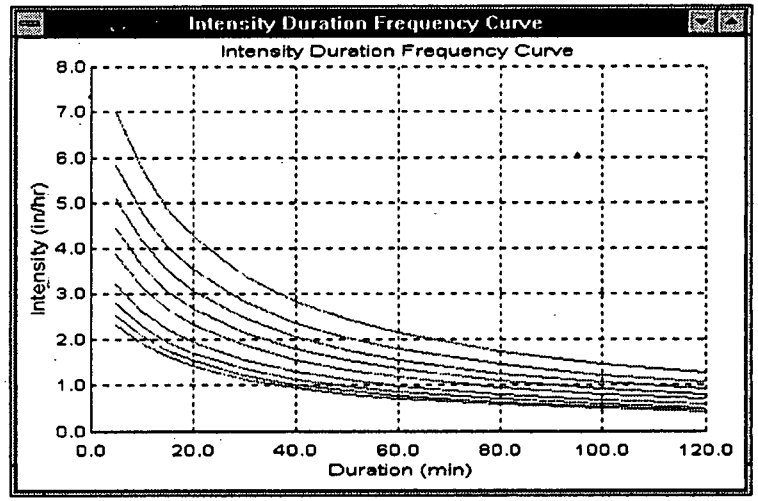
Intensity-Duration-Frequency (IDF) Curves

The rainfall intensities that you will be designing your culverts for are determined by regulatory agencies. Historical rainfall information has been analyzed and compiled into IDF Curves based on the frequency of the storm events. These curves give the engineer a quick reference to determine the intensity of rainfall that will occur at given return periods.

Plot of an IDF 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.

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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 a very simple process 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 of Rainfall Intensities

	Return Periods					
Durations	2 year	5 year	10 year	25 year	50 year	100 year
5 min	7.20	7.86	8.42	9.33	10.07	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 Service's 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

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

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

where i is intensity of rainfall
D is duration of rainfall
a, b, n are equation coefficients.

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Other equations supported in CulvertMaster are:

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

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

where i is intensity of rainfall
D is duration of rainfall
RP is return period
a, b, c, d, m, n are equation coefficients.

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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 the based on the following equations:

$$q_p = q_u A_m Q F_p$$

where q_p is the peak discharge (cfs)
q_u is the unit peak discharge (csm/in)
A_m is the drainage area (sq. mi)
Q is the runoff (in)
F_p is the pond and swamp adjustment factor.

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

where Q is the runoff (in)
P is the rainfall (in)
S is the potential maximum retention after runoff begins (in)
I_a is the initial abstraction (in).

$$I_a = 0.2S$$

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.

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RUNOFF CURVE NUMBERS FOR URBAN AREAS[1]

Cover description: Cover type and hydrologic condition	Average percent impervious area	Curve numbers for hydrologic soil group: A	B	C	D
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	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)		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in Runoff Curve Numbers for Other Agricultural Lands)					

Notes:
[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.

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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 exit 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 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.

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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:

1. 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.
2. Submerged - occurs in higher discharge ranges when the culvert entrance is fully submerged and the culvert is assumed to be operating as an orifice.
3. 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.

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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) or Form (2) Unsubmerged equations vary according to section shape and material.

Inlet Control Design Equations

UNSUBMERGED [1]

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

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

SUBMERGED [3]

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

where HW_i is the headwater depth above inlet control section invert (ft)
D is the interior height of culvert barrel (ft)
H_c is the specific head at critical depth ($d_c + V_c^2/2g$) (ft)
Q is the discharge (ft^3/s)
A is the full cross sectional area of culvert barrel (ft^2)
S is the culvert barrel slope (ft/ft)
K, M, c, Y are constants from the following table.

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

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Constants for Inlet Control Design Equations

CHT NO.	SHAPE AND MAT'L	NOMO-GRAPH SCALE	INLET EDGE DESCRIPTION	UNSUBMERGED FORM	UNSUBMERGED K	UNSUBMERGED M	SUBMERGED C	SUBMERGED Y
1	Circular	1	Square edge w/headwall	1	0.0098	2.0	0.0398	0.67
	Concrete	2	Groove end w/headwall		.0078	2.0	.0292	.74
		3	Groove end projecting		.0045	2.0	.0317	.69
2	Circular	1	Headwall	1	.0078	2.0	.0379	.69
	CMP	2	Mitered to slope		.0210	1.33	.0463	.75
		3	Projecting		.0340	1.50	.0553	.54
3	Circular	A	Beveled ring, 45° bevels	1	.0018	2.50	.0300	.74
		B	Beveled ring, 33.7° bevels		.0018	2.50	.0243	.83
8	Rectangular	1	30° to 75° wingwall flares		.026	1.0	.0385	.81
	Box	2	90° and 15° wingwall flares	1	.061	.75	.0400	.80
		3	0° wingwall flares		.061	.75	.0423	.82
9	Rectangular	1	45° wingwall flare d=.0430	2	.510	.667	.0309	.80
	Box	2	18° - 33.7° wwall flare d=.0830		.486	.667	.0249	.83
10	Rectangular	1	90° headwall w/¾" chamfers	2	.515	.667	.0375	.79
	Box	2	90° headwall w/45° bevels		.495	.667	.0314	.82
		3	90° headwall w/33.7° bevels		.486	.667	.0252	.865
11	Rectangular	1	¾" chamfers;45° skewed headwall	2	.522	.667	.0402	.73
	Box	2	¾" chamfers;30° skewed headwall		.533	.667	.0425	.705
	¾"chamfers	3	¾" chamfers;15° skewed headwall		.545	.667	.04505	.68
		4	45° bevels;10°-45° skewed headwall		.498	.667	.0327	.75
12	Rectangular	1	45° non-offset wingwall flares	2	.497	.667	.0339	.803
	Box	2	18.4° non-offset wingwall flares		.493	.667	.0361	.806
		3	18.4° non-offset wingwall flares 30° skewed barrel		.495	.667	.0386	.71
13	Rectangular	1	45° wingwall flares - offset	2	.497	.667	.0302	.835
	Box	2	33.7° wingwall flares - offset		.495	.667	.0252	.881
	Top Bevels	3	18.4° wingwall flares - offset		.493	.667	.0227	.887
16-19	C M Boxes	1	90° headwall	1	.0083	2.0	.0379	.69
		2	Thick wall projecting		.0145	1.75	.0419	.64
		3	Thin wall projecting		.0340	1.5	.0496	.57
29	Horizontal	1	Square edge w/headwall	1	.0100	2.0	.0398	.67
	Ellipse	2	Groove end w/headwall		.0018	2.5	.0292	.74
	Concrete	3	Groove end projecting		.0045	2.0	.0317	.69
30	Vertical	1	Square edge w/headwall	1	0.0100	2.0	0.0398	0.67
	Ellipse	2	Groove end w/headwall		.0018	2.5	.0292	.74
	Concrete	3	Groove end projecting		.0095	2.0	.0317	.69
34	Pipe Arch	1	90° headwall	1	.0083	2.0	.0379	.69
	18" Corner	2	Mitered to slope		.0300	1.0	.0463	.75
	Radius CM	3	Projecting		.0340	1.5	.0496	.57
35	Pipe Arch	1	Projecting	1	.0296	1.5	.0487	.55
	18" Corner	2	No Bevels		.0087	2.0	.0361	.66
	Radius CM	3	33.7° Bevels		.0030	2.0	.0264	.75
36	Pipe Arch	1	Projecting	1	.0296	1.5	.0487	.55
	31" Corner	2	No Bevels		.0087	2.0	.0361	.66
	Radius CM	3	33.7° Bevels		.0030	2.0	.0264	.75
40-42	Arch CM	1	90° headwall	1	.0083	2.0	.0379	.69
		2	Mitered to slope		.0300	1.0	.0463	.75
		3	Thin wall projecting		.0340	1.5	.0496	.57
55	Circular	1	Smooth tapered inlet throat	2	.534	.555	.0196	.89
		2	Rough tapered inlet throat		.519	.64	.0289	.90
56	Elliptical	1	Tapered inlet-beveled edges	2	.536	.622	.0368	.83
	Inlet Face	2	Tapered inlet-square edges		.5035	.719	.0478	.80
		3	Tapered inlet-thin edge projecting		.547	.80	.0598	.75
57	Rectangular	1	Tapered inlet throat	2	.475	.667	.0179	.97
58	Rectangular	1	Side tapered-less favorable edges	2	.56	.667	.0466	.85
	Concrete	2	Side tapered-more favorable edges		.56	.667	.0378	.87
59	Rectangular	1	Slope tapered-less favorable edges	2	.50	.667	.0466	.65
	Concrete	2	Slope tapered-more favorable edges		.50	.667	.0378	.71

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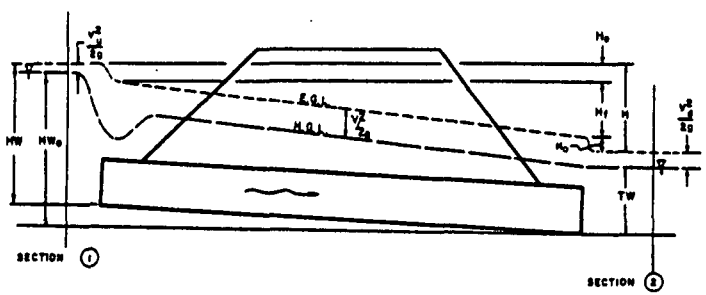
Outlet Control Hydraulics

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 are presented graphically in the figure and the basic energy equation.

Full Flow Energy and Hydraulic Grade Lines



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

HW_o is the headwater depth above the outlet invert, ft (m)

V_u is the approach velocity, ft/s (m/s)

TW is the tailwater depth above the outlet invert, ft (m)

V_d is the exit velocity, ft/s (m/s)

H_L is the sum of all losses including entrance (H_e), friction loss (H_f), exit loss (H_o), and other losses.

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

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

- Partial flow - water surface profiles are computed so the actual flow depths at the entrance and exit of the culvert during part-full conditions are accurately calculated.
- Pressure flow - backwater during full barrel submergence.
- Composite pressure and free surface flows - both sealing and unsealing conditions.
- 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 which uniform flows are undefined.

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Entrance Minor Loss

The entrance loss (H_e) is a function of the barrel velocity head (V) just inside the entrance and is expressed by the equation:

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

The entrance loss coefficient (k_e) according to inlet configuration. Values for the coefficients are presented in the table on the following page.

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ENTRANCE LOSS COEFFICIENTS

Outlet Control, Full or Partly Full Entrance Head Loss

Type and Description	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 of 1/12 barrel dimension, or beveled edges 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
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

*Note: "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.

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

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Exit Minor Loss

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

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

where V_d is the outfall channel velocity, ft/s (m/s).

The discharge velocity can be neglected in which case the exit loss is equal to the barrel velocity head. This is the assumption that CulvertMaster makes.

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, increasing or decreasing, 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 and 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.

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

For a structure operating at a hydraulically steep slope (i.e. constructed slope is greater than critical slope), gradually varied flow profiles will be developed in an upstream direction whenever the controlling tailwater 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.

Usually culverts on a steep slope are entrance or inlet controlled. These structures usually operate under supercritical flow. CulvertMaster 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 downstream direction. In this type of frontwater analysis the depth decreases from critical depth and approaches normal depth at the culvert exit.

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

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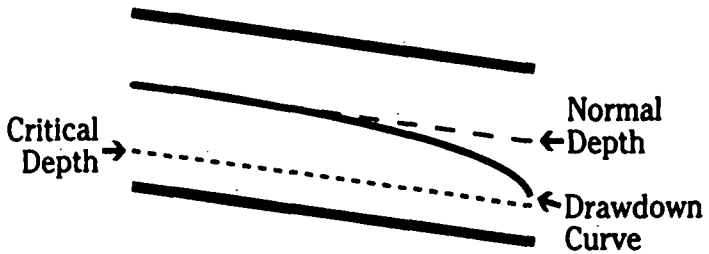
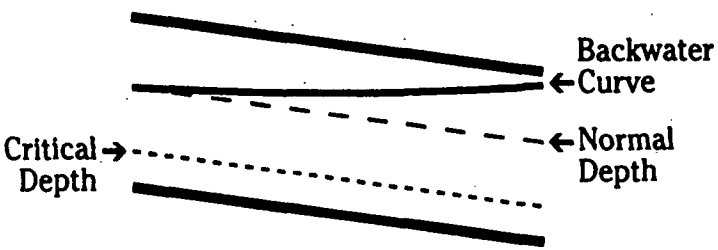
Backwater Analysis

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

Plot of M2 Drawdown Curve



Plot of M1 Backwater Curve



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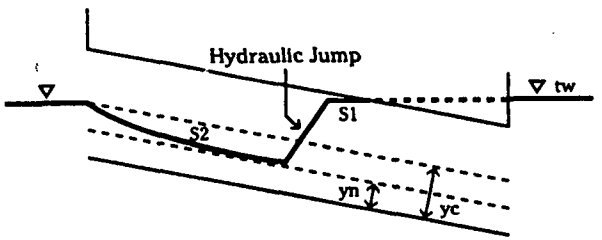
Frontwater Analysis

CulvertMaster will perform a frontwater analysis in a steep culvert operating under supercritical flow which usually means the culvert is inlet controlled. The hydraulic control will be at the upstream end of a hydraulically steep culvert and the gradually varied flow analysis will proceed in a downstream direction until normal depth is achieved, a hydraulic jump occurs, or the end of the culvert is reached. Even though outlet control rarely occurs in supercritical flow situations, the frontwater analysis is still performed for purposes of making an accurate determination of exit velocity.

Plot of Composite Backwater/Frontwater with Hydraulic Jump



where
S1 is the steep slope - Type 1 flow profile classification
S2 is the steep slope - Type 2 flow profile classification
tw is the tailwater elevation
yc is the critical depth in the pipe
yn is the normal depth in the pipe

Direct Step Method

This is a method for solving gradually varied flow profile in which a series of explicit solutions for flow length are made which will result in a specified depth of flow. In the direct step method, the program iterates over a range of depth changes that result from a backwater or drawdown curve and computes the interval distance that would result in the correct amount of headloss producing the energy balance. This interval distance is then summed with all previous intervals to compute the total distance over which the flow profile has 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 that when summed with the previous profile distance will equal the total length of the conduit.

The basic equation for energy balance is:

$$H_1 = H_2 + h_f + h_e$$

where H_1 is the total head at section 1
H_2 is the total head at section 2
h_f is the friction loss
h_e is the eddy loss.

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

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

$$H = Z + \frac{V^2}{2g}$$

where H is the total head
Z is the water surface elevation above the datum
V is the velocity at the end of the section
g is the gravitational constant.

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

$$S_f \Delta x = \tfrac{1}{2}(S_1 + S_2)\Delta x$$

where S_f is the friction slope
Δx is the change in horizontal distance in the section
S_1, S_2 are the friction slopes at the end of the section.

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 has proven to be 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 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:

$$S_f \Delta x = \tfrac{1}{2}(S_1 + S_2)\Delta x$$

yields an average friction slope, S_f, that balances energy between each end of the sub-reach.

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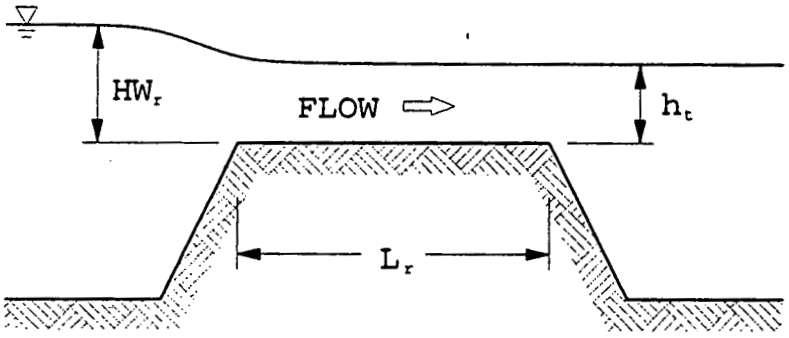
Roadway Overtopping

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.



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

Where Q_o is the overtopping flow rate in ft^3/s (m^3/s)

C_d is the overtopping discharge coefficient

L is the length of the roadway crest, ft (m)

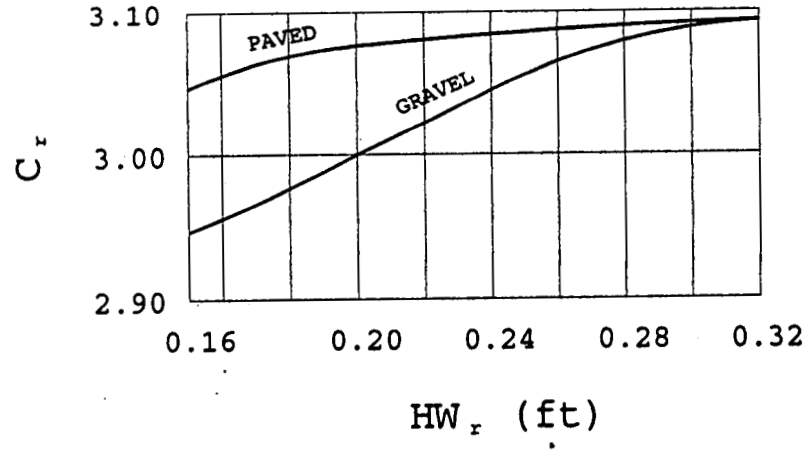
HW_r is the overtopping depth

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

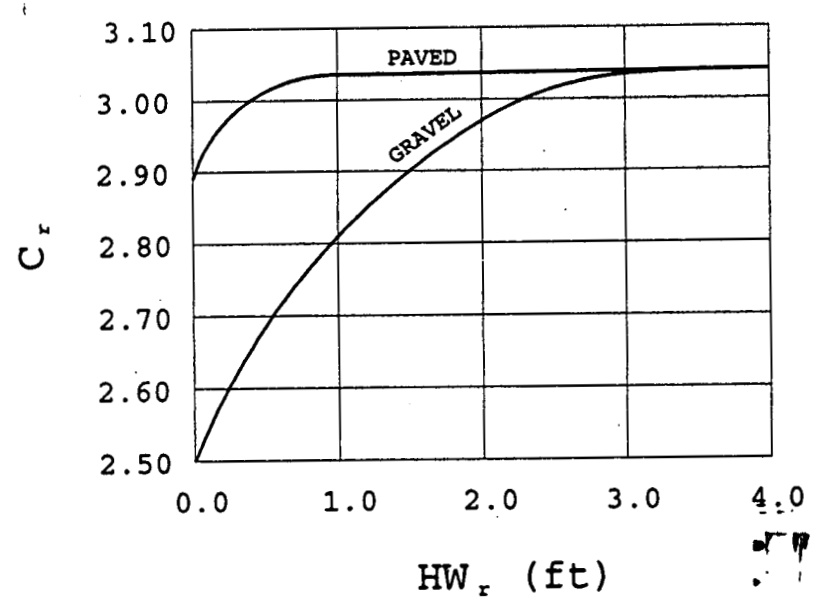
$$C_d = k_t C_r$$

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

Discharge Coefficient for $HW_r/L_r > 0.15$



Discharge Coefficient for $HW_r/L_r \leq 0.15$



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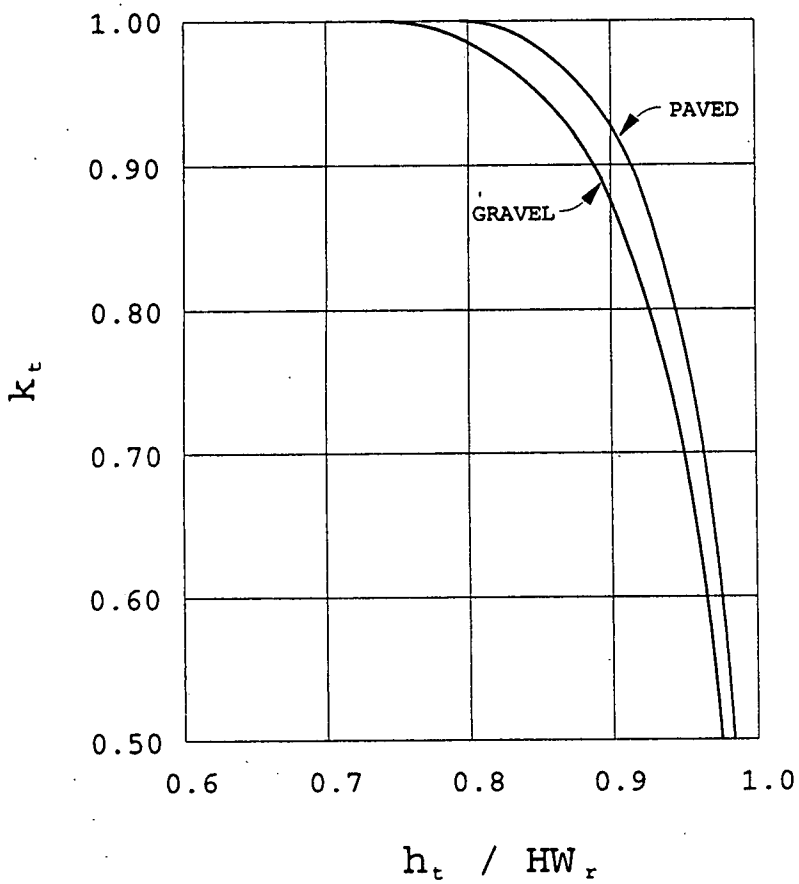
Submergence Factor h_t/HW_r



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

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Tailwater

The tailwater depth and elevation at the culvert discharge channel is an important parameter for computing the response of the culvert. For backwater analyses, the tailwater elevation or critical depth (whichever is higher) is the starting gradually varied flow boundary. CulvertMaster will automatically compute discharge variable tailwater depths using its built-in uniform flow calculation capability.

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:

$$V = CR^x S^y$$

where V is the mean velocity (typically in ft/s or m/s)
R is the hydraulic radius (typically in ft or m)
S is the energy slope (unitless or ft/ft or m/m)
C is a flow resistance factor
x and y are exponents.

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

Since velocity is constant under uniform flow (constant discharge and area conditions), combining the general uniform flow equation with the continuity equation

$$Q = VA$$

results in the equation:

$$Q = ACR^x S^y$$

This equation yields a complete solution for the rate of flow.

where Q is discharge (ft^3/s or m^3/s)
A is cross-sectional flow area (ft^2 or m^2)
R is the hydraulic radius (typically in ft or m)
S is the energy slope (unitless or ft/ft or m/m)
C is a flow resistance factor
x and y are exponents.

Note: Hydraulic radius is related to flow area and further reduction of this equation for specific geometries is often possible.

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Manning's Formula

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

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

Manning's Formula is derived from the Chezy Formula, which computes velocity based on a C coefficient, hydraulic radius, and friction slope. Manning's Formula relates this C coefficient to a roughness coefficient and hydraulic radius. The original format incorporated the roughness coefficient (flow resistance factor):

$$V = CR^{1/2}S^{1/2}$$

where C is the factor of flow resistance
R is the hydraulic radius
S is the energy slope.

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

U.S. Units:

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

where V is the section velocity (ft/s)
R is the hydraulic radius (ft)
S is the energy slope
n is the Manning's n.

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Metric:

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

where V is the section velocity (m/s)
R is the hydraulic radius (m)
S is the energy slope
n is the Manning's n.

Additional Theory

The central reference for practical hydraulic design methods and techniques for the analysis and sizing of highway culverts is the publication *HDS No. 5, Hydraulic Design of Highway Culverts*, Federal Highway Administration, McLean, Virginia, September, 1985.

Haestad Methods, Inc. can provide a complete copy of this 253 page document for our cost of reproduction ($35) plus shipping. If you have a licensed copy of HY-8 or Pond Pack from Haestad Methods, you should already have a copy of this document.

Learning More

More detailed information is available on all of CulvertMaster's features. Look in the on-line Help system for complete details.

Experiment with the available options as well as CulvertMaster's many powerful features. If you have not already used CulvertMaster, run the **One Minute Tutorial** to quickly see how easy it is to use CulvertMaster.

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Written by: JGT Date: 3 May 2000 Reviewed by: Date:

Client: Fluor Fernald, Inc. Project: OSDF Phase III Project No.: GQ1001 Task No.: 04

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ATTACHMENT A-5

USDA-SCS SOIL SURVEY MAP

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

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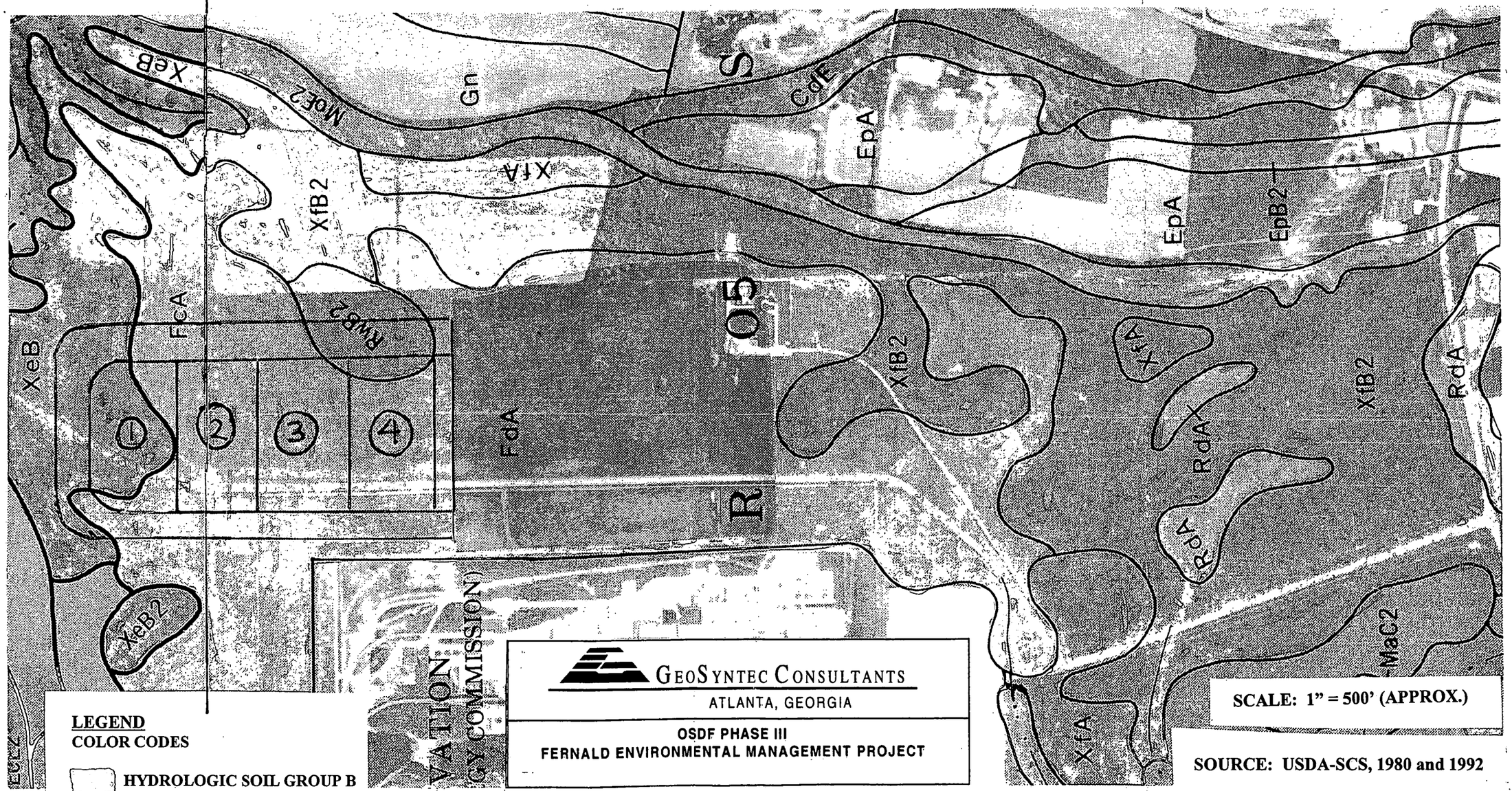

GEOSYNTEC CONSULTANTS
ATLANTA, GEORGIA

OSDF PHASE III
FERNALD ENVIRONMENTAL MANAGEMENT PROJECT

SCALE: 1" = 500' (APPROX.)

SOURCE: USDA-SCS, 1980 and 1992

LEGEND
COLOR CODES

- HYDROLOGIC SOIL GROUP B
- HYDROLOGIC SOIL GROUP C

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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.

Source: USDA-SCS, 1992.

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Soil survey

TABLE 18.--SOIL AND [illegible]R FEATURES--Continued

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				
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.

Source: USDA-SCS, 1992.

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

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				
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.

Source: USDA-SCS, 1992.

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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

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112/123 Jr 5/5/0
RO 5/5/03
Soil
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TABLE 16.--SOIL 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	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				
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.

Source: USDA-SCS, 1986.

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113/123 [illegible] 171

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

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				
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, 1986.

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114/13 SOIL SURVEY

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ATTACHMENT A-7

TR-55 LAND USE CATEGORIES AND RUNOFF CURVE NUMBERS

116/123 5/5/00
RDG 5/5/00

HSG	Soil textures
A	Sand, loamy sand, or sandy loam
B	Silt loam or loam
C	Sandy clay loam
D	Clay loam, silty clay loam, sandy clay, silty clay, or clay

TECHNICAL SPECIFICATIONS FOR TOP SOIL (PHASE 3)

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

RUNON AREAS TO BORROW AREA & BORROW AREA VEGETATION ESTABLISHED

UNVEGETATED BORROW AREA

LINER RUNOUT AND UNVEGETATED FINAL COVER UNDER CONSTRUCTION

B C

[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.

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117/123 JT 5/5/00
RDE 5/5/00

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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] RUNON AREAS NORTH OF OSDF AND OSDF RUNON AREA	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] DISTURBED AREA - CONSTRUCTION SUPPORT	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

[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.

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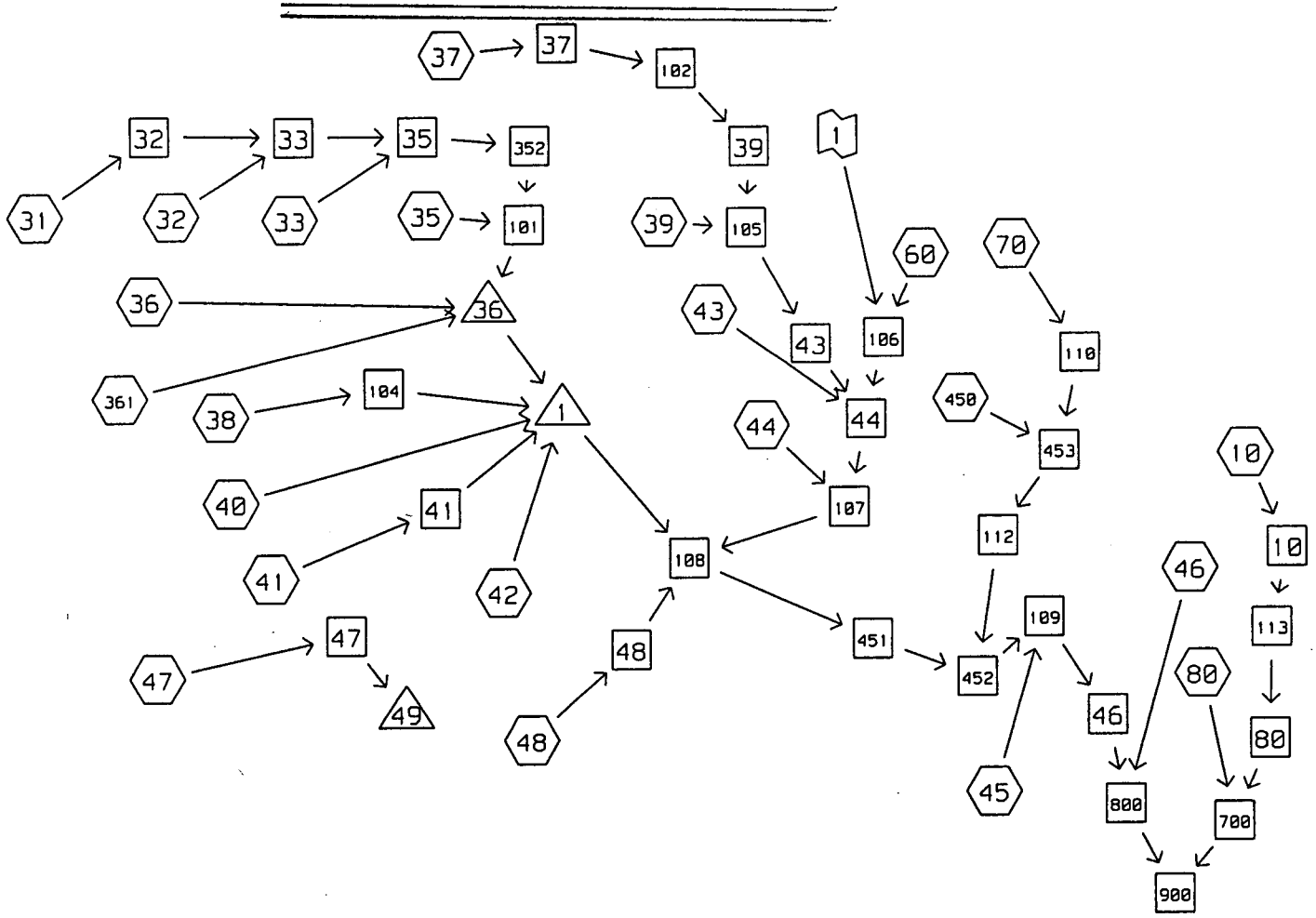
ATTACHMENT A-8

NODAL NETWORK DIAGRAMS

OSDF DESIGN SCENARIO



SUBCATCHMENT REACH POND LINK

Drainage Diagram for OSDF DESIGN SCENARIO
Prepared by GeoSyntec Consultants 7 Apr 00
HydroCAD 5.11 000929 (c) 1986-1999 Applied Microcomputer Systems

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BDE 5/5/00

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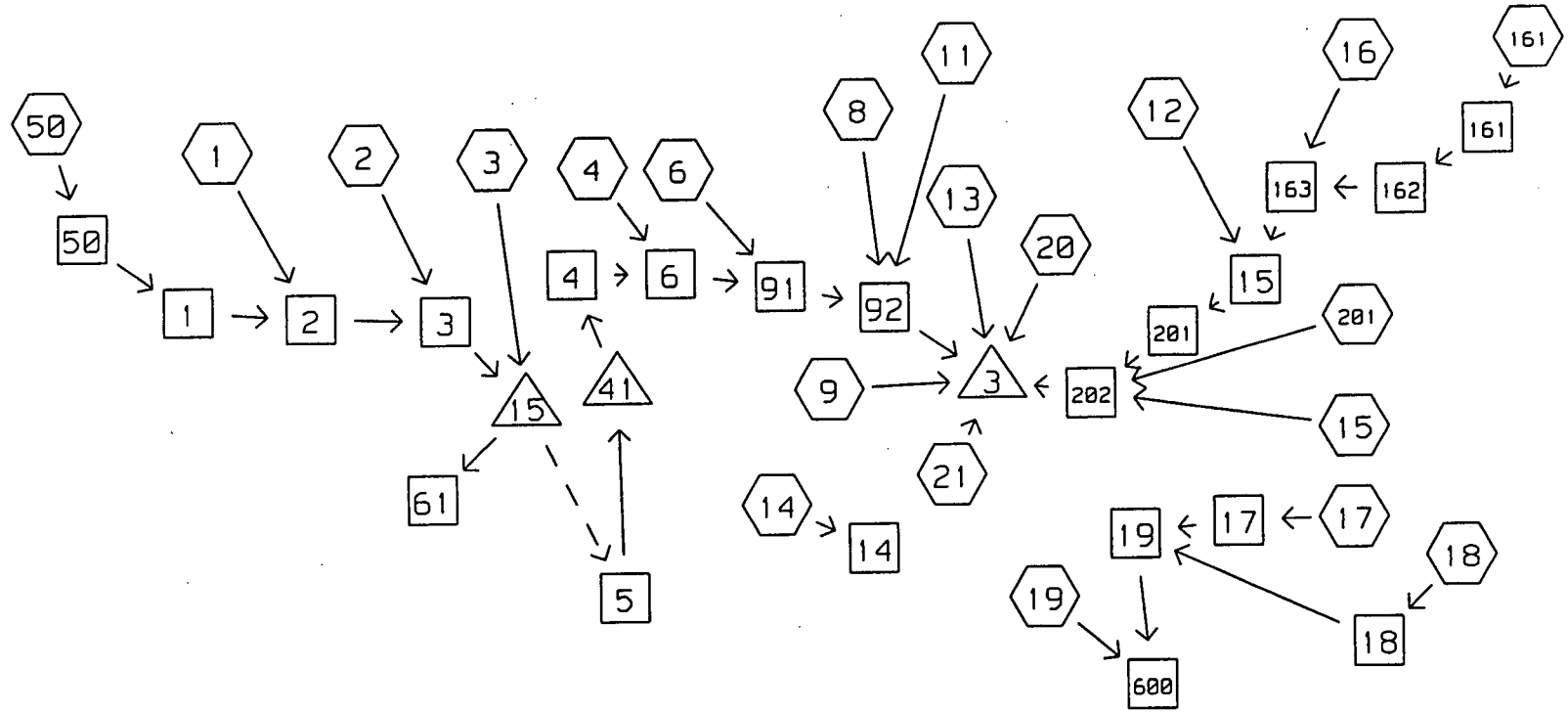
BORROW AREA DESIGN SCENARIO



SUBCATCHMENT REACH POND LINK

Drainage Diagram for BORROW AREA DESIGN SCENARIO
Prepared by GeoSyntec Consultants 7 Apr 00
HydroCAD 5.11 000929 (c) 1986-1999 Applied Microcomputer Systems

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RDE 5/5/00

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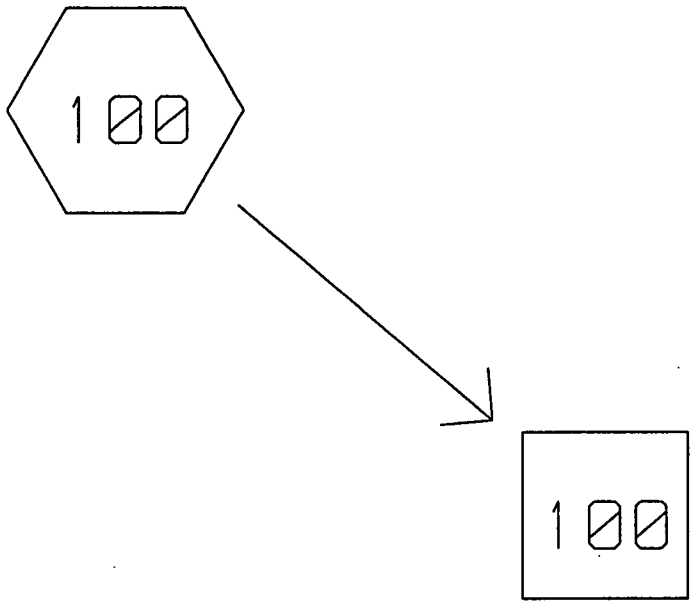
DESIGN CASE A



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121/123 JT 5/5/00
RDE 5/5/00

SUBCATCHMENT REACH POND LINK

Drainage Diagram for DESIGN CASE A
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ATTACHMENT A-9

CMP COVER REQUIREMENTS

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ROE 5/5/00
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CMP COVER REQUIREMENTS

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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

[CONTECH, 1999]

PHASE III 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 and Borrow Area surface-water management (SWM) structures to be constructed as part of the Phase III 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™)

Subcatchment Runoff

For both design scenarios and the design case, the relationships of subcatchments, reaches, and basins are shown in 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.

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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 for each design scenario 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. The Manning's roughness coefficient smooth graded bare soul is selected as 0.022. Data for additional channel parameters are tabulated in Attachment B-5.

Culverts

The Manning's roughness coefficients for CMP, CMPA, RCP, PVC, and HDPE culverts are selected as 0.024, 0.018, 0.013, 0.010, and 0.010, respectively

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Sedimentation Basins

Planimetered areas within contour elevation lines (for calculation of 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 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

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input data for hydraulic analyses in Attachment B-6. A plan view drawing showing the location of new and existing culverts is provided in Attachment B-6.

For evaluation of the structural stability of the one RCP culvert, input parameters include the following:

- γ_s is assumed as 135 pcf;
- B_c is 2.5 ft + (2X3.5 in.)/12 = 3.08 ft for Wall B RCP culvert; and
- I.F. is selected as 1.5.

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REFERENCES

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

HydroCAD. *HydroCADTM: Stormwater Modeling System, Version 5*. Applied Microcomputer Systems, Chocorua, New Hampshire, 1998.

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.

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ATTACHMENT B-1

SCS RAINFALL DISTRIBUTION TYPE

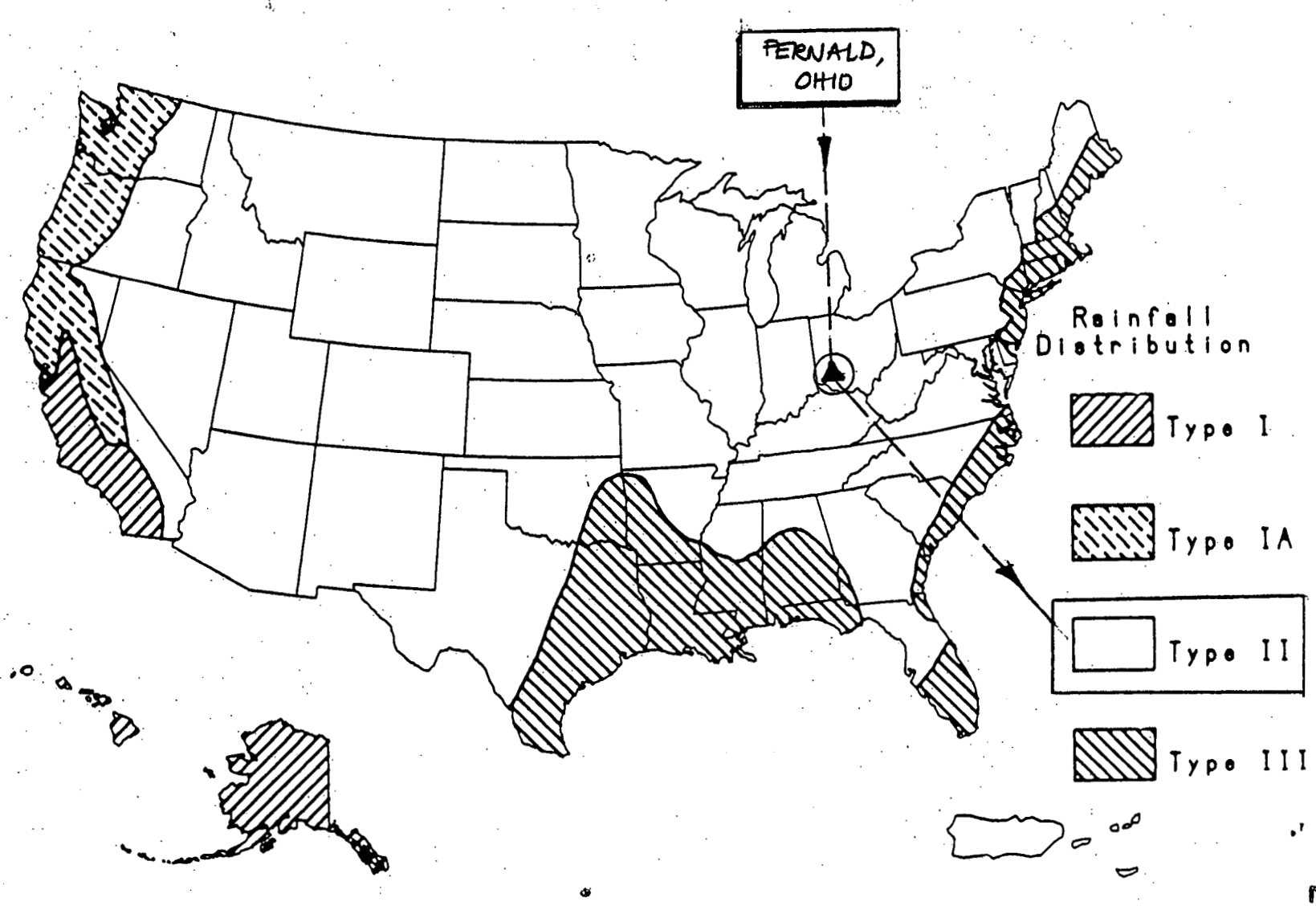

Figure B-2.—Approximate geographic boundaries for SCS rainfall distributions.

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ATTACHMENT B-2

RAINFALL DEPTHS

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RJE 5/5/00
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TABLE 1 - RAINFALL DEPTH FOR A GIVEN DURATION (INCHES)

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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.

REF: PARSONS, "2,000-YEAR FLOOD AND PROBABLE MAXIMUM FLOOD, SITEWIDE FLOOD PLAIN DETERMINATION", AUG 1995.

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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

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Subcatchment Label	Area (acres)	Percent of Total Area %	HSG	Land Use Category	CN	Weighted CN
10	0.95	100	C	Runon Areas North of Borrow Area	79	
31	4.60	100	D	Unvegetated Final Cover System	94	
32	3.72	100	D	Unvegetated Final Cover System	94	
33	1.55	15 85	C B	Disturbed Area - Construction Support Disturbed Area - Construction Support	82 74	
35	2.35	100	C	Disturbed Area - Construction Support	82	
36	4.26	80 20	C B	Disturbed Area - Construction Support Disturbed Area - Construction Support	82 74	
37	1.85	100	C	Disturbed Area - Construction Support	82	
38	4.64	100	C	Disturbed Area - Construction Support	82	
39	4.33	100	C	Disturbed Area - Construction Support	82	
40	3.03	100	C	Disturbed Area - Construction Support	82	
41	1.45	100	C	Disturbed Area - Construction Support	82	
42	2.29	100	N/A	Sedimentation Basin	98	
43	0.81	100	C	Disturbed Area - Construction Support	82	
44	1.04	100	C	Disturbed Area - Construction Support	82	
45	3.58	100	C	Disturbed Area - Construction Support	82	
46	2.83	100	C	Disturbed Area - Construction Support	82	
47	1.64	100	C	Disturbed Area - Construction Support	82	
48	1.94	100	C	Disturbed Area - Construction Support	82	
60	1.30	100	C	Runon Areas North of Borrow Area	79	
70	1.47	100	C	Runon Areas North of Borrow Area	79	
80	0.80	100	C	Runon Areas North of Borrow Area	79	
361	0.11	100	N/A	(Fictitious - for storage upstream of culvert) Sedimentation Basin	98	
450	1.37	100	C	Disturbed Area - Construction Support	82	

N/A - Not Applicable

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RE 5/5/00
GeoSyntec Consultants
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SUBCATCHMENT AREA AND HYDROCAD™ INPUT PARAMETERS FOR THE CALCULATION OF WEIGHTED CN

Borrow Area Design Scenario

Subcatchment Label	Area (acres)	Percent of Total Area %	HSG	Land Use Category	CN	Weighted CN
1	3.22	30	B	Runon Areas East of OSDF	69	
		20	B	Runon Areas North of Borrow Area	69	
		30	C	Runon Areas East of OSDF	79	
		20	C	Runon Areas North of Borrow Area	79	
2	8.64	50	B	Runon Areas East of OSDF	69	
		10	B	Runon Areas North of Borrow Area	69	
		20	C	Runon Areas East of OSDF	79	
		20	C	Runon Areas North of Borrow Area	79	
3	21.99	60	B	Runon Areas East of OSDF	69	
		10	C	Runon Areas East of OSDF	79	
		30	C	Runon Areas North of Borrow Area	79	
4	13.52	20	B	Runon Areas East of OSDF	69	
		10	C	Runon Areas East of OSDF	79	
		65	C	Runon Areas North of Borrow Area	79	
		5	B	Runon Areas North of Borrow Area	69	
6	21.41	15	B	Runon Areas East of OSDF	69	
		35	C	Runon Areas North of Borrow Area	79	
		10	C	Disturbed Area - Construction Support	82	
		30	B	Runon Areas North of Borrow Area	69	
		10	B/C	Stockpile	87	
8	1.81	50	B/C	Stockpile	87	
		50	C	Runon Areas North of Borrow Area	79	
9	1.92	75	C	Runon Areas North of Borrow Area	79	
		10	B	Runon Areas North of Borrow Area	69	
		10	B	Vegetated Borrow Area	61	
		5	C	Vegetated Borrow Area	74	
11	15.54	5	C	Runon Areas to Borrow Area	74	
		5	B	Runon Areas to Borrow Area	61	
		50	B	Vegetated Borrow Area	61	
		15	C	Vegetated Borrow Area	74	
		5	B	Runon Areas North of Borrow Area	69	
		5	C	Runon Areas North of Borrow Area	79	
		15	B/C	Stockpile	87	
12	15.93	5	C	Runon Areas to Borrow Area	74	
		20	B	Runon Areas to Borrow Area	61	
		20	B/C	Stockpile	87	
		50	B	Vegetated Borrow Area	61	
		5	C	Vegetated Borrow Area	74	
13	4.10	100	B	Vegetated Borrow Area	61	
14	1.40	80	B	Vegetated Borrow Area	61	
		20	C	Runon Areas to Borrow Area	79	
15	11.16	30	C	Unvegetated Borrow Area	91	
		70	B	Unvegetated Borrow Area	86	
16	6.19	20	B	Runon Areas to Borrow Area	61	
		35	B	Vegetated Borrow Area	61	
		45	B/C	Stockpile	87	
17	3.63	15	B	Runon Areas to Borrow Area	61	
		85	B	Vegetated Borrow Area	61	
18	6.01	5	B	Runon Areas to Borrow Area	61	
		5	C	Runon Areas to Borrow Area	74	
		90	B	Vegetated Borrow Area	61	
19	2.10	100	B	Vegetated Borrow Area	61	
20	2.08	100	B	Vegetated Borrow Area	61	
21	1.69	100	-	Sedimentation Basin	98	
50	0.58	100	C	Runon Areas North of Borrow Area	79	
161	1.48	100	B	Vegetated Borrow Area	61	
201	0.20	60	B	Vegetated Borrow Area	61	
		40	C	Vegetated Borrow Area	74	
Clay stockpile	15.72	80	B	Unvegetated Borrow Area	86	
		20	C	Unvegetated Borrow Area	91	

N/A - Not Applicable

JH 5/5/01

RAE 5/5/01

13/27

2983

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
100	2.28	100	D	Liner Runout	94	

Written by: JGT Date: 3 May 2000 Reviewed by: Date:

Client: Fluor Fernald, Inc. Project: OSDF Phase III Project No.: GQ1001 Task No.: 04

2983

ATTACHMENT B-4

DATA FOR CALCULATION OF TIME OF CONCENTRATION

JT 5/5/00
RDE 5/5/00
16/27

= 2983

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

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

SUBCATCHMENT LABEL AND DESCRIPTION		SHEET FLOW 1				SHALLOW CONCENTRATED FLOW 1			SHALLOW CONCENTRATED FLOW 2			CHANNEL FLOW 1						CHANNEL FLOW 2						CHANNEL FLOW 3					
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)	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)	Flow Length (ft)	Bottom Width (ft)	Flow Depth (ft)	Sideslopes (ft/ft)	Manning's n	Longitudinal Slope (ft/ft)	Flow Length (ft)	Bottom Width (ft)	Flow Depth (ft)	Sideslopes (ft/ft)	Manning's n	Longitudinal Slope (ft/ft)
1		300	GRASS	0.15	0.007	50	UNPAVED	0.020	-	-	-	650	0	2.0	0.33, 0.20	0.03	0.007	-	-	-	-	-	-	-	-	-	-	-	-
2		300	GRASS	0.15	0.012	520	UNPAVED	0.012	-	-	-	370	0	2.0	0.20, 0.25	0.03	0.011	-	-	-	-	-	-	-	-	-	-	-	-
3		300	GRASS	0.15	0.015	1300	UNPAVED	0.015	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
4		300	GRASS	0.15	0.018	1100	UNPAVED	0.018	-	-	-	300	0	2.0	0.33	0.03	0.003	-	-	-	-	-	-	-	-	-	-	-	-
6		300	GRASS	0.15	0.040	900	UNPAVED	0.018	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
8		100	GRASS	0.15	0.020	-	-	-	-	-	-	300	0	3.0	0.33	0.03	0.008	-	-	-	-	-	-	-	-	-	-	-	-
9		170	GRASS	0.15	0.050	-	-	-	-	-	-	150	10	2.0	0.33	0.03	0.003	-	-	-	-	-	-	-	-	-	-	-	-
11		300	GRASS	0.15	0.030	950	UNPAVED	0.020	-	-	-	370	0	3.0	0.33	0.03	0.011	-	-	-	-	-	-	-	-	-	-	-	-
12		300	GRASS	0.150	0.026	500	UNPAVED	0.016	-	-	-	650	0	0.5	0.04, 0.03	0.03	0.012	80	0	2	0.33	0.03	0.007	-	-	-	-	-	-
13		300	GRASS	0.150	0.050	20	UNPAVED	0.050	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
14		-	-	-	-	-	-	-	-	-	-	600	0	3.0	0.33	0.03	0.013	-	-	-	-	-	-	-	-	-	-	-	-
15		85	SMOOTH	0.011	0.330	215	UNPAVED	0.003	640	UNPAVED	0.0025	325	0	0.5	0.005, 0.33	0.03	0.001	-	-	-	-	-	-	-	-	-	-	-	-
16		300	GRASS	0.150	0.008	220	UNPAVED	0.032	-	-	-	380	0	1.0	0.06, 0.067	0.03	0.029	430	0	1.0	0.12, 0.03	0.03	0.02	-	-	-	-	-	-
17		300	GRASS	0.150	0.020	440	UNPAVED	0.018	-	-	-	380	0	0.5	0.03, 0.10	0.03	0.03	-	-	-	-	-	-	-	-	-	-	-	-
18		300	GRASS	0.150	0.024	200	UNPAVED	0.024	-	-	-	900	0	3.0	0.33	0.03	0.009	-	-	-	-	-	-	-	-	-	-	-	-
19		100	GRASS	0.150	0.07	-	-	-	-	-	-	980	0	3.0	0.20, 0.05	0.03	0.006	-	-	-	-	-	-	-	-	-	-	-	-
20		300	GRASS	0.150	0.050	-	-	-	-	-	-	350	10	3.0	0.07, 0.25	0.03	0.008	320	0	2.0	0.17, 0.33	0.03	0.005	-	-	-	-	-	-
50		60	GRASS	0.15	0.080	-	-	-	-	-	-	80	0	1.5	0.33, 0.20	0.03	0.004	-	-	-	-	-	-	-	-	-	-	-	-
161		300	GRASS	0.150	0.017	100	UNPAVED	0.050	-	-	-	150	0	1.0	0.10, 0.10	0.03	0.002	-	-	-	-	-	-	-	-	-	-	-	-
201		50	GRASS	0.150	0.03	-	-	-	-	-	-	290	0	2.0	0.33	0.03	0.0075	-	-	-	-	-	-	-	-	-	-	-	-

JT 5/5/00
RDE 5/5/00
17/27

2983

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				SHEET FLOW 2				CHANNEL FLOW 1						CHANNEL FLOW 2					
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)	Bottom Width (ft)	Flow Depth (ft)	Sideslopes (ft/ft)	Manning's n	Longitudinal Slope (ft/ft)	Flow Length (ft)	Bottom Width (ft)	Flow Depth (ft)	Sideslopes (ft/ft)	Manning's n	Longitudinal Slope (ft/ft)
100		75	SMOOTH	0.011	0.020	90	SMOOTH	0.011	0.190	590	0	3.0	0.33, 0.02	0.022	0.011	165	0	1.0	0.33, 0.25	0.03	0.015

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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	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
1A	Existing	Borrow Area	Vee	1.6	0.77%	0.030	0	6	4	R1									
1B	Existing	Borrow Area	Vee	2.3	0.67%	0.030	0	5	3	R1									
1C	Existing	Borrow Area	Vee	2	0.45%	0.030	0	4	2	R1									
1D	Existing	Borrow Area	Vee	2.8	0.81%	0.030	0	4	2	R1									
2A	Existing	Borrow Area	Vee	4	0.67%	0.030	0	6	3	R2									
2B	Existing	Borrow Area	Vee	3	1.00%	0.030	0	4	5	R2									
2C	Existing	Borrow Area	Vee	4	1.50%	0.030	0	4	5	R2									
4	Existing	Borrow Area	Vee	2	0.30%	0.030	0	16	3	R4									
5	New	Borrow Area	Trapezoidal	2	0.50%	0.030	8	3	3	R5									
6	Existing	Borrow Area	Trapezoidal	2	0.50%	0.030	10	3	3	R6									
33A	Existing	OSDF	Vee	5	0.45%	0.030	0	6	4	R33									
33B	Existing	OSDF	Vee	6.5	1.10%	0.030	0	5	3	R33									
35	Existing	OSDF	Vee	7.4	1.10%	0.030	0	4	3	R35									
37A	Existing	OSDF	Vee	1.5	1.50%	0.030	0	10	6	R37									
39	Existing	OSDF	Vee	3	1.50%	0.030	0	4	3	R39									
41	New	OSDF	Vee	2	0.75%	0.030	0	3	3	R41									
43	Existing	OSDF	Vee	3.5	0.70%	0.030	0	10	4	R43									
44	Existing	OSDF	Vee	2	2.00%	0.030	0	6	4	R44									
46	Existing	OSDF	Vee	6	0.10%	0.030	0	3	3	R800									
47	Existing	OSDF	Vee	2.5	1.00%	0.030	0	5	6	R47									
48	Existing	OSDF	Vee	2.5	1.00%	0.030	0	5	6	R48									
61	Existing	Borrow Area	Vee	3	1.50%	0.030	0	4	6	R61									
80	Existing	OSDF	Vee	2.5	0.90%	0.030	0	3	3	R700									
91	Existing	Borrow Area	Trapezoidal	2.5	0.30%	0.030	10	3	3	R91									
92	Existing	Borrow Area	Trapezoidal	2.5	0.30%	0.030	10	3	3	R92									
100	New	A	Vee	3	1.50%	0.030	0	4	3	R100									
107	New	OSDF	Trapezoidal	2	1.00%	0.030	8	3	3	R107									
352A	Existing	OSDF	Vee	7	1.30%	0.030	0	4	3	R352									
352B	Existing	OSDF	Vee	3	0.83%	0.030	0	3	4	R352									
451	Existing	OSDF	Vee	4	1.40%	0.030	0	3	3	R451									
452	Existing	OSDF	Vee	5	0.40%	0.030	0	3	3	R452									
453	Existing	OSDF	Vee	1.5	1.50%	0.030	0	6	6	R453									
161	Existing	Borrow Area	Irregular	1.5	0.20%	0.030	20	2	20	R161									
162A	New	Borrow Area	Vee	1.5	4.60%	0.030	0	5	40	R162									
162B	New	Borrow Area	Vee	1	2.30%	0.030	0	5	50	R162									
162C	New	Borrow Area	Vee	1	1.30%	0.030	0	10	40	R162									
162D	New	Borrow Area	Vee	2.3	1.20%	0.030	0	3	3	R162									
162E	New	Borrow Area	Vee	5	13.50%	0.030	0	3	3	R162									
163A	New	Borrow Area	Vee	4	0.70%	0.030	0	3	3	R163									
163B	New	Borrow Area	Vee	4	0.75%	0.030	0	3	3	R163									
201	New	Borrow Area	Vee	3.5	0.75%	0.030	0	3	3	R201									
202	New	Borrow Area	Vee	4	0.50%	0.030	0	3	3	R202									
17A	Existing	Borrow Area	Irregular	1.5	0.25%	0.030	40	3	20	R17									
17B	Existing	Borrow Area	Irregular	1.5	6.00%	0.030	0	10	40	R17									

1. Letters are used to further subdivide a nodal reach to take into consideration geometric variations along the reach.
2. Longitudinal slopes taken from the Drawings.

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19/27

5/3/00

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ATTACHMENT B-6

CULVERT LOCATIONS AND INPUT DATA

JH 5/5/00

GEOSYNTEC CONSULTANTS

2983 RDE 5/5/00 21/27

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) (2)	Design Scenario	Material - Type	Entrance Loss Coefficient (Ke)	Manning's n	Entrance Configuration	Number of Culverts - Diameter	Approximate Length (ft)	Inlet Invert Elevation (ft MSL)	Outlet Invert Elevation (ft MSL)	Slope (%)	Overtopping Elevation (ft MSL)	Entrance Configuration(4)	Entrance Loss Coefficient (Ke)	Number of Culverts - Diameter (4)	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)
1	Existing	Borrow Area	HDPE	0.2	0.01	Flared End Section	1 - 14 in.	80.0	609.60	609.00	0.75	612.0	Groove - Headwall	0.2	1 - 15 in.	R50												
2	Existing	Borrow Area	HDPE	0.2	0.01	Flared End Section	1 - 21 in.	50.0	603.80	603.30	1.00	607.6	Groove - Headwall	0.2	1 - 18 in.	R1												
3a	Existing	OSDF	RCP	0.2	0.013	Projecting	1 - 18 in.	66.5	589.75	589.07	1.02	595.3	Groove - Projecting	0.2	1 - 18 in.	R101												
3b	New	OSDF	CMP	0.9	0.024	Projecting	1 - 30 in.	65.0	589.75	589.00	1.15	595.3	Projecting	0.9	1 - 30 in.	R101					3.2	Off-Highway						
4	Existing	OSDF	CMP	0.9	0.024	Projecting (3)	1 - 24 in.	44.0	586.50	586.00	1.14	589.0	Projecting	0.9	1 - 24 in.	R104												
5	Existing	OSDF	CMP	0.9	0.024	Projecting	2 - 48 in.	114.0	585.30	583	2.02	593.5	Projecting	0.9	2 - 48 in.	P36												
6	Existing	OSDF	CMPA - Arch	0.5	0.018	Flared End Section	13 in. x 17 in.	70.0	587.6	586.6	1.43	590.5	Headwall	0.5	11 in. x 18 in.	R102												
7	Existing	Borrow Area	CMPA - Arch	0.5	0.018	Flared End Section	13 in. x 17 in.	80.0	587.35	586.37	1.23	590.0	Headwall	0.5	11 in. x 18 in.	R61												
8	Existing	OSDF	PVC	0.2	0.01	Concrete Headwall	2 - 15 in.	55.0	581.60	581.30	0.55	585.5	Groove - Headwall	0.2	2 - 15 in.	R106												
9	Existing	OSDF	P - Ellipse, Horizon	0.2	0.013	Concrete Headwall	34 in. x 53 in.	296.0	574.98	573.50	0.50	580.0	Groove - Headwall	0.2	34 in. x 53 in.	R108												
10	Existing	OSDF	CMP - Arch	0.2	0.024	Projecting (3)	13 in. x 17 in.	55.0	581.50	580.50	1.82	584.0	Groove - Projecting	0.2	11 in. x 18 in.	R110												
11a	Existing	Borrow Area	HDPE	0.2	0.01	Projecting	2 - 27 in.	40.0	583.28	583.10	0.45	587.1	Groove - Projecting	0.2	2 - 24 in.	R6												
11b	New	Borrow Area	RCP	0.2	0.013	Projecting	1 - 30 in.	49.0	583.28	583.00	0.57	587.1	Groove - Projecting	0.2	1 - 30 in.	R6					0.7	Off-Highway						
12	Existing	OSDF	RCP	0.2	0.013	Concrete Headwall	2 - 28 in.	30.0	572.12	572.00	0.40	577.0	Groove - Headwall	0.2	2 - 30 in.	R109												
13	Existing	Borrow Area	HDPE	0.2	0.01	Projecting	2 - 18 in.	80.0	583.80	583.50	0.38	587.0	Groove - Projecting	0.2	2 - 18 in.	S8												
14	Existing	Borrow Area	HDPE	0.2	0.01	Projecting	1 - 27 in.	80.0	583.70	583.00	0.88	587.5	Groove - Projecting	0.2	1 - 24 in.	S11												
15	Existing	OSDF	CMP - Arch	0.9	0.024	Projecting	50 in. x 31 in.	112.0	578.60	575.00	3.21	581.9	Projecting	0.9	50 in. x 31 in.	R113												
16	Existing	OSDF	CMP - Arch	0.5	0.024	Concrete Headwall	50 in. x 31 in.	75.0	572.00	569.20	3.73	576.5	Headwall	0.5	50 in. x 31 in.	R700												
17	Existing	OSDF	CMP - Arch	0.5	0.024	Concrete Headwall	72 in. x 44 in.	600.0	573.47	566.90	1.10	578.0	Headwall	0.5	72 in. x 44 in.	R900												
18	Existing	OSDF	RCP	0.2	0.013	Concrete Headwall	1 - 12 in.	116.0	579.00	577.00	1.72	581.5	Groove - Headwall	0.2	1 - 12 in.	R112												
19	New	OSDF	CMP	0.9	0.024	Projecting	2 - 24 in.	60.0	582.00	581.70	0.50	586.0	Projecting	0.9	2 - 24 in.	R43					3 (10)	Off-Highway						
20	Existing	OSDF	PVC	0.2	0.01	Concrete Headwall	2 - 18 in.	60.0	582.70	581.50	2.00	587.0	Groove - Headwall	0.2	2 - 18 in.	R105												
21	New	OSDF	CMP	0.9	0.024	Projecting	2 - 42 in.	84.0	583.20	583.00	0.24	590.0	Projecting	0.9	2 - 42 in.	P1					3.3 (11)	Off-Highway						
22	New	OSDF	CMP	0.9	0.024	Projecting	1 - 24 in.	80.5	583.20	582.70	0.62	589.4	Projecting	0.9	1 - 24 in.	R48					4.2 (11)	Off-Highway						
24	Existing	OSDF	CMP	0.9	0.024	Projecting	1 - 18 in.	38.0	586.39	586.11	0.74	590.0	Projecting	0.9	1 - 18 in.	R41												
25	New	A	CMP	0.9	0.024	Projecting	1 - 24 in.	36.7	586.66	586.11	1.50	591.0	Projecting	0.9	1 - 24 in.	R100					2.3	On-Highway						
26	Existing	OSDF	HDPE	0.9	0.01	Projecting	1 - 18 in.	55.0	583.90	583.60	0.55	585.0	N/A	0.9	N/A	No Flow												
27	New	Borrow	CMP	0.9	0.024	Projecting	1 - 30 in.	62.5	577.81	577.34	0.75	583.7	Projecting	0.9	1 - 36 in.	R15					3.0	Off-Highway						

- CMPA = Asphalted Corrugated Metal Pipe(s)
- CMP = Corrugated Metal Pipe(s)
- RCP = Reinforced Concrete Pipe(s)
- HDPE = High Density Polyethylene
- PVC = Polyvinyl Chloride
- S = Subcatchment
- R = Reach
- P = Pond
- N/A = Not Applicable
- Groove - Headwall = Groove end with headwall
- Groove - Projecting = Groove end projecting
- d_{50} = Average particle diameter

Notes

(1) Existing indicates an existing culvert to be left in place.
(2) New indicates a new culvert to be installed.
(3) Entrance configuration assumed.
(4) Dimensions and entrance configurations used in the Culvertmaster software package were selected to match existing culvert characteristics as closely as possible.
(7) Tailwater elevation represents headwater elevation of Culvert 21 connecting North and South ends of OSDF Basin 1.
(8) Corresponds to peak water level in OSDF Basin 1 for 25-year, 24-hour storm event.
(10) Culvert will be placed along an existing channel by others
(11) Culvert will be placed under existing road by others

2983

ATTACHMENT B-7

BASIN ELEVATION CONTOURS DATA

STAGE-STORAGE INPUT DATA FOR SEDIMENTATION BASINS

OSDF BASIN 1		BORROW AREA BASIN	
Elevation (ft MSL)	Planimetered Area (acres)	Elevation (ft MSL)	Planimetered Area (acres)
582.0	1.26	569.8	0.57
583.0	1.34	570.3	0.64
584.0	1.78	571.0	0.73
585.0	2.07	572.0	0.88
586.0	2.29	573.0	1.04
587.0	2.53	574.0	1.23
588.0	2.78	575.0	1.44
-	-	576.0	1.49
-	-	577.0	1.69
-	-	577.5	1.79

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BS 5/5/00
Rev 5/5/00
24/27

2983

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)	585.8
	Outlet Pipe	Twin 36" CMP		580.1
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)	6.5
SUBCATCHMENT		Total Upstream Drainage area	AREA (acres)	28.0

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JC 5/5/00
ADE 5/5/00
26/27

BORROW AREA BASIN ROUTING INPUT DATA

ITEM		BORROW AREA BASIN		
		DESCRIPTION	PARAMETER VALUE IN UNITS SHOWN	
PRINCIPAL SPILLWAY	Riser Pipe	Twin 48" CMP	ELEVATION (ft MSL)	575.3
	Outlet Pipe	Twin 48" CMP		569.8
EMERGENCY SPILLWAY		NONE		N/A
EMBANKMENT		Cut to existing ground surface		577.5
AVAILABLE STORAGE VOLUME (TO RISER INLET)		To primary spillway riser pipe inlet	VOLUME (acre-ft)	5.5
SUBCATCHMENT		Total Upstream Drainage area	AREA (acres)	130.0

N/A - Not Applicable

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AT 5/5/00
RDE 5/5/00
27/27

PHASE III SURFACE-WATER MANAGEMENT SYSTEM DESIGN

CALCULATIONS

INTRODUCTION

As described in the Procedures Section, the purpose of this calculation package is to design the OSDF and Borrow Area surface-water management (SWM) structures to be constructed as part of the Phase III 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 both design scenarios and the design case. For each design scenario (i.e., OSDF and Borrow Area Design Scenarios) and the Design Case A, a nodal network is prepared, for a total of three nodal networks. 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 Borrow Area Design Scenario and Design Case A nodal networks, HydroCAD™ runs are 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 and Borrow Area Design Scenarios and the Design Case A, respectively.

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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 basins 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).
- For the Borrow Area Basin: 25-year, 24-hour peak water elevations (ft MSL).

Channels

Hydraulic Capacity

For each new and existing 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 which includes minimum available freeboard and example calculations for the computations performed in the spreadsheets. For both new and existing 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 are less than 5 fps.

Culverts

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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. For all existing culverts, the available freeboard exceeds 0.5 ft, with two exceptions. For one of these cases, the calculated freeboard is 0.4 ft. For the remaining case, the road which the culvert crosses is expected to overtop. This road is subject only to site traffic (i.e., no public access), is not used for access to the OSDF, and overtopping of the road will result in no impact to the OSDF structure. It is recommended that no modifications be made to these culverts.

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. Downstream riprap length for the outlet of culverts 3b, 11b, 19, 22, 25, and 27 are 12 ft, 10 ft, 8 ft, 8 ft, 8 ft, and 12 ft, respectively. Upstream riprap length for the inlet of culverts 3b, 11b, 19, 22, 25, and 27 are 5 ft, 5 ft, 4 ft, 4 ft, 4 ft, and 5 ft, respectively. No riprap protection is recommended for culvert 21.

Structural Stability

For each new CMP culvert, minimum required and available cover are tabulated and presented in Attachment C-4B. For all new CMP culverts, the available cover exceeds the minimum required. For the one RCP culvert, a Class IV Wall B reinforced concrete pipe is selected to provide adequate structural stability. Calculations are provided in Attachment C-4B.

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Sedimentation Basins

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 volume. 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.

Borrow Area Sedimentation Basin

The minimum required storage volume based on the total upstream drainage area is calculated (an example calculation is provided in Attachment C-5). The available storage volume exceeds the minimum required storage volume. Analysis results are tabulated and presented in Attachment C-5.

The peak water elevation for the 25-year storm event is obtained from HydroCAD™ output. The peak water elevation for the 25-year storm event 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-1A

HydroCAD™ OUTPUT REPORTS

OSDF DESIGN SCENARIO

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

JK 5/5/00
ROE 5/5/
7/111

2983

POND 1 **OSDF Basin 1**

Qin = 68.02 CFS @ 12.01 HRS, VOLUME= 6.29 AF
Qout= 0.00 CFS @ 0.00 HRS, VOLUME= 0.00 AF, ATTEN=100%, LAG= 0.0 MIN

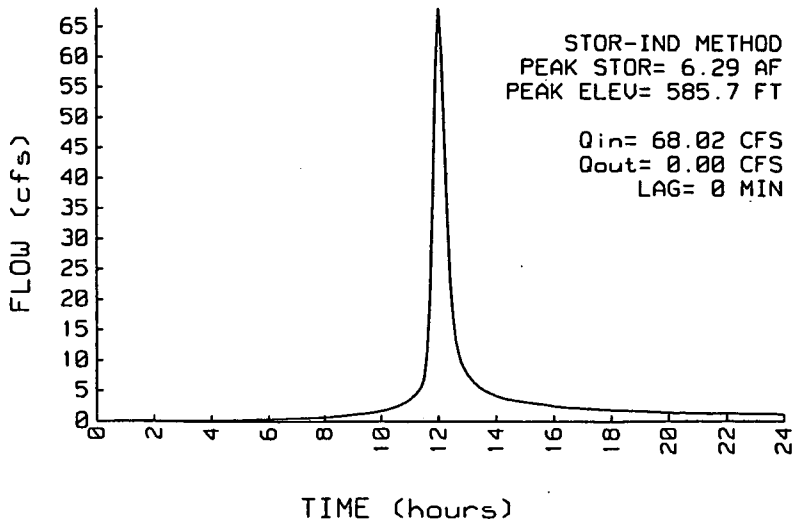
ELEVATION (FT)	CON.AREA (AC)	INC.STOR (AF)	CUM.STOR (AF)	WET.AREA (AC)
582.0	1.26	0.00	0.00	1.26
583.0	1.34	1.30	1.30	1.34
584.0	1.78	1.55	2.85	1.78
585.0	2.07	1.92	4.78	2.07
586.0	2.29	2.18	6.96	2.30
587.0	2.53	2.41	9.37	2.54
588.0	2.78	2.65	12.02	2.79

STOR-IND METHOD
PEAK STORAGE = 6.29 AF
PEAK ELEVATION= 585.7 FT
FLOOD ELEVATION= 588.0 FT
START ELEVATION= 582.0 FT
SPAN= 0-24 HRS, dt=.1 HRS

#	ROUTE	INVERT	OUTLET DEVICES
1	2	585.8'	**48" HORIZONTAL ORIFICE/GRATE X 2** Q=.6 Area SQR(2gH) (Limited to weir flow @ low head)
2	P	580.1'	**36" CULVERT X 2** n=.024 L=61' S=.0187'/' Ke=.7 Cc=.9 Cd=.53
3	P	586.5'	**30' BROAD-CRESTED RECTANGULAR WEIR** Q=C L H^1.5 C=3, 3, 3, 3, 3, 3, 3, 3

Primary Discharge
├──2=Culvert
│ └──1=Orifice/Grate
└──3=Broad-Crested Rectangular Weir

POND 1 INFLOW & OUTFLOW
OSDF Basin 1



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

JI 5/5/00
COE/SPK
9/111

2983

RUNOFF BY SCS TR-20 METHOD: TYPE II 24-HOUR RAINFALL= 4.70 IN, SCS U.H.

RUNOFF SPAN = 0-24 HRS, dt= .10 HRS, 241 POINTS

SUBCAT NUMBER	AREA (ACRE)	Tc (MIN)	--GROUND COVERS (%CN)--	WGT'D CN	C	PEAK (CFS)	Tpeak (HRS)	VOL (AF)
10	.95	11.9	100%79	79	-	3.33	11.99	.20
31	4.60	3.3	100%94	94	-	29.40	11.88	1.54
32	3.72	4.8	100%94	94	-	23.21	11.89	1.24
33	1.55	7.8	100%75	75	-	5.03	11.94	.29
35	2.35	7.5	100%82	82	-	9.84	11.92	.55
36	4.26	12.2	100%80	80	-	15.32	11.99	.93
37	1.85	24.7	100%82	82	-	5.04	12.13	.43
38	4.64	27.7	100%82	82	-	11.72	12.17	1.08
39	4.33	29.0	100%82	82	-	10.67	12.19	1.01
40	3.03	27.8	100%82	82	-	7.64	12.17	.71
41	1.45	12.7	100%82	82	-	5.48	12.00	.34
42	2.29	1.0	100%98	98	-	14.08	11.83	.85
43	.81	15.0	100%82	82	-	2.84	12.02	.19
44	1.04	31.6	100%82	82	-	2.43	12.22	.24
45	3.58	18.3	100%82	82	-	11.28	12.06	.84
46	2.83	20.0	100%82	82	-	8.62	12.08	.66
47	1.64	8.5	100%82	82	-	6.57	11.94	.38
48	1.94	6.0	100%82	82	-	8.77	11.91	.45
60	1.30	10.6	100%79	79	-	4.68	11.98	.28
70	1.47	7.5	100%79	79	-	5.59	11.93	.31
80	.80	10.6	100%79	79	-	2.88	11.98	.17
361	.11	1.0	100%98	98	-	.68	11.83	.04
450	1.37	13.3	100%82	82	-	5.08	12.00	.32

JI 5/5/00
RDE 5/5/0
10/111

2983

REACH ROUTING BY STOR-IND+TRANS METHOD

REACH NO.	DIAM (IN)	BOTTOM WIDTH (FT)	DEPTH (FT)	SIDE SLOPES (FT/FT)		n	LENGTH (FT)	SLOPE (FT/FT)	PEAK VEL. (FPS)	TRAVEL TIME (MIN)	PEAK Qout (CFS)
10	-	-	3.0	.33	.33	.030	5	.0110	2.5	0.0	3.33
32	-	-	3.0	.17	.33	.030	800	.0050	2.8	4.8	22.92
33	-	-	6.0	.17	.33	.030	400	.0090	3.9	1.7	37.02
35	-	-	7.0	.33	.33	.030	270	.0110	4.7	1.0	40.71
37	-	-	1.5	.17	.10	.030	5	.0100	2.1	0.0	5.03
39	-	-	3.0	.33	.33	.030	230	.0150	3.1	1.2	4.83
41	-	-	2.0	.33	.33	.030	5	.0080	2.4	0.0	5.46
43	-	-	3.5	.25	.10	.030	200	.0070	2.5	1.3	15.14
44	-	-	2.0	.17	.25	.030	180	.0200	4.4	.7	21.75
46	-	-	6.0	.33	.33	.030	390	.0010	1.9	3.5	40.52
47	-	-	2.5	.17	.20	.030	5	.0100	2.5	0.0	6.55
48	-	-	2.5	.17	.20	.030	5	.0100	2.6	0.0	8.74
80	-	-	3.0	.33	.33	.030	180	.0090	2.3	1.3	2.99
101	72.0	-	-	-	-	.024	65	.0120	6.8	.2	46.95
102	72.0	-	-	-	-	.024	70	.0140	3.9	.3	4.98
104	72.0	-	-	-	-	.024	44	.0182	5.5	.1	11.68
105	72.0x2	-	-	-	-	.010	60	.0200	11.0	.1	15.49
106	72.0x2	-	-	-	-	.010	55	.0060	6.1	.2	7.47
107	-	8.0	2.0	.33	.33	.030	63	.0100	3.4	.3	24.10
108	72.0x2	-	-	-	-	.013	296	.0050	5.5	.9	25.15
109	72.0x2	-	-	-	-	.013	30	.0040	5.7	.1	42.41
110	72.0	-	-	-	-	.024	55	.0180	4.4	.2	5.45
112	72.0	-	-	-	-	.013	116	.0172	7.9	.2	9.84
113	72.0	-	-	-	-	.024	112	.0320	5.8	.3	3.26
352	-	-	5.0	.33	.33	.030	250	.0130	4.9	.8	39.56

000175

451	-	-	4.0	.33	.33	.030	240	.0140	4.5	.9	25.12
452	-	-	5.0	.33	.33	.030	210	.0140	4.7	.8	31.22
453	-	-	1.5	.17	.17	.030	140	.0150	3.2	.7	9.96
700	72.0	-	-	-	-	.024	75	.0370	6.3	.2	5.68
800	72.0	-	-	-	-	.024	5	.0110	6.6	0.0	47.30
900	72.0	-	-	-	-	.024	600	.0110	6.8	1.5	49.79

JI 5/5/00
ROE 5/5/
12/111

POND ROUTING BY STOR-IND METHOD

POND NO.	START ELEV. (FT)	FLOOD ELEV. (FT)	PEAK ELEV. (FT)	PEAK STORAGE (AF)	PEAK FLOW Qin (CFS)	Qout (CFS)	Qpri (CFS)	Qsec (CFS)	Qout ATTEN. (%)	Qout LAG (MIN)
1	582.0	588.0	585.9	6.68	82.88	1.90			98	429.3
36	586.0	589.0	588.2	.02	62.32	62.24			0	.2
49	585.0	587.0	586.7	.13	6.55	1.83			72	13.1

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JJ 5/5/00
RMD 5/5/
13/111

LINK NO.	NAME	SOURCE	Qout (CFS)
1		REACH 61 from BORROW AREA DESIGN SCENARIO	5.25

JT 5/5/00
RDO 5/9
14/111

2983

SUBCATCHMENT 10

PEAK= 3.33 CFS @ 11.99 HRS, VOLUME= .20 AF

ACRES	CN
.95	79

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	10.9
n=.15 L=50' P2=2.6 in s=.005 '/'		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	1.0
D=3' SS= .33 '/' a=27.27 sq-ft Pw=19.1' r=1.424'		
s=.011 '/' n=.03 V=6.58 fps L=400' Capacity=179.4 cfs		
	Total Length= 450 ft Total Tc=	11.9

SUBCATCHMENT 31

PEAK= 29.40 CFS @ 11.88 HRS, VOLUME= 1.54 AF

ACRES	CN
4.60	94

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	.9
n=.011 L=90' P2=2.6 in s=.05 '/'		
TR-55 SHEET FLOW	**Segment ID:**	.5
n=.011 L=60' P2=2.6 in s=.1 '/'		
TR-55 SHEET FLOW	**Segment ID:**	.8
n=.011 L=150' P2=2.6 in s=.17 '/'		
SHALLOW CONCENTRATED/UPLAND FLOW	**Segment ID:**	.5
Unpaved Kv=16.1345 L=180' s=.17 '/' V=6.65 fps		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	.6
D=1.5' SS= .17 & .33 '/' a=10.03 sq-ft Pw=13.7' r=.73'		
s=.008 '/' n=.03 V=3.59 fps L=120' Capacity=36 cfs		
	Total Length= 600 ft Total Tc=	3.3

JT 5/5/00
RDE 5/5
15/111

2983

SUBCATCHMENT 32

PEAK= 23.21 CFS @ 11.89 HRS, VOLUME= 1.24 AF

ACRES	CN
3.72	94

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	.9
n=.011 L=150' P2=2.6 in s=.13 '/'		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	3.9
D=2' SS= .33 & .17 '/' a=17.83 sq-ft Pw=18.3' r=.973'		
s=.005 '/' n=.03 V=3.44 fps L=800' Capacity=61.3 cfs		
	Total Length= 950 ft Total Tc=	4.8

SUBCATCHMENT 33

PEAK= 5.03 CFS @ 11.94 HRS, VOLUME= .29 AF

ACRES	CN
1.55	75

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	6.4
n=.15 L=140' P2=2.6 in s=.15 '/'		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	1.4
D=2' SS= .33 & .17 '/' a=17.83 sq-ft Pw=18.3' r=.973'		
s=.009 '/' n=.03 V=4.61 fps L=380' Capacity=82.3 cfs		
	Total Length= 520 ft Total Tc=	7.8

JC 5/5/00
KDD 5/5/
16/111

2983

SUBCATCHMENT 35

PEAK= 9.84 CFS @ 11.92 HRS, VOLUME= .55 AF

ACRES	CN
2.35	82

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	6.3
n=.15 L=130' P2=2.6 in s=.13 '/'		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	.3
D=2' SS= .33 '/' a=12.12 sq-ft Pw=12.8' r=.95'		
s=.01 '/' n=.03 V=4.79 fps L=100' Capacity=58 cfs		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	.9
D=2' SS= .33 '/' a=12.12 sq-ft Pw=12.8' r=.95'		
s=.01 '/' n=.03 V=4.79 fps L=250' Capacity=58 cfs		

	Total Length= 480 ft Total Tc=	7.5

SUBCATCHMENT 36

PEAK= 15.32 CFS @ 11.99 HRS, VOLUME= .93 AF

ACRES	CN
4.26	80

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	10.9
n=.15 L=200' P2=2.6 in s=.08 '/'		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	1.3
D=1' SS= .2 '/' a=5 sq-ft Pw=10.2' r=.49'		
s=.02 '/' n=.03 V=4.36 fps L=330' Capacity=21.8 cfs		

	Total Length= 530 ft Total Tc=	12.2

JC 5/5/00
5/8/00
17/111

2983

SUBCATCHMENT 37

PEAK= 5.04 CFS @ 12.13 HRS, VOLUME= .43 AF

ACRES	CN
1.85	82

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	24.1
n=.15 L=250' P2=2.6 in s=.017 '/'		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	.6
D=1' SS= .17 & .1 '/' a=7.94 sq-ft Pw=16' r=.496'		
s=.01 '/' n=.03 V=3.1 fps L=120' Capacity=24.6 cfs		
	Total Length= 370 ft Total Tc=	24.7

SUBCATCHMENT 38

PEAK= 11.72 CFS @ 12.17 HRS, VOLUME= 1.08 AF

ACRES	CN
4.64	82

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	26.2
n=.15 L=300' P2=2.6 in s=.02 '/'		
SHALLOW CONCENTRATED/UPLAND FLOW	**Segment ID:**	1.1
Unpaved Kv=16.1345 L=150' s=.02 '/' V=2.28 fps		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	.4
D=2' SS= .1 & .01 '/' a=220 sq-ft Pw=220.1' r=1'		
s=.018 '/' n=.03 V=6.64 fps L=170' Capacity=1461.5 cfs		
	Total Length= 620 ft Total Tc=	27.7

JI 5/5/00
ROD SP
18/111

2983

SUBCATCHMENT 39

PEAK= 10.67 CFS @ 12.19 HRS, VOLUME= 1.01 AF

ACRES	CN
4.33	82

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	24.8
n=.15 L=300' P2=2.6 in s=.023 '/'		
SHALLOW CONCENTRATED/UPLAND FLOW	**Segment ID:**	2.8
Unpaved Kv=16.1345 L=300' s=.012 '/' V=1.77 fps		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	1.4
D=3' SS= .33 '/' a=27.27 sq-ft Pw=19.1' r=1.424'		
s=.01 '/' n=.03 V=6.27 fps L=520' Capacity=171 cfs		

	Total Length= 1120 ft Total Tc=	29.0

SUBCATCHMENT 40

PEAK= 7.64 CFS @ 12.17 HRS, VOLUME= .71 AF

ACRES	CN
3.03	82

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	26.2
n=.15 L=300' P2=2.6 in s=.02 '/'		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	1.6
D=1' SS= .1 '/' a=10 sq-ft Pw=20.1' r=.498'		
s=.01 '/' n=.03 V=3.11 fps L=300' Capacity=31.1 cfs		

	Total Length= 600 ft Total Tc=	27.8

JC 5/5/00
RDE 5/8/
19/111

2983

SUBCATCHMENT 41

PEAK= 5.48 CFS @ 12.00 HRS, VOLUME= .34 AF

ACRES	CN
1.45	82

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	11.4
n=.15 L=150' P2=2.6 in s=.04 '/'		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	1.3
D=2' SS= .33 '/' a=12.12 sq-ft Pw=12.8' r=.95'		
s=.008 '/' n=.03 V=4.28 fps L=330' Capacity=51.9 cfs		

	Total Length= 480 ft Total Tc=	12.7

SUBCATCHMENT 42

PEAK= 14.08 CFS @ 11.83 HRS, VOLUME= .85 AF

ACRES	CN
2.29	98

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
DIRECT ENTRY	**Segment ID:**	1.0

SUBCATCHMENT 43

PEAK= 2.84 CFS @ 12.02 HRS, VOLUME= .19 AF

ACRES	CN
.81	82

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	14.2
n=.15 L=140' P2=2.6 in s=.02 '/'		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	.8
D=2' SS= .25 & .1 '/' a=28 sq-ft Pw=28.3' r=.988'		
s=.007 '/' n=.03 V=4.11 fps L=200' Capacity=115.1 cfs		

	Total Length= 340 ft Total Tc=	15.0

JC 7/5/00
RDE 5/5/00
20/111

2983

SUBCATCHMENT 44

PEAK= 2.43 CFS @ 12.22 HRS, VOLUME= .24 AF

ACRES	CN
1.04	82

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	31.1
n=.15 L=300' P2=2.6 in s=.013 '/'		
SHALLOW CONCENTRATED/UPLAND FLOW	**Segment ID:**	.3
Unpaved Kv=16.1345 L=40' s=.02 '/' V=2.28 fps		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	.2
W=8' D=2' SS= .33 '/' a=28.12 sq-ft Pw=20.8' r=1.354'		
s=.01 '/' n=.03 V=6.06 fps L=63' Capacity=170.5 cfs		

	Total Length= 403 ft Total Tc=	31.6

SUBCATCHMENT 45

PEAK= 11.28 CFS @ 12.06 HRS, VOLUME= .84 AF

ACRES	CN
3.58	82

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	17.0
n=.15 L=210' P2=2.6 in s=.029 '/'		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	1.3
D=2' SS= .33 '/' a=12.12 sq-ft Pw=12.8' r=.95'		
s=.014 '/' n=.03 V=5.66 fps L=450' Capacity=68.6 cfs		

	Total Length= 660 ft Total Tc=	18.3

JT 3/5/00
BDS 3/5/00
21/111

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SUBCATCHMENT 46

PEAK= 8.62 CFS @ 12.08 HRS, VOLUME= .66 AF

ACRES	CN
2.83	82

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	15.9
n=.15 L=180' P2=2.6 in s=.025 '/'		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	.2
D=2' SS= .2 '/' a=20 sq-ft Pw=20.4' r=.981'		
s=.08 '/' n=.03 V=13.83 fps L=150' Capacity=276.6 cfs		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	3.9
D=2' SS= .33 '/' a=12.12 sq-ft Pw=12.8' r=.95'		
s=.001 '/' n=.03 V=1.51 fps L=350' Capacity=18.3 cfs		
	Total Length= 680 ft Total Tc=	20.0

SUBCATCHMENT 47

PEAK= 6.57 CFS @ 11.94 HRS, VOLUME= .38 AF

ACRES	CN
1.64	82

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	6.4
n=.15 L=150' P2=2.6 in s=.17 '/'		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	2.1
D=2' SS= .17 & .2 '/' a=21.76 sq-ft Pw=22.1' r=.983'		
s=.01 '/' n=.03 V=4.9 fps L=630' Capacity=106.6 cfs		
	Total Length= 780 ft Total Tc=	8.5

Data for OSDF DESIGN SCENARIO
TYPE II 24-HOUR RAINFALL= 4.70 IN
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6 Apr 00

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SUBCATCHMENT 48

PEAK= 8.77 CFS @ 11.91 HRS, VOLUME= .45 AF

ACRES	CN
1.94	82

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	2.5
n=.15 L=50' P2=2.6 in s=.2 '/'		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	3.5
D=2' SS= .17 & .2 '/' a=21.76 sq-ft Pw=22.1' r=.983'		
s=.01 '/' n=.03 V=4.9 fps L=1030' Capacity=106.6 cfs		
	Total Length= 1080 ft Total Tc=	6.0

SUBCATCHMENT 60

PEAK= 4.68 CFS @ 11.98 HRS, VOLUME= .28 AF

ACRES	CN
1.30	79

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	10.2
n=.15 L=140' P2=2.6 in s=.046 '/'		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	.4
D=2' SS= .25 & .17 '/' a=19.76 sq-ft Pw=20.2' r=.979'		
s=.015 '/' n=.03 V=5.98 fps L=130' Capacity=118.3 cfs		
	Total Length= 270 ft Total Tc=	10.6

SUBCATCHMENT 70

PEAK= 5.59 CFS @ 11.93 HRS, VOLUME= .31 AF

ACRES	CN
1.47	79

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	7.3
n=.15 L=90' P2=2.6 in s=.044 '/'		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	
D=3' SS= .2 & .11 '/' a=63.41 sq-ft Pw=42.7' r=1.484'		
s=.007 '/' n=.03 V=5.39 fps L=80' Capacity=341.9 cfs		
	Total Length= 170 ft Total Tc=	7.5

Data for OSDF DESIGN SCENARIO
TYPE II 24-HOUR RAINFALL= 4.70 IN
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SUBCATCHMENT 80

PEAK= 2.88 CFS @ 11.98 HRS, VOLUME= .17 AF

ACRES	CN
.80	79

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	9.9
n=.15 L=100' P2=2.6 in s=.025 '/'		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	.7
D=2' SS= .33 '/' a=12.12 sq-ft Pw=12.8' r=.95'		
s=.009 '/' n=.03 V=4.54 fps L=180' Capacity=55 cfs		
	Total Length= 280 ft Total Tc=	10.6

SUBCATCHMENT 361

PEAK= .68 CFS @ 11.83 HRS, VOLUME= .04 AF

ACRES	CN
.11	98

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
DIRECT ENTRY	**Segment ID:**	1.0

SUBCATCHMENT 450

PEAK= 5.08 CFS @ 12.00 HRS, VOLUME= .32 AF

ACRES	CN
1.37	82

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	12.2
n=.15 L=100' P2=2.6 in s=.015 '/'		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	1.1
D=1' SS= .17 '/' a=5.88 sq-ft Pw=11.9' r=.493'		
s=.015 '/' n=.03 V=3.79 fps L=250' Capacity=22.3 cfs		
	Total Length= 350 ft Total Tc=	13.3

JT 5/5/00
RPG 5/5/00
24/111

REACH 10

2983

Qin = 3.33 CFS @ 11.99 HRS, VOLUME= .20 AF
Qout= 3.33 CFS @ 11.99 HRS, VOLUME= .20 AF, ATTEN= 0%, LAG= 0.0 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.30	.27	.39
.60	1.09	2.45
.90	2.45	7.23
1.29	5.04	18.89
1.80	9.82	45.94
2.40	17.45	98.93
3.00	27.27	179.37

0' x 3' CHANNEL
SIDE SLOPE= .33 '/'
n= .03
LENGTH= 5 FT
SLOPE= .011 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .65 FT
PEAK VELOCITY= 2.5 FPS
TRAVEL TIME = 0.0 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 32 **East sedimentation ditch**

Qin = 29.40 CFS @ 11.88 HRS, VOLUME= 1.54 AF
Qout= 22.92 CFS @ 12.00 HRS, VOLUME= 1.53 AF, ATTEN= 22%, LAG= 7.5 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.30	.40	.39
.60	1.60	2.47
.90	3.61	7.29
1.29	7.42	19.04
1.80	14.44	46.30
2.40	25.67	99.70
3.00	40.11	180.78

0' x 3' CHANNEL
S/S= .17 & .33 '/'
n= .03
LENGTH= 800 FT
SLOPE= .005 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= 1.38 FT
PEAK VELOCITY= 2.8 FPS
TRAVEL TIME = 4.8 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 33 **East sedimentation ditch**

Qin = 40.35 CFS @ 11.92 HRS, VOLUME= 2.78 AF
Qout= 37.02 CFS @ 11.98 HRS, VOLUME= 2.77 AF, ATTEN= 8%, LAG= 3.7 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.0	0.00
.60	1.6	3.32
1.20	6.4	21.07
1.80	14.4	62.11
2.58	29.7	162.22
3.60	57.8	394.39
4.80	102.7	849.37
6.00	160.4	1540.01

0' x 6' CHANNEL
S/S= .17 & .33 '/'
n= .03
LENGTH= 400 FT
SLOPE= .009 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= 1.43 FT
PEAK VELOCITY= 3.9 FPS
TRAVEL TIME = 1.7 MIN
SPAN= 0-24 HRS, dt=.1 HRS

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RDS 5/5/00
25/111

2983

REACH 35 Southeast sedimentation ditch

Qin = 41.73 CFS @ 11.98 HRS, VOLUME= 3.06 AF
Qout= 40.71 CFS @ 12.00 HRS, VOLUME= 3.06 AF, ATTEN= 2%, LAG= 1.6 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.0	0.00
.70	1.5	3.70
1.40	5.9	23.50
2.10	13.4	69.29
3.01	27.5	180.97
4.20	53.5	439.97
5.60	95.0	947.54
7.00	148.5	1718.00

0' x 7' CHANNEL
SIDE SLOPE= .33 '/'
n= .03
LENGTH= 270 FT
SLOPE= .011 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= 1.68 FT
PEAK VELOCITY= 4.7 FPS
TRAVEL TIME = 1.0 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 37

Qin = 5.04 CFS @ 12.13 HRS, VOLUME= .43 AF
Qout= 5.03 CFS @ 12.13 HRS, VOLUME= .43 AF, ATTEN= 0%, LAG= .1 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.15	.18	.16
.30	.71	.99
.45	1.61	2.93
.65	3.30	7.65
.90	6.43	18.61
1.20	11.44	40.07
1.50	17.87	72.65

0' x 1.5' CHANNEL
S/S= .17 & .1 '/'
n= .03
LENGTH= 5 FT
SLOPE= .01 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .53 FT
PEAK VELOCITY= 2.1 FPS
TRAVEL TIME = 0.0 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 39

Qin = 4.98 CFS @ 12.14 HRS, VOLUME= .43 AF
Qout= 4.83 CFS @ 12.19 HRS, VOLUME= .43 AF, ATTEN= 3%, LAG= 2.5 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.30	.27	.45
.60	1.09	2.87
.90	2.45	8.45
1.29	5.04	22.06
1.80	9.82	53.64
2.40	17.45	115.52
3.00	27.27	209.46

0' x 3' CHANNEL
SIDE SLOPE= .33 '/'
n= .03
LENGTH= 230 FT
SLOPE= .015 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .71 FT
PEAK VELOCITY= 3.1 FPS
TRAVEL TIME = 1.2 MIN
SPAN= 0-24 HRS, dt=.1 HRS

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RDE 5/8/00
26/111

2983

REACH 41 **Ditch to S-Basin 1**

Qin = 5.48 CFS @ 12.00 HRS, VOLUME= .34 AF
Qout= 5.46 CFS @ 12.00 HRS, VOLUME= .34 AF, ATTEN= 0%, LAG= 0.0 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.20	.12	.11
.40	.48	.71
.60	1.09	2.09
.86	2.24	5.47
1.20	4.36	13.29
1.60	7.76	28.62
2.00	12.12	51.88

0' x 2' CHANNEL
SIDE SLOPE= .33 '/'
n= .03
LENGTH= 5 FT
SLOPE= .008 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .86 FT
PEAK VELOCITY= 2.4 FPS
TRAVEL TIME = 0.0 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 43

Qin = 15.49 CFS @ 12.19 HRS, VOLUME= 1.44 AF
Qout= 15.14 CFS @ 12.23 HRS, VOLUME= 1.44 AF, ATTEN= 2%, LAG= 2.3 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.35	.86	1.10
.70	3.43	7.00
1.05	7.72	20.64
1.51	15.86	53.92
2.10	30.87	131.09
2.80	54.88	282.31
3.50	85.75	511.86

0' x 3.5' CHANNEL
S/S= .25 & .1 '/'
n= .03
LENGTH= 200 FT
SLOPE= .007 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .92 FT
PEAK VELOCITY= 2.5 FPS
TRAVEL TIME = 1.3 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 44

Qin = 21.93 CFS @ 12.19 HRS, VOLUME= 4.15 AF
Qout= 21.75 CFS @ 12.21 HRS, VOLUME= 4.14 AF, ATTEN= 1%, LAG= 1.2 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.20	.20	.29
.40	.79	1.87
.60	1.78	5.51
.86	3.65	14.38
1.20	7.12	34.97
1.60	12.65	75.31
2.00	19.76	136.55

0' x 2' CHANNEL
S/S= .17 & .25 '/'
n= .03
LENGTH= 180 FT
SLOPE= .02 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .98 FT
PEAK VELOCITY= 4.4 FPS
TRAVEL TIME = .7 MIN
SPAN= 0-24 HRS, dt=.1 HRS

JZ 5/5/00
RDE 5/5/00
27/111

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REACH 46

Qin = 42.41 CFS @ 12.08 HRS, VOLUME= 7.19 AF
Qout= 40.52 CFS @ 12.20 HRS, VOLUME= 7.15 AF, ATTEN= 4%, LAG= 7.4 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.0	0.00
.60	1.1	.74
1.20	4.4	4.70
1.80	9.8	13.85
2.58	20.2	36.17
3.60	39.3	87.94
4.80	69.8	189.40
6.00	109.1	343.40

0' x 6' CHANNEL
SIDE SLOPE= .33 '/'
n= .03
LENGTH= 390 FT
SLOPE= .001 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= 2.69 FT
PEAK VELOCITY= 1.9 FPS
TRAVEL TIME = 3.5 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 47

Qin = 6.57 CFS @ 11.94 HRS, VOLUME= .38 AF
Qout= 6.55 CFS @ 11.94 HRS, VOLUME= .38 AF, ATTEN= 0%, LAG= .1 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.25	.34	.42
.50	1.36	2.64
.75	3.06	7.80
1.08	6.29	20.36
1.50	12.24	49.50
2.00	21.76	106.61
2.50	34.01	193.30

0' x 2.5' CHANNEL
S/S= .17 & .2 '/'
n= .03
LENGTH= 5 FT
SLOPE= .01 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .68 FT
PEAK VELOCITY= 2.5 FPS
TRAVEL TIME = 0.0 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 48

Qin = 8.77 CFS @ 11.91 HRS, VOLUME= .45 AF
Qout= 8.74 CFS @ 11.91 HRS, VOLUME= .45 AF, ATTEN= 0%, LAG= 0.0 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.25	.34	.42
.50	1.36	2.64
.75	3.06	7.80
1.08	6.29	20.36
1.50	12.24	49.50
2.00	21.76	106.61
2.50	34.01	193.30

0' x 2.5' CHANNEL
S/S= .17 & .2 '/'
n= .03
LENGTH= 5 FT
SLOPE= .01 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .77 FT
PEAK VELOCITY= 2.6 FPS
TRAVEL TIME = 0.0 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 80

Qin = 3.26 CFS @ 12.00 HRS, VOLUME= .20 AF
Qout= 2.99 CFS @ 12.03 HRS, VOLUME= .20 AF, ATTEN= 8%, LAG= 2.0 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.30	.27	.35
.60	1.09	2.22
.90	2.45	6.54
1.29	5.04	17.09
1.80	9.82	41.55
2.40	17.45	89.49
3.00	27.27	162.25

0' x 3' CHANNEL
SIDE SLOPE= .33 '/'
n= .03
LENGTH= 180 FT
SLOPE= .009 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .67 FT
PEAK VELOCITY= 2.3 FPS
TRAVEL TIME = 1.3 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 101 **Culvert under scraper road**

Qin = 47.24 CFS @ 12.00 HRS, VOLUME= 3.61 AF
Qout= 46.95 CFS @ 12.01 HRS, VOLUME= 3.61 AF, ATTEN= 1%, LAG= .2 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.60	1.47	5.25
1.20	4.03	22.01
1.80	7.13	49.21
4.20	21.14	210.39
4.80	24.25	245.63
5.40	26.80	267.83
5.64	27.58	270.31
5.82	28.03	267.82
6.00	28.27	251.30

72" PIPE
n= .024
LENGTH= 65 FT
SLOPE= .012 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= 1.76 FT
PEAK VELOCITY= 6.8 FPS
TRAVEL TIME = .2 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 102 **11" X 18" ASPHALT LINED CMP**

Qin = 5.03 CFS @ 12.13 HRS, VOLUME= .43 AF
Qout= 4.98 CFS @ 12.14 HRS, VOLUME= .43 AF, ATTEN= 1%, LAG= .6 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.60	1.47	5.67
1.20	4.03	23.77
1.80	7.13	53.15
4.20	21.14	227.25
4.80	24.25	265.31
5.40	26.80	289.29
5.64	27.58	291.97
5.82	28.03	289.28
6.00	28.27	271.43

72" PIPE
n= .024
LENGTH= 70 FT
SLOPE= .014 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .52 FT
PEAK VELOCITY= 3.9 FPS
TRAVEL TIME = .3 MIN
SPAN= 0-24 HRS, dt=.1 HRS

JR 5/5/00
RDE 5/8/00
29/111

2983

REACH 104 **24" CMP**

Qin = 11.72 CFS @ 12.17 HRS, VOLUME= 1.08 AF
Qout= 11.68 CFS @ 12.17 HRS, VOLUME= 1.08 AF, ATTEN= 0%, LAG= .2 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.60	1.47	6.46
1.20	4.03	27.10
1.80	7.13	60.61
4.20	21.14	259.11
4.80	24.25	302.50
5.40	26.80	329.84
5.64	27.58	332.90
5.82	28.03	329.82
6.00	28.27	309.48

72" PIPE
n= .024
LENGTH= 44 FT
SLOPE= .0182 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .75 FT
PEAK VELOCITY= 5.5 FPS
TRAVEL TIME = .1 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 105 **2-18" PVC**

Qin = 15.51 CFS @ 12.19 HRS, VOLUME= 1.44 AF
Qout= 15.49 CFS @ 12.19 HRS, VOLUME= 1.44 AF, ATTEN= 0%, LAG= .2 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.60	2.94	32.51
1.20	8.05	136.37
1.80	14.27	304.95
4.20	42.28	1303.76
4.80	48.50	1522.13
5.40	53.61	1659.68
5.64	55.16	1675.07
5.82	56.05	1659.60
6.00	56.55	1557.22

72" PIPE X 2
n= .01
LENGTH= 60 FT
SLOPE= .02 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .29 FT
PEAK VELOCITY= 11.0 FPS
TRAVEL TIME = .1 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 106 **2-15" PVC**

Qin = 7.51 CFS @ 12.00 HRS, VOLUME= 2.52 AF
Qout= 7.47 CFS @ 12.01 HRS, VOLUME= 2.52 AF, ATTEN= 0%, LAG= .3 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.60	2.94	17.81
1.20	8.05	74.69
1.80	14.27	167.03
4.20	42.28	714.10
4.80	48.50	833.71
5.40	53.61	909.05
5.64	55.16	917.47
5.82	56.05	909.00
6.00	56.55	852.93

72" PIPE X 2
n= .01
LENGTH= 55 FT
SLOPE= .006 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .25 FT
PEAK VELOCITY= 6.1 FPS
TRAVEL TIME = .2 MIN
SPAN= 0-24 HRS, dt=.1 HRS

JT 5/5/00
ROC 5/5/00
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2983

REACH 107

Qin = 24.19 CFS @ 12.21 HRS, VOLUME= 4.38 AF
Qout= 24.10 CFS @ 12.22 HRS, VOLUME= 4.38 AF, ATTEN= 0%, LAG= .6 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.20	1.72	2.77
.40	3.68	9.05
.60	5.89	18.33
.86	9.12	34.81
1.20	13.96	64.08
1.60	20.56	110.40
2.00	28.12	170.51

8' x 2' CHANNEL
SIDE SLOPE= .33 '/'
n= .03
LENGTH= 63 FT
SLOPE= .01 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .69 FT
PEAK VELOCITY= 3.4 FPS
TRAVEL TIME = .3 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 108 34" X 53" RCP

Qin = 25.32 CFS @ 12.21 HRS, VOLUME= 5.76 AF
Qout= 25.15 CFS @ 12.23 HRS, VOLUME= 5.75 AF, ATTEN= 1%, LAG= 1.3 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.60	2.94	12.50
1.20	8.05	52.45
1.80	14.27	117.29
4.20	42.28	501.45
4.80	48.50	585.44
5.40	53.61	638.34
5.64	55.16	644.26
5.82	56.05	638.31
6.00	56.55	598.93

72" PIPE X 2
n= .013
LENGTH= 296 FT
SLOPE= .005 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .79 FT
PEAK VELOCITY= 5.5 FPS
TRAVEL TIME = .9 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 109 2-28" RCP

Qin = 42.42 CFS @ 12.08 HRS, VOLUME= 7.20 AF
Qout= 42.41 CFS @ 12.08 HRS, VOLUME= 7.19 AF, ATTEN= 0%, LAG= .2 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.60	2.94	11.18
1.20	8.05	46.91
1.80	14.27	104.91
4.20	42.28	448.51
4.80	48.50	523.63
5.40	53.61	570.95
5.64	55.16	576.24
5.82	56.05	570.92
6.00	56.55	535.70

72" PIPE X 2
n= .013
LENGTH= 30 FT
SLOPE= .004 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= 1.12 FT
PEAK VELOCITY= 5.7 FPS
TRAVEL TIME = .1 MIN
SPAN= 0-24 HRS, dt=.1 HRS

RJC 5/5/00
RDC 5/5/00
31/111

2983

REACH 110 13" X 17" CMP

Qin = 5.59 CFS @ 11.93 HRS, VOLUME= .31 AF
Qout= 5.45 CFS @ 11.94 HRS, VOLUME= .31 AF, ATTEN= 3%, LAG= .5 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.60	1.47	6.43
1.20	4.03	26.95
1.80	7.13	60.27
4.20	21.14	257.68
4.80	24.25	300.84
5.40	26.80	328.02
5.64	27.58	331.06
5.82	28.03	328.01
6.00	28.27	307.77

72" PIPE
n= .024
LENGTH= 55 FT
SLOPE= .018 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .50 FT
PEAK VELOCITY= 4.4 FPS
TRAVEL TIME = .2 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 112 12" RCP TRANSITION TO 10" HDPE

Qin = 9.96 CFS @ 11.99 HRS, VOLUME= .63 AF
Qout= 9.84 CFS @ 11.99 HRS, VOLUME= .63 AF, ATTEN= 1%, LAG= .4 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.60	1.47	11.60
1.20	4.03	48.64
1.80	7.13	108.77
4.20	21.14	465.02
4.80	24.25	542.91
5.40	26.80	591.97
5.64	27.58	597.46
5.82	28.03	591.94
6.00	28.27	555.43

72" PIPE
n= .013
LENGTH= 116 FT
SLOPE= .0172 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .51 FT
PEAK VELOCITY= 7.9 FPS
TRAVEL TIME = .2 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 113 50" X 31" CMP

Qin = 3.33 CFS @ 11.99 HRS, VOLUME= .20 AF
Qout= 3.26 CFS @ 12.00 HRS, VOLUME= .20 AF, ATTEN= 2%, LAG= .5 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.60	1.47	8.57
1.20	4.03	35.94
1.80	7.13	80.36
4.20	21.14	343.57
4.80	24.25	401.12
5.40	26.80	437.36
5.64	27.58	441.42
5.82	28.03	437.34
6.00	28.27	410.36

72" PIPE
n= .024
LENGTH= 112 FT
SLOPE= .032 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .23 FT
PEAK VELOCITY= 5.8 FPS
TRAVEL TIME = .3 MIN
SPAN= 0-24 HRS, dt=.1 HRS

JT 5/5/00
ROE 5/5/00
32/111

2983

REACH 352

Qin = 40.71 CFS @ 12.00 HRS, VOLUME= 3.06 AF
Qout= 39.56 CFS @ 12.02 HRS, VOLUME= 3.06 AF, ATTEN= 3%, LAG= 1.3 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.50	.76	1.64
1.00	3.03	10.42
1.50	6.82	30.71
2.15	14.01	80.21
3.00	27.27	195.00
4.00	48.48	419.95
5.00	75.76	761.42

0' x 5' CHANNEL
SIDE SLOPE= .33 '/'
n= .03
LENGTH= 250 FT
SLOPE= .013 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= 1.63 FT
PEAK VELOCITY= 4.9 FPS
TRAVEL TIME = .8 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 451

Qin = 25.15 CFS @ 12.23 HRS, VOLUME= 5.75 AF
Qout= 25.12 CFS @ 12.25 HRS, VOLUME= 5.74 AF, ATTEN= 0%, LAG= 1.7 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.40	.48	.94
.80	1.94	5.96
1.20	4.36	17.58
1.72	8.96	45.91
2.40	17.45	111.61
3.20	31.03	240.36
4.00	48.48	435.80

0' x 4' CHANNEL
SIDE SLOPE= .33 '/'
n= .03
LENGTH= 240 FT
SLOPE= .014 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= 1.34 FT
PEAK VELOCITY= 4.5 FPS
TRAVEL TIME = .9 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 452

Qin = 32.31 CFS @ 12.04 HRS, VOLUME= 6.37 AF
Qout= 31.22 CFS @ 12.09 HRS, VOLUME= 6.36 AF, ATTEN= 3%, LAG= 2.9 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.50	.76	1.70
1.00	3.03	10.81
1.50	6.82	31.87
2.15	14.01	83.23
3.00	27.27	202.36
4.00	48.48	435.80
5.00	75.76	790.16

0' x 5' CHANNEL
SIDE SLOPE= .33 '/'
n= .03
LENGTH= 210 FT
SLOPE= .014 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= 1.49 FT
PEAK VELOCITY= 4.7 FPS
TRAVEL TIME = .8 MIN
SPAN= 0-24 HRS, dt=.1 HRS

JR 6/5/00
RDE 5/5/00
33/111

2983

REACH 453

Qin = 10.18 CFS @ 11.97 HRS, VOLUME= .63 AF
Qout= 9.96 CFS @ 11.99 HRS, VOLUME= .63 AF, ATTEN= 2%, LAG= 1.2 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.15	.13	.14
.30	.53	.90
.45	1.19	2.65
.65	2.45	6.92
.90	4.76	16.81
1.20	8.47	36.21
1.50	13.24	65.65

0' x 1.5' CHANNEL
SIDE SLOPE= .17 '/'
n= .03
LENGTH= 140 FT
SLOPE= .015 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .73 FT
PEAK VELOCITY= 3.2 FPS
TRAVEL TIME = .7 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 700 **50" X 31" CMP**

Qin = 5.76 CFS @ 12.00 HRS, VOLUME= .37 AF
Qout= 5.68 CFS @ 12.01 HRS, VOLUME= .37 AF, ATTEN= 1%, LAG= .3 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.60	1.47	9.21
1.20	4.03	38.64
1.80	7.13	86.41
4.20	21.14	369.44
4.80	24.25	431.32
5.40	26.80	470.29
5.64	27.58	474.65
5.82	28.03	470.27
6.00	28.27	441.26

72" PIPE
n= .024
LENGTH= 75 FT
SLOPE= .037 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .37 FT
PEAK VELOCITY= 6.3 FPS
TRAVEL TIME = .2 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 800 **72" x 44" CMP**

Qin = 47.30 CFS @ 12.17 HRS, VOLUME= 7.81 AF
Qout= 47.30 CFS @ 12.17 HRS, VOLUME= 7.81 AF, ATTEN= 0%, LAG= 0.0 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.60	1.47	5.02
1.20	4.03	21.07
1.80	7.13	47.12
4.20	21.14	201.44
4.80	24.25	235.18
5.40	26.80	256.43
5.64	27.58	258.81
5.82	28.03	256.42
6.00	28.27	240.60

72" PIPE
n= .024
LENGTH= 5 FT
SLOPE= .011 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= 1.80 FT
PEAK VELOCITY= 6.6 FPS
TRAVEL TIME = 0.0 MIN
SPAN= 0-24 HRS, dt=.1 HRS

JC 5/5/00
RDE 5/5/00
34/111

2983

REACH 900 **72" X 44" CMP**

Qin = 50.90 CFS @ 12.15 HRS, VOLUME= 8.18 AF
Qout= 49.79 CFS @ 12.20 HRS, VOLUME= 8.16 AF, ATTEN= 2%, LAG= 3.3 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.60	1.47	5.02
1.20	4.03	21.07
1.80	7.13	47.12
4.20	21.14	201.44
4.80	24.25	235.18
5.40	26.80	256.43
5.64	27.58	258.81
5.82	28.03	256.42
6.00	28.27	240.60

72" PIPE
n= .024
LENGTH= 600 FT
SLOPE= .011 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= 1.85 FT
PEAK VELOCITY= 6.8 FPS
TRAVEL TIME = 1.5 MIN
SPAN= 0-24 HRS, dt=.1 HRS

POND 1 OSDF Basin 1

Qin = 82.88 CFS @ 12.00 HRS, VOLUME= 7.56 AF
Qout= 1.90 CFS @ 19.16 HRS, VOLUME= .92 AF, ATTEN= 98%, LAG= 429.3 MIN

ELEVATION (FT)	CON.AREA (AC)	INC.STOR (AF)	CUM.STOR (AF)	WET.AREA (AC)
582.0	1.26	0.00	0.00	1.26
583.0	1.34	1.30	1.30	1.34
584.0	1.78	1.55	2.85	1.78
585.0	2.07	1.92	4.78	2.07
586.0	2.29	2.18	6.96	2.30
587.0	2.53	2.41	9.37	2.54
588.0	2.78	2.65	12.02	2.79

STOR-IND METHOD
PEAK STORAGE = 6.68 AF
PEAK ELEVATION= 585.9 FT
FLOOD ELEVATION= 588.0 FT
START ELEVATION= 582.0 FT
SPAN= 0-24 HRS, dt=.1 HRS
Tdet= 664.3 MIN (.92 AF)

#	ROUTE	INVERT	OUTLET DEVICES
1	2	585.8'	**48" HORIZONTAL ORIFICE/GRATE X 2** Q=.6 Area SQR(2gH) (Limited to weir flow @ low head)
2	P	580.1'	**36" CULVERT X 2** n=.024 L=61' S=.0187'/' Ke=.7 Cc=.9 Cd=.53
3	P	586.5'	**30' BROAD-CRESTED RECTANGULAR WEIR** Q=C L H^1.5 C=3, 3, 3, 3, 3, 3, 3, 3

Primary Discharge
├──2=Culvert
│ └──1=Orifice/Grate
└──3=Broad-Crested Rectangular Weir

POND 36

Qin = 62.32 CFS @ 12.00 HRS, VOLUME= 4.58 AF
Qout= 62.24 CFS @ 12.00 HRS, VOLUME= 4.58 AF, ATTEN= 0%, LAG= .2 MIN

ELEVATION (FT)	CON.AREA (AC)	INC.STOR (AF)	CUM.STOR (AF)	WET.AREA (AC)
586.0	0.00	0.00	0.00	0.00
587.0	.01	0.00	0.00	.01
588.0	.01	.01	.01	.01
589.0	.01	.01	.02	.01

STOR-IND METHOD
PEAK STORAGE = .02 AF
PEAK ELEVATION= 588.2 FT
FLOOD ELEVATION= 589.0 FT
START ELEVATION= 586.0 FT
SPAN= 0-24 HRS, dt=.1 HRS
Tdet= .3 MIN (4.58 AF)

#	ROUTE	INVERT	OUTLET DEVICES
1	P	586.0'	**48" CULVERT X 2** n=.024 L=114' S=.02'/' Ke=.7 Cc=.9 Cd=.53

JH 5/5/00
RDB 5/5/00
36/111

2983

POND 49

Qin = 6.55 CFS @ 11.94 HRS, VOLUME= .38 AF
Qout= 1.83 CFS @ 12.16 HRS, VOLUME= .37 AF, ATTEN= 72%, LAG= 13.1 MIN

ELEVATION (FT)	CON.AREA (AC)	INC.STOR (AF)	CUM.STOR (AF)	WET.AREA (AC)
585.0	.03	0.00	0.00	.03
586.0	.08	.05	.05	.08
587.0	.14	.11	.16	.14

STOR-IND METHOD
PEAK STORAGE = .13 AF
PEAK ELEVATION= 586.7 FT
FLOOD ELEVATION= 587.0 FT
START ELEVATION= 585.0 FT
SPAN= 0-24 HRS, dt=.1 HRS
Tdet= 50.4 MIN (.37 AF)

#	ROUTE	INVERT	OUTLET DEVICES
1	P	585.0'	12" CULVERT
			n=.024 L=200' S=.006'/' Ke=.5 Cc=.9 Cd=.6

5/5/00
5/5/00
37/111

2983

LINK 1

Qout= 5.25 CFS @ 12.51 HRS, VOLUME= 2.24 AF, SPAN= 0-24 HRS, dt=.1 HRS

REACH 61 from BORROW AREA DESIGN SCENARIO

JT 5/5/00
KDE 5/5/00

Written by: JGT Date: 3 May 2000 Reviewed by: Date:

Client: Fluor Fernald, Inc. Project: OSDF Phase III Project No.: GQ1001 Task No.: 04

2983

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

000203

JH 5/5/00
ODE 5/5/
39/111

2983

POND 1 **OSDF Basin 1**

Qin = 105.3 CFS @ 12.00 HRS, VOLUME= 9.50 AF
Qout= 5.91 CFS @ 14.05 HRS, VOLUME= 2.83 AF, ATTEN= 94%, LAG= 122.9 MIN

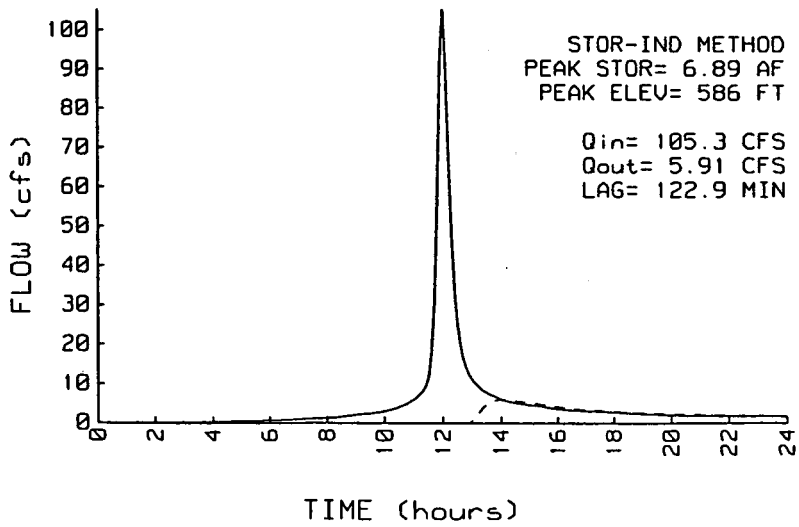
ELEVATION (FT)	CON.AREA (AC)	INC.STOR (AF)	CUM.STOR (AF)	WET.AREA (AC)
582.0	1.26	0.00	0.00	1.26
583.0	1.34	1.30	1.30	1.34
584.0	1.78	1.55	2.85	1.78
585.0	2.07	1.92	4.78	2.07
586.0	2.29	2.18	6.96	2.30
587.0	2.53	2.41	9.37	2.54
588.0	2.78	2.65	12.02	2.79

STOR-IND METHOD
PEAK STORAGE = 6.89 AF
PEAK ELEVATION= 586.0 FT
FLOOD ELEVATION= 588.0 FT
START ELEVATION= 582.0 FT
SPAN= 0-24 HRS, dt=.1 HRS
Tdet= 397.9 MIN (2.82 AF)

#	ROUTE	INVERT	OUTLET DEVICES
1	2	585.8'	**48" HORIZONTAL ORIFICE/GRATE X 2** Q=.6 Area SQR(2gH) (Limited to weir flow @ low head)
2	P	580.1'	**36" CULVERT X 2** n=.024 L=61' S=.0187'/' Ke=.7 Cc=.9 Cd=.53
3	P	586.5'	**30' BROAD-CRESTED RECTANGULAR WEIR** Q=C L H^1.5 C=3, 3, 3, 3, 3, 3, 3, 3

Primary Discharge
├──2=Culvert
│ └──1=Orifice/Grate
└──3=Broad-Crested Rectangular Weir

POND 1 INFLOW & OUTFLOW
OSDF Basin 1



2983

ATTACHMENT C-1B

HydroCAD™ OUTPUT REPORT

BORROW AREA DESIGN SCENARIO
25-YEAR, 24-HOUR STORM EVENT

JY 5/5/00
RDE 5/5/00
41/111

RUNOFF BY SCS TR-20 METHOD: TYPE II 24-HOUR RAINFALL= 4.70 IN, SCS U.H.

RUNOFF SPAN = 0-24 HRS, dt= .10 HRS, 241 POINTS

2983

SUBCAT NUMBER	AREA (ACRE)	Tc (MIN)	--GROUND COVERS (%CN)--	WGT'D CN	C	PEAK (CFS)	Tpeak (HRS)	VOL (AF)
1	3.80	42.9	100%74	74	-	5.35	12.37	.67
2	8.64	38.2	100%73	73	-	12.68	12.31	1.46
3	21.99	40.4	100%73	73	-	31.06	12.34	3.71
4	13.52	37.7	100%77	77	-	23.56	12.30	2.65
6	21.41	26.7	100%76	76	-	44.64	12.16	4.06
8	1.81	11.8	100%83	83	-	7.20	11.99	.44
9	1.92	12.2	100%76	76	-	6.02	11.99	.37
11	15.54	34.0	100%69	69	-	20.36	12.27	2.24
12	15.93	33.0	100%68	68	-	20.21	12.25	2.20
13	4.10	18.2	100%61	61	-	4.97	12.09	.41
14	1.41	1.4	100%65	65	-	3.55	11.87	.17
15	11.16	26.7	100%88	88	-	34.42	12.15	3.13
16	6.19	41.9	100%73	73	-	8.48	12.36	1.04
17	3.63	31.5	100%61	61	-	3.09	12.25	.36
18	6.01	28.1	100%58	58	-	4.43	12.22	.50
19	2.10	9.9	100%61	61	-	3.38	11.99	.21
20	2.08	20.6	100%61	61	-	2.37	12.11	.21
21	1.69	1.0	100%98	98	-	10.39	11.83	.63
50	.58	6.4	100%79	79	-	2.34	11.91	.12
161	1.48	30.2	100%61	61	-	1.31	12.24	.15
201	.20	6.5	100%66	66	-	.46	11.92	.03

JT 5/5/00
RDE 5/5/
42/111

REACH ROUTING BY STOR-IND+TRANS METHOD

2983

REACH NO.	DIAM (IN)	BOTTOM WIDTH (FT)	DEPTH (FT)	SIDE SLOPES (FT/FT)		n	LENGTH (FT)	SLOPE (FT/FT)	PEAK VEL. (FPS)	TRAVEL TIME (MIN)	PEAK Qout (CFS)
1	-	-	2.0	.20	.33	.030	880	.0070	1.7	8.8	1.56
2	-	-	3.0	.20	.25	.030	530	.0110	2.7	3.3	6.17
3	-	-	3.0	.33	.33	.030	830	.0120	3.9	3.6	17.98
4	-	-	2.0	.05	.33	.030	320	.0030	2.1	2.5	41.17
5	-	8.0	2.0	.33	.33	.030	305	.0050	3.2	1.6	42.55
6	-	10.0	2.0	.33	.33	.030	500	.0050	3.4	2.5	55.91
14	-	-	3.0	.33	.33	.030	5	.0130	2.7	0.0	3.55
15	72.0	-	-	-	-	.024	63	.0075	5.1	.2	28.46
17	-	-	1.5	.03	.10	.030	5	.0300	2.3	0.0	3.09
18	-	-	3.0	.33	.33	.030	5	.0090	2.5	0.0	4.43
19	-	-	3.0	.20	.05	.030	980	.0060	1.7	9.8	6.22
50	-	-	1.5	.33	.20	.030	5	.0040	1.4	.1	2.32
61	-	-	3.0	.17	.25	.030	325	.0150	2.7	2.0	4.80
91	-	10.0	4.0	.33	.33	.030	280	.0030	3.0	1.5	68.81
92	-	10.0	2.5	.33	.33	.030	540	.0030	3.2	2.8	80.88
161	-	-	1.5	.10	.10	.030	5	.0020	.8	.1	1.31
162	-	-	2.0	.12	.03	.030	670	.0200	1.5	7.4	1.13
163	-	-	4.0	.33	.33	.030	182	.0070	2.7	1.1	9.41
201	-	-	3.5	.33	.33	.030	290	.0075	3.6	1.3	28.22
202	-	-	4.0	.17	.33	.030	391	.0050	3.4	1.9	55.76
600	72.0	-	-	-	-	.024	5	.0050	3.0	0.0	6.67

YI 5/5/00
RDE 5/5
43/111

2983

POND ROUTING BY STOR-IND METHOD

POND NO.	START ELEV. (FT)	FLOOD ELEV. (FT)	PEAK ELEV. (FT)	PEAK STORAGE (AF)	PEAK FLOW Qin (CFS)	Qout (CFS)	Qpri (CFS)	Qsec (CFS)	Qout ATTEN. (%)	Qout LAG (MIN)
3	569.8	577.5	576.4	7.18	141.1	101.7			28	28.2
15	587.4	590.1	589.6	.01	47.92	47.72	4.82	42.90	0	.1
41	586.0	589.0	588.3	.64	42.55	41.93			1	3.4

JC 5/5/00
CDE 5/5/00
44/111

2983

SUBCATCHMENT 1

PEAK= 5.35 CFS @ 12.37 HRS, VOLUME= .67 AF

ACRES	CN
3.80	74

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	39.8
n=.15 L=300' P2=2.6 in s=.007 '/'		
SHALLOW CONCENTRATED/UPLAND FLOW	**Segment ID:**	.4
Unpaved Kv=16.1345 L=50' s=.02 '/' V=2.28 fps		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	2.7
D=2' SS= .33 & .2 '/' a=16.06 sq-ft Pw=16.6' r=.969'		
s=.007 '/' n=.03 V=4.06 fps L=650' Capacity=65.2 cfs		
	Total Length= 1000 ft Total Tc=	42.9

SUBCATCHMENT 2

PEAK= 12.68 CFS @ 12.31 HRS, VOLUME= 1.46 AF

ACRES	CN
8.64	73

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	32.1
n=.15 L=300' P2=2.6 in s=.012 '/'		
SHALLOW CONCENTRATED/UPLAND FLOW	**Segment ID:**	4.9
Unpaved Kv=16.1345 L=520' s=.012 '/' V=1.77 fps		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	1.2
D=2' SS= .2 & .25 '/' a=18 sq-ft Pw=18.4' r=.976'		
s=.011 '/' n=.03 V=5.11 fps L=370' Capacity=92 cfs		
	Total Length= 1190 ft Total Tc=	38.2

JA 5/5/00
RDE 5/5/00
45/111

2983

SUBCATCHMENT 3

PEAK= 31.06 CFS @ 12.34 HRS, VOLUME= 3.71 AF

ACRES	CN
21.99	73

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	29.4
n=.15 L=300' P2=2.6 in s=.015 '/'		
SHALLOW CONCENTRATED/UPLAND FLOW	**Segment ID:**	11.0
Unpaved Kv=16.1345 L=1300' s=.015 '/' V=1.98 fps		
	Total Length= 1600 ft Total Tc=	40.4

SUBCATCHMENT 4

PEAK= 23.56 CFS @ 12.30 HRS, VOLUME= 2.65 AF

ACRES	CN
13.52	77

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	27.3
n=.15 L=300' P2=2.6 in s=.018 '/'		
SHALLOW CONCENTRATED/UPLAND FLOW	**Segment ID:**	8.5
Unpaved Kv=16.1345 L=1100' s=.018 '/' V=2.16 fps		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	1.9
D=2' SS= .33 '/' a=12.12 sq-ft Pw=12.8' r=.95'		
s=.003 '/' n=.03 V=2.62 fps L=300' Capacity=31.8 cfs		
	Total Length= 1700 ft Total Tc=	37.7

SUBCATCHMENT 6

PEAK= 44.64 CFS @ 12.16 HRS, VOLUME= 4.06 AF

ACRES	CN
21.41	76

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	19.8
n=.15 L=300' P2=2.6 in s=.04 '/'		
SHALLOW CONCENTRATED/UPLAND FLOW	**Segment ID:**	6.9
Unpaved Kv=16.1345 L=900' s=.018 '/' V=2.16 fps		
	Total Length= 1200 ft Total Tc=	26.7

000210

JY 5/5/00
RDE 5/5/00
46/111

2983

SUBCATCHMENT 8

PEAK= 7.20 CFS @ 11.99 HRS, VOLUME= .44 AF

ACRES	CN
1.81	83

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	10.9
n=.15 L=100' P2=2.6 in s=.02 '/'		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	.9
D=3' SS= .33 '/' a=27.27 sq-ft Pw=19.1' r=1.424'		
s=.008 '/' n=.03 V=5.61 fps L=300' Capacity=153 cfs		
	Total Length= 400 ft Total Tc=	11.8

SUBCATCHMENT 9

PEAK= 6.02 CFS @ 11.99 HRS, VOLUME= .37 AF

ACRES	CN
1.92	76

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	11.5
n=.15 L=170' P2=2.6 in s=.05 '/'		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	.7
W=10' D=2' SS= .33 '/' a=32.12 sq-ft Pw=22.8' r=1.411'		
s=.003 '/' n=.03 V=3.41 fps L=150' Capacity=109.6 cfs		
	Total Length= 320 ft Total Tc=	12.2

8/5/5/00
RDB 5/8
47/111

2983

SUBCATCHMENT 11

PEAK= 20.36 CFS @ 12.27 HRS, VOLUME= 2.24 AF

ACRES	CN
15.54	69

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	26.2
n=.15 L=300' P2=2.6 in s=.02 '/'		
SHALLOW CONCENTRATED/UPLAND FLOW	**Segment ID:**	6.9
Unpaved Kv=16.1345 L=950' s=.02 '/' V=2.28 fps		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	.9
D=3' SS= .33 '/' a=27.27 sq-ft Pw=19.1' r=1.424'		
s=.011 '/' n=.03 V=6.58 fps L=370' Capacity=179.4 cfs		

	Total Length= 1620 ft Total Tc=	34.0

SUBCATCHMENT 12

PEAK= 20.21 CFS @ 12.25 HRS, VOLUME= 2.20 AF

ACRES	CN
15.93	68

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	23.6
n=.15 L=300' P2=2.6 in s=.026 '/'		
SHALLOW CONCENTRATED/UPLAND FLOW	**Segment ID:**	4.1
Unpaved Kv=16.1345 L=500' s=.016 '/' V=2.04 fps		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	5.0
D=.5' SS= .04 & .03 '/' a=7.29 sq-ft Pw=29.2' r=.25'		
s=.012 '/' n=.03 V=2.15 fps L=650' Capacity=15.7 cfs		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	.3
D=2' SS= .33 '/' a=12.12 sq-ft Pw=12.8' r=.95'		
s=.007 '/' n=.03 V=4 fps L=80' Capacity=48.5 cfs		

	Total Length= 1530 ft Total Tc=	33.0

01 5/5/00
48/111

2983

SUBCATCHMENT 13

PEAK= 4.97 CFS @ 12.09 HRS, VOLUME= .41 AF

ACRES	CN
4.10	61

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	18.1
n=.15 L=300' P2=2.6 in s=.05 '/'		
SHALLOW CONCENTRATED/UPLAND FLOW	**Segment ID:**	.1
Unpaved Kv=16.1345 L=20' s=.05 '/' V=3.61 fps		
	Total Length= 320 ft Total Tc=	18.2

SUBCATCHMENT 14

PEAK= 3.55 CFS @ 11.87 HRS, VOLUME= .17 AF

ACRES	CN
1.41	65

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
RECT/VEE/TRAP CHANNEL	**Segment ID:**	1.4
D=3' SS= .33 '/' a=27.27 sq-ft Pw=19.1' r=1.424'		
s=.013 '/' n=.03 V=7.15 fps L=600' Capacity=195 cfs		

SUBCATCHMENT 15

PEAK= 34.42 CFS @ 12.15 HRS, VOLUME= 3.13 AF

ACRES	CN
11.16	88

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	.4
n=.011 L=85' P2=2.6 in s=.33 '/'		
SHALLOW CONCENTRATED/UPLAND FLOW	**Segment ID:**	4.4
Unpaved Kv=16.1345 L=215' s=.0025 '/' V=.81 fps		
SHALLOW CONCENTRATED/UPLAND FLOW	**Segment ID:**	13.2
Unpaved Kv=16.1345 L=640' s=.0025 '/' V=.81 fps		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	8.7
D=.5' SS= .01 & .33 '/' a=12.88 sq-ft Pw=51.6' r=.25'		
s=.001 '/' n=.03 V=.62 fps L=325' Capacity=8 cfs		
	Total Length= 1265 ft Total Tc=	26.7

000213

JT 5/5/00
RDE 5/5/00
49/111

2983

SUBCATCHMENT 16

PEAK= 8.48 CFS @ 12.36 HRS, VOLUME= 1.04 AF

ACRES	CN
6.19	73

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	37.8
n=.15 L=300' P2=2.6 in s=.008 '/'		
SHALLOW CONCENTRATED/UPLAND FLOW	**Segment ID:**	1.3
Unpaved Kv=16.1345 L=220' s=.032 '/' V=2.89 fps		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	1.2
D=1' SS= .06 & .07 '/' a=15.48 sq-ft Pw=31' r=.499'		
s=.029 '/' n=.03 V=5.31 fps L=380' Capacity=82.1 cfs		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	1.6
D=1' SS= .12 & .03 '/' a=20.83 sq-ft Pw=41.7' r=.499'		
s=.02 '/' n=.03 V=4.41 fps L=430' Capacity=91.8 cfs		
	Total Length= 1330 ft Total Tc=	41.9

SUBCATCHMENT 17

PEAK= 3.09 CFS @ 12.25 HRS, VOLUME= .36 AF

ACRES	CN
3.63	61

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	26.2
n=.15 L=300' P2=2.6 in s=.02 '/'		
SHALLOW CONCENTRATED/UPLAND FLOW	**Segment ID:**	3.4
Unpaved Kv=16.1345 L=440' s=.018 '/' V=2.16 fps		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	1.9
D=.5' SS= .03 & .1 '/' a=5.42 sq-ft Pw=21.7' r=.25'		
s=.03 '/' n=.03 V=3.4 fps L=380' Capacity=18.4 cfs		
	Total Length= 1120 ft Total Tc=	31.5

JY 5/5/00 ROF 5/3/ 50/111

2983

SUBCATCHMENT 18

PEAK= 4.43 CFS @ 12.22 HRS, VOLUME= .50 AF

ACRES	CN
6.01	58

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	24.3
n=.15 L=300' P2=2.6 in s=.024 '/'		
SHALLOW CONCENTRATED/UPLAND FLOW	**Segment ID:**	1.3
Unpaved Kv=16.1345 L=200' s=.024 '/' V=2.5 fps		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	2.5
D=3' SS= .33 '/' a=27.27 sq-ft Pw=19.1' r=1.424'		
s=.009 '/' n=.03 V=5.95 fps L=900' Capacity=162.2 cfs		
	Total Length= 1400 ft Total Tc=	28.1

SUBCATCHMENT 19

PEAK= 3.38 CFS @ 11.99 HRS, VOLUME= .21 AF

ACRES	CN
2.10	61

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	6.6
n=.15 L=100' P2=2.6 in s=.07 '/'		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	3.3
D=3' SS= .2 & .05 '/' a=112.5 sq-ft Pw=75.4' r=1.493'		
s=.006 '/' n=.03 V=5.01 fps L=980' Capacity=563.8 cfs		
	Total Length= 1080 ft Total Tc=	9.9

JI 5/5/00
RDE 5/5
5/11

2983

SUBCATCHMENT 20

PEAK= 2.37 CFS @ 12.11 HRS, VOLUME= .21 AF

ACRES	CN
2.08	61

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	18.1
n=.15 L=300' P2=2.6 in s=.05 '/'		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	.9
W=10' D=3' SS= .07 & .25 '/' a=112.29 sq-ft Pw=65.3' r=1.719'		
s=.008 '/' n=.03 V=6.36 fps L=350' Capacity=713.8 cfs		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	1.6
D=2' SS= .17 & .33 '/' a=17.83 sq-ft Pw=18.3' r=.973'		
s=.005 '/' n=.03 V=3.44 fps L=320' Capacity=61.3 cfs		
	Total Length= 970 ft Total Tc=	20.6

SUBCATCHMENT 21

PEAK= 10.39 CFS @ 11.83 HRS, VOLUME= .63 AF

ACRES	CN
1.69	98

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
DIRECT ENTRY	**Segment ID:**	1.0

SUBCATCHMENT 50

PEAK= 2.34 CFS @ 11.91 HRS, VOLUME= .12 AF

ACRES	CN
.58	79

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	4.1
n=.15 L=60' P2=2.6 in s=.08 '/'		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	2.3
D=1.5' SS= .33 & .2 '/' a=9.03 sq-ft Pw=12.4' r=.726'		
s=.004 '/' n=.03 V=2.53 fps L=350' Capacity=22.9 cfs		
	Total Length= 410 ft Total Tc=	6.4

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RJO 5/5
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2983

SUBCATCHMENT 161

PEAK= 1.31 CFS @ 12.24 HRS, VOLUME= .15 AF

ACRES	CN
1.48	61

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	27.9
n=.15 L=300' P2=2.6 in s=.017 '/'		
SHALLOW CONCENTRATED/UPLAND FLOW	**Segment ID:**	.5
Unpaved Kv=16.1345 L=100' s=.05 '/' V=3.61 fps		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	1.8
D=1' SS= .1 '/' a=10 sq-ft Pw=20.1' r=.498'		
s=.002 '/' n=.03 V=1.39 fps L=150' Capacity=13.9 cfs		

	Total Length= 550 ft Total Tc=	30.2

SUBCATCHMENT 201

PEAK= .46 CFS @ 11.92 HRS, VOLUME= .03 AF

ACRES	CN
.20	66

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	5.3
n=.15 L=50' P2=2.6 in s=.03 '/'		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	1.2
D=2' SS= .33 '/' a=12.12 sq-ft Pw=12.8' r=.95'		
s=.0075 '/' n=.03 V=4.14 fps L=290' Capacity=50.2 cfs		

	Total Length= 340 ft Total Tc=	6.5

HI 5/5/00
PDE 5/5
53/111

2983

REACH 1

Qin = 2.32 CFS @ 11.91 HRS, VOLUME= .12 AF
Qout= 1.56 CFS @ 12.15 HRS, VOLUME= .12 AF, ATTEN= 33%, LAG= 14.5 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.20	.16	.14
.40	.64	.89
.60	1.45	2.63
.86	2.97	6.86
1.20	5.78	16.69
1.60	10.28	35.94
2.00	16.06	65.16

0' x 2' CHANNEL
S/S= .2 & .33 '/'
n= .03
LENGTH= 880 FT
SLOPE= .007 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .49 FT
PEAK VELOCITY= 1.7 FPS
TRAVEL TIME = 8.8 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 2

Qin = 6.29 CFS @ 12.31 HRS, VOLUME= .79 AF
Qout= 6.17 CFS @ 12.41 HRS, VOLUME= .79 AF, ATTEN= 2%, LAG= 6.4 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.30	.41	.58
.60	1.62	3.71
.90	3.65	10.94
1.29	7.49	28.57
1.80	14.58	69.47
2.40	25.92	149.61
3.00	40.50	271.26

0' x 3' CHANNEL
S/S= .2 & .25 '/'
n= .03
LENGTH= 530 FT
SLOPE= .011 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .70 FT
PEAK VELOCITY= 2.7 FPS
TRAVEL TIME = 3.3 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 3

Qin = 18.64 CFS @ 12.34 HRS, VOLUME= 2.25 AF
Qout= 17.98 CFS @ 12.46 HRS, VOLUME= 2.24 AF, ATTEN= 4%, LAG= 7.2 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.30	.27	.40
.60	1.09	2.56
.90	2.45	7.56
1.29	5.04	19.73
1.80	9.82	47.98
2.40	17.45	103.33
3.00	27.27	187.35

0' x 3' CHANNEL
SIDE SLOPE= .33 '/'
n= .03
LENGTH= 830 FT
SLOPE= .012 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= 1.24 FT
PEAK VELOCITY= 3.9 FPS
TRAVEL TIME = 3.6 MIN
SPAN= 0-24 HRS, dt=.1 HRS

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REACH 4

Qin = 41.93 CFS @ 12.50 HRS, VOLUME= 3.56 AF
Qout= 41.17 CFS @ 12.59 HRS, VOLUME= 3.56 AF, ATTEN= 2%, LAG= 5.9 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.20	.46	.27
.40	1.84	1.70
.60	4.15	5.01
.86	8.52	13.09
1.20	16.58	31.83
1.60	29.48	68.55
2.00	46.06	124.30

0' x 2' CHANNEL
S/S= .05 & .33 '/'
n= .03
LENGTH= 320 FT
SLOPE= .003 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= 1.32 FT
PEAK VELOCITY= 2.1 FPS
TRAVEL TIME = 2.5 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 5

Qin = 42.90 CFS @ 12.39 HRS, VOLUME= 3.98 AF
Qout= 42.55 CFS @ 12.44 HRS, VOLUME= 3.98 AF, ATTEN= 1%, LAG= 2.9 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.20	1.72	1.96
.40	3.68	6.40
.60	5.89	12.96
.86	9.12	24.61
1.20	13.96	45.31
1.60	20.56	78.06
2.00	28.12	120.57

8' x 2' CHANNEL
SIDE SLOPE= .33 '/'
n= .03
LENGTH= 305 FT
SLOPE= .005 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= 1.16 FT
PEAK VELOCITY= 3.2 FPS
TRAVEL TIME = 1.6 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 6

Qin = 57.87 CFS @ 12.54 HRS, VOLUME= 6.21 AF
Qout= 55.91 CFS @ 12.63 HRS, VOLUME= 6.20 AF, ATTEN= 3%, LAG= 5.3 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.20	2.12	2.44
.40	4.48	7.91
.60	7.09	15.91
.86	10.84	29.93
1.20	16.36	54.48
1.60	23.76	92.68
2.00	32.12	141.54

10' x 2' CHANNEL
SIDE SLOPE= .33 '/'
n= .03
LENGTH= 500 FT
SLOPE= .005 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= 1.23 FT
PEAK VELOCITY= 3.4 FPS
TRAVEL TIME = 2.5 MIN
SPAN= 0-24 HRS, dt=.1 HRS

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RPE 5/5/00
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REACH 14

Qin = 3.55 CFS @ 11.87 HRS, VOLUME= .17 AF
Qout= 3.55 CFS @ 11.87 HRS, VOLUME= .17 AF, ATTEN= 0%, LAG= 0.0 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.30	.27	.42
.60	1.09	2.67
.90	2.45	7.86
1.29	5.04	20.54
1.80	9.82	49.94
2.40	17.45	107.55
3.00	27.27	195.00

0' x 3' CHANNEL
SIDE SLOPE= .33 '/'
n= .03
LENGTH= 5 FT
SLOPE= .013 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .64 FT
PEAK VELOCITY= 2.7 FPS
TRAVEL TIME = 0.0 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 15 36" CMP

Qin = 28.56 CFS @ 12.30 HRS, VOLUME= 3.38 AF
Qout= 28.46 CFS @ 12.30 HRS, VOLUME= 3.38 AF, ATTEN= 0%, LAG= .4 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.60	1.47	4.15
1.20	4.03	17.40
1.80	7.13	38.91
4.20	21.14	166.33
4.80	24.25	194.19
5.40	26.80	211.74
5.64	27.58	213.70
5.82	28.03	211.73
6.00	28.27	198.67

72" PIPE
n= .024
LENGTH= 63 FT
SLOPE= .0075 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= 1.51 FT
PEAK VELOCITY= 5.1 FPS
TRAVEL TIME = .2 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 17

Qin = 3.09 CFS @ 12.25 HRS, VOLUME= .36 AF
Qout= 3.09 CFS @ 12.25 HRS, VOLUME= .36 AF, ATTEN= 0%, LAG= .1 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.15	.49	.74
.30	1.95	4.72
.45	4.39	13.91
.65	9.01	36.33
.90	17.55	88.33
1.20	31.20	190.23
1.50	48.75	344.91

0' x 1.5' CHANNEL
S/S= .03 & .1 '/'
n= .03
LENGTH= 5 FT
SLOPE= .03 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .24 FT
PEAK VELOCITY= 2.3 FPS
TRAVEL TIME = 0.0 MIN
SPAN= 0-24 HRS, dt=.1 HRS

JI 5/5/00
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REACH 18

Qin = 4.43 CFS @ 12.22 HRS, VOLUME= .50 AF
Qout= 4.43 CFS @ 12.22 HRS, VOLUME= .50 AF, ATTEN= 0%, LAG= .1 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.30	.27	.35
.60	1.09	2.22
.90	2.45	6.54
1.29	5.04	17.09
1.80	9.82	41.55
2.40	17.45	89.49
3.00	27.27	162.25

0' x 3' CHANNEL
SIDE SLOPE= .33 '/'
n= .03
LENGTH= 5 FT
SLOPE= .009 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .75 FT
PEAK VELOCITY= 2.5 FPS
TRAVEL TIME = 0.0 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 19

Qin = 7.53 CFS @ 12.23 HRS, VOLUME= .86 AF
Qout= 6.22 CFS @ 12.54 HRS, VOLUME= .85 AF, ATTEN= 17%, LAG= 18.6 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.0	0.00
.30	1.1	1.21
.60	4.5	7.71
.90	10.1	22.74
1.29	20.8	59.38
1.80	40.5	144.37
2.40	72.0	310.93
3.00	112.5	563.75

0' x 3' CHANNEL
S/S= .2 & .05 '/'
n= .03
LENGTH= 980 FT
SLOPE= .006 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .53 FT
PEAK VELOCITY= 1.7 FPS
TRAVEL TIME = 9.8 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 50

Qin = 2.34 CFS @ 11.91 HRS, VOLUME= .12 AF
Qout= 2.32 CFS @ 11.91 HRS, VOLUME= .12 AF, ATTEN= 1%, LAG= .1 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.15	.09	.05
.30	.36	.31
.45	.81	.92
.65	1.67	2.41
.90	3.25	5.86
1.20	5.78	12.61
1.50	9.03	22.87

0' x 1.5' CHANNEL
S/S= .33 & .2 '/'
n= .03
LENGTH= 5 FT
SLOPE= .004 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .63 FT
PEAK VELOCITY= 1.4 FPS
TRAVEL TIME = .1 MIN
SPAN= 0-24 HRS, dt=.1 HRS

JT 5/5/00
RDE 5/5
57/111
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REACH 61

Qin = 4.82 CFS @ 12.39 HRS, VOLUME= 1.97 AF
Qout= 4.80 CFS @ 12.46 HRS, VOLUME= 1.96 AF, ATTEN= 1%, LAG= 4.0 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.30	.44	.75
.60	1.78	4.77
.90	4.00	14.06
1.29	8.22	36.73
1.80	16.01	89.29
2.40	28.46	192.29
3.00	44.47	348.65

0' x 3' CHANNEL
S/S= .17 & .25 '/'
n= .03
LENGTH= 325 FT
SLOPE= .015 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .60 FT
PEAK VELOCITY= 2.7 FPS
TRAVEL TIME = 2.0 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 91

Qin = 70.10 CFS @ 12.59 HRS, VOLUME= 10.26 AF
Qout= 68.81 CFS @ 12.63 HRS, VOLUME= 10.25 AF, ATTEN= 2%, LAG= 2.4 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.40	4.48	6.13
.80	9.94	20.40
1.20	16.36	42.20
1.72	26.16	82.25
2.40	41.45	156.24
3.20	63.03	277.91
4.00	88.48	441.09

10' x 4' CHANNEL
SIDE SLOPE= .33 '/'
n= .03
LENGTH= 280 FT
SLOPE= .003 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= 1.56 FT
PEAK VELOCITY= 3.0 FPS
TRAVEL TIME = 1.5 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 92

Qin = 82.07 CFS @ 12.28 HRS, VOLUME= 12.92 AF
Qout= 80.88 CFS @ 12.40 HRS, VOLUME= 12.88 AF, ATTEN= 1%, LAG= 6.9 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.25	2.69	2.75
.50	5.76	8.99
.75	9.20	18.21
1.08	14.25	34.57
1.50	21.82	63.64
2.00	32.12	109.63
2.50	43.94	169.33

10' x 2.5' CHANNEL
SIDE SLOPE= .33 '/'
n= .03
LENGTH= 540 FT
SLOPE= .003 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= 1.70 FT
PEAK VELOCITY= 3.2 FPS
TRAVEL TIME = 2.8 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 161

2983

Qin = 1.31 CFS @ 12.24 HRS, VOLUME= .15 AF
Qout= 1.31 CFS @ 12.24 HRS, VOLUME= .15 AF, ATTEN= 0%, LAG= .2 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.15	.23	.09
.30	.90	.56
.45	2.03	1.65
.65	4.16	4.32
.90	8.10	10.50
1.20	14.40	22.62
1.50	22.50	41.01

0' x 1.5' CHANNEL
SIDE SLOPE= .1 '/'
n= .03
LENGTH= 5 FT
SLOPE= .002 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .40 FT
PEAK VELOCITY= .8 FPS
TRAVEL TIME = .1 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 162

Qin = 1.31 CFS @ 12.24 HRS, VOLUME= .15 AF
Qout= 1.13 CFS @ 12.49 HRS, VOLUME= .14 AF, ATTEN= 14%, LAG= 15.1 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.20	.83	1.26
.40	3.33	7.98
.60	7.50	23.52
.86	15.41	61.42
1.20	30.00	149.32
1.60	53.33	321.58
2.00	83.33	583.06

0' x 2' CHANNEL
S/S= .12 & .03 '/'
n= .03
LENGTH= 670 FT
SLOPE= .02 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .18 FT
PEAK VELOCITY= 1.5 FPS
TRAVEL TIME = 7.4 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 163

Qin = 9.49 CFS @ 12.38 HRS, VOLUME= 1.19 AF
Qout= 9.41 CFS @ 12.42 HRS, VOLUME= 1.19 AF, ATTEN= 1%, LAG= 2.1 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.40	.48	.66
.80	1.94	4.22
1.20	4.36	12.43
1.72	8.96	32.46
2.40	17.45	78.92
3.20	31.03	169.96
4.00	48.48	308.16

0' x 4' CHANNEL
SIDE SLOPE= .33 '/'
n= .03
LENGTH= 182 FT
SLOPE= .007 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= 1.06 FT
PEAK VELOCITY= 2.7 FPS
TRAVEL TIME = 1.1 MIN
SPAN= 0-24 HRS, dt=.1 HRS

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RDO 5/5/
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REACH 201

Qin = 28.46 CFS @ 12.30 HRS, VOLUME= 3.38 AF
Qout= 28.22 CFS @ 12.34 HRS, VOLUME= 3.38 AF, ATTEN= 1%, LAG= 2.2 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.35	.37	.48
.70	1.48	3.06
1.05	3.34	9.01
1.51	6.86	23.53
2.10	13.36	57.22
2.80	23.76	123.22
3.50	37.12	223.41

0' x 3.5' CHANNEL
SIDE SLOPE= .33 '/'
n= .03
LENGTH= 290 FT
SLOPE= .0075 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= 1.59 FT
PEAK VELOCITY= 3.6 FPS
TRAVEL TIME = 1.3 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 202

Qin = 57.22 CFS @ 12.22 HRS, VOLUME= 6.54 AF
Qout= 55.76 CFS @ 12.29 HRS, VOLUME= 6.52 AF, ATTEN= 3%, LAG= 3.8 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.40	.71	.84
.80	2.85	5.33
1.20	6.42	15.70
1.72	13.18	41.01
2.40	25.67	99.70
3.20	45.63	214.73
4.00	71.30	389.32

0' x 4' CHANNEL
S/S= .17 & .33 '/'
n= .03
LENGTH= 391 FT
SLOPE= .005 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= 1.90 FT
PEAK VELOCITY= 3.4 FPS
TRAVEL TIME = 1.9 MIN
SPAN= 0-24 HRS, dt=.1 HRS

REACH 600

Qin = 6.67 CFS @ 12.54 HRS, VOLUME= 1.06 AF
Qout= 6.67 CFS @ 12.54 HRS, VOLUME= 1.06 AF, ATTEN= 0%, LAG= 0.0 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.60	1.47	3.39
1.20	4.03	14.20
1.80	7.13	31.77
4.20	21.14	135.81
4.80	24.25	158.56
5.40	26.80	172.88
5.64	27.58	174.49
5.82	28.03	172.88
6.00	28.27	162.21

72" PIPE
n= .024
LENGTH= 5 FT
SLOPE= .005 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= .78 FT
PEAK VELOCITY= 3.0 FPS
TRAVEL TIME = 0.0 MIN
SPAN= 0-24 HRS, dt=.1 HRS

JR 5/5/00
60/111

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POND 3 **Borrow Area Basin**

Qin = 141.1 CFS @ 12.32 HRS, VOLUME= 21.01 AF
Qout= 101.7 CFS @ 12.79 HRS, VOLUME= 15.36 AF, ATTEN= 28%, LAG= 28.2 MIN

ELEVATION (FT)	CON.AREA (AC)	INC.STOR (AF)	CUM.STOR (AF)	WET.AREA (AC)
569.8	.57	0.00	0.00	.57
570.3	.64	.30	.30	.64
571.0	.73	.48	.78	.73
572.0	.88	.80	1.59	.88
573.0	1.04	.96	2.54	1.04
574.0	1.23	1.13	3.68	1.23
575.0	1.44	1.33	5.01	1.44
576.0	1.49	1.46	6.48	1.50
577.0	1.69	1.59	8.07	1.70
577.5	1.79	.87	8.94	1.80

STOR-IND METHOD
PEAK STORAGE = 7.18 AF
PEAK ELEVATION= 576.4 FT
FLOOD ELEVATION= 577.5 FT
START ELEVATION= 569.8 FT
SPAN= 0-24 HRS, dt=.1 HRS
Tdet= 152.4 MIN (15.36 AF)

#	ROUTE	INVERT	OUTLET DEVICES
1	2	575.3'	**48" HORIZONTAL ORIFICE/GRATE X 2** Q=.6 Area SQR(2gH) (Limited to weir flow @ low head)
2	P	569.8'	**48" CULVERT X 2** n=.024 L=200' S=.004'/' Ke=.7 Cc=.9 Cd=.53

Primary Discharge
└─2=Culvert
  └─1=Orifice/Grate

POND 15

Qin = 47.92 CFS @ 12.39 HRS, VOLUME= 5.95 AF
Qout= 47.72 CFS @ 12.39 HRS, VOLUME= 5.95 AF, ATTEN= 0%, LAG= .1 MIN
Qpri= 4.82 CFS @ 12.39 HRS, VOLUME= 1.97 AF
Qsec= 42.90 CFS @ 12.39 HRS, VOLUME= 3.98 AF

ELEVATION (FT)	CON.AREA (AC)	INC.STOR (AF)	CUM.STOR (AF)	WET.AREA (AC)
587.4	0.00	0.00	0.00	0.00
589.0	.01	.01	.01	.01
590.1	.01	.01	.02	.01

STOR-IND METHOD
PEAK STORAGE = .01 AF
PEAK ELEVATION= 589.6 FT
FLOOD ELEVATION= 590.1 FT
START ELEVATION= 587.4 FT
SPAN= 0-24 HRS, dt=.1 HRS
Tdet= .4 MIN (5.92 AF)

#	ROUTE	INVERT	OUTLET DEVICES
1	P	587.4'	**15" CULVERT** n=.024 L=80' S=.012'/' Ke=.7 Cc=.9 Cd=.53
2	S	588.0'	**7' BROAD-CRESTED RECTANGULAR WEIR** Q=C L H^1.5 C=3, 3, 3, 3, 3, 3, 3, 3

000225

Primary Discharge
└─1=Culvert

Secondary Discharge
└─2=Broad-Crested Rectangular Weir

JR 5/5/00
RDF 5/5
61/111

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POND 41

Qin = 42.55 CFS @ 12.44 HRS, VOLUME= 3.98 AF
Qout= 41.93 CFS @ 12.50 HRS, VOLUME= 3.56 AF, ATTEN= 1%, LAG= 3.4 MIN

ELEVATION (FT)	CON.AREA (AC)	INC.STOR (AF)	CUM.STOR (AF)	WET.AREA (AC)
586.0	.05	0.00	0.00	.05
587.0	.17	.10	.10	.17
588.0	.48	.31	.42	.48
589.0	1.28	.85	1.26	1.28

STOR-IND METHOD
PEAK STORAGE = .64 AF
PEAK ELEVATION= 588.3 FT
FLOOD ELEVATION= 589.0 FT
START ELEVATION= 586.0 FT
SPAN= 0-24 HRS, dt=.1 HRS
Tdet= 53 MIN (3.56 AF)

#	ROUTE	INVERT	OUTLET DEVICES
1	P	588.0'	**100' BROAD-CRESTED RECTANGULAR WEIR**
			Q=C L H^1.5 C=3, 3, 3, 3, 3, 3, 3, 3

2983

ATTACHMENT C-1C

HydroCAD™ OUTPUT REPORT

DESIGN CASE A
25-YEAR, 24-HOUR STORM EVENT

JT 5/5/00
ROC 5/5
63/111

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RUNOFF BY SCS TR-20 METHOD: TYPE II 24-HOUR RAINFALL= 4.70 IN, SCS U.H.

RUNOFF SPAN = 0-24 HRS, dt= .10 HRS, 241 POINTS

SUBCAT NUMBER	AREA (ACRE)	Tc (MIN)	--GROUND COVERS (%CN)--	WGT'D CN	C	PEAK (CFS)	Tpeak (HRS)	VOL (AF)
100	2.28	3.4	100%94	94	-	14.57	11.88	.76

2983

Jr 5/5/00
60 5/5/0
64/111

REACH ROUTING BY STOR-IND+TRANS METHOD

REACH NO.	DIAM (IN)	BOTTOM WIDTH (FT)	DEPTH (FT)	SIDE SLOPES (FT/FT)		n	LENGTH (FT)	SLOPE (FT/FT)	PEAK VEL. (FPS)	TRAVEL TIME (MIN)	PEAK Qout (CFS)
100	-	-	3.0	.25	.33	.030	5	.0150	3.9	0.0	14.55

JT 5/5/00
RDG 5/5/00
65/111

2983

SUBCATCHMENT 100

PEAK= 14.57 CFS @ 11.88 HRS, VOLUME= .76 AF

ACRES	CN
2.28	94

SCS TR-20 METHOD
TYPE II 24-HOUR
RAINFALL= 4.70 IN
SPAN= 0-24 HRS, dt=.1 HRS

Method	Comment	Tc (min)
TR-55 SHEET FLOW	**Segment ID:**	1.1
n=.011 L=75' P2=2.6 in s=.02 '/'		
TR-55 SHEET FLOW	**Segment ID:**	.5
n=.011 L=90' P2=2.6 in s=.19 '/'		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	1.1
D=3' SS= .33 & .02 '/' a=238.64 sq-ft Pw=159.6' r=1.495'		
s=.01 '/' n=.022 V=8.83 fps L=590' Capacity=2107.6 cfs		
RECT/VEE/TRAP CHANNEL	**Segment ID:**	.7
D=1' SS= .25 & .33 '/' a=3.52 sq-ft Pw=7.3' r=.481'		
s=.015 '/' n=.03 V=3.72 fps L=165' Capacity=13.1 cfs		
	Total Length= 920 ft Total Tc=	3.4

JT 5/15/02
RDE 5/31/02
66/111

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REACH 100

Qin = 14.57 CFS @ 11.88 HRS, VOLUME= .76 AF
Qout= 14.55 CFS @ 11.88 HRS, VOLUME= .76 AF, ATTEN= 0%, LAG= 0.0 MIN

DEPTH (FT)	END AREA (SQ-FT)	DISCH (CFS)
0.00	0.00	0.00
.30	.32	.53
.60	1.27	3.35
.90	2.85	9.88
1.29	5.85	25.80
1.80	11.39	62.73
2.40	20.25	135.09
3.00	31.64	244.94

0' x 3' CHANNEL
S/S= .25 & .33 '/'
n= .03
LENGTH= 5 FT
SLOPE= .015 FT/FT

STOR-IND+TRANS METHOD
PEAK DEPTH= 1.01 FT
PEAK VELOCITY= 3.9 FPS
TRAVEL TIME = 0.0 MIN
SPAN= 0-24 HRS, dt=.1 HRS

ATTACHMENT C-2

WEIGHTED CURVE NUMBER CALCULATIONS

YC 5/5/00
ROC 5/5/00
68/111

2983

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
10	0.95	100	C	Runon Areas North of Borrow Area	79	79
31	4.60	100	D	Unvegetated Final Cover System	94	94
32	3.72	100	D	Unvegetated Final Cover System	94	94
33	1.55	15 85	C B	Disturbed Area - Construction Support Disturbed Area - Construction Support	82 74	75
35	2.35	100	C	Disturbed Area - Construction Support	82	82
36	4.26	80 20	C B	Disturbed Area - Construction Support Disturbed Area - Construction Support	82 74	80
37	1.85	100	C	Disturbed Area - Construction Support	82	82
38	4.64	100	C	Disturbed Area - Construction Support	82	82
39	4.33	100	C	Disturbed Area - Construction Support	82	82
40	3.03	100	C	Disturbed Area - Construction Support	82	82
41	1.45	100	C	Disturbed Area - Construction Support	82	82
42	2.29	100	N/A	Sedimentation Basin	98	98
43	0.81	100	C	Disturbed Area - Construction Support	82	82
44	1.04	100	C	Disturbed Area - Construction Support	82	82
45	3.58	100	C	Disturbed Area - Construction Support	82	82
46	2.83	100	C	Disturbed Area - Construction Support	82	82
47	1.64	100	C	Disturbed Area - Construction Support	82	82
48	1.94	100	C	Disturbed Area - Construction Support	82	82
60	1.30	100	C	Runon Areas North of Borrow Area	79	79
70	1.47	100	C	Runon Areas North of Borrow Area	79	79
80	0.80	100	C	Runon Areas North of Borrow Area	79	79
361	0.11	100	N/A	(Fictitious - for storage upstream of culvert) Sedimentation Basin	98	98
450	1.37	100	C	Disturbed Area - Construction Support	82	82

N/A - Not Applicable

JT 5/5/00

GeoSyntec Consultants

RDC 5/5/00

69/111

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RESULTS FOR THE CALCULATION OF WEIGHTED CN
Borrow Area Design Scenario

Subcatchment Label	Area (acres)	Percent of Total Area %	HSG	Land Use Category	CN	Weighted CN
1	3.22	30	B	Runon Areas East of OSDF	69	
		20	B	Runon Areas North of Borrow Area	69	
		30	C	Runon Areas East of OSDF	79	
		20	C	Runon Areas North of Borrow Area	79	74
2	8.64	50	B	Runon Areas East of OSDF	69	
		10	B	Runon Areas North of Borrow Area	69	
		20	C	Runon Areas East of OSDF	79	
		20	C	Runon Areas North of Borrow Area	79	73
3	21.99	60	B	Runon Areas East of OSDF	69	
		10	C	Runon Areas East of OSDF	79	
		30	C	Runon Areas North of Borrow Area	79	73
4	13.52	20	B	Runon Areas East of OSDF	69	
		10	C	Runon Areas East of OSDF	79	
		65	C	Runon Areas North of Borrow Area	79	
		5	B	Runon Areas North of Borrow Area	69	77
6	21.41	15	B	Runon Areas East of OSDF	69	
		35	C	Runon Areas North of Borrow Area	79	
		10	C	Disturbed Area - Construction Support	82	
		30	B	Runon Areas North of Borrow Area	69	
		10	B/C	Stockpile	87	76
8	1.81	50	B/C	Stockpile	87	
		50	C	Runon Areas North of Borrow Area	79	83
9	1.92	75	C	Runon Areas North of Borrow Area	79	
		10	B	Runon Areas North of Borrow Area	69	
		10	B	Vegetated Borrow Area	61	
		5	C	Vegetated Borrow Area	74	76
11	15.54	5	C	Runon Areas to Borrow Area	74	
		5	B	Runon Areas to Borrow Area	61	
		50	B	Vegetated Borrow Area	61	
		15	C	Vegetated Borrow Area	74	
		5	B	Runon Areas North of Borrow Area	69	
		5	C	Runon Areas North of Borrow Area	79	
		15	B/C	Stockpile	87	69
12	15.93	5	C	Runon Areas to Borrow Area	74	
		20	B	Runon Areas to Borrow Area	61	
		20	B/C	Stockpile	87	
		50	B	Vegetated Borrow Area	61	
		5	C	Vegetated Borrow Area	74	68
13	4.10	100	B	Vegetated Borrow Area	61	61
14	1.40	80	B	Vegetated Borrow Area	61	
		20	C	Runon Areas to Borrow Area	79	65
15	11.16	30	C	Unvegetated Borrow Area	91	
		70	B	Unvegetated Borrow Area	86	88
16	6.19	20	B	Runon Areas to Borrow Area	61	
		35	B	Vegetated Borrow Area	61	
		45	B/C	Stockpile	87	73
17	3.63	15	B	Runon Areas to Borrow Area	61	
		85	B	Vegetated Borrow Area	61	61
18	6.01	5	B	Runon Areas to Borrow Area	61	
		5	C	Runon Areas to Borrow Area	74	
		90	B	Vegetated Borrow Area	61	58
19	2.10	100	B	Vegetated Borrow Area	61	61
20	2.08	100	B	Vegetated Borrow Area	61	61
21	1.69	100	-	Sedimentation Basin	98	98
50	0.58	100	C	Runon Areas North of Borrow Area	79	79
161	1.48	100	B	Vegetated Borrow Area	61	61
201	0.20	60	B	Vegetated Borrow Area	61	
		40	C	Vegetated Borrow Area	74	66
Clay stockpile	15.72	80	B	Unvegetated Borrow Area	86	
		20	C	Unvegetated Borrow Area	91	87

N/A - Not Applicable

000234

JT 5/5/00

GeoSyntec Consultants

RDE 5/5/00

70/111

2983

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
100	2.28	100	D	Liner Runout	94	94

000235

EXAMPLE CALCULATION FOR WEIGHTED CURVE NUMBER OF SUBCATCHMENT 3

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 3 grouped according to Hydrological Soil Group (HSG), the contributing percentage of the total area for subcatchment area 3, and land use.

HSG	Percent of Area %	Land Use Description	CN
B	60	Runon Area east of OSDF	69
C	10	Runon Area east of OSDF	79
C	30	Runon Areas North of Borrow Area	79

CALCULATIONS

$$\text{Weighted CN} = (69)(0.60) + (79)(0.10) + (79)(0.30)$$

$$= 73$$

2983

ATTACHMENT C-3

ANALYSIS RESULTS FOR CHANNELS

SUMMARY OF CHANNEL ANALYSES RESULTS

Channel Identification			Channel Characteristics							Hydrologic Calculations		Hydraulic Calculations							
Channel Name[1]	Status	Design Scenario	Section Shape	Available Flow Depth (ft)	Longitudinal Slope[5] (%)	Manning n	Bottom Width B (ft)	Side Slope M_1:1	Side Slope M_2:1	HydroCAD Node	HydroCAD Q[2] (cfs)	Area of Flow A (sq ft)	Perimeter P (ft)	Hydraulic Radius, R (ft)	Peak Flow Depth[3] Y (ft)	Estimated Q[3] (cfs)	Channel Freeboard[6] (ft)	Peak Flow Velocity[7] (fps)	Lining[4] Type
1A	Existing	Borrow Area	Vee	1.6	0.77%	0.030	0	6	4	R1	1.6	0.99	4.54	0.22	0.44	1.56	1.16	-	-
1B	Existing	Borrow Area	Vee	2.3	0.67%	0.030	0	5	3	R1	1.6	0.99	4.11	0.24	0.50	1.56	1.80	-	-
1C	Existing	Borrow Area	Vee	2	0.45%	0.030	0	4	2	R1	1.6	1.08	3.82	0.28	0.60	1.56	1.40	-	-
1D	Existing	Borrow Area	Vee	2.8	0.81%	0.030	0	4	2	R1	1.6	0.87	3.42	0.25	0.54	1.56	2.26	-	-
2A	Existing	Borrow Area	Vee	4	0.67%	0.030	0	6	3	R2	6.2	2.85	7.36	0.39	0.80	6.17	3.20	-	-
2B	Existing	Borrow Area	Vee	3	1.00%	0.030	0	4	5	R2	6.2	2.45	6.81	0.36	0.74	6.17	2.26	-	-
2C	Existing	Borrow Area	Vee	4	1.50%	0.030	0	4	5	R2	6.2	2.11	6.31	0.33	0.68	6.17	3.32	-	-
4	Existing	Borrow Area	Vee	2	0.30%	0.030	0	16	3	R4	41.2	19.14	27.25	0.70	1.42	41.16	0.58	-	-
5	New	Borrow Area	Trapezoidal	2	0.50%	0.030	8	3	3	R5	42.6	13.31	15.33	0.87	1.16	42.55	0.84	3.2	Grass
6	Existing	Borrow Area	Trapezoidal	2	0.50%	0.030	10	3	3	R6	55.9	16.61	17.69	0.94	1.22	55.91	0.78	-	-
33A	Existing	OSDF	Vee	5	0.45%	0.030	0	6	4	R33	37.0	13.00	16.46	0.79	1.61	37.01	3.39	-	-
33B	Existing	OSDF	Vee	6.5	1.10%	0.030	0	5	3	R33	37.0	8.84	12.28	0.72	1.49	37.01	5.01	-	-
35	Existing	OSDF	Vee	7.4	1.10%	0.030	0	4	3	R35	40.7	9.22	11.83	0.78	1.62	40.70	5.78	-	-
37A	Existing	OSDF	Vee	1.5	1.50%	0.030	0	10	6	R37	5.0	2.07	8.21	0.25	0.51	5.03	0.99	-	-
39	Existing	OSDF	Vee	3	1.50%	0.030	0	4	3	R39	4.8	1.66	5.02	0.33	0.69	4.83	2.31	-	-
41	New	OSDF	Vee	2	0.75%	0.030	0	3	3	R41	5.5	2.28	5.52	0.41	0.87	5.46	1.13	2.4	Grass
43	Existing	OSDF	Vee	3.5	0.70%	0.030	0	10	4	R43	15.1	6.10	13.23	0.46	0.93	15.13	2.57	-	-
44	Existing	OSDF	Vee	2	2.00%	0.030	0	6	4	R44	21.8	4.99	10.19	0.49	1.00	21.74	1.00	-	-
46	Existing	OSDF	Vee	6	0.10%	0.030	0	3	3	R800	40.5	21.88	17.08	1.28	2.70	40.53	3.30	-	-
47	Existing	OSDF	Vee	2.5	1.00%	0.030	0	5	6	R47	6.6	2.69	7.81	0.34	0.70	6.55	1.80	-	-
48	Existing	OSDF	Vee	2.5	1.00%	0.030	0	5	6	R48	8.7	3.33	8.71	0.38	0.78	8.73	1.72	-	-
61	Existing	Borrow Area	Vee	3	1.50%	0.030	0	4	6	R61	4.8	1.79	6.10	0.29	0.60	4.80	2.40	-	-
80	Existing	OSDF	Vee	2.5	0.90%	0.030	0	3	3	R700	3.0	1.36	4.25	0.32	0.67	2.99	1.83	-	-
91	Existing	Borrow Area	Trapezoidal	2.5	0.30%	0.030	10	3	3	R91	68.8	22.98	19.89	1.16	1.56	68.82	0.94	-	-
92	Existing	Borrow Area	Trapezoidal	2.5	0.30%	0.030	10	3	3	R92	80.9	25.77	20.78	1.24	1.70	80.89	0.80	-	-
100	New	A	Vee	3	1.50%	0.030	0	4	3	R100	14.6	3.79	7.59	0.50	1.04	14.54	1.96	3.8	Grass
107	New	OSDF	Trapezoidal	2	1.00%	0.030	8	3	3	R107	24.1	7.07	12.43	0.57	0.70	24.09	1.30	3.4	Grass
352A	Existing	OSDF	Vee	7	1.30%	0.030	0	4	3	R352	39.6	8.48	11.34	0.75	1.56	39.55	5.44	-	-
352B	Existing	OSDF	Vee	3	0.83%	0.030	0	3	4	R352	39.6	10.03	12.33	0.81	1.69	39.55	1.31	-	-
451	Existing	OSDF	Vee	4	1.40%	0.030	0	3	3	R451	25.1	5.68	8.70	0.65	1.38	25.11	2.62	-	-
452	Existing	OSDF	Vee	5	0.40%	0.030	0	3	3	R452	31.2	10.70	11.94	0.90	1.89	31.22	3.11	-	-
453	Existing	OSDF	Vee	1.5	1.50%	0.030	0	6	6	R453	10.0	3.22	8.92	0.36	0.73	9.95	0.77	-	-
161	Existing	Borrow Area	Irregular	1.5	0.20%	0.030	20	2	20	R161	1.3	2.54	22.65	0.11	0.12	1.31	1.38	-	-
162A	New	Borrow Area	Vee	1.5	4.60%	0.030	0	5	40	R162	1.1	0.57	7.20	0.08	0.16	1.13	1.34	2.0	Grass
162B	New	Borrow Area	Vee	1	2.30%	0.030	0	5	50	R162	1.1	0.78	9.28	0.08	0.17	1.13	0.83	1.4	Grass
162C	New	Borrow Area	Vee	1	1.30%	0.030	0	10	40	R162	1.1	0.94	9.73	0.10	0.19	1.13	0.81	1.2	Grass
162D	New	Borrow Area	Vee	2.3	1.20%	0.030	0	3	3	R162	1.1	0.59	2.80	0.21	0.44	1.13	1.86	1.9	Grass
162E	New	Borrow Area	Vee	5	13.50%	0.030	0	3	3	R162	1.1	0.24	1.78	0.13	0.28	1.13	4.72	4.8	Grass
163A	New	Borrow Area	Vee	4	0.70%	0.030	0	3	3	R163	9.4	3.53	6.86	0.51	1.08	9.41	2.92	2.7	Grass
163B	New	Borrow Area	Vee	4	0.75%	0.030	0	3	3	R163	9.4	3.44	6.77	0.51	1.07	9.41	2.93	2.7	Grass
201	New	Borrow Area	Vee	3.5	0.75%	0.030	0	3	3	R201	28.2	7.83	10.22	0.77	1.62	28.21	1.88	3.6	Grass
202	New	Borrow Area	Vee	4	0.50%	0.030	0	3	3	R202	55.8	15.20	14.24	1.07	2.25	55.76	1.75	3.7	Grass
17A	Existing	Borrow Area	Irregular	1.5	0.25%	0.030	40	3	20	R17	3.1	5.12	42.87	0.12	0.12	3.09	1.38	-	-
17B	Existing	Borrow Area	Irregular	1.5	6.00%	0.030	0	10	40	R17	3.1	1.13	10.65	0.11	0.21	3.09	1.29	-	-

1. Letters are used to further subdivide a nodal reach to take into consideration geometric variations along the reach.
2. Peak flow rates calculated by HydroCAD for each reach. Included as attachment C-1A, C-1B, or C-1C.
3. Calculated flow rates using an iterative procedure and compared with flow from Hydrocad.
4. Maximum permissible velocity is 5fps for grass lined channels (evaluated only for new channels).
5. Longitudinal slopes taken from the Drawings.
6. Calculated as the difference between minimum available flow depth and peak flow depth.
7. Existing channels evaluated only for hydraulic capacity.

Written by: J. TAM. Date: 2000/4/6 Reviewed by: RDE Date: 00/05/05

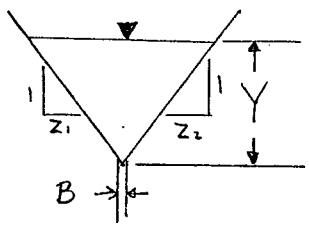
Client: Fluor Fernald, Inc Project: OSDF - PHASE 3 Project/Proposal No.: GQ1001 Task No.: 03

EXAMPLE CALCULATION FOR ANALYSIS OF CHANNEL 41

2983

Peak Discharge (25-year, 24-hour storm event), Q_{max} = 45.5 cfs
Base Width, B = 0.00 ft
Left Side Slope, M_1 = 3 Horizontal : 1 Vertical
Right Side Slope, M_2 = 3 Horizontal : 1 Vertical
Manning's Roughness Coefficient, n = 0.030
Longitudinal channel Slope, S_o = 0.0075 ft/ft.

Channel Geometry



Assume Depth of Flow $Y = 0.87$ ft

AREA OF FLOW

$$A = \frac{1}{2} Y \left(2B + (Z_1 Y + Z_2 Y)\right)$$

$$= YB + Z_1 \frac{Y^2}{2} + Z_2 \frac{Y^2}{2}$$

$$= (0.87\,ft)\,0.00\,ft + 3 \times \frac{(0.87\,ft)^2}{2} + 3 \times \frac{(0.87\,ft)^2}{2}$$

$$= 2.27$$

$$= \underline{\underline{2.3\ ft^2}}$$

Written by: J. TAM Date: 2000 / 4 / 6 (YY MM DD) Reviewed by: _____ Date: __/__/__ (YY MM DD)

Client: Fluor Fernald, Inc. Project: OSDF - Phase 3 Project/Proposal No.: GQ1001 Task No.: 03

WETTED PERIMETER

$$P = B + \sqrt{(Y)^2 + (Z_1 Y)^2} + \sqrt{(Y)^2 + (Z_2 Y)^2}$$

2983

$$= 0 + \sqrt{(0.87\,ft)^2 + (3 \times 0.87\,ft)^2} + \sqrt{(0.87\,ft)^2 + (3 \times 0.87\,ft)^2}$$

$$= 2.736 + 2.736$$

$$= 5.47$$

$$= \underline{\underline{5.5\ ft}}$$

HYDRAULIC RADIUS

$$R = \frac{A}{P}$$

$$= \frac{2.27\ ft^2}{5.47\ ft}$$

$$= 0.42\,ft$$

$$= \underline{\underline{0.4\,ft}}$$

AVERAGE VELOCITY

$$V = \frac{Q}{A} = \frac{\frac{1.49}{n} \cdot A \times R^{2/3} \times S_o^{1/2}}{A} = \frac{1.49}{n} \times R^{2/3} \times S_o^{1/2}$$

$$= \frac{1.49}{0.030} \times (0.42\,ft)^{2/3} \times (0.0075)^{1/2}$$

$$= 2.41\ ft/s$$

$$= \underline{\underline{2.4\ ft/s}}$$

DISCHARGE (Verification)

$$Q_{max} = A \times V$$

$$= 2.27\,ft^2 \times 2.4\,ft/s$$

$$= 5.45$$

$$= \underline{\underline{5.5\ cfs}}$$

2983

ATTACHMENT C-4A

Culvertmaster® OUTPUT REPORTS FOR CULVERTS

JC 5/5/00
RDE 5/5/00
77/111

Culvert Designer/Analyzer Report
Culvert1

2983

Design:Trial-1

Solve For: Headwater Elevation

Culvert Summary			
Allowable HW Elevation	612.00 ft	Storm Event	Design
Computed Headwater Elevation	610.50 ft	Discharge	2.34 cfs
Headwater Depth/ Height	0.72	Tailwater Elevation	N/A ft
Inlet Control HW Elev	610.47 ft	Control Type	Entrance Control
Outlet Control HW Elev	610.50 ft		

Grades			
Upstream Invert	609.60 ft	Downstream Invert	609.00 ft
Length	80.00 ft	Constructed Slope	0.007500 ft/ft

Hydraulic Profile			
Profile	S2	Depth, Downstream	0.49 ft
Slope Type	Steep	Normal Depth	0.49 ft
Flow Regime	Supercritical	Critical Depth	0.61 ft
Velocity Downstream	5.28 ft/s	Critical Slope	0.003345 ft/ft

Section			
Section Shape	Circular	Mannings Coefficient	0.010
Section Material	PVC	Span	1.25 ft
Section Size	15 inch	Rise	1.25 ft
Number Sections	1		

Outlet Control Properties			
Outlet Control HW Elev	610.50 ft	Upstream Velocity Head	0.24 ft
Ke	0.20	Entrance Loss	0.05 ft

Inlet Control Properties			
Inlet Control HW Elev	610.47 ft	Flow Control	N/A
Inlet Type	Groove end w/headwall	Area Full	1.2 ft^2
K	0.00780	HDS 5 Chart	1
M	2.00000	HDS 5 Scale	2
C	0.02920	Equation Form	1
Y	0.74000		

000242

JT 5/5/00
ROE 5/5/00
78/111

Culvert Designer/Analyzer Report
Culvert2

2983

Design:Trial-1

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	607.60	ft	Storm Event	Design	
Computed Headwater Elevation	604.63	ft	Discharge	2.32	cfs
Headwater Depth/ Height	0.56		Tailwater Elevation	N/A	ft
Inlet Control HW Elev	604.60	ft	Control Type	Entrance Control	
Outlet Control HW Elev	604.63	ft			

Grades					
Upstream Invert	603.80	ft	Downstream Invert	603.30	ft
Length	50.00	ft	Constructed Slope	0.010000	ft/ft

Hydraulic Profile					
Profile	S2		Depth, Downstream	0.43	ft
Slope Type	Steep		Normal Depth	0.42	ft
Flow Regime	Supercritical		Critical Depth	0.58	ft
Velocity Downstream	5.63	ft/s	Critical Slope	0.002949	ft/ft

Section					
Section Shape	Circular		Mannings Coefficient	0.010	
Section Material	PVC		Span	1.50	ft
Section Size	18 inch		Rise	1.50	ft
Number Sections	1				

Outlet Control Properties					
Outlet Control HW Elev	604.63	ft	Upstream Velocity Head	0.21	ft
Ke	0.20		Entrance Loss	0.04	ft

Inlet Control Properties					
Inlet Control HW Elev	604.60	ft	Flow Control	N/A	
Inlet Type	Groove end w/headwall		Area Full	1.8	ft^2
K	0.00780		HDS 5 Chart	1	
M	2.00000		HDS 5 Scale	2	
C	0.02920		Equation Form	1	
Y	0.74000				

000243

JT 5/5/00
KOO 5/5/00
79/111

Culvert Designer/Analyzer Report
Culvert3a

2983

Design:Trial-1

Solve For: Headwater Elevation

Culvert Summary				
Allowable HW Elevation	595.30 ft	Storm Event	Design	
Computed Headwater Elevation	594.44 ft	Discharge	18.00 cfs	
Headwater Depth/ Height	3.13	Tailwater Elevation	N/A ft	
Inlet Control HW Elev	594.07 ft	Control Type	Outlet Control	
Outlet Control HW Elev	594.44 ft			

Grades			
Upstream Invert	589.75 ft	Downstream Invert	589.07 ft
Length	66.50 ft	Constructed Slope	0.010226 ft/ft

Hydraulic Profile			
Profile	CompositeM2Pressure	Depth, Downstream	1.45 ft
Slope Type	Mild	Normal Depth	N/A ft
Flow Regime	Subcritical	Critical Depth	1.45 ft
Velocity Downstream	10.29 ft/s	Critical Slope	0.025777 ft/ft

Section			
Section Shape	Circular	Mannings Coefficient	0.013
Section Material	Concrete	Span	1.50 ft
Section Size	18 inch	Rise	1.50 ft
Number Sections	1		

Outlet Control Properties			
Outlet Control HW Elev	594.44 ft	Upstream Velocity Head	1.61 ft
Ke	0.20	Entrance Loss	0.32 ft

Inlet Control Properties			
Inlet Control HW Elev	594.07 ft	Flow Control	N/A
Inlet Type	Groove end projecting	Area Full	1.8 ft²
K	0.00450	HDS 5 Chart	1
M	2.00000	HDS 5 Scale	3
C	0.03170	Equation Form	1
Y	0.69000		

000244

JC 5/5/00
RDE 5/5/00
80/11

Culvert Designer/Analyzer Report
Culvert3b

Design:Trial-1

2983

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	595.30	ft	Storm Event	Design	
Computed Headwater Elevation	593.25	ft	Discharge	29.24	cfs
Headwater Depth/ Height	1.40		Tailwater Elevation	N/A	ft
Inlet Control HW Elev	593.11	ft	Control Type	Outlet Control	
Outlet Control HW Elev	593.25	ft			

Grades					
Upstream Invert	589.75	ft	Downstream Invert	589.00	ft
Length	65.00	ft	Constructed Slope	0.011538	ft/ft

Hydraulic Profile					
Profile	M2		Depth, Downstream	1.84	ft
Slope Type	Mild		Normal Depth	N/A	ft
Flow Regime	Subcritical		Critical Depth	1.84	ft
Velocity Downstream	7.53	ft/s	Critical Slope	0.021679	ft/ft

Section					
Section Shape	Circular		Mannings Coefficient	0.024	
Section Material	CMP		Span	2.50	ft
Section Size	30 inch		Rise	2.50	ft
Number Sections	1				

Outlet Control Properties					
Outlet Control HW Elev	593.25	ft	Upstream Velocity Head	0.56	ft
Ke	0.90		Entrance Loss	0.50	ft

Inlet Control Properties					
Inlet Control HW Elev	593.11	ft	Flow Control	N/A	
Inlet Type	Projecting		Area Full	4.9	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				

000245

JM 5/5/00
ROE 5/5/00
81/111

2983

Culvert Designer/Analyzer Report
Culvert4

Design:Trial-1

Solve For: Headwater Elevation

Culvert Summary				
Allowable HW Elevation	589.00	ft	Storm Event	Design
Computed Headwater Elevation	588.63	ft	Discharge	11.72 cfs
Headwater Depth/ Height	1.07		Tailwater Elevation	586.40 ft
Inlet Control HW Elev	588.53	ft	Control Type	Outlet Control
Outlet Control HW Elev	588.63	ft		

Grades				
Upstream Invert	586.50	ft	Downstream Invert	586.00 ft
Length	44.00	ft	Constructed Slope	0.011364 ft/ft

Hydraulic Profile				
Profile	M2		Depth, Downstream	1.23 ft
Slope Type	Mild		Normal Depth	1.48 ft
Flow Regime	Subcritical		Critical Depth	1.23 ft
Velocity Downstream	5.79	ft/s	Critical Slope	0.018836 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	588.63	ft	Upstream Velocity Head	0.35 ft
Ke	0.90		Entrance Loss	0.32 ft

Inlet Control Properties				
Inlet Control HW Elev	588.53	ft	Flow Control	N/A
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			

000246

JR 5/5/00
RDE 5/5/00
82/111

Culvert Designer/Analyzer Report
Culvert5

2983

Design:Trial-1

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	593.50	ft	Storm Event	Design	
Computed Headwater Elevation	588.14	ft	Discharge	62.30	cfs
Headwater Depth/ Height	0.71		Tailwater Elevation	586.40	ft
Inlet Control HW Elev	587.73	ft	Control Type	Entrance Control	
Outlet Control HW Elev	588.14	ft			

Grades					
Upstream Invert	585.30	ft	Downstream Invert	583.00	ft
Length	114.00	ft	Constructed Slope	0.020175	ft/ft

Hydraulic Profile					
Profile	CompositeS1S2		Depth, Downstream	3.40	ft
Slope Type	Steep		Normal Depth	1.45	ft
Flow Regime	N/A		Critical Depth	1.66	ft
Velocity Downstream	2.74	ft/s	Critical Slope	0.012435	ft/ft

Section					
Section Shape	Circular		Mannings Coefficient	0.024	
Section Material	CMP		Span	4.00	ft
Section Size	48 inch		Rise	4.00	ft
Number Sections	2				

Outlet Control Properties					
Outlet Control HW Elev	588.14	ft	Upstream Velocity Head	0.62	ft
Ke	0.90		Entrance Loss	0.56	ft

Inlet Control Properties					
Inlet Control HW Elev	587.73	ft	Flow Control	N/A	
Inlet Type	Projecting		Area Full	25.1	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				

000247

JT 5/5/00

RDE 5/5/00

83/111

Culvert Designer/Analyzer Report
Culvert6

2983

Design:Trial-1

Solve For: Headwater Elevation

Culvert Summary				
Allowable HW Elevation	590.50 ft	Storm Event	Design	
Computed Headwater Elevation	589.05 ft	Discharge	5.03 cfs	
Headwater Depth/ Height	1.58	Tailwater Elevation	N/A ft	
Inlet Control HW Elev	589.05 ft	Control Type	Inlet Control	
Outlet Control HW Elev	588.90 ft			

Grades			
Upstream Invert	587.60 ft	Downstream Invert	586.60 ft
Length	70.00 ft	Constructed Slope	0.014286 ft/ft

Hydraulic Profile			
Profile	S2	Depth, Downstream	0.92 ft
Slope Type	Steep	Normal Depth	1.01 ft
Flow Regime	Supercritical	Critical Depth	0.68 ft
Velocity Downstream	4.30 ft/s	Critical Slope	0.012854 ft/ft

Section			
Section Shape	Arch	Mannings Coefficient	0.018
Section Material	Concrete	Span	1.50 ft
Section Size	18.0 x 11.0 inch	Rise	0.92 ft
Number Sections	1		

Outlet Control Properties			
Outlet Control HW Elev	588.90 ft	Upstream Velocity Head	0.41 ft
Ke	0.50	Entrance Loss	0.20 ft

Inlet Control Properties			
Inlet Control HW Elev	589.05 ft	Flow Control	N/A
Inlet Type	Square edge w/headwall	Area Full	1.1 ft^2
K	0.00980	HDS 5 Chart	0
M	2.00000	HDS 5 Scale	0
C	0.03980	Equation Form	1
Y	0.67000		

000248

JC 5/5/01

RDE 5/5/01
84/111

Culvert Designer/Analyzer Report
Culvert7

2983

Design:Trial-1

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	590.00	ft	Storm Event	Design	
Computed Headwater Elevation	588.73	ft	Discharge	4.82	cfs
Headwater Depth/ Height	1.51		Tailwater Elevation	N/A	ft
Inlet Control HW Elev	588.73	ft	Control Type	Inlet Control	
Outlet Control HW Elev	588.66	ft			

Grades					
Upstream Invert	587.35	ft	Downstream Invert	586.37	ft
Length	80.00	ft	Constructed Slope	0.012250	ft/ft

Hydraulic Profile					
Profile	M2		Depth, Downstream	0.67	ft
Slope Type	Mild		Normal Depth	0.92	ft
Flow Regime	Subcritical		Critical Depth	0.67	ft
Velocity Downstream	5.02	ft/s	Critical Slope	0.012277	ft/ft

Section					
Section Shape	Arch		Mannings Coefficient	0.018	
Section Material	Concrete		Span	1.50	ft
Section Size	18.0 x 11.0 inch		Rise	0.92	ft
Number Sections	1				

Outlet Control Properties					
Outlet Control HW Elev	588.66	ft	Upstream Velocity Head	0.26	ft
Ke	0.50		Entrance Loss	0.13	ft

Inlet Control Properties					
Inlet Control HW Elev	588.73	ft	Flow Control	N/A	
Inlet Type	Square edge w/headwall		Area Full	1.1	ft^2
K	0.00980		HDS 5 Chart	0	
M	2.00000		HDS 5 Scale	0	
C	0.03980		Equation Form	1	
Y	0.67000				

000249

JC 5/5/10

ROE 5/5/10
85/111

Culvert Designer/Analyzer Report
Culvert8

2983

Design:Trial-1

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	585.50	ft	Storm Event	Design	
Computed Headwater Elevation	582.79	ft	Discharge	7.51	cfs
Headwater Depth/ Height	0.95		Tailwater Elevation	N/A	ft
Inlet Control HW Elev	582.79	ft	Control Type	Inlet Control	
Outlet Control HW Elev	582.78	ft			

Grades					
Upstream Invert	581.60	ft	Downstream Invert	581.30	ft
Length	55.00	ft	Constructed Slope	0.005455	ft/ft

Hydraulic Profile					
Profile	S2		Depth, Downstream	0.70	ft
Slope Type	Steep		Normal Depth	0.70	ft
Flow Regime	Supercritical		Critical Depth	0.78	ft
Velocity Downstream	5.29	ft/s	Critical Slope	0.003891	ft/ft

Section					
Section Shape	Circular		Mannings Coefficient	0.010	
Section Material	PVC		Span	1.25	ft
Section Size	15 inch		Rise	1.25	ft
Number Sections	2				

Outlet Control Properties					
Outlet Control HW Elev	582.78	ft	Upstream Velocity Head	0.33	ft
Ke	0.20		Entrance Loss	0.07	ft

Inlet Control Properties					
Inlet Control HW Elev	582.79	ft	Flow Control	N/A	
Inlet Type	Groove end w/headwall		Area Full	2.5	ft^2
K	0.00780		HDS 5 Chart	1	
M	2.00000		HDS 5 Scale	2	
C	0.02920		Equation Form	1	
Y	0.74000				

000250

JL 5/5/00
RDE 5/5/00
86/111

Culvert Designer/Analyzer Report
Culvert9

2983

Design:Trial-1

Solve For: Headwater Elevation

Culvert Summary				
Allowable HW Elevation	580.00 ft		Storm Event	Design
Computed Headwater Elevation	576.89 ft		Discharge	25.32 cfs
Headwater Depth/ Height	0.67		Tailwater Elevation	0.00 ft
Inlet Control HW Elev	576.80 ft		Control Type	Entrance Control
Outlet Control HW Elev	576.89 ft			

Grades				
Upstream Invert	574.98 ft		Downstream Invert	573.50 ft
Length	296.00 ft		Constructed Slope	0.005000 ft/ft

Hydraulic Profile				
Profile	S2		Depth, Downstream	1.18 ft
Slope Type	Steep		Normal Depth	1.18 ft
Flow Regime	Supercritical		Critical Depth	1.31 ft
Velocity Downstream	6.52 ft/s		Critical Slope	0.003359 ft/ft

Section				
Section Shape	Horizontal Ellipse		Mannings Coefficient	0.013
Section Material	Concrete		Span	4.44 ft
Section Size	34x53 inch		Rise	2.83 ft
Number Sections	1			

Outlet Control Properties				
Outlet Control HW Elev	576.89 ft		Upstream Velocity Head	0.50 ft
Ke	0.20		Entrance Loss	0.10 ft

Inlet Control Properties				
Inlet Control HW Elev	576.80 ft		Flow Control	N/A
Inlet Type	Groove end with headwall		Area Full	10.2 ft²
K	0.00180		HDS 5 Chart	29
M	2.50000		HDS 5 Scale	2
C	0.02920		Equation Form	1
Y	0.74000			

000251

JT 5/5/00

ROC 5/5/00
87/111

Culvert Designer/Analyzer Report
Culvert10

Design:Trial-1

Solve For: Headwater Elevation

2983

Culvert Summary				
Allowable HW Elevation	584.00 ft		Storm Event	Design
Computed Headwater Elevation	583.53 ft		Discharge	5.59 cfs
Headwater Depth/ Height	2.22		Tailwater Elevation	N/A ft
Inlet Control HW Elev	582.95 ft		Control Type	Outlet Control
Outlet Control HW Elev	583.53 ft			

Grades				
Upstream Invert	581.50 ft		Downstream Invert	580.50 ft
Length	55.00 ft		Constructed Slope	0.018182 ft/ft

Hydraulic Profile				
Profile	CompositeM2Pressure		Depth, Downstream	0.72 ft
Slope Type	Mild		Normal Depth	N/A ft
Flow Regime	Subcritical		Critical Depth	0.72 ft
Velocity Downstream	5.46 ft/s		Critical Slope	0.025957 ft/ft

Section				
Section Shape	Arch		Mannings Coefficient	0.024
Section Material	Concrete		Span	1.50 ft
Section Size	18.0 x 11.0 inch		Rise	0.92 ft
Number Sections	1			

Outlet Control Properties				
Outlet Control HW Elev	583.53 ft		Upstream Velocity Head	0.41 ft
Ke	0.20		Entrance Loss	0.08 ft

Inlet Control Properties				
Inlet Control HW Elev	582.95 ft		Flow Control	N/A
Inlet Type	Groove end projecting		Area Full	1.1 ft^2
K	0.00450		HDS 5 Chart	0
M	2.00000		HDS 5 Scale	0
C	0.03170		Equation Form	1
Y	0.69000			

000252

JL 5/5/00

RDE 5/5/00

88/111

Culvert Designer/Analyzer Report
culvert11a

2983

Design:Trial-1

Solve For: Headwater Elevation

Culvert Summary				
Allowable HW Elevation	586.50 ft	Storm Event	Design	
Computed Headwater Elevation	585.94 ft	Discharge	40.00 cfs	
Headwater Depth/ Height	1.33	Tailwater Elevation	0.00 ft	
Inlet Control HW Elev	585.94 ft	Control Type	Inlet Control	
Outlet Control HW Elev	585.90 ft			

Grades				
Upstream Invert	583.28 ft	Downstream Invert	583.10 ft	
Length	40.00 ft	Constructed Slope	0.004500 ft/ft	

Hydraulic Profile				
Profile	M2	Depth, Downstream	1.61 ft	
Slope Type	Mild	Normal Depth	1.67 ft	
Flow Regime	Subcritical	Critical Depth	1.61 ft	
Velocity Downstream	7.40 ft/s	Critical Slope	0.004806 ft/ft	

Section				
Section Shape	Circular	Mannings Coefficient	0.010	
Section Material	PVC	Span	2.00 ft	
Section Size	24 inch	Rise	2.00 ft	
Number Sections	2			

Outlet Control Properties				
Outlet Control HW Elev	585.90 ft	Upstream Velocity Head	0.80 ft	
Ke	0.20	Entrance Loss	0.16 ft	

Inlet Control Properties				
Inlet Control HW Elev	585.94 ft	Flow Control	N/A	
Inlet Type	Groove end projecting	Area Full	6.3 ft^2	
K	0.00450	HDS 5 Chart	1	
M	2.00000	HDS 5 Scale	3	
C	0.03170	Equation Form	1	
Y	0.69000			

000253

YT 5/5/00

RDC 5/5/00
89/111

Culvert Designer/Analyzer Report
culvert11b

Design:Trial-2

Solve For: Headwater Elevation

2983

Culvert Summary					
Allowable HW Elevation	586.50	ft	Storm Event	Design	
Computed Headwater Elevation	585.42	ft	Discharge	17.87	cfs
Headwater Depth/ Height	0.85		Tailwater Elevation	0.00	ft
Inlet Control HW Elev	585.35	ft	Control Type	Entrance Control	
Outlet Control HW Elev	585.42	ft			

Grades					
Upstream Invert	583.28	ft	Downstream Invert	583.00	ft
Length	49.00	ft	Constructed Slope	0.005714	ft/ft

Hydraulic Profile					
Profile	S2		Depth, Downstream	1.36	ft
Slope Type	Steep		Normal Depth	1.36	ft
Flow Regime	Supercritical		Critical Depth	1.43	ft
Velocity Downstream	6.54	ft/s	Critical Slope	0.004865	ft/ft

Section					
Section Shape	Circular		Mannings Coefficient	0.013	
Section Material	Concrete		Span	2.50	ft
Section Size	30 inch		Rise	2.50	ft
Number Sections	1				

Outlet Control Properties					
Outlet Control HW Elev	585.42	ft	Upstream Velocity Head	0.59	ft
Ke	0.20		Entrance Loss	0.12	ft

Inlet Control Properties					
Inlet Control HW Elev	585.35	ft	Flow Control	Unsubmerged	
Inlet Type	Groove end projecting		Area Full	4.9	ft^2
K	0.00450		HDS 5 Chart	1	
M	2.00000		HDS 5 Scale	3	
C	0.03170		Equation Form	1	
Y	0.69000				

000254

JT 5/5/00
ROE 5/5/00
90/111

Culvert Designer/Analyzer Report
Culvert12

2983

Design:Trial-1

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	577.00	ft	Storm Event	Design	
Computed Headwater Elevation	574.49	ft	Discharge	42.40	cfs
Headwater Depth/ Height	0.95		Tailwater Elevation	N/A	ft
Inlet Control HW Elev	574.49	ft	Control Type	Inlet Control	
Outlet Control HW Elev	574.48	ft			

Grades					
Upstream Invert	572.12	ft	Downstream Invert	572.00	ft
Length	30.00	ft	Constructed Slope	0.004000	ft/ft

Hydraulic Profile					
Profile	M2		Depth, Downstream	1.56	ft
Slope Type	Mild		Normal Depth	1.72	ft
Flow Regime	Subcritical		Critical Depth	1.56	ft
Velocity Downstream	6.56	ft/s	Critical Slope	0.005210	ft/ft

Section					
Section Shape	Circular		Mannings Coefficient	0.013	
Section Material	Concrete		Span	2.50	ft
Section Size	30 inch		Rise	2.50	ft
Number Sections	2				

Outlet Control Properties					
Outlet Control HW Elev	574.48	ft	Upstream Velocity Head	0.57	ft
Ke	0.20		Entrance Loss	0.11	ft

Inlet Control Properties					
Inlet Control HW Elev	574.49	ft	Flow Control	N/A	
Inlet Type	Groove end w/headwall		Area Full	9.8	ft^2
K	0.00780		HDS 5 Chart	1	
M	2.00000		HDS 5 Scale	2	
C	0.02920		Equation Form	1	
Y	0.74000				

000255

JR 5/5/00

RDE 5/5/00
91/111

Culvert Designer/Analyzer Report
Culvert13

Design:Trial-1

2983

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	587.00	ft	Storm Event	Design	
Computed Headwater Elevation	584.86	ft	Discharge	7.20	cfs
Headwater Depth/ Height	0.71		Tailwater Elevation	N/A	ft
Inlet Control HW Elev	584.82	ft	Control Type	Entrance Control	
Outlet Control HW Elev	584.86	ft			

Grades					
Upstream Invert	583.80	ft	Downstream Invert	583.50	ft
Length	80.00	ft	Constructed Slope	0.003750	ft/ft

Hydraulic Profile					
Profile	S2		Depth, Downstream	0.69	ft
Slope Type	Steep		Normal Depth	0.69	ft
Flow Regime	Supercritical		Critical Depth	0.72	ft
Velocity Downstream	4.55	ft/s	Critical Slope	0.003130	ft/ft

Section					
Section Shape	Circular		Mannings Coefficient	0.010	
Section Material	PVC		Span	1.50	ft
Section Size	18 inch		Rise	1.50	ft
Number Sections	2				

Outlet Control Properties					
Outlet Control HW Elev	584.86	ft	Upstream Velocity Head	0.28	ft
Ke	0.20		Entrance Loss	0.06	ft

Inlet Control Properties					
Inlet Control HW Elev	584.82	ft	Flow Control	N/A	
Inlet Type	Groove end projecting		Area Full	3.5	ft^2
K	0.00450		HDS 5 Chart	1	
M	2.00000		HDS 5 Scale	3	
C	0.03170		Equation Form	1	
Y	0.69000				

000256

JT 5/5/00

ROE 5/5/00
92/111

Culvert Designer/Analyzer Report
Culvert14

2983

Design:Trial-1

Solve For: Headwater Elevation

Culvert Summary				
Allowable HW Elevation	587.50 ft	Storm Event	Design	
Computed Headwater Elevation	586.41 ft	Discharge	20.40 cfs	
Headwater Depth/ Height	1.35	Tailwater Elevation	N/A ft	
Inlet Control HW Elev	586.41 ft	Control Type	Inlet Control	
Outlet Control HW Elev	586.36 ft			

Grades			
Upstream Invert	583.70 ft	Downstream Invert	583.00 ft
Length	80.00 ft	Constructed Slope	0.008750 ft/ft

Hydraulic Profile			
Profile	S2	Depth, Downstream	1.33 ft
Slope Type	Steep	Normal Depth	1.28 ft
Flow Regime	Supercritical	Critical Depth	1.62 ft
Velocity Downstream	9.16 ft/s	Critical Slope	0.004913 ft/ft

Section			
Section Shape	Circular	Mannings Coefficient	0.010
Section Material	PVC	Span	2.00 ft
Section Size	24 inch	Rise	2.00 ft
Number Sections	1		

Outlet Control Properties			
Outlet Control HW Elev	586.36 ft	Upstream Velocity Head	0.87 ft
Ke	0.20	Entrance Loss	0.17 ft

Inlet Control Properties			
Inlet Control HW Elev	586.41 ft	Flow Control	N/A
Inlet Type	Groove end projecting	Area Full	3.1 ft²
K	0.00450	HDS 5 Chart	1
M	2.00000	HDS 5 Scale	3
C	0.03170	Equation Form	1
Y	0.69000		

000257

JT 5/1/00

ROE 5/5/00
93/111

Culvert Designer/Analyzer Report
Culvert15

2983

Design:Trial-1

Solve For: Headwater Elevation

Culvert Summary				
Allowable HW Elevation	581.90 ft	Storm Event	Design	
Computed Headwater Elevation	579.28 ft	Discharge	3.30 cfs	
Headwater Depth/ Height	0.26	Tailwater Elevation	N/A ft	
Inlet Control HW Elev	579.11 ft	Control Type	Entrance Control	
Outlet Control HW Elev	579.28 ft			

Grades			
Upstream Invert	578.60 ft	Downstream Invert	575.00 ft
Length	112.00 ft	Constructed Slope	0.032143 ft/ft

Hydraulic Profile			
Profile	S2	Depth, Downstream	0.32 ft
Slope Type	Steep	Normal Depth	0.32 ft
Flow Regime	Supercritical	Critical Depth	0.40 ft
Velocity Downstream	4.08 ft/s	Critical Slope	0.013316 ft/ft

Section			
Section Shape	Arch	Mannings Coefficient	0.024
Section Material	Aluminized CR 3x1 Corrugations Historic	Span	4.17 ft
Section Size	50 x 31 inch	Rise	2.58 ft
Number Sections	1		

Outlet Control Properties			
Outlet Control HW Elev	579.28 ft	Upstream Velocity Head	0.15 ft
Ke	0.90	Entrance Loss	0.13 ft

Inlet Control Properties			
Inlet Control HW Elev	579.11 ft	Flow Control	N/A
Inlet Type	18" CR, projecting	Area Full	8.4 ft²
K	0.03400	HDS 5 Chart	'34
M	1.50000	HDS 5 Scale	3
C	0.04960	Equation Form	1
Y	0.57000		

000258

JI 5/5/00
RJE 5/5/00
94/111

Culvert Designer/Analyzer Report
Culvert16

2983

Design:Trial-1

Solve For: Headwater Elevation

Culvert Summary				
Allowable HW Elevation	576.50 ft	Storm Event	Design	
Computed Headwater Elevation	572.84 ft	Discharge	5.80 cfs	
Headwater Depth/ Height	0.33	Tailwater Elevation	N/A ft	
Inlet Control HW Elev	572.70 ft	Control Type	Entrance Control	
Outlet Control HW Elev	572.84 ft			

Grades			
Upstream Invert	572.00 ft	Downstream Invert	569.20 ft
Length	75.00 ft	Constructed Slope	0.037333 ft/ft

Hydraulic Profile			
Profile	S2	Depth, Downstream	0.41 ft
Slope Type	Steep	Normal Depth	0.41 ft
Flow Regime	Supercritical	Critical Depth	0.54 ft
Velocity Downstream	5.20 ft/s	Critical Slope	0.012407 ft/ft

Section			
Section Shape	Arch	Mannings Coefficient	0.024
Section Material	Aluminized Corrugations Historic	Span	4.17 ft
Section Size	50 x 31 inch	Rise	2.58 ft
Number Sections	1		

Outlet Control Properties			
Outlet Control HW Elev	572.84 ft	Upstream Velocity Head	0.20 ft
Ke	0.50	Entrance Loss	0.10 ft

Inlet Control Properties			
Inlet Control HW Elev	572.70 ft	Flow Control	N/A
Inlet Type	18" CR, 90 ° headwall	Area Full	8.4 ft²
K	0.00830	HDS 5 Chart	34
M	2.00000	HDS 5 Scale	1
C	0.03790	Equation Form	1
Y	0.69000		

000259

JC 5/5/00

RDE 5/5/00
95/111

2983

Culvert Designer/Analyzer Report
Culvert17

Design:Trial-1

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	578.00	ft	Storm Event	Design	
Computed Headwater Elevation	576.02	ft	Discharge	50.90	cfs
Headwater Depth/ Height	0.69		Tailwater Elevation	N/A	ft
Inlet Control HW Elev	575.74	ft	Control Type	Entrance Control	
Outlet Control HW Elev	576.02	ft			

Grades					
Upstream Invert	573.50	ft	Downstream Invert	566.90	ft
Length	600.00	ft	Constructed Slope	0.011000	ft/ft

Hydraulic Profile					
Profile	S2		Depth, Downstream	1.51	ft
Slope Type	Steep		Normal Depth	1.51	ft
Flow Regime	Supercritical		Critical Depth	1.53	ft
Velocity Downstream	6.58	ft/s	Critical Slope	0.010590	ft/ft

Section					
Section Shape	Arch		Mannings Coefficient	0.024	
Section Material	Aluminized CR 3x1 Corrugations Historic		Span	6.00	ft
Section Size	72 x 44 inch		Rise	3.67	ft
Number Sections	1				

Outlet Control Properties					
Outlet Control HW Elev	576.02	ft	Upstream Velocity Head	0.66	ft
Ke	0.50		Entrance Loss	0.33	ft

Inlet Control Properties					
Inlet Control HW Elev	575.74	ft	Flow Control	N/A	
Inlet Type	18" CR, 90 ° headwall		Area Full	17.2	ft²
K	0.00830		HDS 5 Chart	34	
M	2.00000		HDS 5 Scale	1	
C	0.03790		Equation Form	1	
Y	0.69000				

000200

JA 5/5/00
RDE 5/5/00
96/111

Culvert Designer/Analyzer Report
Culvert18

2983

Design:Trial-1

Solve For: Headwater Elevation

Culvert Summary				
Allowable HW Elevation	581.50 ft		Storm Event	Design
Computed Headwater Elevation	590.16 ft		Discharge	10.00 cfs
Headwater Depth/ Height	11.16		Tailwater Elevation	N/A ft
Inlet Control HW Elev	584.47 ft		Control Type	Outlet Control
Outlet Control HW Elev	590.16 ft			

Grades				
Upstream Invert	579.00 ft		Downstream Invert	577.00 ft
Length	116.00 ft		Constructed Slope	0.017241 ft/ft

Hydraulic Profile				
Profile	CompositeM2Pressure		Depth, Downstream	0.99 ft
Slope Type	Mild		Normal Depth	N/A ft
Flow Regime	Subcritical		Critical Depth	0.99 ft
Velocity Downstream	12.74 ft/s		Critical Slope	0.073796 ft/ft

Section				
Section Shape	Circular		Mannings Coefficient	0.013
Section Material	Concrete		Span	1.00 ft
Section Size	12 inch		Rise	1.00 ft
Number Sections	1			

Outlet Control Properties				
Outlet Control HW Elev	590.16 ft		Upstream Velocity Head	2.52 ft
Ke	0.20		Entrance Loss	0.50 ft

Inlet Control Properties				
Inlet Control HW Elev	584.47 ft		Flow Control	N/A
Inlet Type	Groove end w/headwall		Area Full	0.8 ft²
K	0.00780		HDS 5 Chart	1
M	2.00000		HDS 5 Scale	2
C	0.02920		Equation Form	1
Y	0.74000			

000261

JA 5/5/00
RAE 5/5/00
97/111

Culvert Designer/Analyzer Report
Culvert19

2983

Design:Trial-1

Solve For: Headwater Elevation

Culvert Summary				
Allowable HW Elevation	586.00 ft	Storm Event	Design	
Computed Headwater Elevation	583.71 ft	Discharge	15.50 cfs	
Headwater Depth/ Height	0.85	Tailwater Elevation	N/A ft	
Inlet Control HW Elev	583.53 ft	Control Type	Outlet Control	
Outlet Control HW Elev	583.71 ft			

Grades			
Upstream Invert	582.00 ft	Downstream Invert	581.70 ft
Length	60.00 ft	Constructed Slope	0.005000 ft/ft

Hydraulic Profile			
Profile	M2	Depth, Downstream	0.99 ft
Slope Type	Mild	Normal Depth	1.48 ft
Flow Regime	Subcritical	Critical Depth	0.99 ft
Velocity Downstream	5.00 ft/s	Critical Slope	0.016542 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	2		

Outlet Control Properties			
Outlet Control HW Elev	583.71 ft	Upstream Velocity Head	0.18 ft
Ke	0.90	Entrance Loss	0.16 ft

Inlet Control Properties			
Inlet Control HW Elev	583.53 ft	Flow Control	N/A
Inlet Type	Projecting	Area Full	6.3 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		

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2983

Culvert Designer/Analyzer Report
Culvert20

Design:Trial-1

Solve For: Headwater Elevation

Culvert Summary				
Allowable HW Elevation	587.00 ft	Storm Event	Design	
Computed Headwater Elevation	584.41 ft	Discharge	15.50 cfs	
Headwater Depth/ Height	1.14	Tailwater Elevation	N/A ft	
Inlet Control HW Elev	584.41 ft	Control Type	Inlet Control	
Outlet Control HW Elev	584.38 ft			

Grades			
Upstream Invert	582.70 ft	Downstream Invert	581.50 ft
Length	60.00 ft	Constructed Slope	0.020000 ft/ft

Hydraulic Profile			
Profile	S2	Depth, Downstream	0.71 ft
Slope Type	Steep	Normal Depth	0.66 ft
Flow Regime	Supercritical	Critical Depth	1.08 ft
Velocity Downstream	9.43 ft/s	Critical Slope	0.004287 ft/ft

Section			
Section Shape	Circular	Mannings Coefficient	0.010
Section Material	PVC	Span	1.50 ft
Section Size	18 inch	Rise	1.50 ft
Number Sections	2		

Outlet Control Properties			
Outlet Control HW Elev	584.38 ft	Upstream Velocity Head	0.50 ft
Ke	0.20	Entrance Loss	0.10 ft

Inlet Control Properties			
Inlet Control HW Elev	584.41 ft	Flow Control	N/A
Inlet Type	Groove end w/headwall	Area Full	3.5 ft²
K	0.00780	HDS 5 Chart	1
M	2.00000	HDS 5 Scale	2
C	0.02920	Equation Form	1
Y	0.74000		

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Culvert Designer/Analyzer Report
Culvert21

Design:Trial-1

Solve For: Headwater Elevation

2983

Culvert Summary					
Allowable HW Elevation	590.00	ft	Storm Event	Design	
Computed Headwater Elevation	586.99	ft	Discharge	82.90	cfs
Headwater Depth/ Height	1.08		Tailwater Elevation	585.90	ft
Inlet Control HW Elev	586.44	ft	Control Type	Outlet Control	
Outlet Control HW Elev	586.99	ft			

Grades					
Upstream Invert	583.20	ft	Downstream Invert	583.00	ft
Length	84.00	ft	Constructed Slope	0.002381	ft/ft

Hydraulic Profile					
Profile	M2		Depth, Downstream	2.90	ft
Slope Type	Mild		Normal Depth	N/A	ft
Flow Regime	Subcritical		Critical Depth	2.00	ft
Velocity Downstream	4.86	ft/s	Critical Slope	0.014824	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	586.99	ft	Upstream Velocity Head	0.31	ft
Ke	0.90		Entrance Loss	0.28	ft

Inlet Control Properties					
Inlet Control HW Elev	586.44	ft	Flow Control	N/A	
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				

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Culvert Designer/Analyzer Report
Culvert22

Design:Trial-1

Solve For: Headwater Elevation

2983

Culvert Summary			
Allowable HW Elevation	589.40 ft	Storm Event	Design
Computed Headwater Elevation	585.03 ft	Discharge	8.80 cfs
Headwater Depth/ Height	0.92	Tailwater Elevation	N/A ft
Inlet Control HW Elev	584.86 ft	Control Type	Outlet Control
Outlet Control HW Elev	585.03 ft		

Grades			
Upstream Invert	583.20 ft	Downstream Invert	582.70 ft
Length	80.50 ft	Constructed Slope	0.006211 ft/ft

Hydraulic Profile			
Profile	M2	Depth, Downstream	1.06 ft
Slope Type	Mild	Normal Depth	1.50 ft
Flow Regime	Subcritical	Critical Depth	1.06 ft
Velocity Downstream	5.21 ft/s	Critical Slope	0.017045 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	585.03 ft	Upstream Velocity Head	0.20 ft
Ke	0.90	Entrance Loss	0.18 ft

Inlet Control Properties			
Inlet Control HW Elev	584.86 ft	Flow Control	N/A
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		

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Culvert Designer/Analyzer Report
Culvert24

Design:Trial-1

Solve For: Headwater Elevation

2983

Culvert Summary					
Allowable HW Elevation	590.00	ft	Storm Event	Design	
Computed Headwater Elevation	588.00	ft	Discharge	5.50	cfs
Headwater Depth/ Height	1.07		Tailwater Elevation	586.44	ft
Inlet Control HW Elev	587.87	ft	Control Type	Outlet Control	
Outlet Control HW Elev	588.00	ft			

Grades					
Upstream Invert	586.39	ft	Downstream Invert	586.11	ft
Length	38.00	ft	Constructed Slope	0.007368	ft/ft

Hydraulic Profile					
Profile	M2		Depth, Downstream	0.90	ft
Slope Type	Mild		Normal Depth	N/A	ft
Flow Regime	Subcritical		Critical Depth	0.90	ft
Velocity Downstream	4.94	ft/s	Critical Slope	0.020415	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	588.00	ft	Upstream Velocity Head	0.19	ft
Ke	0.90		Entrance Loss	0.17	ft

Inlet Control Properties					
Inlet Control HW Elev	587.87	ft	Flow Control	N/A	
Inlet Type	Projecting		Area Full	1.8	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				

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Culvert Designer/Analyzer Report
Culvert25

2983

Design:Trial-1

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	590.00	ft	Storm Event	Design	
Computed Headwater Elevation	589.13	ft	Discharge	14.60	cfs
Headwater Depth/ Height	1.23		Tailwater Elevation	586.40	ft
Inlet Control HW Elev	589.05	ft	Control Type	Outlet Control	
Outlet Control HW Elev	589.13	ft			

Grades					
Upstream Invert	586.66	ft	Downstream Invert	586.11	ft
Length	36.70	ft	Constructed Slope	0.014986	ft/ft

Hydraulic Profile					
Profile	M2		Depth, Downstream	1.38	ft
Slope Type	Mild		Normal Depth	1.59	ft
Flow Regime	Subcritical		Critical Depth	1.38	ft
Velocity Downstream	6.33	ft/s	Critical Slope	0.021158	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	589.13	ft	Upstream Velocity Head	0.47	ft
Ke	0.90		Entrance Loss	0.42	ft

Inlet Control Properties					
Inlet Control HW Elev	589.05	ft	Flow Control	N/A	
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				

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Culvert Designer/Analyzer Report
Culvert27

2983

Design:Trial-1

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	583.00	ft	Storm Event	Design	
Computed Headwater Elevation	581.49	ft	Discharge	28.60	cfs
Headwater Depth/ Height	1.47		Tailwater Elevation	N/A	ft
Inlet Control HW Elev	581.11	ft	Control Type	Outlet Control	
Outlet Control HW Elev	581.49	ft			

Grades					
Upstream Invert	577.81	ft	Downstream Invert	577.34	ft
Length	62.50	ft	Constructed Slope	0.007520	ft/ft

Hydraulic Profile					
Profile	CompositeM2Pressure		Depth, Downstream	1.82	ft
Slope Type	Mild		Normal Depth	N/A	ft
Flow Regime	Subcritical		Critical Depth	1.82	ft
Velocity Downstream	7.46	ft/s	Critical Slope	0.021302	ft/ft

Section					
Section Shape	Circular		Mannings Coefficient	0.024	
Section Material	CMP		Span	2.50	ft
Section Size	30 inch		Rise	2.50	ft
Number Sections	1				

Outlet Control Properties					
Outlet Control HW Elev	581.49	ft	Upstream Velocity Head	0.53	ft
Ke	0.90		Entrance Loss	0.47	ft

Inlet Control Properties					
Inlet Control HW Elev	581.11	ft	Flow Control	N/A	
Inlet Type	Projecting		Area Full	4.9	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				

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ATTACHMENT C-4B

ANALYSIS RESULTS FOR CULVERTS

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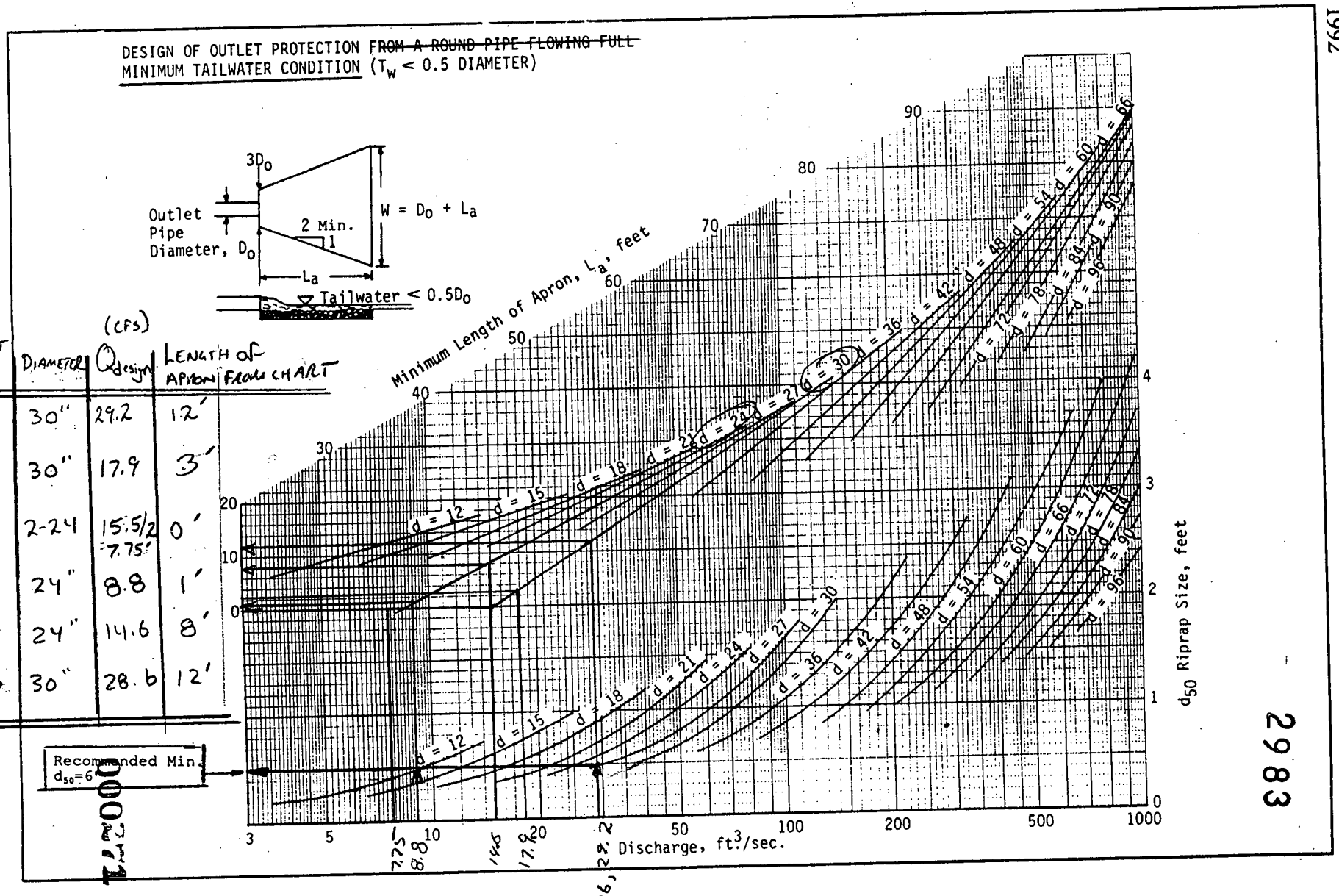
SUMMARY OF CULVERT ANALYSES RESULTS

CULVERT IDENTIFICATION			PHYSICAL CHARACTERISTICS OF CULVERT						CULVERT PROFILE				CULVERTMASTER® MODELING CHARACTERISTICS			HYDRAULIC CAPACITY					STRUCTURAL CAPACITY				OUTLET PROTECTION			
Culvert Name	Status (1)(2)	Design Scenario	Material - Type	Entrance Loss Coefficient (Ke)	Manning's n	Entrance Configuration	Number of Culverts - Diameter	Approximate Length (ft)	Inlet Invert Elevation (ft MSL)	Outlet Invert Elevation (ft MSL)	Slope (%)	Overtopping Elevation (ft MSL)	Entrance Configuration(4)	Entrance Loss Coefficient (Ke)	Number of Culverts - Diameter (4)	HydroCad Node / Peak Flow Rate (cfs)	Calculated Freeboard (ft)	Tailwater Elevation (6) (ft MSL)	Calculated Headwater Depth - Inlet Control (9) (ft MSL)	Calculated Headwater Depth - Outlet Control (9) (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)
1	Existing	Borrow Area	HDPE	0.2	0.01	Flared End Section	1 - 14 in.	80.0	609.60	609.00	0.75	612.0	Groove - Headwall	0.2	1 - 15 in.	R50/2.3	1.5	N/A	610.47	610.50								
2	Existing	Borrow Area	HDPE	0.2	0.01	Flared End Section	1 - 21 in.	50.0	603.80	603.30	1.00	607.6	Groove - Headwall	0.2	1 - 18 in.	R1/2.3	3.0	N/A	604.60	604.63								
3a	Existing	OSDF	RCP	0.2	0.013	Projecting	1 - 18 in.	66.5	589.75	589.07	1.02	595.3	Groove - Projecting	0.2	1 - 18 in.	R101/18.0 (5)	0.8	N/A	594.07	594.44								
3b	**New**	**OSDF**	**CMP**	**0.9**	**0.024**	**Projecting**	**1 - 30 in.**	**65.0**	**589.75**	**589.00**	**1.15**	**595.3**	**Projecting**	**0.9**	**1 - 30 in.**	**R101/29.2 (5)**	**2.0**	**N/A**	**593.11**	**593.25**	**3.2**	**Off-Highway**	**3**	**Yes**	**7.5**	**5**	**12**	**6**
4	Existing	OSDF	CMP	0.9	0.024	Projecting (3)	1 - 24 in.	44.0	586.50	586.00	1.14	589.0	Projecting	0.9	1 - 24 in.	R104/11.7	0.4	586.4 (7)	588.53	588.63								
5	Existing	OSDF	CMP	0.9	0.024	Projecting	2 - 48 in.	114.0	585.30	583	2.02	593.5	Projecting	0.9	2 - 48 in.	P36/62.3	5.4	586.4 (7)	587.73	588.14								
6	Existing	OSDF	CMPA - Arch	0.5	0.018	Flared End Section	13 in. x 17 in.	70.0	587.6	586.6	1.43	590.5	Headwall	0.5	11 in. x 18 in.	R102/5.0	1.5	N/A	589.05	588.90								
7	Existing	Borrow Area	CMPA - Arch	0.5	0.018	Flared End Section	13 in. x 17 in.	80.0	587.35	586.37	1.23	590.0	Headwall	0.5	11 in. x 18 in.	R61/4.8	1.3	N/A	588.73	588.66								
8	Existing	OSDF	PVC	0.2	0.01	Concrete Headwall	2 - 15 in.	55.0	581.60	581.30	0.55	585.5	Groove - Headwall	0.2	2 - 15 in.	R106/7.5	2.7	N/A	582.79	582.78								
9	Existing	OSDF	RCP - Ellipse, Horizonta	0.2	0.013	Concrete Headwall	34 in. x 53 in.	296.0	574.98	573.50	0.50	580.0	Groove - Headwall	0.2	34 in. x 53 in.	R108/25.3	3.1	N/A	576.80	576.89								
10	Existing	OSDF	CMP - Arch	0.2	0.024	Projecting (3)	13 in. x 17 in.	55.0	581.50	580.50	1.82	584.0	Groove - Projecting	0.2	11 in. x 18 in.	R110/5.6	0.5	N/A	582.95	583.53								
11a	Existing	Borrow Area	HDPE	0.2	0.01	Projecting	2 - 27 in.	40.0	583.28	583.10	0.45	587.1	Groove - Projecting	0.2	2 - 24 in.	R6/40.0 (5)	1.2	N/A	585.94	585.90								
11b	**New**	**Borrow Area**	**RCP**	**0.2**	**0.013**	**Projecting**	**1 - 30 in.**	**49.0**	**583.28**	**583.00**	**0.57**	**587.1**	**Groove - Projecting**	**0.2**	**1 - 30 in.**	**R6/17.9 (5)**	**1.7**	**N/A**	**585.35**	**585.42**	**0.7**	**Off-Highway**	**Note 12**	**Yes**	**6.5**	**5**	**10**	**6**
12	Existing	OSDF	RCP	0.2	0.013	Concrete Headwall	2 - 28 in.	30.0	572.12	572.00	0.40	577.0	Groove - Headwall	0.2	2 - 30 in.	R109/42.4	2.5	N/A	574.49	574.48								
13	Existing	Borrow Area	HDPE	0.2	0.01	Projecting	2 - 18 in.	80.0	583.80	583.50	0.38	587.0	Groove - Projecting	0.2	2 - 18 in.	S8/7.2	2.1	N/A	584.82	584.86								
14	Existing	Borrow Area	HDPE	0.2	0.01	Projecting	1 - 27 in.	80.0	583.70	583.00	0.88	587.5	Groove - Projecting	0.2	1 - 24 in.	S11/20.4	1.1	N/A	586.41	586.36								
15	Existing	OSDF	CMP - Arch	0.9	0.024	Projecting	50 in. x 31 in.	112.0	578.60	575.00	3.21	581.9	Projecting	0.9	50 in. x 31 in.	R113/3.3	2.6	N/A	579.11	579.28								
16	Existing	OSDF	CMP - Arch	0.5	0.024	Concrete Headwall	50 in. x 31 in.	75.0	572.00	569.20	3.73	576.5	Headwall	0.5	50 in. x 31 in.	R700/5.8	3.7	N/A	572.70	572.84								
17	Existing	OSDF	CMP - Arch	0.5	0.024	Concrete Headwall	72 in. x 44 in.	600.0	573.47	566.90	1.10	578.0	Headwall	0.5	72 in. x 44 in.	R900/50.9	2.0	N/A	575.74	576.02								
18	Existing	OSDF	RCP	0.2	0.013	Concrete Headwall	1 - 12 in.	116.0	579.00	577.00	1.72	581.5	Groove - Headwall	0.2	1 - 12 in.	R112/10.0	8.7	N/A	584.47	590.16								
19	**New**	**OSDF**	**CMP**	**0.9**	**0.024**	**Projecting**	**2 - 24 in.**	**60.0**	**582.00**	**581.70**	**0.50**	**586.0**	**Projecting**	**0.9**	**2 - 24 in.**	**R43/15.5**	**2.3**	**N/A**	**583.53**	**583.71**	**3 (10)**	**Off-Highway**	**3**	**Yes**	**5.0**	**4**	**8**	**6**
20	Existing	OSDF	PVC	0.2	0.01	Concrete Headwall	2 - 18 in.	60.0	582.70	581.50	2.00	587.0	Groove - Headwall	0.2	2 - 18 in.	R105/15.5	2.6	N/A	584.41	584.38								
21	**New**	**OSDF**	**CMP**	**0.9**	**0.024**	**Projecting**	**2 - 42 in.**	**84.0**	**583.20**	**583.00**	**0.24**	**590.0**	**Projecting**	**0.9**	**2 - 42 in.**	**P1/82.9**	**3.0**	**585.9 (8)**	**586.44**	**586.99**	**3.3 (11)**	**Off-Highway**	**3**	**Yes**	**4.9**	**0**	**0**	**0**
22	**New**	**OSDF**	**CMP**	**0.9**	**0.024**	**Projecting**	**1 - 24 in.**	**80.5**	**583.20**	**582.70**	**0.62**	**589.4**	**Projecting**	**0.9**	**1 - 24 in.**	**R48/8.8**	**4.4**	**N/A**	**584.86**	**585.03**	**4.2 (11)**	**Off-Highway**	**3**	**Yes**	**5.2**	**5**	**8**	**6**
24	Existing	OSDF	CMP	0.9	0.024	Projecting	1 - 18 in.	38.0	586.39	586.11	0.74	590.0	Projecting	0.9	1 - 18 in.	R41/5.5	2.0	586.4 (7)	587.87	588.00								
25	**New**	**A**	**CMP**	**0.9**	**0.024**	**Projecting**	**1 - 24 in.**	**36.7**	**586.66**	**586.11**	**1.50**	**591.0**	**Projecting**	**0.9**	**1 - 24 in.**	**R100/14.6**	**1.9**	**586.4 (7)**	**589.05**	**589.13**	**2.3**	**On-Highway**	**2**	**Yes**	**6.3**	**4**	**8**	**6**
26	Existing	OSDF	HDPE	0.9	0.01	Projecting	1 - 18 in.	55.0	583.90	583.60	0.55	585.0	N/A	0.9	N/A	No Flow	N/A	N/A	N/A	N/A								
27	**New**	**Borrow Area**	**CMP**	**0.9**	**0.024**	**Projecting**	**1 - 30 in.**	**62.5**	**577.81**	**577.34**	**0.75**	**583.7**	**Projecting**	**0.9**	**1 - 30 in.**	**R15/28.6**	**2.2**	**N/A**	**581.11**	**581.49**	**3.0**	**Off-Highway**	**3**	**Yes**	**7.5**	**6**	**12**	**6**

- CMPA = Asphalted Corrugated Metal Pipe(s)
- CMP = Corrugated Metal Pipe(s)
- RCP = Reinforced Concrete Pipe(s)
- HDPE = High Density Polyethylene
- PVC = Polyvinyl Chloride
- S = Subcatchment
- R = Reach
- P = Pond
- N/A = Not Applicable
- Groove - Headwall = Groove end with headwall
- Groove - Projecting = Groove end projecting
- d_{50} = Average particle diameter

Notes
(1) Existing indicates an existing culvert to be left in place.
(2) New indicates a new culvert to be installed.
(3) Entrance configuration assumed.
(4) Dimensions and entrance configurations used in the Culvertmaster software package were selected to match existing culvert characteristics as closely as possible.
(5) Combined flow rates through pipes (a) and (b) represent HydroCAD design flow rate.
(6) Tailwater depth accounted for only in cases where culvert outlet impacted by water level in a sedimentation basin.
(7) Tailwater elevation represents headwater elevation of Culvert 21 connecting North and South ends of OSDF Basin 1.
(8) Corresponds to peak water level in OSDF Basin 1 for 25-year, 24-hour storm event.
(9) Headwater depths calculated using Culvertmaster software package. Summary output presented for each culvert in Attachment C-4A.
(10) Culvert will be placed along an existing channel by others
(11) Culvert will be placed under existing road by others
(12) See calculations provided in this Attachment.
(13) Culvert 21 connects the north and south portions of OSDF Sedimentation Basin #1, no outlet protection is recommended.

OUTLET PROTECTION FOR NEW CULVERTS



CULVERT NAME	DIAMETER	Q_{design} (CFS)	LENGTH OF APRON FROM CHART
36	30"	29.2	12'
11b	30"	17.9	3'
19	2-24	15.5/2 = 7.75'	0'
22	24"	8.8	1'
25	24"	14.6	8'
27	30"	28.6	12'

Source: USDA-SCS

Plate 3.18-3

1992

3.18

2983

106/111

901

2/5/92

Written by: DAN PASS Date: 2000/5/4 Reviewed by: RDB Date: 00/05/05

Client: FF Project: OSDF PHASE III Project/Proposal No.: GQ1001 Task No.: 04

2983

CULVERT 11b STRUCTURAL STABILITY - RCP

STEP 1: CALCULATION OF EARTH LOAD, W_E

$$W_E = 135\,pcf \times 1\,ft \times 3.08\,ft$$

$$= 415.80\ lb/LF$$

STEP 2: CALCULATION OF LIVE LOAD ACTING ON CULVERT, W_L

LENGTH AND WIDTH OF TIRE FOOTPRINT

$$A_t = 0.01 \times 95{,}000\,lb/2$$

$$= 475.00\ in^2 = 3.299\ ft^2$$

$$l_F = \sqrt{\frac{3.299\,ft^2}{2.5}}$$

$$= 1.149\ ft$$

$$W_F = 1.149\,ft \times 2.5$$

$$= 2.873\ ft$$

PRESSURE FROM LIVE LOAD ACTING AT HORIZONTAL PLANE AT LEVEL OF THE CULVERT CROWN, ω

$$L_{LL} = 2.873\,ft + 1.75 \times 1\,ft$$

$$= 4.623\ ft$$

$$W_{LL} = 1.149\,ft + 1.75 \times 1\,ft$$

$$= 2.899\ ft$$

$$A_{LL} = 4.623\,ft \times 2.899\,ft$$

$$= 13.40\,ft^2$$

$$\omega = \left(\frac{1 + 0.5}{13.40\,ft^2}\right) \times 95{,}000\,lb/2$$

$$= 5{,}317.2\ psf$$

000272

JT 5/5/00

Written by: DAN PASS Date 2000 / 5 / 4 (YY MM DD) Reviewed by: RDE Date: 00 / 05 / 05 (YY MM DD)

Client: FF Project: OSDF PHASE III Project/Proposal No.: GQ1001 Task No.: 04

2983

TOTAL LIVE LOAD ACTING ON PIPE, W_T

$$W_T = 5{,}317.2\ psf \times 4.623\ ft \times 2.899\ ft$$

$$= 71{,}261.5\ lb$$

LIVE LOAD ACTING ON CULVERT, W_L

$$L_e = 4.623\ ft + 1.75\ \frac{(3 \times 3.08\ ft)}{4}$$

$$= 8.666\ ft$$

$$W_L = \frac{71{,}261.5\ lb}{8.666\ ft}$$

$$= 8{,}223.1\ lb/LF$$

STEP 3: SELECTION OF BEDDING FACTOR, B_F

SELECTED AS 1.9 (SEE PROCEDURES SECTION OF CALCULATION PACKAGE)

STEP 4: SELECTION OF CULVERT CLASS

$$D_{0.01} = \left(\frac{8{,}223.1\ lb/LF + 415.80\ lb/LF}{1.9 \times 2.5\ ft} \right) 1.0$$

$$= 1819\ lb/LF$$

FROM ASTM C 76 FOR CLASS IV RCP D-LOAD = 2000 lb/LF

1819 lb/LF < 2000 lb/ft

THUS, SELECT CLASS IV WALL B RCP CULVERT

ATTACHMENT C-5

ANALYSIS RESULTS FOR BASIN OUTLET STRUCTURES

ITEM		BORROW AREA BASIN		
		DESCRIPTION	PARAMETER VALUE IN UNITS SHOWN	
PRINCIPAL SPILLWAY	Riser Pipe	Twin 48" CMP	ELEVATION (ft MSL)	575.3
	Outlet Pipe	Twin 48" CMP		569.8
EMERGENCY SPILLWAY		NONE		N/A
EMBANKMENT		Cut to existing ground surface		577.5
AVAILABLE STORAGE VOLUME (TO RISER INLET)		To primary spillway riser pipe inlet	VOLUME (acre-ft)	5.5
SUBCATCHMENT		Total Upstream Drainage area	AREA (acres)	130.0

N/A - Not Applicable

BORROW AREA BASIN ROUTING RESULTS

DESIGN PARAMETER	Value
25-YEAR 24-HOUR PEAK WATER ELEVATION (ft MSL)	576.4
REQUIRED STORAGE VOLUME BASED ON DRAINAGE AREA (acre-ft)	5.4

Notes

1. Minimum Basin volume of 5.5 acre-ft exceeds disturbed area-based volume of 5.4 acre-ft.
2. A freeboard greater than 1ft is maintained for the 25-yr, 24-hour storm event.
3. Calculation for volume based on drainage area.

(1800 cf/acre-year) x (acre-ft/43,560 ft^3) x (130.0 acres of upstream drainage area) x (1 year/cleanout) = 5.4 acre-ft/cleanout

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ITEM		OSDF BASIN 1		
		DESCRIPTION	PARAMETER VALUE IN UNITS SHOWN	
PRINCIPAL SPILLWAY	Riser Pipe	Twin 48" CMP	ELEVATION (ft MSL)	585.8
	Outlet Pipe	Twin 36" CMP		580.1
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)	6.5
SUBCATCHMENT		Total Upstream Drainage area	AREA (acres)	28.0

OSDF BASIN 1 ROUTING RESULTS

DESIGN PARAMETER	Value
10-YEAR 24-HOUR RUNOFF VOLUME (acre-ft)	6.3
25-YEAR 24-HOUR PEAK WATER ELEVATION (ft MSL)	585.9
100-YEAR 24-HOUR PEAK WATER ELEVATION (ft MSL)	586.0
REQUIRED STORAGE VOLUME BASED ON DRAINAGE AREA (acre-ft)	3.5

Notes

1. Minimum Sedimentation Basin volume of 6.5 acre-ft exceeds runoff volume of 6.3 acre-ft and disturbed area-based volume of 3.5 acre-ft.
2. Flow does not enter emergency spillway for the 25-year, 24-hour storm event.
3. A freeboard greater than 1ft 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 (28.0 acres of upstream drainage area) x (1 year/cleanout) = 3.5 acre-ft/cleanout

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15. BORROW AREA

15.6 Surface-Water Management During Development
15.7 Borrow Area Development Subarea Volume Verification
15.8 Borrow Area Stockpile Area Capacity Calculation

Written by: WHN Date: 00 / 05 / 12 Reviewed by: Vaner Linde Date: 00 / 05 / 15

Client: Fluor Fernald, Inc. Project: OSDF-Phase III Proj./Proposal No.: GQ1001 Task No.: 04

ADDENDUM TO SECTION 15.1

(Added To Revision 1A Package)

Borrow Area required volume calculations performed in this Section are based on construction of eight (8) OSDF cells. The maximum size of the OSDF has been reduced to seven (7) cells. The updated strategy for developing, managing, and restoring the Borrow Area to support construction/filling/closure of 7 OSDF cells is presented in the document prepared by U. S. Department of Energy (DOE) titled "*On-Site Disposal Facility Borrow Area Strategy Report, Revision 0*", Fernald Environmental Management Project, DOE Fernald Area Office, Fernald, OH, 2000. In the Borrow Area Strategy Report, the Borrow Area required volume for construction of 7 OSDF cells is presented.

Written by: WHN Date: 00 / 05 / 12 Reviewed by: Vander Linde Date: 00 / 05 / 15

Client: Fluor Fernald, Inc. Project: OSDF-Phase III Proj./Proposal No.: GQ1001 Task No.: 04

ADDENDUM TO SECTION 15.2

(Added To Revision 1A Package)

Borrow Area capacity verification calculations presented in this Section were prepared for the earthworks components of eight (8) OSDF cells. The maximum size of the OSDF has been reduced to seven (7) cells. The updated strategy for developing, managing, and restoring the Borrow Area to support construction/filling/closure of 7 OSDF cells is presented in the document prepared by U. S. Department of Energy (DOE) titled, *"On-Site Disposal Facility Borrow Area Strategy Report, Revision 0"*, Fernald Environmental Management Project, (DOE) Fernald Area Office, Fernald, OH, 2000. The Borrow Area subareas to support construction/filling/closure of 7 OSDF cells are presented in the document prepared by GeoSyntec Consultants titled, *"Borrow Area Management and Restoration Plan, On-Site Disposal Facility, Revision 1D"*, Fernald Environmental Management Project, DOE Fernald Area Office, Fernald, OH, 2000.

Borrow area capacity verification calculations for construction/filing/closure of 7 OSDF cells and based on the updated strategy presented in the Borrow Area Strategy Report are presented in Section 15.7 of the OSDF Calculation Package, entitled *"Borrow Area Development Subarea Volume Verification"*. Section 15.7 is presented in Volume V of the OSDF Final Design Calculation Package, Revision 1A.

15.6 Surface-Water Management During Development

GEOSYNTEC CONSULTANTS

Written by: DGP Date: 00 / 05 / 09 Reviewed by: Vander Linde Date: 00 / 05 / 15

Client: Fluor Fernald, Inc. Project: OSDF-Phase III Proj./Proposal No.: GQ1001 Task No.: 04

SURFACE-WATER MANAGEMENT DURING DEVELOPMENT

Surface-water management system design calculations for staged development of the Borrow Area are presented as part of Section 12 of the OSDF Calculation Package, entitled *"Surface-Water Management During OSDF Construction/Filling/Closure"*.

15.7 Borrow Area Development Subarea Volume Verification

COMPUTATION COVER SHEET

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SUBJECT OF COMPUTATIONS Borrow Area Development Subarea Volume Verification

Computations By:
(Cognizant Engineer)

Signature: William H. Nagel — Date: 05 APRIL 2000

Printed Name and Title: William H. Nagel, Staff Engineer

Assumptions and Procedures Checked By:
(Checker)

Signature: Douglas T. Mandeville — Date: 05 April 2000

Printed Name and Title: Douglas T. Mandeville, Staff Engineer

Computations Checked by:

Signature: Douglas T. Mandeville — Date: 05 April 2000

Printed Name and Title: Douglas T. Mandeville, Staff Engineer

Computations Backchecked by:
(Cognizant Engineer)

Signature: William H. Nagel — Date: 05 APRIL 2000

Printed Name and Title: William H. Nagel, Staff Engineer

Approved by:
(PDP)

Signature: [illegible] — Date: 02 May 2000

Printed Name and Title: R. David Espinoza, Senior Project Engineer

Approved by:
(DTL and TETL)

Signature: J F Beck — Date: 8 May 2000

Printed Name and Title: J.F. Beck, PRINCIPAL

Record of Revision (Number and initial all revisions)

Rev. No. DLVL 16 MAY 2000	Reason	Date	By	Checked	Approval
Ø A	Pre-Final Submittal to Fluor Fernald, Inc./DOE	5 APRIL 2000	WHN	DLVL FOR DTM	JFB
1 A	FINAL SUBMITTAL TO FLUOR FERNALD, INC. /DOE	16 MAY 2000	WHN	DLVL FOR DTM	JFB
				000283	

Written By :CGK/WHN Date: 22 Feb 2000 Reviewed by: NTM Date: 4/5/00

Client: FF Project: Fernald OSDF Phase III Project/Proposal No.: GQ1001 Task No.: 06

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BORROW AREA DEVELOPMENT
SUBAREA VOLUME VERIFICATION

EXECUTIVE SUMMARY

This calculation package was prepared to verify the volumes of borrow material within each subarea of the East Field Borrow Area (borrow area) as calculated by CADD. The verification was achieved by measuring the areas at different elevations, and conducting the contour-area (average-end-area) method to estimate the volumes. The contours were measured from elevation contours shown on the *On-Site Disposal Facility, Phase III Construction Drawings* [GeoSyntec, 2000]. The following table presents the CADD-calculated volumes versus the hand calculated volumes for each subarea of borrow area development:

Subarea	CADD-Calculated Volume (bcy)	Hand Calculated Volume (bcy)	Difference (CADD vs. Contour-Area)
1	105,016	98,684	6.4%
2	139,786	132,752	5.3%
3	137,302	128,001	7.3%
4	173,968	173,344	0.4%
5	104,472	103,318	1.1%
6	70,218	70,758	-0.8%
7	67,196	68,512	-1.9%
8	106,038	106,099	-0.1%

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In general, there is good agreement between the two methods used. Particularly for subareas 4, 5, 6, 7 and 8, the difference is smaller than 1.9%. For subareas 1, 2 and 3, the existing condition drawings used in the calculation includes stockpiles. The contours representing the stockpiles were neglected and

Written By :CGK/WHN Date: 22 Feb 2000 Reviewed by: DTM Date: 4/5/00

Client: FF Project: Fernald OSDF Phase III Project/Proposal No.: GQ1001 Task No.: 06

new contour lines within the stockpile area were interpolated. As a result, the discrepancy between the hand- and CADD- calculated volumes is larger 2983

Based on the results above, the CADD-calculated volumes are considered reasonably accurate.

Written By :CGK/WHN Date: 22 Feb 2000 Reviewed by: DTM Date: 4/5/00

Client: FF Project: Fernald OSDF Phase III Project/Proposal No.: GQ1001 Task No.: 06

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BORROW AREA DEVELOPMENT
SUBAREA VOLUME VERIFICATION

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Written By :CGK/BN Date: 22 Feb 2000 Reviewed by: DTM Date: 4/5/00

Client: FF Project: Fernald OSDF Phase III Project/Proposal No.: GQ1001 Task No.: 06

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BORROW AREA DEVELOPMENT SUBAREA VOLUME VERIFICATION

CALCULATION PROCEDURES

Estimate the volume of material within each subarea of development in the East Field Borrow Area (borrow area) using the contour-area method, as detailed in the steps below:

1. Estimate plane areas of material removed at specific elevations

Estimate the plane areas of borrow material removed at the top and bottom of the subarea excavation by measuring the areas enclosed by the excavation limits as shown on Borrow Area Subarea 1, 2, 3, 4, 5, 6, 7 and 8 drawings presented in Appendix A. Measure these areas with a planimeter.

2. Estimate volume of each subarea

The total volume of borrow material removed can be divided into "slices" whose tops and bottoms correspond to planes at specific elevations. Estimate the volume of borrow material removed in a "slice" of height Δz by using one of the following formulas:

For volume of material in a "slice" whose bottom is the plane bounded by the uppermost contour in a section:

$V_z = 1/3\ A_z \Delta z$ (i.e., volume of a pyramid)

Where: V_z = volume of material removed between elevation 'z' and 'z+Δz'

A_z = plane area of material removed at elevation 'z'

Δz = elevation increment between contours

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Written By :CGK/BN Date: 22 Feb 2000 Reviewed by: DTM Date: 4/5/00

Client: FF Project: Fernald OSDF Phase III Project/Proposal No.: GQ1001 Task No.: 06

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For volume of material in a "slice" whose top is the plane bounded by the contour at elevation 'z+Δz' and whose bottom is the plane at elevation 'z':

$V_z = (A_z + A_{z+\Delta z})\Delta z/2$ (i.e., "average-end-area" volume)

Where: $A_{z+\Delta z}$ = plane area of material removed at elevation 'z+Δz'

Note that the average-end-area (contour-area) volume is <u>not</u> the true volume of the estimated subarea. Equations for the true volume of the estimated volume, or *prismoidal volume*, can be found in most surveying texts. The contour-area volume is accurate enough for the purposes of this calculation, and for most earthwork volume estimates [Wolf and Brinker, 1989].

3. Compare contour-area method volumes to CADD volumes

Compare the volumes estimated from the contour-area method calculations to the volumes estimated from CADD. If both volume estimates for each subarea are reasonably close (within 15 percent of each other), the CADD volume is considered to be accurate. Otherwise, one or both of the volumes is/are inaccurate.

Written By :CGK/BN Date: 22 Feb 2000 Reviewed by: DTM Date: 4/5/00

Client: FF Project: Fernald OSDF Phase III Project/Proposal No.: GQ1001 Task No.: 06

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REFERENCES

GeoSyntec Consultants, "*Final Design Package*," Fernald Environmental Management Project, DOE Fernald Area Office, Fernald, OH, 1997.

GeoSyntec Consultants, "*Borrow Area Management and Restoration Plan, On-Site Disposal Facility*," Fernald Environmental Management Project, Fernald Area Office, Fernald, OH, 2000.

GeoSyntec Consultants, "*On-Site Disposal Facility, Phase III Construction Drawings*," Fernald Environmental Management Project, DOE Fernald Area Office, Fernald, OH 2000 [in progress].

GeoSyntec Consultants, "*Borrow Area Strategy Report*," Fernald Environmental Management Project, Fernald Area Office, Fernald, OH, 1999.

Wolf, P.R., and Brinker, R.C., "*Elementary Surveying*," 8th ed., Harper and Row Publishers, New York, NY, 1989, 696 p.

Written By :CGK/BN Date: 22 Feb 2000 Reviewed by: DTM Date: 4/5/00

Client: FF Project: Fernald OSDF Phase III Project/Proposal No.: GQ1001 Task No.: 06

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BORROW AREA DEVELOPMENT
SUBAREA VOLUME VERIFICATION

DATA VERIFICATION

The existing contours and proposed layouts of each subarea within the East Field Borrow Area (borrow area) were obtained from *On-Site Disposal Facility, Phase III Construction Drawings* [GeoSyntec, 2000]. These drawings were used to prepare both the CADD and the contour-area method volume estimates of each subarea within the borrow area.

CADD computer software utilizes Softdesk®, Inc. Civil/Survey together with Autodesk®, Inc. AutoCAD Release 14 to generate volume estimations from three-dimensional planes. A summary of the CADD estimated volumes for each subarea are presented below:

Subarea	CADD-Calculated Volume (bcy)
1	105,016
2	139,786
3	137,302
4	173,968
5	104,472
6	70,218
7	67,196
8	106,038

Documentation for the CADD software used to prepare the CADD volumes is attached in Appendix B.

Written By :CGK/BN Date: 22 Feb 2000 Reviewed by: DTM Date: 4/5/00

Client: FF Project: Fernald OSDF Phase III Project/Proposal No.: GQ1001 Task No.: 06

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BORROW AREA DEVELOPMENT
SUBAREA VOLUME VERIFICATION

CALCULATIONS

Volumes for each subarea calculated by the contour-area-method are presented on the attached spreadsheets. The estimated hand-calculation contour-area-method volumes for each subarea are presented below:

Subarea	Contour-Area-Hand Calculated Volume (bcy)
1	98,684
2	132,752
3	128,001
4	173,344
5	103,318
6	70,758
7	68,512
8	106,099

To follow is planimeter generated data and corresponding volume calculations using the contour-area-method for borrow area subarea 1 through subarea 8, inclusive. The data has been broken up and is presented within the following sub-groupings for added clarity:

- Hand calculated contour-area data using the planimeter for various contour elevations using the borrow area drawings presented in Appendix A [attached].
- Excel Spreadsheet printout that calculates individual subarea volumes using the contour-area-method utilizing the planimetered data [above].

EXAMPLE OF HAND CALCULATION USING END-AREA METHOD

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PHASE III

scale: 1"=50' (I:600)
* indicates that an adjustment was made with respect to the area to NOT include existing stockpiles, portions of the ramp and haul roads within the subarea.

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SUBAREA 1

(HAND PLANIMETER DATA)

ATTEMPT	CONTOUR ELEVATION (FT)	PLANIMETERED AREA* SF	ACRES	
1	576	139850.19	3.210	
2	576	139578.94	3.204	
average	576 (D)	139714.57 (A)	3.207	223018.24
1	577	306321.91 **	7.032	
2	577	306321.91 **	7.032	
average	577 (C)	306321.91 (B) **	7.032	323701.46 **
1	578	340771.01	7.823	
2	578	341391.02	7.837	
average	578	341081.02	7.830	674198.82
1	580	333214.68	7.649	
2	580	333020.93	7.645	
average	580	333117.81	7.647	618379.05
1	582	285571.21	6.556	
2	582	284951.27	6.541	
average	582	285261.24	6.549	427630.22
1	584	142291.47	3.266	
2	584	142446.48	3.27	
average	584	142368.98	3.268	189140.72
1	586	46907.37	1.077	
2	586	46636.12	1.071	
average	586	46771.75	1.074	72676.39
1	588	25885.27	0.594	
2	588	25924.02	0.595	
average	588	25904.65	0.595	37801.02
1	590	11586.37	0.266	
2	590	12206.38	0.28	
average	590	11896.38	0.273	45318.58
1	592	33286.6 **	0.764	
2	592	33557.8 **	0.77	
average	592	33422.20	0.767	52603.60
1	594	19026.4	0.437	
2	594	19336.4	0.444	
average	594	19181.40	0.441	

**Added 577 Contour to better define contour for improved Overall Volume
** Added 592 & 594 Contours (3/00)

END-AREA METHOD

$\left(\frac{A+B}{2}\right)(C-D)$ EQUATION - FORMULA

Σ TOTAL

Σ total = 2664468.09 CF
= 98684.00 CY ← RESULT USING END AREA METHOD

target = 105016 CY ← CADD GENERATED TARGET VALUE
% Diff = 6.4 % ← VARIANCE

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NOTE: REFERENCE ATTACHED SUBAREA DRAWING

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PHASE III

SUBAREA 1

scale: 1"=50' (I:600)
* indicates that an adjustment was made with respect to the area to NOT include existing stockpiles, portions of the ramp and haul roads within the subarea.

**Added 577 Contour to better define contour for improved Overall Volume
** Added 592 & 594 Contours (3/00)

ATTEMPT	CONTOUR ELEVATION	PLANIMETERED AREA* SF		ACRES		
1	576	139850.19		3.210		
2	576	139578.94		3.204		
average	576	139714.57		3.207	223018.24	
1	577	306321.91	**	7.032		
2	577	306321.91	**	7.032		
average	577	306321.91	**	7.032	323701.46	**
1	578	340771.01		7.823		
2	578	341391.02		7.837		
average	578	341081.02		7.830	674198.82	
1	580	333214.68		7.649		
2	580	333020.93		7.645		
average	580	333117.81		7.647	618379.05	
1	582	285571.21		6.556		
2	582	284951.27		6.541		
average	582	285261.24		6.549	427630.22	
1	584	142291.47		3.266		
2	584	142446.48		3.27		
average	584	142368.98		3.268	189140.72	
1	586	46907.37		1.077		
2	586	46636.12		1.071		
average	586	46771.75		1.074	72676.39	
1	588	25885.27		0.594		
2	588	25924.02		0.595		
average	588	25904.65		0.595	37801.02	
1	590	11586.37		0.266		
2	590	12206.38		0.28		
average	590	11896.38		0.273	45318.58	
1	592	33286.6	**	0.764		
2	592	33557.8	**	0.77		
average	592	33422.20		0.767	52603.60	
1	594	19026.4		0.437		
2	594	19336.4		0.444		
average	594	19181.40		0.441		

total = 2664468.09 CF
= 98684.00 CY

target = 105016 CY
% Diff = 6.4 %

NTM 4/5/00

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FF
PHASE III

SUBAREA 2

scale: 1"=50' (I:600)
* indicates that an adjustment was made with respect to the area to NOT include existing stockpiles, ramp and haul roads within the subarea.

ATTEMPT	CONTOUR ELEVATION	PLANIMETERED AREA* SF	ACRES			
3	570	251335.09	5.770			
1	570	251257.59	5.768			
2	570	251180.09	5.766			total = 3584315.09 CF
average	570	251257.59	5.768		505557.09	= 132752.411 CY
1	572	254783.88	5.849			target = 139786 CY
2	572	253815.12	5.827			% Diff = 5.3 %
average	572	254299.50	5.838		517879.72	0.18874064
1	574	263735.22	6.054			
2	574	263425.21	6.047			
average	574	263580.22	6.051		274546.58	
1	575	285667.94	6.558	Bottom		
2	575	285357.94	6.551	Bottom		
average	575	285512.94	6.555	Bottom	0.00	
1	575	314343.24	7.2162	Top		
2	575	314033.24	7.2092	Top		
average	575	314188.24	7.213	Top	304907.52	
1	576	295704.30	6.788			
2	576	295549.30	6.785			
average	576	295626.80	6.787		586351.68	
1	578	290666.75	6.673			
2	578	290783.00	6.675			
average	578	290724.88	6.674		516833.46	
1	580	225953.58	5.187			
2	580	226263.58	5.194			
average	580	226108.58	5.191		390720.28	
1	582	164107.94	3.767			
2	582	165115.45	3.79			
average	582	164611.70	3.779		248486.94	
1	584	83894.62	1.926			
2	584	83855.87	1.925			
average	584	83875.25	1.926		128341.33	
1	586	44485.46	1.021			
2	586	44446.71	1.020			
average	586	44466.09	1.021		66844.44	
1	588	22591.48	0.519			
2	588	22165.23	0.509			
average	588	22378.36	0.514		28423.42	
1	590	6006.31	0.138			
2	590	6083.81	0.14			
average	590	6045.06	0.139		9726.35	
1	592	3720.04	0.085			
2	592	3642.54	0.084			
average	592	3681.29	0.085		5696.31	
1	594	2053.77	0.047			
2	594	1976.27	0.045			
average	594	2015.02	0.046			

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PHASE III

scale: 1"=50' (I:600)

SUBAREA 3

ATTEMPT	CONTOUR ELEVATION	PLANIMETERED AREA* SF	ACRES			
1	576	252691.35	5.800			
2	576	252148.85	5.788		total =	3456031.85 CF
average	576	252420.10	5.794	247159.73	=	128001.179 CY
1	577	241647.49	5.547		target =	137302 CY
2	577	242151.24	5.559		% Diff =	7.3 %
average	577	241899.37	5.553	243581.78		
1	578	245251.28	5.630			
2	578	244825.02	5.620			
3	578	245716.28	5.641			
average	578	245264.19	5.630	497819.92		
1	580	252652.60	5.800			
2	580	252458.85	5.795			
average	580	252555.73	5.798	512202.78		
1	582	260015.18	5.969			
2	582	259278.92	5.952			
average	582	259647.05	5.961	525203.54		
1	584	265440.24	6.093			
2	584	265672.74	6.099			
average	584	265556.49	6.096	527354.19		
1	586	261681.45	6.007			
2	586	261913.95	6.013			
average	586	261797.70	6.010	462989.77		
1	588	201463.32	4.625			
2	588	200920.82	4.612			
average	588	201192.07	4.619	303590.00		
1	590	102339.80	2.349			
2	590	102456.05	2.352			
average	590	102397.93	2.351	136130.15		
1	592	33751.6	0.775			
2	592	33712.84	0.774			
average	592	33732.22	0.774			

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DTM 4/5/00

FF scale: 1"=50' (I:600)
PHASE III

SUBAREA 4

ATTEMPT	CONTOUR ELEVATION	PLANIMETERED AREA* SF	ACRES	
1	576	13950.14	0.320	
2	576	14105.15	0.324	
average	576	14027.65	0.322	128874.14
1	577	243585.01	5.592	
2	577	243856.26	5.598	
average	577	243720.64	5.595	246626.92
1	578	249552.57	5.729	
2	578	249513.82	5.728	
average	578	249533.20	5.729	511369.65
1	580	261952.70	6.013	
2	580	261720.20	6.008	
average	580	261836.45	6.011	535472.40
1	582	273694.07	6.283	
2	582	273577.82	6.280	
average	582	273635.95	6.282	559691.40
1	584	286132.95	6.569	
2	584	285977.95	6.564	
average	584	286055.45	6.567	584104.15
1	586	297796.82	6.836	
2	586	298300.57	6.848	
average	586	298048.70	6.842	579260.35
1	588	281095.40	6.453	
2	588	281327.90	6.458	
average	588	281211.65	6.456	526908.56
1	590	245638.78	5.639	
2	590	245755.03	5.642	
average	590	245696.91	5.641	418271.81
1	592	172284.27	3.955	
2	592	172865.53	3.968	
average	592	172574.90	3.962	278770.37
1	594	106253.59	2.439	
2	594	106137.34	2.437	
average	594	106195.47	2.438	169819.87
1	596	63318.15	1.454	
2	596	63930.65	1.447	
average	596	63624.40	1.451	90207.17
1	598	26544.02	0.609	
2	598	26621.52	0.611	
average	598	26582.77	0.610	36929.13
1	600	10385.11	0.238	
2	600	10307.61	0.237	
average	600	10346.36	0.238	13969.53
1	602	3603.79	0.083	
2	602	3642.54	0.084	
average	602	3623.17	0.083	

total = 4680275.40 CF
= 173343.533 CY

target = 173968 CY
% Diff = 0.4 %

FF scale: 1"=50' (I:600)
PHASE III

2983 (7 OF 24)

DTM 4/5/00

SUBAREA 5

ATTEMPT	CONTOUR ELEVATION	PLANIMETERED AREA* SF	ACRES			
1	576	174880.55	4.015			
2	576	174841.80	4.014		total =	2789583.09
average	576	174861.18	4.015	350516.74	=	103317.892
1	578	175655.56	4.032		target =	104472
2	578	175655.56	4.032		% Diff =	1.1
average	578	175655.56	4.032	352473.63		
1	580	176856.82	4.060			
2	580	176779.32	4.058			
average	580	176818.07	4.059	354488.65		
1	581			Skip		
2	581			Skip		
average	581	0.00	0.000	Skip		
1	582	177631.83	4.078			
2	582	177709.33	4.080			
average	582	177670.58	4.079	352803.01		
1	584	175190.55	4.022			
2	584	175074.30	4.019			
average	584	175132.43	4.021	336934.72		
1	586	161589.16	3.709			
2	586	162015.42	3.719			
average	586	161802.29	3.714	309538.19		
1	588	147949.02	3.396			
2	588	147522.77	3.387			
average	588	147735.90	3.392	277394.73		
1	590	129658.83	2.976			
2	590	129658.83	2.976			
average	590	129658.83	2.976	224054.81		
1	592	94279.72	2.164			
2	592	94512.23	2.170			
average	592	94395.98	2.167	151281.57		
1	594	56769.34	1.303			
2	594	57001.84	1.309			
average	594	56885.59	1.306	80097.08		
1	596	23095.23	1.226			
2	596	23327.74	1.232			
average	596	23211.49	1.229	32414.71		
1	598	9183.85	0.211			
2	598	9222.6	0.212			
average	598	9203.23	0.211	13795.15		
1	600	4572.54	0.105			
2	600	4611.3	0.106			
average	600	4591.92	0.105	6200.07		
1	602	1588.77	0.037			
2	602	1627.52	0.037			
average	602	1608.15	0.037			

DTM 4/5/00

FF scale: 1"=50' (I:600)
PHASE III

SUBAREA 6

ATTEMPT	CONTOUR ELEVATION	PLANIMETERED AREA* SF	ACRES	
1	576	14647.65	0.336	
2	576	14608.90	0.335	
average	576	14628.28	0.336	55480.88
1	577	96333.49	2.211	
2	577	96333.49	2.211	
average	577	96333.49	2.211	96314.12
1	578	96255.99	2.210	
2	578	96333.49	2.211	
average	578	96294.74	2.211	192395.73
1	580	96023.49	2.204	
2	580	96178.49	2.208	
average	580	96100.99	2.206	192143.86
1	582	95984.74	2.204	
2	582	96100.99	2.206	
average	582	96042.87	2.205	191988.86
1	584	95829.74	2.200	
2	584	96062.24	2.205	
average	584	95945.99	2.203	191775.73
1	586	95674.74	2.196	
2	586	95984.74	2.203	
average	586	95829.74	2.200	190128.84
1	588	94279.72	2.164	
2	588	94318.47	2.165	
average	588	94299.10	2.165	179588.73
1	590	85367.13	1.960	
2	590	85212.13	1.956	
average	590	85289.63	1.958	160000.40
1	592	74633.27	1.713	
2	592	74788.27	1.717	
average	592	74710.77	1.715	139288.32
1	594	64480.67	1.480	
2	594	64674.42	1.485	
average	594	64577.55	1.482	122218.77
1	596	57621.84	1.323	
2	596	57660.60	1.324	
average	596	57641.22	1.323	103618.57
1	598	46035.48	1.057	
2	598	45919.22	1.054	
average	598	45977.35	1.055	68452.58
1	600	22475.23	0.516	
2	600	22475.23	0.516	
average	600	22475.23	0.516	27067.16
1	602	4611.3	0.106	
2	602	4572.55	0.105	
average	602	4591.93	0.105	

total = 1910462.51 CF
= 70757.8708 CY

target = 70218 CY
% Diff = -0.8 %
(EXCEEDS)

FF
PHASE III

scale: 1"=50' (I:600)

2983 (9 OF 24)

OTM 4/5/00

SUBAREA 7

ATTEMPT	CONTOUR ELEVATION	PLANIMETERED AREA* SF	ACRES		
1	577	33441.6	0.767		
2	577	33557.8	0.770		total = 1849818.13 CF
average	577	33499.7	0.769	52564.90	= 68511.782 CY
1	578	71727.0	1.647		target = 67196 CY
2	578	71533.2	1.642		% Diff = -1.9 %
average	578	71630.1	1.645	144926.50	(EXCEEDS)
1	580	73277.0	1.682		
2	580	73315.8	1.683		
average	580	73296.4	1.683	148879.05	
1	582	75640.8	1.736		
2	582	75524.5	1.734		
average	582	75582.7	1.735	152947.80	
1	584	77423.3	1.777		
2	584	77307.0	1.775		
average	584	77365.2	1.776	156822.85	
1	586	79322.1	1.821		
2	586	79593.3	1.827		
average	586	79457.7	1.824	161085.40	
1	588	81608.3	1.873		
2	588	81647.1	1.874		
average	588	81627.7	1.874	165309.20	
1	590	83817.1	1.924		
2	590	83545.9	1.918		
average	590	83681.5	1.921	169494.25	
1	592	85677.1	1.967		
2	592	85948.4	1.973		
average	592	85812.8	1.970	172284.25	
1	594	86335.9	1.982		
2	594	86607.1	1.988		
average	594	86471.5	1.985	166452.35	
1	596	79787.1	1.832		
2	596	80174.6	1.840		
average	596	79980.9	1.836	141070.85	
1	598	61186.9	1.405		
2	598	60993.1	1.400		
average	598	61090.0	1.403	106602.35	
1	600	45570.5	1.046		
2	600	45454.2	1.043		
average	600	45512.4	1.045	68626.95	
1	602	23017.7	0.528		
2	602	23211.5	0.533		
average	602	23114.6	0.531	34352.25	
1	604	11276.4	0.259		
2	604	11198.9	0.257		
average	604	11237.7	0.258	8399.18	
1	605	5541.3	0.127		
2	605	5580.1	0.128		
average	605	5560.7	0.128		

FF scale: 1"=50' (I:600)
PHASE III

2983 (10 OF 24)

SUBAREA 8

DTM 4/5/00

ATTEMPT	CONTOUR ELEVATION	PLANIMETERED AREA* SF	ACRES		
1	578	99666.0	2.288		
2	578	99937.3	2.294		total = 2864681.46 CF
average	578	99801.7	2.291	207837.80	= 106099.313 CY
1	580	107997.4	2.479		target = 106038 CY
2	580	108074.9	2.481		% Diff = -0.1 %
average	580	108036.2	2.480	223783.60	(EXCEEDS)
1	582	115708.7	2.656		
2	582	115786.2	2.658		
average	582	115747.5	2.657	239729.35	
1	584	124040.0	2.847		
2	584	123923.8	2.844		
average	584	123981.9	2.846	256527.65	
1	586	132720.1	3.047		
2	586	132371.4	3.039		
average	586	132545.8	3.043	273732.85	
1	588	141090.2	3.239		
2	588	141284.0	3.243		
average	588	141187.1	3.241	290492.35	
1	590	148956.5	3.419		
2	590	149654.0	3.435		
average	590	149305.3	3.427	307077.50	
1	592	158024.1	3.628		
2	592	157520.4	3.616		
average	592	157772.3	3.622	323255.85	
1	594	165425.5	3.798		
2	594	165541.7	3.800		
average	594	165483.6	3.799	301013.10	
1	596	135626.4	3.113		
2	596	135432.6	3.109		
average	596	135529.5	3.111	206384.61	
1	598	70797.0	1.625		
2	598	70913.2	1.628		
average	598	70855.1	1.627	120785.01	
1	600	49833.0	1.144		
2	600	50026.8	1.148		
average	600	49929.9	1.146	70719.50	
1	602	20886.5	0.479		
2	602	20692.7	0.475		
average	602	20789.6	0.477	31659.10	
1	604	10927.6	0.250		
2	604	10811.4	0.248		
average	604	10869.5	0.249	11683.20	
1	606	813.6	0.019		
2	606	813.8	0.019		
average	606	813.7	0.019		

Written by: B. NAGEL Date: 00/FEB/21 (YY MM DD) Reviewed by: DTM Date: 00/04/05 (YY MM DD)

Client: FF Project: PHASE III OSDF Project/Proposal No.: GQ1001 Task No.: 03

SUB-AREA 1 SCALE: 1" = 50' (1:600)

ATTEMPT	CONTOUR ELEVATION (FT)	PLANIMETERED AREA (*) SF	ACRES	+ ADDITIONAL AREA SF	ACRES	+ SF	ACRES
1	576	139,850.19	3.210				
2	↓	139,578.94	3.204				
AVE	576	139,714.57	3.207				
1	577	306,321.91	7.032				
2	↓	306,321.91	7.032				
AVE	577	306,321.91	7.032				
1	578	340,771.01	7.823				
2	↓	341,391.02	7.837				
AVE	578	341,081.02	7.830				
1	580	333,214.68	7.649				
2	↓	333,020.93	7.645				
AVE	580	333,117.81	7.647				
1	582	285,571.21	6.556				
2	↓	284,951.27	6.541				
AVE	582	285,261.24	6.549				
1	584 **	132,565.11	3.043	6,200.06	0.142	3,603.79	0.083
2	↓	132,720.12	3.047	+ 6,083.81	0.140	+ 3,565.04	0.082
AVE	584	132,642.62	3.045	6,141.94	0.141	3,584.42	0.082
1	586	39,564.16	0.908	7,323.83	0.168		
2	↓	39,292.91	0.902	+ 7,362.58	0.169		
AVE	586	39,428.54	0.905	7,343.21	0.169		
1	588	17,941.44	0.412	7,943.83	0.182		
2	↓	17,980.19	0.413	+ 7,943.83	0.182		
AVE	588	17,960.82	0.413	7,943.83	0.182		
1	590	11,586.37	0.266				
2	↓	12,206.38	0.280				
AVE	590	11,896.38	0.273				
1	592	33,286.6	0.764				
2	↓	33,557.8	0.770				
AVE	592	33,422.2	0.767				
1	594	19,026.4	0.437				
2	↓	19,336.4	0.444				
AVE	594	19,181.4	0.440				

(*) ADJUSTMENT WAS PERFORMED/MA TO NOT INCLUDE EXISTING STOCKPILES AND PORTION OF RAMP FEATURE WITHIN SUB-AREA. (IE, QUANTITIES ARE NOT REFLECTED IN VOLUME ESTIMAT

** NOTE 3 CONTOUR ELEV. 584 PLANES/AREAS EXISTING ON SUB-AREA 1 DRAWING. (2 EA. FOR 586 AND 588 CONTOUR ELEV.'S).

AVE. TOTAL → 12,368.98 SF

AVE TOTAL → 46,771.75 SF

AVE. TOTAL → 25904.65 SF

Added 24 March 2000. (BN)

000301

Written by: B. NAGEL Date: 00/FEB/22 (YY MM DD) Reviewed by: DTM Date: 00/04/05 (YY MM DD)

Client: FF Project: OSDF PHASE III Project/Proposal No.: GQ1001 Task No.: 03

SUBAREA 2 SCALE 1" = 50' (1:600)

* ADJUSTMENT MADE TO SURFACE CONTOURS TO NOT INCLUDE/EXCLUDE STOCKPILES WITHIN SUBAREA.

ATTEMPT	CONTOUR ELEVATION (FT)	PLANIMETER AREA SF	PLANIMETER AREA ACRE	+ ADDITIONAL AREAS SF	ADDITIONAL AREAS ACRE	
1	570	251,257.59	5.768			
2	↓	251,186.09	5.766			
3		251,335.09	5.770			
AVE.	570	251,257.59	5.768			
1	572	254,783.88	5.849			LOWER "BENCH" PLATEAU PORTION
2	↓	253,815.12	5.827			
AVE	572	254,299.50	5.838			
1	574	263,735.22	6.054			
2	↓	263,425.21	6.047			
AVE	574	263,580.22	6.051			
1	575 (BOTTOM)	285,667.94	6.558			
2	↓	285,357.94	6.551			
AVE	575 (BOTTOM)	285,512.94	6.554			
1	575 (TOP)	+28,675.296 314,343.24	+0.6582/88 ×			UPPER "BENCH" PLATEAU PORTION
2	↓	314,033.24	×			
AVE	575 (TOP)	314,188.24	7.213			
1	576	295,704.30	6.788			
2	↓	295,549.30	6.785			
AVE	576	295,626.80	6.787			
1	578	290,666.75	6.673			
2	↓	290,783.00	6.675			
AVE	578	290,724.88	6.674			
1	580	225,953.58	5.187			
2	↓	226,263.58	5.194			
AVE	580	226,108.58	5.191			
1	582	164,107.94	3.767			
2	↓	165,115.45	3.790			
AVE	582	164,611.70	3.779			
1	584	83,894.62	1.926			
2	↓	83,855.87	1.925			
AVE	584	83,875.25	1.926			

000202

Written by: B. NAGEL Date: 00/FEB/22 (YY MM DD) Reviewed by: MM Date: 00/04/05 (YY MM DD)

Client: FF Project: OSDF PHASE III Project/Proposal No.: GQ1001 Task No.: 03

SUBAREA 2 (CONTINUED) SCALE 1" = 50' (1:600)

* ADJUSTMENT MADE TO SURFACE CONTOURS TO EXCLUDE STOCKPILES WITHIN SUBAREA.

ATTEMPT	CONTOUR ELEVATION (FT)	PLANIMETER AREA SF	PLANIMETER AREA ACRES	+ ADDITIONAL AREAS SF	+ ADDITIONAL AREAS ACRES
1	586	44,485.46	1.021		
2	↓	44,446.71	1.020		
AVE	586	44,466.09	1.021		
1	588	22,591.48	0.519		
2	↓	22,165.23	0.509		
AVE	588	22,378.36	0.514		
1	590	6,006.31	0.138		
2	↓	6,083.81	0.140		
AVE	590	6,045.06	0.139		
1	592	3,720.04	0.085		
2	↓	3,642.54	0.084		
AVE	592	3,681.29	0.085		
1	594	2,053.77	0.047		
2	↓	1,976.27	0.045		
AVE	594	2,015.02	0.046		

Written by: B. NAGEL Date: 00/FEB/23 (YY MM DD) Reviewed by: DTM Date: 00/04/05 (YY MM DD)

Client: FF Project: OSDF PHASE III Project/Proposal No.: GQ1001 Task No.: 03

SUBAREA 3 SCALE 1" = 50' (1:600)

ATTEMPT	CONTOUR ELEVATION (FT)	PLANIMETER AREA SF	PLANIMETER AREA ACRES	+ ADDITIONAL AREAS SF	ADDITIONAL AREAS ACRES
1	576	252,691.35	5.800		
2	↓	252,148.85	5.788		
AVE	576	252,420.10	5.795		
1	577	241,647.49	5.547		
2	↓	242,151.24	5.559		
AVE	577	241,899.37	5.553		
3	578	245,251.28	5.630		
1	578	244,825.02	5.620		
2	↓	245,716.28	5.641		
AVE	578	245,264.19	5.630		
1	580	252,652.60	5.800		
2	↓	252,458.85	5.795		
AVE	580	252,555.73	5.798		
1	582	260,015.18	5.969		
2	↓	259,278.92	5.952		
AVE	582	259,647.05	5.961		
1	584	265,440.24	6.093		
2	↓	265,672.74	6.099		
AVE	584	265,556.49	6.096		
1	586	261,681.45	6.007		
2	↓	261,913.95	6.013		
AVE	586	261,797.70	6.010		
1	588	201,463.32	4.625		
2	↓	200,920.82	4.612		
AVE	588	201,192.07	4.619		
1	590	102,339.80	2.349		
2	↓	102,456.05	2.352		
AVE	590	102,397.93	2.351		
1	592	5,115.05	0.11742	28,481.54	0.6538
2	↓	5,231.30	0.12009 +	28,636.55	0.6574
AVE	592	5,173.18	0.11876	28,559.05	0.65563

000004

Written by: B. NAGEL Date: 00/FEB/23 (YY MM DD) Reviewed by: OTM Date: 00/04/05 (YY MM DD)

Client: FF Project: OSDF PHASE III Project/Proposal No.: 6Q1001 Task No.: 03

SUBAREA 4 SCALE 1" = 50' (1:600)

ATTEMPT	CONTOUR ELEVATION (FT)	PLANIMETER AREA SF	PLANIMETER AREA ACRES	+ ADDITIONAL AREAS SF	ADDITIONAL AREAS ACRES
1	576	13,950.14	0.320		
2	↓	14,105.15	0.324		
AVE	576	14,027.65	0.322		
1	577	243,585.01	5.592		
2	↓	243,856.26	5.598		
AVE	577	243,720.64	5.595		
1	578	249,552.57	5.729		
2	↓	249,513.82	5.728		
AVE	578	249,533.20	5.728		
1	580	261,952.70	6.013		
2	↓	261,720.20	6.008		
AVE	580	261,836.45	6.011		
1	582	273,694.07	6.283		
2	↓	273,577.82	6.280		
AVE	582	273,635.95	6.282		
1	584	286,132.95	6.569		
2	↓	285,977.95	6.564		
AVE	584	286,055.45	6.567		
1	586	297,796.82	6.836		
2	↓	298,300.57	6.848		
AVE	586	298,048.70	6.842		
1	588	281,095.40	6.453		
2	↓	281,327.90	6.458		
AVE	588	281,211.65	6.456		
1	590	245,638.78	5.639		
2	↓	245,755.03	5.642		
AVE	590	245,696.91	5.640		
1	592	172,284.27	3.955		
2	↓	172,865.53	3.968		
AVE	592	172,574.90	3.962		

Written by: B. NAGEL Date: 00/FEB/23 (YY MM DD) Reviewed by: MM Date: 00/04/05 (YY MM DD)

Client: EF Project: OSDF PHASE III Project/Proposal No.: GQ1001 Task No.: 03

SUBAREA 4 (CONTINUED) SCALE: 1" = 50' (1:600)

ATTEMPT	CONTOUR ELEVATION (FT)	PLANIMETER AREA SF	PLANIMETER AREA ACRES
1	594	106,253.59	2.439
2	↓	106,137.34	2.437
AVE	594	106,195.47	2.438
1	596	63,318.15	1.454
2	↓	62,930.65	1.447
AVE	596	63,124.40	1.4951
1	598	26,544.02	0.609
2	↓	26,621.52	0.611
AVE	598	26,582.77	0.610
1	600	10,385.11	0.238
2	↓	10,307.61	0.237
AVE	600	10,346.36	0.238
1	602	3603.79	0.0827
2	↓	3642.54	0.0836
AVE	602	3623.17	0.0832

Written by: B. NAGR Date: 00/FEB/23 (YY MM DD) Reviewed by: DTM Date: 00/04/05 (YY MM DD)

Client: FF Project: CSDF PHASE III Project/Proposal No.: GQ1001 Task No.: 03

SUBAREA 5 SCALE 1" = 50' (1:600)

ATTEMPT	CONTOUR ELEVATION (FT)	PLANIMETER AREA SF	ACRES	+ ADDITIONAL AREAS SF	ACRES
1	576	174,880.55	4.015		
2	↓	174,841.80	4.014		
AVE	576	174,861.18	4.014		
1	578	175,655.56	4.032		
2	↓	175,655.56	4.032		
AVE	578	175,655.56	4.032		
1	580	176,856.82	4.060		
2	↓	176,779.32	4.058		
AVE	580	176,818.07	4.059		
1	582	177,631.83	4.078		
2	↓	177,709.33	4.080		
AVE	582	177,670.58	4.079		
1	584	175,190.55	4.022		
2	↓	175,074.30	4.019		
AVE	584	175,132.43	4.020		
1	586	161,589.16	3.709		
2	↓	162,015.42	3.719		
AVE.	586	161,802.29	3.714		
1	588	147,949.02	3.396		
2	↓	147,522.77	3.387		
AVE	588	147,735.90	3.392		
1	590	129,658.83	2.976		
2	↓	129,658.83	2.976		
AVE	590				
1	592	94,279.72	2.164		
2	↓	94,512.23	2.170		
AVE	592	94,395.98	2.167		
1	594	56,769.34	1.303		
2	↓	57,001.84	1.309		
AVE	594	56,885.59	1.306		

000007

Written by: B. NAGEL Date: 00/FEB/23 (YY MM DD) Reviewed by: MTM Date: 00/09/05 (YY MM DD)

Client: FF Project: OSDF PHASE III Project/Proposal No.: GQ1001 Task No.: 03

SUBAREA 5 (CONTINUED) SCALE: 1" = 50' (1:600)

ATTEMPT	CONTOUR ELEVATION (FT)	PLANIMETER AREA SF	PLANIMETER AREA ACRES	+	ADDITIONAL AREA SF	ADDITIONAL AREA ACRES
1	596	3,371.28	0.0774		19,723.95	0.452
2	↓	3,371.28	0.0774	+	19,956.46	0.458
AVE	596	3,371.28	0.0774		19,840.21	0.455
1	598	658.76	0.0151		8525.09	0.1957
2	↓	658.76	0.0151	+	8,563.84	0.1966
AVE	598	658.76	0.0151		8,544.47	0.1962
1	600	232.50	0.00533		4,456.30	0.1023
2	↓	155.00	0.00356	+	4,340.04	0.0996
AVE	600	193.75	0.00445		4,398.17	0.1010
1	602	1,588.77	0.0365			
2	↓	1,627.52	0.0374			
AVE	602	1,608.15	0.0369			

Written by: B. NAGEL Date: 00/FEB/23 (YY MM DD) Reviewed by: MTM Date: 00/04/05 (YY MM DD)

Client: FF Project: OSDF PHASE III Project/Proposal No.: GQ1001 Task No.: 03

SUBAREA 6 SCALE 1" = 50' (1:600)

ATTEMPT	CONTOUR ELEVATION (FT)	PLANIMETER AREA SF	PLANIMETER AREA ACRES	+ ADDITIONAL AREAS SF	ADDITIONAL AREAS ACRES
1	576	14,647.65	0.3363		
2	↓	14,608.90	0.3354		
AVE	576	14,628.28	0.3358		
1	577	96,333.49	2.2114		
2	↓	96,333.49	2.2114		
AVE	577	96,333.49	2.2114		
1	578	96,255.99	2.2097		
2	↓	96,333.49	2.2114		
AVE	578	96,294.74	2.2106		
1	580	96,023.49	2.2043		
2	↓	96,178.49	2.2079		
AVE	580	96,100.99	2.2062		
1	582	95,984.74	2.2035		
2	↓	96,100.99	2.2061		
AVE	582	96,042.87	2.2048		
1	584	95,829.74	2.1999		
2	↓	96,062.24	2.2052		
AVE	584	95945.99	2.2026		
1	586	95,674.74	2.1963		
2	↓	95,984.74	2.2034		
AVE	586	95,829.74	2.1999		
1	588	94,279.72	2.1643		
2	↓	94,318.47	2.1652		
AVE	588	94,299.10	2.1648		
1	590	85,367.13	1.9597		
2	↓	85,212.13	1.9561		
AVE	590	85,289.63	1.9580		
1	592	74,633.27	1.71329		
2	↓	74,788.27	1.71685		
AVE	592	74,710.77	1.71512		

GEOSYNTEC CONSULTANTS

Written by: B. NAGEL Date: 00/FEB/23 (YY MM DD) Reviewed by: DTM Date: 00/04/05 (YY MM DD)

Client: FF Project: CSDF PHASE III Project/Proposal No.: GQ1001 Task No.: 03

SUBAREA 6 (CONTINUED) SCALE 1" = 50' (1:600)

ATTEMPT	CONTOUR ELEVATION (FT)	PLANIMETER AREA SF	PLANIMETER AREA ACRES	+ ADDITIONAL AREAS SF	ADDITIONAL AREAS ACRES
1	594	64,480.67	1.4802		
2	↓	64,674.42	1.4847		
AVE	594	64,577.55	1.4825		
1	596	57,621.84	1.3228		
2	↓	57,660.60	1.3237		
AVE	596	57,641.22	1.3233		
1	598	46,035.48	1.0568		
2	↓	45,919.22	1.0541		
AVE.	598	45,977.35	1.0555		
1	600	16,430.17	0.3772	5,890.06	0.1352
2	↓	16,585.17	0.3807	6,045.06	0.1388
AVE	600	16,507.67	0.3790	+ 5967.56	0.1370
1	602	4,533.80	0.1041	116.25	0.002669
2	↓	4,456.30	0.1023	77.50	0.001779
AVE	602	4,495.05	0.1032	+ 96.88	0.00222

Written by: B. NAGEL Date: 00/03/22 (YY MM DD) Reviewed by: DTM Date: 00/04/05 (YY MM DD)

Client: FF Project: OSDF PHASE III Project/Proposal No.: GQ1001 Task No.: 06

SUBAREA 7

ATTEMPT	CONTOUR ELEVATION (FT)	PLANIMETER AREA SF	PLANIMETER AREA ACRES
1	577	33,441.6	0.767
2	↓	33,557.8	0.770
AVE	577	33,499.7	0.769
1	578	71,727.0	1.647
2	↓	71,533.2	1.642
AVE	578	71,630.1	1.644
1	580	73,277.0	1.682
2	↓	73,315.8	1.683
AVE	580	73,296.4	1.683
1	582	75,640.8	1.736
2	↓	75,524.5	1.734
AVE	582	75,582.7	1.735
1	584	77,423.3	1.777
2	↓	77,307.0	1.775
AVE	584	77,365.2	1.776
1	586	79,322.1	1.821
2	↓	79,593.3	1.827
AVE	586	79,457.7	1.824
1	588	81,608.3	1.873
2	↓	81,647.1	1.874
AVE	588	81,627.7	1.874
1	590	83,817.1	1.924
2	↓	83,545.9	1.918
AVE	590	83,681.5	1.921
1	592	85677.1	1.967
2	↓	85948.4	1.973
AVE	592	85812.8	1.970
1	594	86,335.9	1.982
2	↓	86,607.1	1.988
AVE	594	86,471.5	1.985

000311

Written by: B. NAGEL Date: 00/03/23 (YY MM DD) Reviewed by: DTM Date: 00/04/05 (YY MM DD)

Client: FF Project: OSDF PHASE III Project/Proposal No.: GQ1001 Task No.: 06

SUBAREA 7 (CONTINUED)

ATTEMPT	CONTOUR ELEVATION (FT)	PLANIMETER AREA SF	ACRES
1	596	79,787.1	1.832
2	↓	80,174.6	1.840
AVE	596	79,980.9	1.836
1	598	61,186.9	1.405
2	↓	60,993.1	1.400
AVE	598	61 090.0	1.402
1	600	45,570.5	1.046
2	↓	45,454.2	1.043
AVE	600	45,512.4	1.045
1	602	23,017.7	0.528
2	↓	23,211.5	0.533
AVE	602	23,114.6	0.531
1	604	11,276.4	0.259
2	↓	11,198.9	0.257
AVE	604	11,237.7	0.258
1	605	5,541.3	0.127
2	↓	5,580.1	0.128
AVE	605	5,560.7	0.128

Written by: B. NAGEL Date: 00/03/23 (YY MM DD) Reviewed by: DTM Date: 00/04/05 (YY MM DD)

Client: FF Project: OSDF PHASE III Project/Proposal No.: GQ1001 Task No.: 06

SUBAREA 8

ATTEMPT	CONTOUR ELEVATION (FT)	PLANIMETERED AREA SF	ACRES
1	578	99,666.0	2.288
2	↓	99,937.3	2.294
AVE	578	99,801.7	2.291
1	580	107,997.4	2.479
2	↓	108,074.9	2.481
AVE	580	108,036.2	2.480
1	582	115,708.7	2.656
2	↓	115,786.2	2.658
AVE	582	115,747.5	2.657
1	584	124,040.03	2.847
2	↓	123,923.77	2.844
AVE	584	123,981.9	2.846
1	586	132,720.1	3.047
2	↓	132,371.4	3.039
AVE	586	132,545.8	3.043
1	588	141,090.2	3.239
2	↓	141,284.0	3.243
AVE	588	141,187.1	3.241
1	590	148,956.5	3.419
2	↓	149,654.0	3.435
AVE	590	149,305.3	3.428
1	592	158,024.1	3.628
2	↓	157,520.4	3.616
AVE	592	157,772.3	3.622
1	594	165,425.5	3.798
2	↓	165,541.7	3.800
AVE	594	165,483.6	3.799
1	596	135,626.4	3.113
2	↓	135,432.6	3.109
AVE	596	135,529.5	3.111

000313

GEOSYNTEC CONSULTANTS

Written by: B. NAGEL Date: 00/03/23 (YY MM DD) Reviewed by: DTM Date: 00/04/05 (YY MM DD)

Client: FF Project: OSDF PHASE III Project/Proposal No.: GQ1001 Task No.: 06

SUB-AREA 8 (CONTINUED)

ATTEMPT	CONTOUR ELEVATION (FT)	PLANIMETERED AREA SF	ACRES
1	598	70,796.98	1.625
2	↓	70,913.23	1.628
AVE	598	70,855.11	1.627
1	600	49,833.0	1.144
2	↓	50,026.8	1.148
AVE	600	49,929.9	1.146
1	602	20,886.5	0.479
2	↓	20,692.7	0.475
AVE	602	20,789.6	0.477
1	604	10,927.6	0.250
2	↓	10,811.4	0.248
AVE	604	10,869.5	0.250
1	606	813.6	0.019
2	↓	813.8	0.019
AVE	606	813.7	0.019

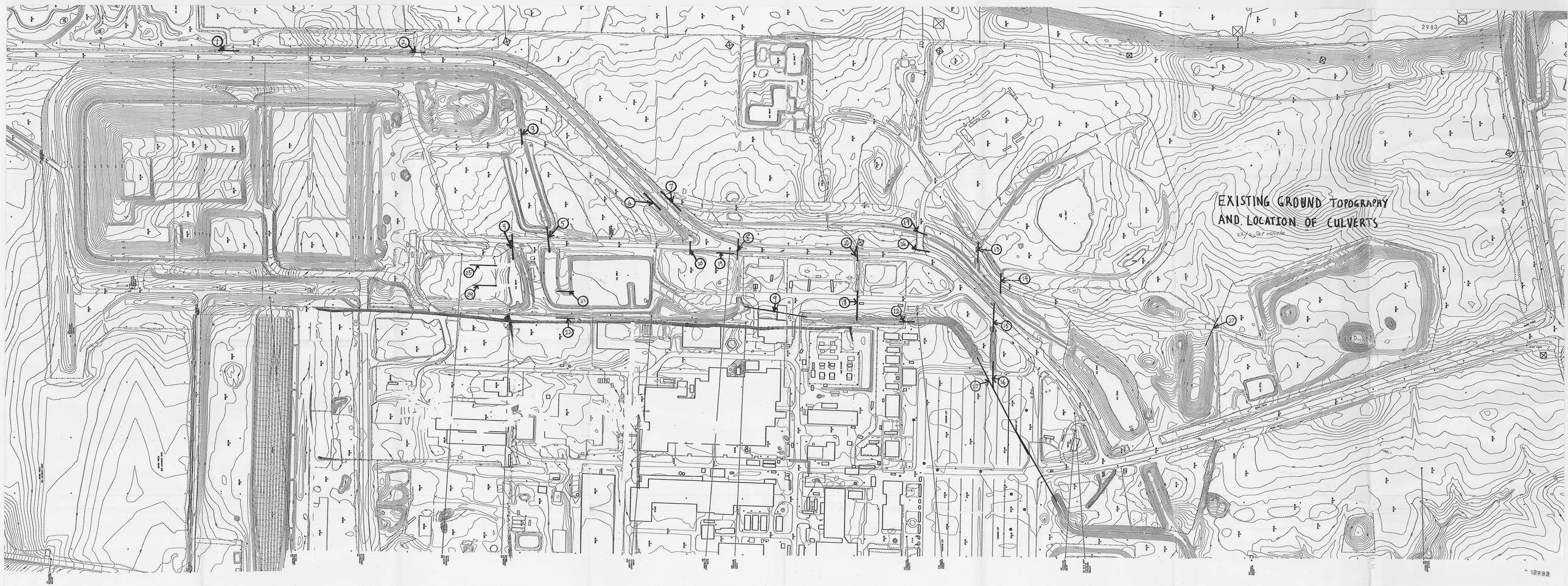
Written By :CGK/BN Date: 22 Feb 2000 Reviewed by: DTM Date: 4/5/00

Client: FF Project: Fernald OSDF Phase III Project/Proposal No.: GQ1001 Task No.: 06

BORROW AREA DEVELOPMENT
SUBAREA VOLUME VERIFICATION

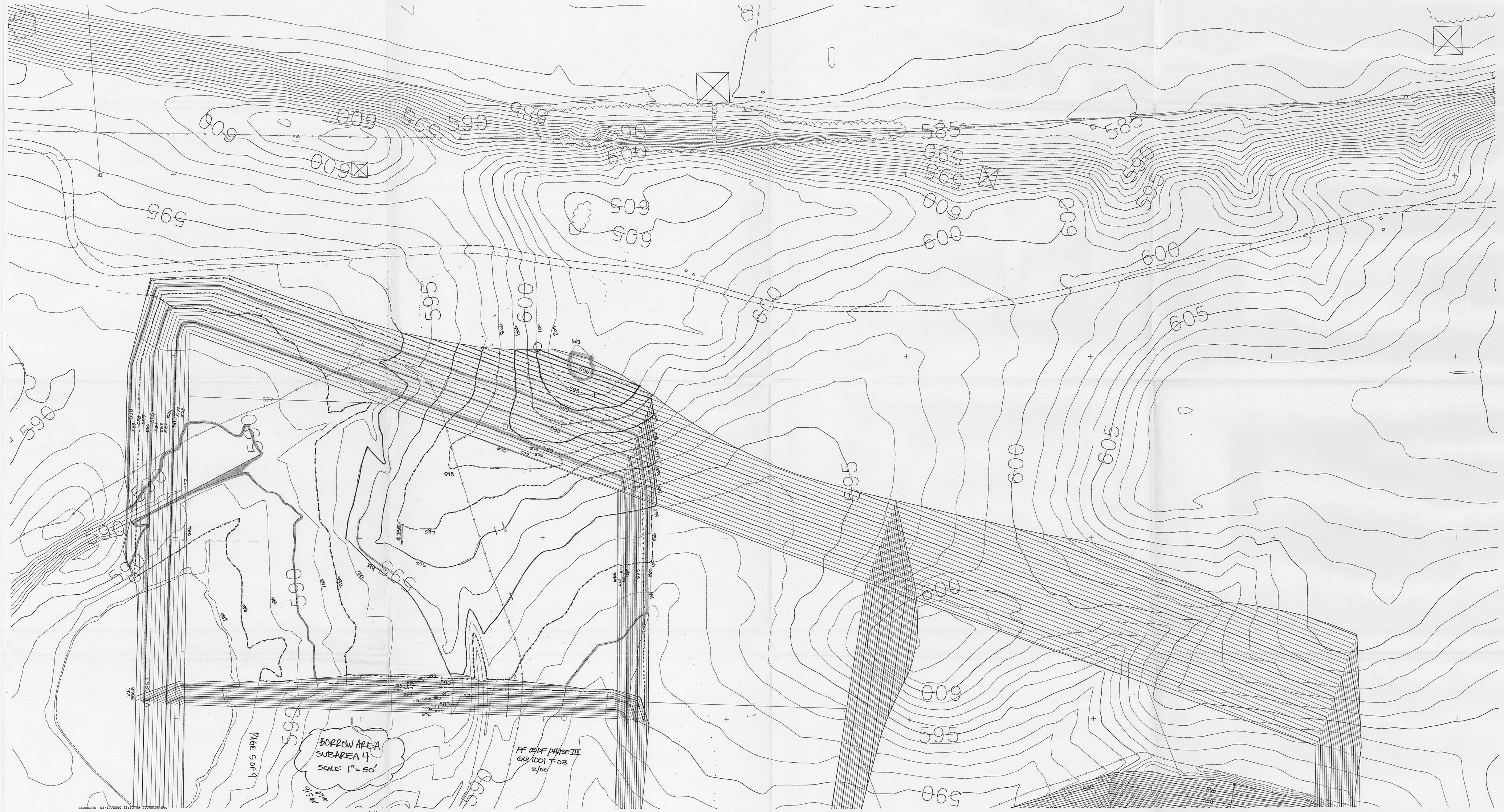
APPENDIX A

BORROW AREA DRAWINGS
FOR SUBAREAS 1 THROUGH 8

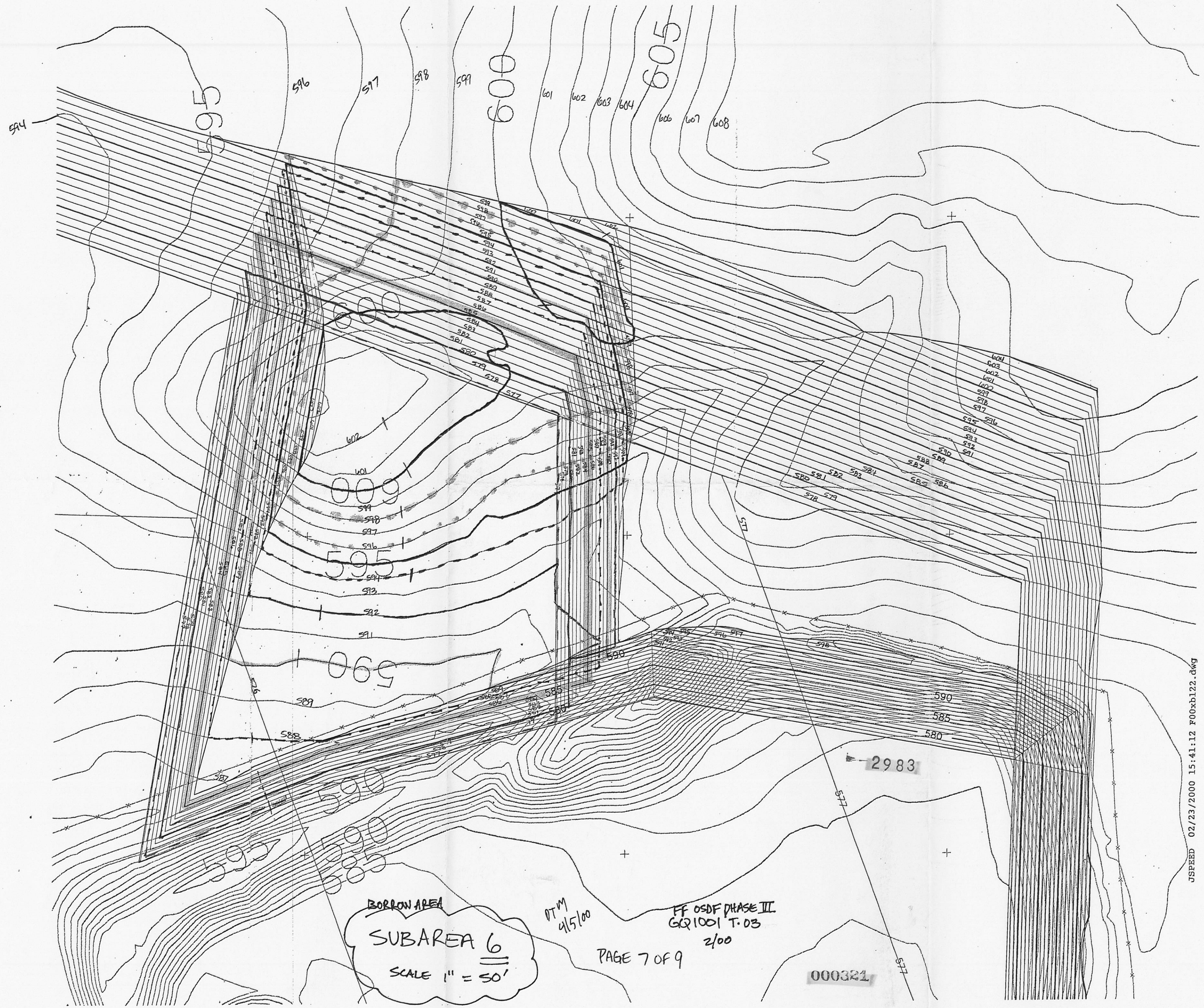

EXISTING GROUND TOPOGRAPHY
AND LOCATION OF CULVERTS
2983

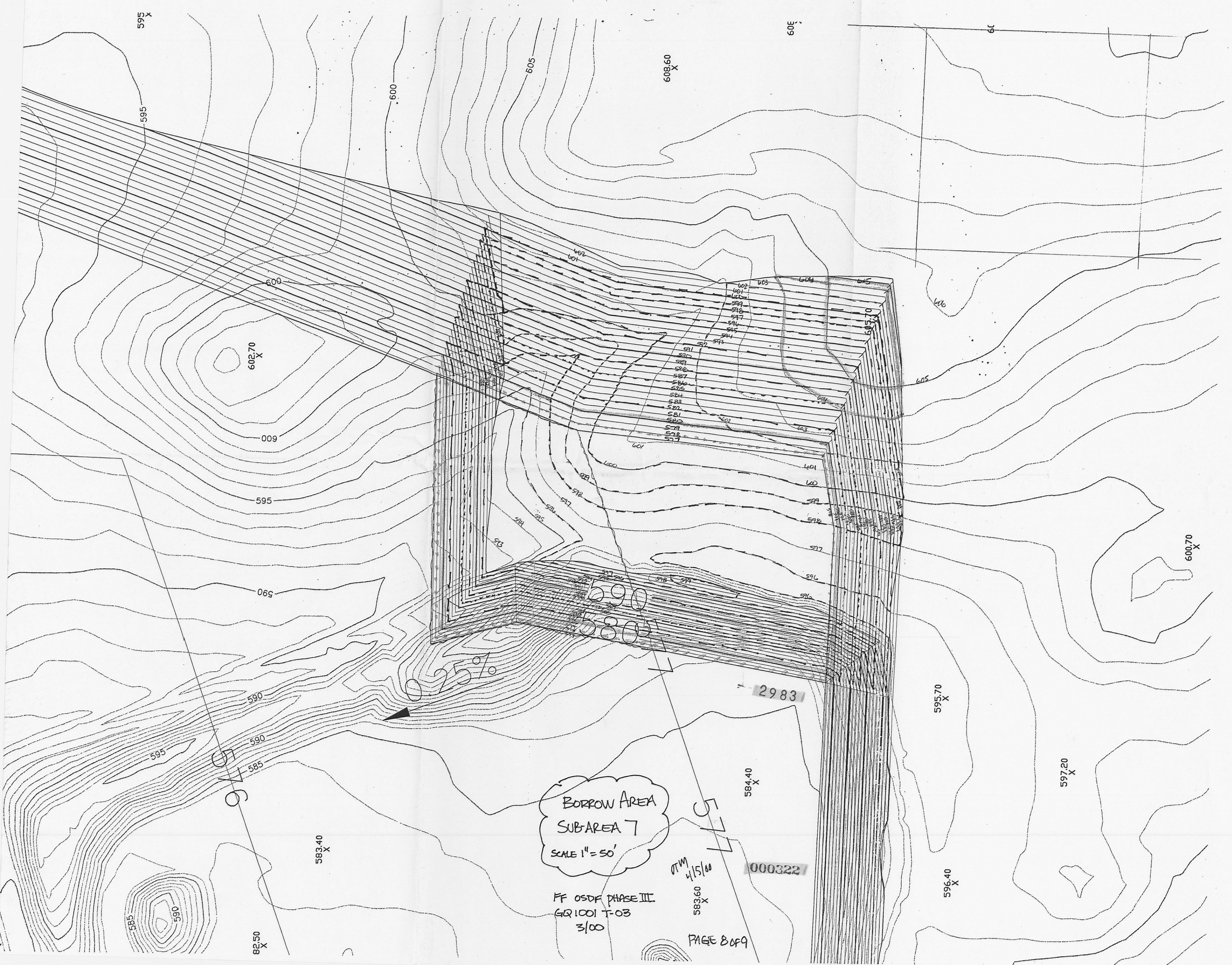
FF OSDF PHASE III
GQR1001 T-03
2/00
BORROW AREA
SUBAREA 2
SCALE: 1"=50'
FF OSDF PHASE III
GQR1001 T-03 2/00
PAGE 3 OF 9
DENSE TREES
DENSE TREES
V.E. 570.3
V.E. 570.3
V.E. 577.2
570 ELEVATION CONTINUE
ATTENTION
EAST
2S1 / 257.59
2S1 / 150.59
2S1 / 535.59
ACRES
5.708
5.746
5.770
570 ELEVATION
CONTINUE (PORTION)
ACRES
6.578
6.598
6.651

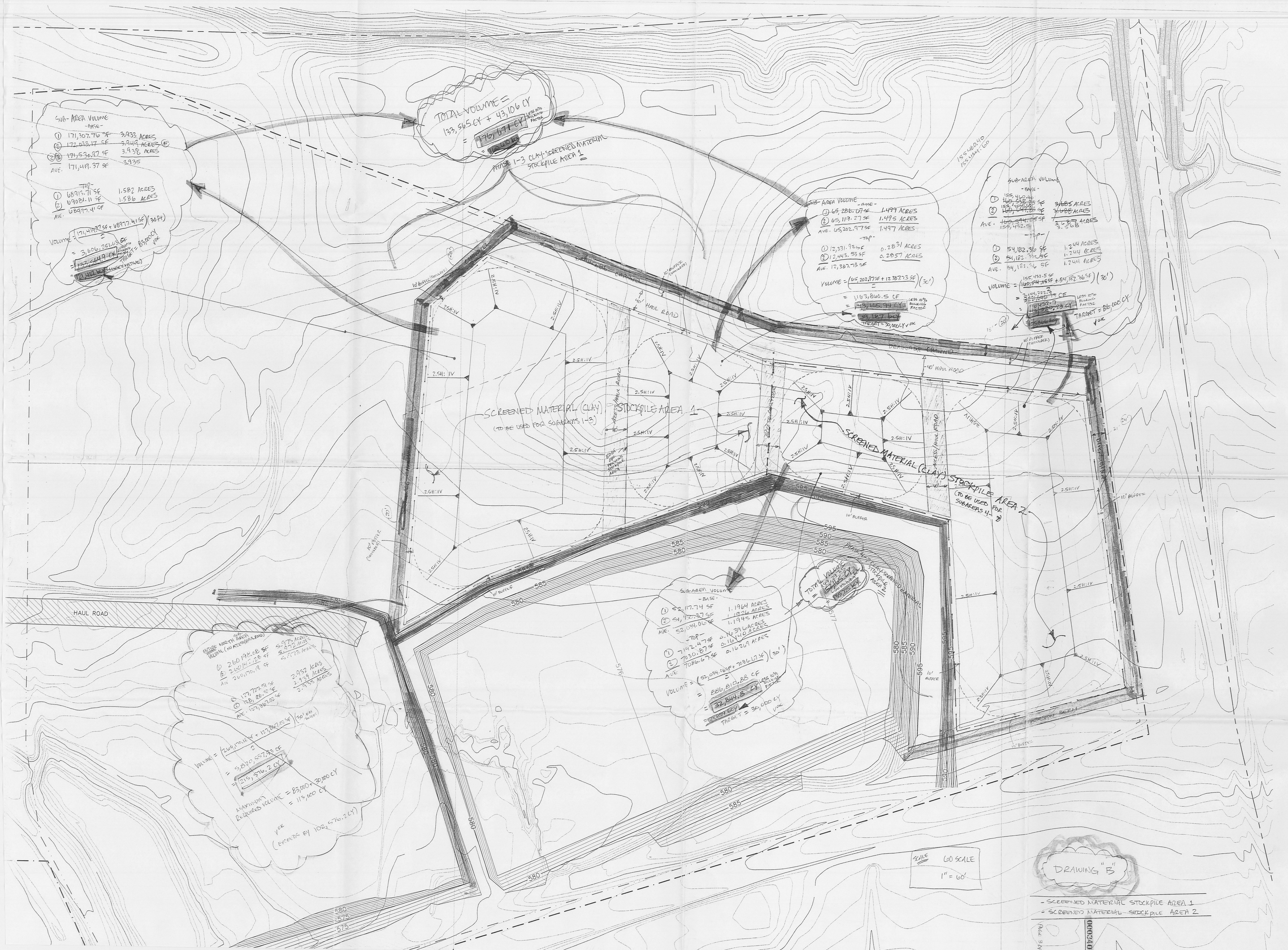
BORROW AREA SUBAREA 3
SCALE: 1"=50'
FF OSDF PHASE III GQ1001 T.03 2/00
DTM 4/5/00
PAGE 4 OF 9
2983
DENSE TREES
W.E. 570.3
W.E. 586.8


BORROW AREA
SUBAREA 4
SCALE: 1" = 50'
FF 05DF PHASE III
669/1001 T-03
2/00
PAGE 5 OF 9
WEST TREE
595
600
605
590
585
580
577

50 SCALE
BORROW AREA
SUBAREA 5
SCALE: 1" = 50'
FF OSDF PHASE III
GQ1001 T:03
2/00
DTM
4/5/00
PAGE 6 OF 9
ATTEMPT | AREA SF | ACRES
1 | 301,478.11 | 6.921
2 | 301,168.10 | 6.914
3 | 302,756.07 | 6.950
4 | 302,679.37 | 6.948
DENSE TREES
W.E. 577.2
W.E. 570.3
W.E. 586.7
W.E. 586.8
575 CONTOUR (UPPER)
570 ELEVATION CONTOUR
RAMP
PARKING
605
600
595
590
585
580
575
2983
000320


BORROW AREA
SUBAREA 6
SCALE 1" = 50'
RTM 9/5/00
FF OSDF PHASE III
GQ1001 T.03
2/00
PAGE 7 OF 9
000321
2983
JSPEED 02/23/2000 15:41:12 F00xb122.dwg


BORROW AREA
SUBAREA 7
SCALE 1" = 50'
FF OSDF PHASE III
GQ1001 T-03
3/00
PAGE 8 OF 9
000322
2983
0.25%
595
590
585
600
605
608.60
602.70
595.70
597.20
596.40
600.70
584.40
583.60
583.40
582.50


TOTAL VOLUME = 133,565 CY + 43,106 CY = 176,671 CY
LESS 10% BULKING FACTOR
PHASE 1-3 CLAY SCREENED MATERIAL STOCKPILE AREA 1
SUB-AREA VOLUME
-BASE-
① 171,307.76 SF 3.933 ACRES
② 172,033.17 SF 3.949 ACRES
③ 171,536.97 SF 3.938 ACRES
AVE. 171,419.37 SF 3.935
-TOP-
① 68913.71 SF 1.582 ACRES
② 69081.11 SF 1.586 ACRES
AVE. 68997.41 SF
VOLUME = (171,419.37 SF + 68997.41 SF)/2 (30 FT)
= 3,606,231.63 CF
= 133,564.9 CY
= 176,671 CY
VOL
SUB-AREA VOLUME
-BASE-
① 65,286.07 SF 1.499 ACRES
② 65,119.27 SF 1.495 ACRES
AVE. 65,202.97 SF 1.497 ACRES
-TOP-
① 12,331.93 SF 0.2831 ACRES
② 12,443.53 SF 0.2857 ACRES
AVE. 12,387.73 SF
VOLUME = (65,202.97 SF + 12,387.73 SF)/2 (30')
= 1,163,860.5 CF
= 43,106.94 CY
TARGET = 30,000 CY VOL
SUB-AREA VOLUME
-BASE-
① 155,452.5 SF 3.685 ACRES
② 155,452.5 SF 3.568 ACRES
AVE. 155,452.5 3.568 ACRES
-TOP-
① 54,182.36 SF 1.244 ACRES
② 54,182.36 SF 1.244 ACRES
AVE. 54,182.36 SF 1.244 ACRES
VOLUME = (155,452.5 SF + 54,182.36 SF)/2 (30')
= 3,214,777.9 CF
= 119,252.7 CY
TARGET = 86,000 CY
VOL
LESS 10% BULKING FACTOR
DRAINAGE CHANNEL
40' HAUL ROAD
10' BUFFER (MINIMUM)
2.5H:1V
SCREENED MATERIAL (CLAY) STOCKPILE AREA 1
(TO BE USED FOR SUBAREAS 1-3)
ACCESS/HAUL ROAD
EDGE OF PROPOSED STOCKPILE AREA
SCREENED MATERIAL (CLAY) STOCKPILE AREA 2
(TO BE USED FOR SUBAREAS 4-7)
10' BUFFER
HAUL ROAD
SUB-AREA VOLUME
-BASE-
① 52,117.74 SF 1.1964 ACRES
② 51,950.37 SF 1.1926 ACRES
AVE. 52,034.06 SF 1.1945 ACRES
-TOP-
① 7142.47 SF 0.1639 ACRES
② 7030.87 SF 0.1614 ACRES
AVE. 7086.67 SF 0.1627 ACRES
VOLUME = (52,034.06 SF + 7086.67 SF)/2 (30')
= 886,810.88 CF
= 32,844.8 CY
TARGET = 30,000 CY
VOL
LESS 10% BULKING FACTOR
TOTAL VOLUME = 52,034.06 CY + 32,844.8 CY = 84,878.8 CY
SCREENED MATERIAL STOCKPILE AREA 2
FUTURE NORTH SUB-AREA VOLUME (NO ACCESS/HAUL ROAD)
① 260,192.08 SF 5.973 ACRES
② 260,147.28 SF 5.972 ACRES
AVE. 260,170.18 SF 5.973 ACRES
① 127,727.91 SF 2.932 ACRES
② 128,118.05 SF 2.941 ACRES
AVE. 127,862.05 SF 2.935 ACRES
VOLUME = (260,170.18 SF + 127,862.05 SF)/2 (30' AVG. HEIGHT)
= 5,820,557.98 CF
= 215,576.2 CY
MAXIMUM REQUIRED VOLUME = 83,000 + 30,000 CY = 113,000 CY
VOL
(EXCEEDS BY 102,576.2 CY)
SCALE
60 SCALE
1" = 60'
DRAWING "B"
- SCREENED MATERIAL STOCKPILE AREA 1
- SCREENED MATERIAL STOCKPILE AREA 2

Written By :CGK/BN Date: 22 Feb 2000 Reviewed by: DTM Date: 4/5/00

Client: FF Project: Fernald OSDF Phase III Project/Proposal No.: GQ1001 Task No.: 06

2983

BORROW AREA DEVELOPMENT
SUBAREA VOLUME VERIFICATION

APPENDIX B

COMPUTER SOFTWARE USED TO PREPARE CALCULATION PACKAGE

1. Autodesk®, Inc., AutoCAD® Release 14 Users Guide,

2. SOFTDESK®, Inc., SOFTDESK® Civil/Survey: Earthworks Reference Manual, Henner, NH, November 1994.

Written By :CGK Date: 22 Feb 2000 Reviewed by: MM Date: 4/5/00

Client: FF Project: Fernald OSDF Phase III Project/Proposal No.: GQ1030 Task No.: 06

2983

BORROW AREA DEVELOPMENT
SUBAREA VOLUME VERIFICATION

APPENDIX B

COMPUTER SOFTWARE USED TO PREPARE CALCULATION PACKAGE

1. **Autodesk®, Inc., AutoCAD® Release 14 Users Guide,**

2. **SOFTDESK®, Inc., SOFTDESK® Civil/Survey: Earthworks Reference Manual, Henner, NH, November 1994.**

15.8 Borrow Area Stockpile Area Capacity Calculation

2983

COMPUTATION COVER SHEET

SUBJECT OF COMPUTATIONS Borrow Area Stockpile Area Capacity Calculation

Computations By:
(Cognizant Engineer)

Signature: William H. Nagel — Date: 05 APRIL 2000

Printed Name and Title: William H. Nagel, Staff Engineer

Assumptions and Procedures Checked By:
(Checker)

Signature: Douglas T. Mandeville — Date: 05 April 2000

Printed Name and Title: Douglas T. Mandeville, Staff Engineer

Computations Checked by:

Signature: Douglas T. Mandeville — Date: 05 April 2000

Printed Name and Title: Douglas T. Mandeville, Staff Engineer

Computations Backchecked by:
(Cognizant Engineer)

Signature: William H. Nagel — Date: 05 APRIL 2000

Printed Name and Title: William H. Nagel, Staff Engineer

Approved by:
(PDP)

Signature: [illegible] — Date: 02 May 2000

Printed Name and Title: R. David Espinoza, Senior Project Engineer

Approved by:
(DTL and TETL)

Signature: J F Beck — Date: 5 May 2000

Printed Name and Title: J.F. Beck, PRINCIPAL

Record of Revision (Number and initial all revisions)

Rev. No. DLVL 16 MAY 2000	Reason	Date	By	Checked	Approval
0A	Pre-Final Submittal to Fluor Fernald, Inc./DOE	5 APRIL 2000	WHN	DLVL FOR DTM	JFB
1A	FINAL SUBMITTAL TO FLUOR FERNALD, INC./DOE	16 MAY 2000	WHN	DLVL FOR DTM	JFB

Written By :CGK/WHN Date: 4 April 2000 Reviewed by: DTM Date: 4/5/00

Client: FF Project: FERNALD OSDF Project/Proposal No.: GQ1001 Task No.: 06

STOCKPILE AREA CAPACITY CALCULATION

EXECUTIVE SUMMARY

PURPOSE OF ANALYSIS

The purpose of this package is to calculate the capacity of the stockpile areas within the East Field Borrow Area (Borrow Area) at the Fernald Environmental Management Project (FEMP) On-Site Disposal Facility (OSDF). The calculation was used to verify that the allotted stockpile areas are large enough to hold the required volume during borrow area excavation.

METHODS OF ANALYSIS

The stockpile area capacity is calculated by the following procedure:

Step 1: Determine the worst-case scenario (most material to be stockpiled) for each stockpile area within a given phase of borrow area development.

Step 2: Determine the configuration of the stockpiles within the stockpile area, accounting for drainage, haul roads, and maximum slope angle.

Step 3: Calculate the volume of the stockpiles within the stockpile area.

The capacity of stockpiled material is then compared to the actual volume from the borrow area to evaluate whether there is adequate space in the stockpile area for the stockpiled soil.

Written By: CGK / WHN Date: 4 April 2000 Reviewed by: DTM Date: 4/5/00

Client: FF Project: FERNALD OSDF Project/Proposal No.: GQ1001 Task No.: 06

2983

CONCLUSIONS

There are three stockpile areas to be used within the borrow area: i) Screened Material (Clay) Stockpile Area 1, ii) Screened Material (Clay) Stockpile Area 2, and iii) South Stockpile Area. The capacity of each stockpile area was determined and compared to the volume of soil coming from the borrow area. Two stockpiles were assumed for each stockpile area corresponding to two separate subarea source locations. The following stockpile areas were calculated and compared to their required volumes:

Stockpile Area	Initial Construction Season (Source 1) Stockpile		Following Construction Season (Source 2) Stockpile	
	Capacity (bcy)	Required (bcy)	Capacity (bcy)	Required (bcy)
Screened Material Stockpile Area 1	121,422	83,000	39,187	30,000
Screened Material Stockpile Area 2	105,866	86,000	29,859	30,000

Stockpile Area	Ordinary Borrow Material		Topsoil	
	Capacity (bcy)	Required (bcy)	Capacity (bcy)	Required (bcy)
South Stockpile Area	80,870	86,000	7,236	8,000

The capacity of Screened Material Stockpile Areas 1 and 2 each meet or exceed the incoming material volumes from the borrow area (with the exception of Screened Material Stockpile Area 2, where the capacity is within 0.5 percent of the required volume). The capacity of the South Stockpile Area is approximately 6.7 percent less than the required volume from the borrow area (6.3 percent for ordinary borrow and 10.6 percent for topsoil). However, the analysis is conservative in that it assumes

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the South Stockpile Area will stockpile ordinary borrow soil and topsoil in the entire subarea during one construction season when in fact the topsoil and ordinary borrow will be used for OSDF construction at the same time it is excavated. In addition, the size and extent of the South Stockpile Area is constrained on all sides by fixed boundary limits. Therefore, the size of the South Stockpile Area and Screened Material Stockpile Areas 1 and 2 are of sufficient capacity to temporarily hold the required volumes of material excavated from the borrow area.

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STOCKPILE AREA CAPACITY CALCULATION

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STOCKPILE AREA CAPACITY CALCULATION

CALCULATION PROCEDURES

The maximum amount of material to be stockpiled is determined by taking the development of a subarea with the largest quantity of excavated material and stockpiling the material in a stockpile area. For the South Stockpile Area (Appendix A, Drawing A, Attached), the largest quantity of excavated material is determined over one construction season (one subarea of development). This is a conservative approach because the ordinary borrow soil and topsoil removed from a subarea within a given construction season are to be used for OSDF construction during that same year. Hence, it is unlikely that the entire amount of ordinary borrow soil and topsoil from a given subarea will be stockpiled at the same time. Two stockpiles were assumed for the South Stockpile Area: one for ordinary borrow soil and one for topsoil.

For Screened Material Stockpile Area 1 (Appendix A, Drawing B, attached), the largest quantity of excavated material is determined by summing the largest quantity of excavated material over two consecutive construction seasons (two subareas of development) within development of subareas 1, 2 and 3 of the borrow area. Similarly, the largest quantity of excavated material for Screened Material Stockpile Area 2 (Appendix A, Drawing B, attached) is determined by summing the largest quantity of excavated material over two consecutive construction seasons (two subareas of development) within development of subareas 4, 5, 6 and 7 of the borrow area. (Borrow area subareas 1, 2, 3, 4, 5, 6, 7 and 8 are presented in the Borrow Area Management and Restoration Plan (BAMR) figures and in the OSDF Phase III Borrow Area Stockpile Area Capacity Calculation package included herein.) This approach is conservative because stockpiled screened material (clay) from the first construction season will be (i.e., adding the volume of stockpile for two consecutive seasons) removed and used for OSDF construction as screened material (clay) when the second construction season material is stockpiled. Hence, it is unlikely that the entire amount of clay from two subareas will be stockpiled at the same time. Two

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stockpiles were assumed for each screened material stockpile area: one for screened material (clay) from the first construction season and one for screened material (clay) from the second construction season.

The volume of material placed in the stockpile area during development of the borrow area is estimated using the end-area method, as detailed in the steps below:

1. Estimate plane areas of stockpiled material at specific locations

Estimate the plane areas at the top and bottom of each stockpile by measuring stockpile areas as shown on Drawings A and B. Measure these areas with a planimeter.

2. Estimate volumes of each stockpile

The total volume of each stockpile can be calculated from measuring the area of the planes at the top and bottom of each stockpile. Estimate the volume in a stockpile of height Δz by using one of the following formulas:

For a stockpile that comes to a point at the top (i.e., pyramidal shape), the volume (V_z) is calculated as:

$$V_z = 1/3\ A_z \Delta z$$

Where: A_z = plane area of stockpile at the base

Δz = height of the stockpile (typically 30 feet)

For a stockpile with areas at elevation 'z+Δz' and at elevation 'z' equal to $A_{z+\Delta z}$ and A_z, respectively, the stockpile volume (V_z) is calculated as:

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$V_z = (A_z + A_{z+\Delta z})\Delta z/2$ (i.e., "average-end-area" volume)

Note that the average-end-area volume is not the true volume of the proposed stockpile. Equations for the true volume of the estimated volume, or *prismoidal volume*, can be found in most surveying texts. The average-end-area volume is accurate enough for the purposes of this calculation, and for most earthwork volume estimates [Wolf and Brinker, 1989].

REFERENCES

GeoSyntec Consultants, "*Final Design Package*," Fernald Environmental Management Project, DOE Fernald Area Office, Fernald, OH, 1997.

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GeoSyntec Consultants, "*Borrow Area Strategy Report*," Fernald Environmental Management Project, Fernald Area Office, Fernald, OH, 1999.

Wolf, P.R., and Brinker, R.C., "*Elementary Surveying*," 8th ed., Harper and Row Publishers, New York, NY, 1989, 696 p.

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STOCKPILE AREA CAPACITY CALCULATION

DATA VERIFICATION

ASSUMPTIONS

The following assumptions were made for orientation of the stockpile areas:

1. The South Stockpile Area will have two separate stockpiles separated by a haul road: one for topsoil and the other for ordinary borrow;
2. The South Stockpile Area shall contain ordinary borrow soil and topsoil from one subarea during one construction season;
3. The Screened Material Stockpile Areas will each have two separate stockpiles separated by a haul road: one for clay from a subarea excavation during the current construction season and one from a subarea excavation from the previous construction season;
4. Haul roads are 40 feet wide;
5. Stockpiles shall be a maximum of 30 feet in height and have maximum side slopes of 2.5H:1V (Horizontal:Vertical);
6. All existing structures/appurtenances within the stockpile area shall be removed;
7. All stockpiles are separated from cut slopes by a minimum distance of 50 feet;
8. A 10-foot wide buffer is placed between the stockpiles and the stockpile area boundaries; and
9. The calculated stockpile volumes are conservatively reduced by 10% to take into consideration any additional bulking of the excavated soils.

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REQUIRED VOLUMES

The following required volumes were calculated for each stockpile:

	Initial Construction Season Volume (bcy)	Following Construction Season Volume (bcy)
South Stockpile Area	94,000*	-------
Screened Material Stockpile Area 1	83,000	30,000
Screened Material Stockpile Area 2	86,000	30,000

* 94,000 bcy is comprised of 86,000 bcy of ordinary borrow material and 8,000 bcy topsoil.

Volumes for each subarea of borrow area development were obtained from the Borrow Area Strategy Report (BASR) [GeoSyntec, 1999]. The purpose of the BASR is to provide a general overall plan for borrow area excavation and restoration grading. A summary of subarea volumes obtained from the BASR is presented in Table 1 (attached).

AVAILABLE VOLUMES

The calculations for the volume available for screened material stockpile areas 1 and 2, and the south stockpile area are presented in Appendix A and are summarized in the tables below.

Stockpile Area	Initial Construction Season (Source 1) Stockpile	Following Construction Season (Source 2) Stockpile
	Capacity (bcy)	Capacity (bcy)
Screened Material Stockpile Area 1	121,422	39,187
Screened Material Stockpile Area 2	105,866	29,859

000336

Stockpile Area	Ordinary Borrow Material	Topsoil
	Capacity (bcy)	Capacity (bcy)
South Stockpile Area	80,870	7,236

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BORROW AREA DEVELOPMENT BY SUB-AREA (Quantities in Thousands)

clay conversion factor	=	1.24
common borrow conversion factor		1.00
topsoil conversion factor	=	1.02

BORROW AREA PHASE	CALENDAR YEAR	CLAY CONSTRUCTION ACTIVITIES:		CLAY MATERIAL:			ORDINARY BORROW AND TOPSOIL**		TOTAL BORROW EXCAVATION	TOTAL
		PROCESS CLAY	HAUL & PLACE CLAY	EXCAVATE (bcy)	PROCESS/STOCKPILE (icy)	PLACE (icy)	EXCAVATE (bcy)	PLACE (icy)	(bcy)	(icy)
--	2000	C1, L4	NONE	83	67	0	0	0	83	0
Base Bid	2001	C-2	C1, L4	30	24	67	94	94	124	161
1	2002	L-5, C-3	C-2	81	65	24	45	45	126	69
2	2003	L-6, C-4	L-5, C-3	86	69	65	84	84	170	149
3	2004	C-5	L-6, C-4	30	24	69	80	80	110	149
4	2005	C-6	C-5	33	27	24	45	45	78	69
5	2006	NONE	C-6	—	—	27	56	56	56	83
6	contingency									
--										
TOTALS:		L=Liner, C=Cap		343	277	277	404	404	747	681

** SEE BELOW FOR BREAKDOWN of ORDINARY BORROW AND TOPSOIL:

ORDINARY BORROW MATERIAL:		TOPSOIL:		TOTAL	TOTAL
EXCAVATE (bcy)	PLACE (icy)	EXCAVATE (bcy)	PLACE (icy)	(bcy)	(icy)
0	0	0	0	0	0
86 +	86	8	8	= 94	94
38	38	7	7	45	45
77	77	7	7	84	84
73	73	7	7	80	80
38	38	7	7	45	45
48	48	8	8	56	56
360	360	44	43	404	403

KEY:
CRITICAL CONDITION

- SCREENED MATERIAL STOCKPILE: AREA 1
  "WORST CASE" CONDITION FOR PHASE 1-3 CLAY STOCKPILE FOR 2-YR PERIOD.
  83,000 CY + 30,000 CY = 113,000 CY
- SCREENED MATERIAL STOCKPILE: AREA 2
  "WORST CASE" CONDITION FOR PHASE 4-7 CLAY STOCKPILE FOR 2-YR PERIOD.
  86,000 CY + 30,000 CY = 116,000 CY
- SOUTH STOCKPILE AREA
  "WORST CASE" CONDITION FOR MAX. 1-YR PERIOD
  (i) ORDINARY BORROW = 86,000 CY (MAX. STORAGE)
  (ii) TOPSOIL = 8,000 CY (MAX. STORAGE)

TABLE 1 SUBAREA VOLUMES

(INFORMATION OBTAINED FROM BORROW AREA STRATEGY REPORT (BASR) 2/2/00 [GEOSYNTEC, 1999].

GeoSyntec Consultants
BA development

000237

029883

DTM 4/15/00

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STOCKPILE AREA CAPACITY CALCULATION

APPENDIX A

SOUTH STOCKPILE, SCREENED MATERIAL STOCKPILE AREA 1 AND SCREENED MATERIAL STOCKPILE AREA 2 DRAWINGS

570 ELEVATION CONTOUR
575 ELEVATION CONTOUR (APPROX)
BORROW AREA
SUBAREA 1
SCALE: 1" = 50'
OSDF PHASE III
GQ1001 T-03
2/00
FF OSDF PHASE III
GQ1001 T-03
2/00
50 SCALE
PARKING
RAMP
DENSE TREES
V.E. 570.3
V.E. 577.2
V.E. 586.7
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IPTIONS:

EXISTING CHANNEL SEGMENT WITHIN STOCKPILE AREA TO BE FILLED OVER. (IE, TO BECOME USABLE SPACE).

) ACCESS/HAUL ROAD WIDTH SET AT MAX. 40-FT (ASSUMED ADEQUATE)

) HAUL/TRANSPORT ROAD (ALONG SOUTHERNMOST EDGE OF STOCKPILE) WAS RELOCATED FROM WITHIN STOCKPILE AREA TO OUTSIDE LIMITS OF STOCKPILE ~~AS SHOWN AT RIGHT~~ AND THEN RELOCATED TO SOUTH SIDE OF STOCKPILE AREA AS SHOWN. ~~IT IS NOTED~~ THAT THE ROAD MAY, AT THE DISCRETION OF THE CONSTRUCTION MANAGER, BE RELOCATED ~~MOVED OVER~~ SLIGHTLY TO UTILIZE EXISTING ACCESS ROAD ~~AND PARKING AREA~~.

4) TOP SOIL AND ORDINARY BORROW MATERIAL STOCKPILES WILL BE SEGREGATED BY A HAUL/ACCESS ROAD.

5) STOCKPILE DIMENSIONS: 2.5H:1V SIDE SLOPES @ 75' LENGTHS TO ATTAIN MAX. STOCKPILE HEIGHT OF 30-FT.

TES: ALL AREAS (~~OUTLINED~~ IN HIGHLIGHTER) HAVE BEEN ESTIMATED USING PLANIMETER, MINIMUM (2) ITERATIONS USING ~~PARTIO~~ AVERAGE VALUES.

TOPSOIL STOCKPILE

- BASE -

(1) 19,474.41 SF = 0.4471 ACRES
(2) 19,306.99 SF = 0.4432 ACRES
AVE. 19,390.7 SF 0.4451 ACRES

- MID-SLOPE (15' HEIGHT) -

(1) 4575.65 SF = 0.1050 ACRES
(2) 4631.45 SF = 0.1063 ACRES
AVE. 4603.55 SF 0.1057 ACRES

- TOP OF SLOPE (30' HEIGHT) -

(1) 55.80 SF
(2) 55.80 SF
AVE. 55.80 SF

$$\text{VOLUME} = \left(\frac{19{,}390.7\text{ SF} + 4603.55\text{ SF}}{2}\right)(15\text{-FT}) + \left(\frac{4603.55\text{ SF} + 55.80\text{ SF}}{2}\right)(15\text{ FT})$$

= 214,902 CF
= 7959.3 CY - LESS 10% BULKING FACTOR (TO BE CONSERVATIVE)
= 7,236 CY ~~TARGET~~ = 8,000 CY = VOL

NOTE: STOCKPILE FOOT PRINT CONSIDERED IS LARGE CIRCLE; CONSIDERABLE ADDITIONAL SPACE EXISTS OUTSIDE OF THIS AREA WITHIN THE DESIGNATED TOPSOIL STOCKPILING AREA.

ORDINARY BORROW MATERIAL STOCKPILE

- BASE -

(1) 120,696.64 SF = 2.771 ACRES
(2) 120,585.04 SF = 2.768 ACRES
AVE. 120,640.84 SF 2.769 ACRES

- TOP (HEIGHT = 30') -

(1) 39,339.41 SF = 0.9031 ACRES
(2) 39,618.41 SF = 0.9095 ACRES
AVE. 39,478.91 SF 0.9063 ACRES

$$\text{VOLUME} = \left(\frac{120{,}640.84\text{ SF} + 39{,}478.91\text{ SF}}{2}\right)(30\text{-FT})$$

= 2,401,796.25 CF
= 88,955.4 CY LESS 10% BULKING FACTOR (TO BE CONSERVATIVE)
= 80,870 CY ~~TARGET~~ = 86,000 CY

VOL BEFORE APPLYING BULKING FACTOR
(10% SHORT FALL WITH BULKING FACTOR)

60 scale