

RUN-ON AND RUN-OFF CONTROL SYSTEM PLAN

CHOCTAW GENERATION LIMITED PARTNERSHIP, L.L.P.
RED HILLS OPERATION
2391 PENSACOLA ROAD
ACKERMAN, MS 39735
(662) 387-5758



ENVIRONMENTAL COMPLIANCE & SAFETY, INC.

Post Office Box 356
Sherman, Mississippi 38869
Office: (662) 840-5945
Fax: (662) 840-5965
www.envirocomp.net

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Record of Revisions

Run-On and Run-Off Control System Plan

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1.0 INTRODUCTION AND CERTIFICATION

1.1 SITE DESCRIPTION AND REGULATORY APPLICABILITY

The Choctaw Generation Limited Partnership, L.L.L.P. – Red Hills Operations (Red Hills) is located near the City of Ackerman in Choctaw County, Mississippi. Red Hills is in north central Mississippi on a 170-acre site. Red Hills is bounded on the south by Pensacola Road and is about ½ mile west of US Highway 9. Figure 1 shows the location of the site. Red Hills operates a single unit electrical generation facility designed to generate electricity for dispatch to the Tennessee Valley Authority (TVA) electrical system. The primary boiler fuel is lignite coal. As a result of combusting lignite coal, ash is created and must be disposed or re-purposed. Red Hills owns and operates an existing Ash Management Unit (AMU) for the placement and disposal of ash. The AMU is considered a Coal Combustion Residuals (CCR) Unit in accordance with 40 CFR Part 257, Subpart D. The AMU is located in the northeastern portion of property and consists of three (3) cells encompassing a total of approximately 90 acres of the Red Hills site. Figure 2 shows an aerial view of the site, Figure 3 shows a diagram of the Run-On and Run-Off flow directions, and Figure 4 shows a topographical diagram of the AMU.

The ash generated at the site has been approved by the Mississippi Department of Environmental Quality (MDEQ) for beneficial use as a road construction stabilizer. The approval of the ash for beneficial use has reduced the amount of ash stored in the AMU annually and will extend the life of the AMU. The majority of the ash generated is transported to the adjacent North American Coal – Red Hills Mine to be used for road stabilization and construction.

This site is required to comply with the Coal Combustion Residue Rule (40 CFR Part 257, Subpart D). As an existing CCR Unit, Red Hills must be in compliance with the following requirements:

- Design, construct, operate, and maintain a run-on control system to prevent flow onto the active portion of the CCR Unit during the peak discharge from a 24-hour, 25-year storm as required by §257.81(a)(1);
- Design, construct, operate, and maintain a run-off control system from the active portion of the CCR Unit to collect and control at least the water volume resulting from a 24-hour, 25-year storm as required by §257.81(a)(2);
- Handle run-off from the active portion of the CCR Unit in accordance with the surface water requirements under §257.3-3 as required by §257.81(b) and NPDES Permit No. MS0054496; and
- Develop a Run-on and Run-off Control System Plan that documents how the run-on and run-off control systems have been designed and constructed to meet applicable requirements of §257.81.

1.2 PROFESSIONAL ENGINEER CERTIFICATION

After a review of the existing Run-On and Run-Off Control System, it is believed that the system has been designed and constructed to meet the requirements of 40 CFR 257, Subpart D. The owner or operator of

a CCR Unit must design, construct, operate, and maintain a storm water management system that consists of sufficient run-on and run-off controls. The system should be effective in preventing flow onto the active portion of the CCR Unit during peak discharge from a 24-hour, 25-year storm while collecting and controlling run-off of at least the water volume resulting from a 24-hour, 25-year storm. In addition, the system should be able to handle run-off from the CCR Unit in accordance with the surface water requirements under §257.3-3.

I hereby certify that I am familiar with the provisions of the final rule to regulate the disposal of Coal Combustion Residuals as solid waste under Subtitle D of the Resource Conservation and Recovery Act (RCRA). I also attest that I, or an agent under my supervision, have reviewed the Run-on and Run-off controls for the CCR Unit, the Run-on and Run-off control locations, and that the design of the system appears to be adequate to comply with the CCR requirements.



Brian S. Ketchum, P.E.
Senior Engineer
Environmental Compliance & Safety, Inc.



10/14/16

Date

State of Mississippi
Registration No. 13372
(Seal)



BSK

2.0 STORM WATER MANAGEMENT SYSTEM

2.1 ACCESS CONTROLS

Perimeter fencing around the entire Red Hills site provides a physical enclosure of the AMU and limits individual and vehicle access. All fences will be of a chain-link security type. Access to the site is controlled through the main plant gate by the use of a slide-card access system or remotely operated by the Control Room via camera system. The main gate (and all other gates) will be secured 24 hours a day to allow only access by the plant personnel or other authorized personnel.

2.2 SYSTEM COMPONENTS

The AMU consists of three (3) cells which, if completely utilized, will bring the AMU to a final crowned configuration. The cells will be filled to an interim grade of approximately 580 feet mean sea level (msl). When all three (3) cells are filled to an elevation of approximately 580 ft msl, then additional fill will be added to the entire footprint of the cells to bring final grades to approximately 640 ft msl, which is the AMU closure final grade. The AMU uses the significant natural relief of the existing land to route storm water along natural drainage pathways and graded runoff storm water collection system to a sump at a low area near the AMU Basin.

The storm water management system consists of two (2) primary components: the AMU Basin and the perimeter drainage system. The first component, the AMU Basin, provides containment of any storm water that has the potential to come in contact with the ash once placed in the AMU. Leachate generated from the AMU will also be discharged to the AMU Basin by way of a leachate sump. The second component, the perimeter drainage system, consists of a ditch designed to capture all flow coming in contact with the landfill surface, a center berm to provide containment and separation of run-on and runoff, and an exterior ditch designed to intercept any flow that would flow onto the AMU footprint. This configuration (interior ditch, berm, exterior ditch) was constructed in phases as a permanent structure around the entire AMU and is currently being maintained. Facility diagrams showing the Run-On and Run-Off flow directions and the AMU topography are included as Figure 3 and Figure 4, respectively.

2.2.1 Ash Management Unit Basin

The AMU Basin was constructed with a liner system consisting of high density polyethylene (HDPE) flexible membrane and a geosynthetic clay liner having a hydraulic conductivity of at least 1×10^{-7} cm/s. The AMU Basin has been sized to handle storm water flow from the cells for a 25-year, 24-hour storm event, and any leachate generated from the AMU. The AMU Basin included the construction of an outlet structure at the northwest end of the AMU Basin, an inlet structure along the southern berm, and an inlet structure at the eastern end of the AMU Basin. The AMU Basin inlet and outlet structures are designed to provide adequate horizontal separation between the inlet and the discharge point. Top of berm elevation

for the Basin is approximately 490 ft msl. The design maximum operating water level for the AMU Basin is 487 ft msl.

The leachate collection sump and pump station is located south of the AMU Basin within or near the area designated as Cell I. The leachate collection sump area was lined with a compacted clay liner and a flexible membrane liner. The sump and pump station was designed to accommodate leachate generated from the entire AMU (Cells I, II, and III). Leachate collection lines will transport leachate generated in the cells to the leachate sump. Leachate stored in the sump is pumped to the AMU Basin. Currently, water from the AMU can be pumped to the power plant for treatment and use as makeup water, or recycled for ash hydration and dust control on the AMU.

2.2.2 Run-On Control Ditch System

Run-on controls consist of perimeter ditches that carry storm water drainage around the cells to the natural low point of the main ditch on the west side and east side of the AMU. Grading was required to develop appropriate ditch drainage profiles. The primary components of the storm water run-on management system include an up-gradient and down-gradient perimeter diversion dike and swales adjacent to access roadways, and culverts. The run-on control system minimizes and/or eliminates the quantity of storm water coming into contact with the ash, and therefore, minimizes the quantity of storm water that needs to be collected and conveyed to the AMU Basin. The primary components of the run-on control system and their functions are described below:

Perimeter Diversion Dike

The main component of the run-on control system is the perimeter diversion dike. This dike was installed both up-gradient and down-gradient of the fill area to divert run-on generated from outside the fill area to a discharge point down-gradient of the AMU Basin. Overland flow is routed to the natural points of the main ditch that surrounds the AMU. The dike was constructed using on-site soils prior to initiating AMU filling operations. The dike is designed to convey the storm water generated during the 25-year, 24-hour storm event.

Roadway Swales and Culverts

The AMU includes a central access roadway for incoming ash material. The access roads extend through each of the cells to the active disposal area. The access roads were constructed on a compacted base nine-inches above grade, and include storm water swales on each side of the road.

2.2.3 Run-Off Controls

Run-off controls from the cells consist of interior ditches that carry storm water drainage within the limits of ash fill area and flow into the AMU Basin. As with the run-on, these interior ditches are graded in certain areas to achieve the desired drainage slope. The run-off control also features a perimeter ditch with a high point located at the southwestern interface between Cell I and Cell II to route the flow around the east and west sides of the AMU. Storm water run-off that flows into an interior ditch along the

western portion of Cell I is routed to a point at the southwestern corner of the AMU Basin, then flows parallel to the southern berm of the Basin and discharges to the AMU Basin inlet structure near the leachate sump. Run-off from Cell I to the east of the divide flows along the eastern side of Cell I and around Cell II and Cell III to an inlet structure at the eastern end of the AMU Basin. The primary components of the storm water run-off management system are detailed below and include perimeter storm water collection swales, side slope diversion swales, downchutes, and the AMU Basin:

Perimeter Storm Water Collection Swales

Perimeter storm water collection swales were created within each active cell by filling ash to the edge of the perimeter diversion dikes. These swales collect run-off generated within the cell and route the flow to the AMU Basin. The swales are designed to convey water generated as a result of the 25-year, 24-hour storm event.

Side Slope Diversion Swales

Temporary diversion swales are constructed on external slopes to collect and divert run-off to the AMU Basin. Diversion swales are created, as the ash is placed, by constructing a bench with the ash. Ash placement continues upwards from the bench. Permanent diversion swales will be constructed on the final cap of each cell to collect and divert the run-off to the AMU Basin.

Downchutes

Downchutes may be used to collect the run-off discharge from the diversion swales. Temporary downchutes may be constructed of riprap, and final downchutes may be constructed of riprap and underlain by a geotextile filter fabric to prevent erosion of the underlying final cover. The downchutes discharge to a stilling structure for energy dissipation. The stilling structure will then discharge run-off to the perimeter swale and to the AMU Basin.

AMU Basin

The AMU Basin serves dual roles of peak flow attenuation and enhancement of storm water run-off quality by sedimentation. The dimensions of the Basin are approximately 865 feet long by 310 feet wide by 18 feet deep. The AMU Basin is designed to handle run-off from the active cells of the AMU. The size of the Basin is configured to contain the entire run-off for the 25-year, 24-hour storm event with two (2) feet of siltation and one foot of freeboard.

The AMU Basin was constructed by damming off the north and south ends of the main drainage valley on the west side of the AMU and performing some excavation of the Basin bottom area. The Basin berms were constructed with the spoil from the Basin excavation. Inlets and outlets are lined with riprap to dissipate energy. The Basin liner system was constructed meeting the same specifications as the bottom liner system of AMU Cell I. The primary Basin outlet consists of a trash rack attached to a vertical pipe and an anti-vortex device. The vertical pipe is attached to a horizontal pipe, which is securely fixed to the Basin bottom and discharged through the Basin wall at the western corner of the Basin. The secondary Basin outlet utilizes a weir spillway which also discharges at the west corner of the Basin.

2.2.4 Liner and Leachate System

Leachate collecting in the sump from the AMU leachate collection system is pumped through a metering station into the AMU Basin. The liner and leachate collection system consisted of an approved liner system (varied for each cell) with a hydraulic conductivity of at least 1×10^{-7} cm/s. The liner systems were constructed in accordance with MDEQ approval. The surface of the liner was not left exposed for an extended period of time to prevent drying and cracking. If desiccation cracking occurred, the surface was wetted, mixed, and re-compacted.

Leachate collection piping was installed in trenches within a gravel envelope. Both the gravel around the pipe and the pipe provide transport capacity of any leachate generated in the cell. The trenches drain by gravity to the leachate collection sump. Cleanouts provide upstream access for future pipe cleaning if required, and downstream access is at the leachate sump. A minimum 12-inch thick granular leachate drainage layer with a minimum hydraulic conductivity of 1×10^{-3} cm/s was placed over the liner system for each cell. The leachate collection system in Cell I and Cell II traverses the entire bottom surface of the respective cells; however, approval of the Cell III design limited the leachate collection system to roughly 25% of the cell bottom surface (approximately seven (7) acres) at the lowest elevation grade.

2.3 DESIGN CALCULATIONS

Surface water hydrologic and hydraulic calculations have been performed to establish design peak flows, run-off volumes, channel capacities, minimum channel dimensions, and slopes required to pass the design peak flows of a 25-year, 24-hour storm event. The design calculations were developed during the solid waste permitting process. Malcolm Pirnie (later acquired by Arcadis) prepared the calculations, and a copy is attached in Appendix A. The facility layout and storm water diversion systems ensure that no up gradient run-off will enter the landfill facility as run-on. Therefore, storm water considerations for the landfill are dictated by direct precipitation falling on the facility. Run-on from areas outside the fill area are diverted from coming in contact with ash, and run-off from the fill areas is captured and diverted to the AMU Basin. Storm water run-on collected by the diversion dikes will be routed around areas where ash has been placed to a discharge point down-gradient of the AMU Basin. Both the run-on (run-off from outside the fill area) and run-off (from within the fill area) controls were designed to convey the flows generated during a 25-year, 24-hour duration storm event in accordance with the CCR and Mississippi Nonhazardous Waste Management Regulations. Storm water discharges from the site are permitted under MDEQ Baseline Storm Water General Permit No. MSR001656, and the (potential) discharge of storm water collected in the AMU Basin is permitted under NPDES Direct Discharge Permit No. MS0053881. However, the AMU system is designed as a zero discharge system in compliance with §257.3-3.

3.0 AMENDMENTS AND INSPECTIONS

3.1 AMENDMENTS

The Run-on and Run-off Control System Plan may be amended at any time provided that the revised plan is placed in the facility's operating record as required by §257.105(g)(3). An amendment must be made whenever there is a change in conditions that would substantially affect the written plan in effect. CCR regulations do not specify a timeline for completing an amendment once the need for an amendment is identified; however, the amended Plan must be placed in the site operating record "within a reasonable amount of time".

3.2 5-YEAR PLAN REVISION

At a minimum, a periodic Run-on and Run-off Control System Plan must be prepared every five (5) years. The deadline for completion of any subsequent plan is five (5) years from the date when the previous plan was completed. The Plan is considered complete once it is placed in the facility's operating record.

3.3 ROUTINE INSPECTIONS AND MONITORING

Weekly and annual inspections of the CCR landfill are required under §257.84. In addition, monthly storm water inspections are required by MDEQ Baseline Operating Permit No. MSR001656. These inspections include the run-on and run-off control features of the facility. During these routine inspections, qualified personnel must look to identify signs of distress or malfunction of the run-on and run-off control features (i.e., erosion, debris accumulation, liner damage, AMU basin fluid level). If a deficiency is observed, the owner or operator must remedy the deficiency in accordance with §257.97. The Run-on and Run-off control features are inspected weekly and annually as part of the facility's CCR Landfill Weekly Inspection Checklist (see Appendix B) in accordance with §257.84. In addition, the facility conducts monthly storm water inspections in accordance with MDEQ Baseline Operating Permit No. MSR001656. In the event of a discharge from the AMU Basin, sampling requirements specified in NPDES Permit No. MS0054496 will be implemented.

4.0 RECORDKEEPING AND NOTIFICATIONS

4.1 RECORDKEEPING

The Run-On and Run-Off Control System Plan must comply with the recordkeeping, notification, and website requirements described in 40 CFR 257, Subpart D. Copies of records are kept on site. Unless specified otherwise, records must be retained for at least five (5) years from the date of occurrence, measurement, maintenance, corrective action, report, record, or study as required by §257.105(b).

In accordance with the requirements of §257.105(g)(3), the Run-On and Run-Off Control System Plan (§257.81(c)), CCR inspections and storm water inspections must be placed in the site operating record as they become available.

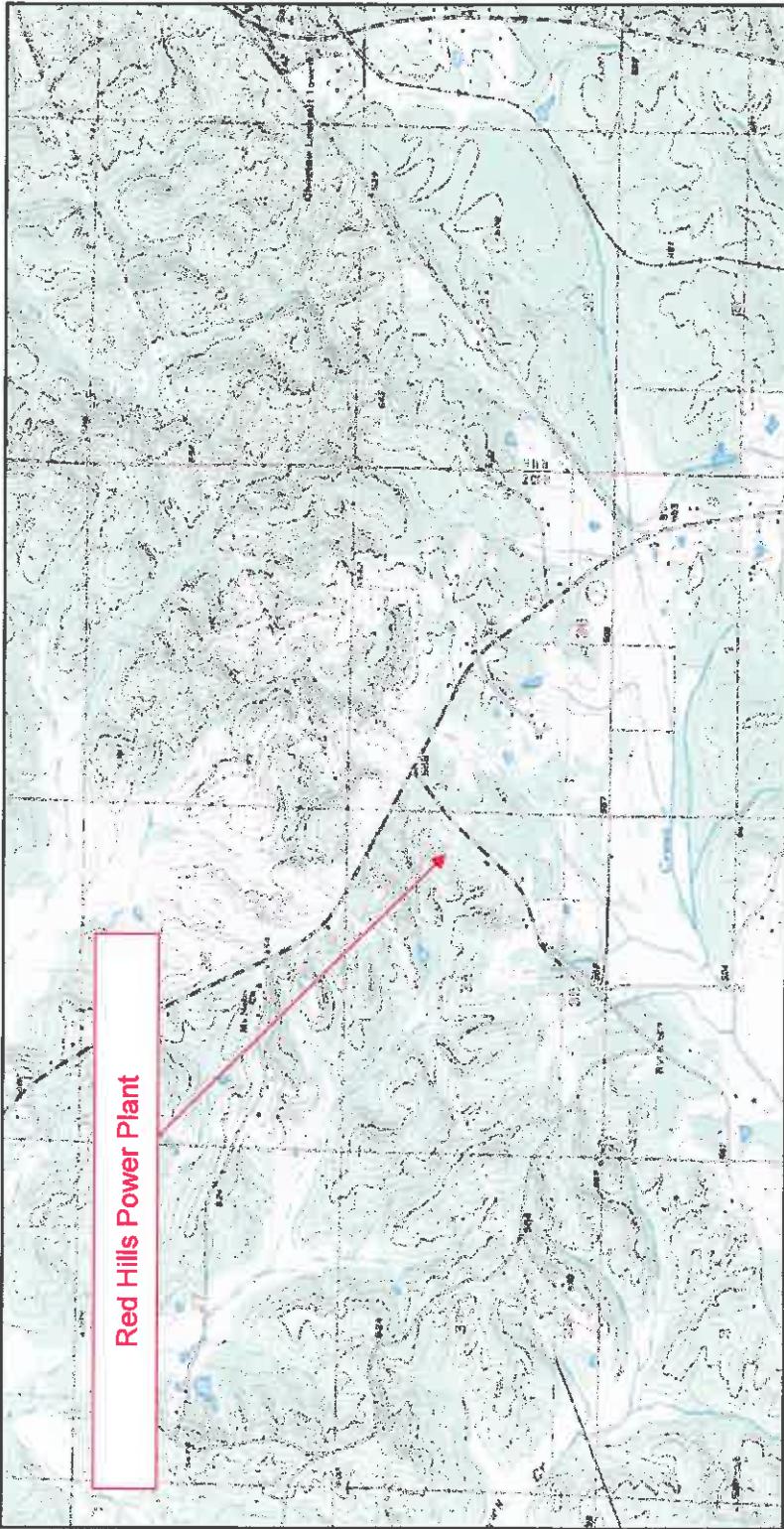
4.2 NOTIFICATIONS

The facility must comply with the notification requirements specified in §257.106(g)(3), which states the owner or operator of a CCR Unit must notify the State Director and/or appropriate Tribal authority of the availability of the initial and subsequent periodic Run-On and Run-Off Control System Plan.

With regards to the CCR Unit on site, Red Hills is required to *notify the MDEQ* of availability of the aforementioned initial and subsequent periodic Run-On and Run-Off Control System Plan when this information has been placed in the site's operating record and on their publicly accessible internet site.

FIGURES

FIGURE 1
SITE LOCATION MAP



Legend:	Drawn By: JTB	Checked By: BSK
Date: 10/14/2016	Scale: 1:24,000	
Project No.:		Drawing No: N/A
Source: Digital-Topo-maps.com		P.O. Box 356 Sherman, Mississippi 38869 (662) 840-5945
Figure 1: Site Location Map		

FIGURE 2
AERIAL SITE MAP



Legend:	Drawn By: JTB	Checked By: BSK
Project No.:	Date: 10/14/2016	Scale: 1:24,000
		Drawing No: N/A
Source:	Red Hills Power Plant 2391 Pensacola Road Ackerman, Mississippi	
Google Earth (2016)		

ECS
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Figure 2: Aerial Site Map

FIGURE 3
RUN-ON AND RUN-OFF FLOW DIAGRAM



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Red Hills Power Plant
2391 Pensacola Road
Ackerman, MS 39735

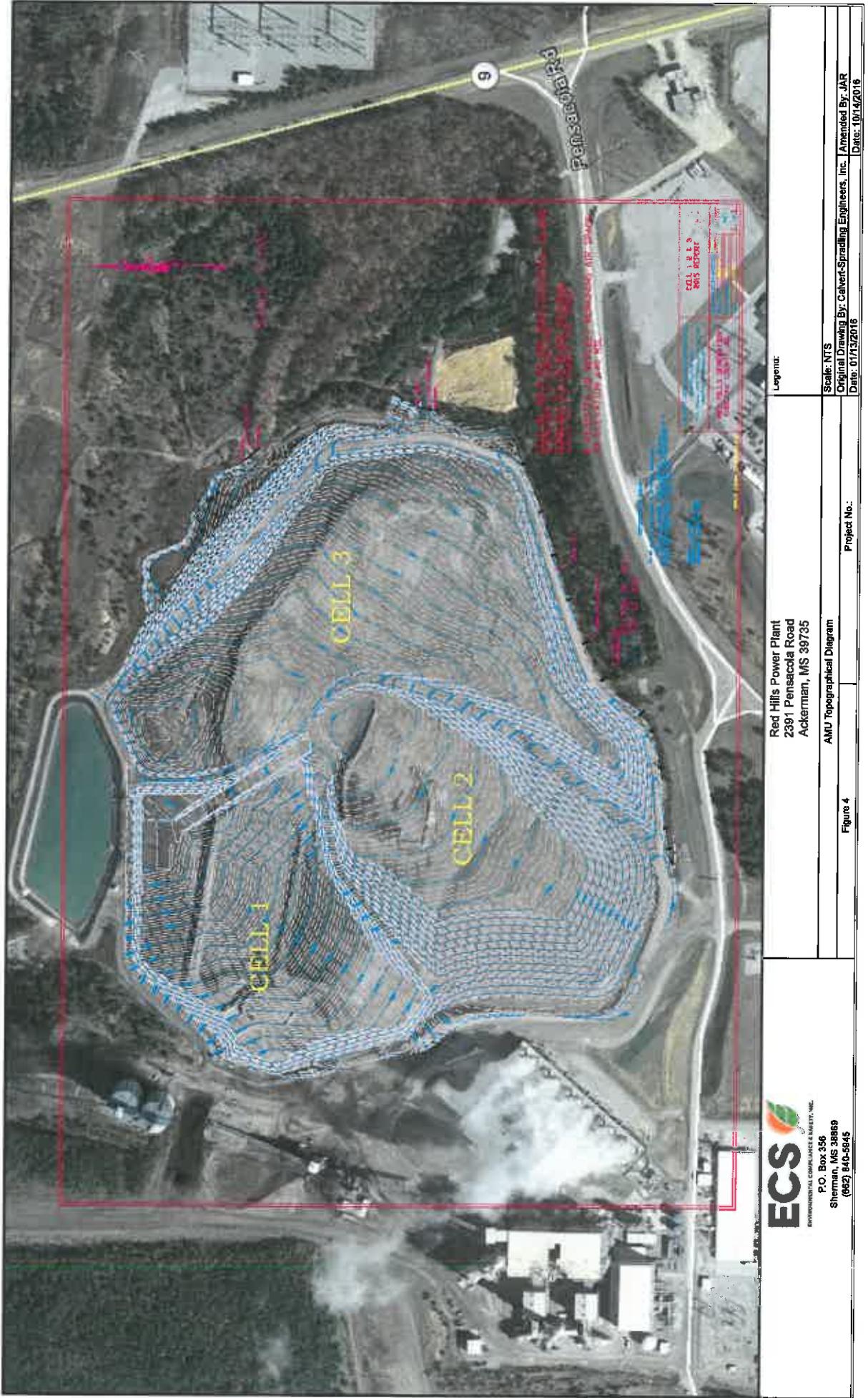
Run-On and Run-Off Flow Diagram

Figure 3

Project No.:

Legend:	
Drainage Ditch	AMU Berm
Surface Water Flow	AMU Drainage System
Storm Drain	NTS
	Drawn By: JAR
	Date: 10/14/2016

FIGURE 4
AMU TOPOGRAPHICAL DIAGRAM



APPENDICES

APPENDIX A

DESIGN CALCULATIONS

Section 1
Design of Perimeter Diversion Dike

APPENDIX A

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check 8/4/97
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Assumptions

Choctaw Generation Facility - Ash Disposal Site - 10year Grading Plan - Design of Dikes
Subsidiary of Red Hills Generation Facility
Job No. 0874097110
Date Calc 5-Aug-97

(R)

PROBLEM STATEMENT:

Design temporary and permanent diversion swales and dikes for the final 10-year grading plan to accommodate the 25-year, 24-hour storm event.

ASSUMPTIONS:

Assumptions include:

1. Following "Planning & Design Manual for the Control of Erosion, Sediment & Stormwater" by Mississippi Department of Environmental Quality, Mississippi SCS & USSCS, for:
 - a. Sediment Basin (temporary practices)
 - b. Diversion (permanent practices)
 - c. Diversion (temporary practices)
2. The diversion dike was designed for the worst-case condition which is when cell I is closed and Cell II is being constructed (initial condition). The following dikes will be constructed according to this worst-case design.
3. Diversions will be designed to be stable under the as-built, bare soil condition, according to the velocities set forth in the P & D Manual: 4 fps for clay, 3 fps for silt, and 2.5 fps for sandy soils.
4. All sediment and erosion control features are designed for the 25-year, 24-hour storm event.
5. CN based on Soil Group D (assuming silty clay), for runoff, agricultural lands, woods, good condition was assumed for a CN = 77. For final condition runoff, grass fair condition was assumed, for a CN of 79. See Table 6-5B of Mississippi Erosion and Sediment Control Manual.
6. Manning's n for sheet flow is based on woods surface description, with dense underbrush (0.8), for runoff, and for runoff (closed cells), n = 0.15 short grass.
7. Design of all swales is based on the proposed sequential filling plans (attached).
8. Drainage areas were determined using the Planix planimeter.
9. Stability calculations based on "Open-Channel Hydraulics" by Ven Te Chow, Section 7-20. The process for designing for stability is the following:
 1. Assume value of n and determine VR from Figure 7-14.
 2. Select permissible velocity from SCS or Table 7-6.
 3. Compute VR using Manning's Formula:
$$VR = (1.49 * R^{3/4} * S^{1/2})/n$$
 4. Check the computed VR against the VR in the n-VR curve. Continue until equal or within approximately 3% error.
 5. Compute the water area by $A = Q/V$.
 6. Determine various sections, using the R and A just calculated, comparing configurations of various channels with the R & A. The R & A will give you the bottom width and expected depth for various side slopes. Pick one that is reasonable.
10. Perform capacity calculations. This method compares velocity and is usually deeper:
 1. Assume a depth y and compute area A and hydraulic radius R (geometric formula).
 2. Compute velocity V by $V = Q/A$, and VR by $VR = V * R$.
 3. From this VR and n-VR curve (higher vegetal retardance), find n.
 4. Using this n and Manning's formula, compute V.
 5. Compare this V with the V in Step 2. Continue until equal or within 3% error. Note that this value should be equal to or less than the assumed velocity in the stability calculations. If not, then go back to stability and change the bottom or side slopes (or both), and continue through the steps again.
11. Add the proper freeboard. In Mississippi, add 0.3 ft.

REFERENCES:

1. "Planning & Design Manual for the Control of Erosion, Sediment & Stormwater" by Mississippi Department of Environmental Quality, Mississippi SCS & USSCS.
2. "Open-Channel Hydraulics" by Ven Te Chow, Section 7-20.
3. "Urban Hydrology for Small Watersheds" by the USDA, SCS, Technical Release 55 (TR-55).

APPENDIX A

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Summary

Summary of Design Criteria for Diversion Dikes

Swale Name	calculated			Flow (cfs)	Actual Velocity (fps) ²	Recommended Velocity (fps) ³
	Base Slope	Side slope	Depth (ft) ¹			
D-1	0.003	4.0	2.9	162.54	3.0	4.0
D-2	0.003	4.0	4.1	403.77	4.3	8.0
D-3	0.010	4.0	1.6	15.32	1.6	4.0 (reinforced)

¹Includes 0.3 foot freeboard.

²Actual calculated velocity based on the actual flow and configuration determined in Stability.

³Maximum velocity recommended by the Mississippi Stormwater Design Manual, or by stabilizing material, such as Enkamat. Based on stability analysis. If above 4 fps, then reinforcement is necessary to obtain capacity velocities.

APPENDIX A

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Tc

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10-Year Grading Plan - Design of Dikes
 Red Hills Generation Facility - Ash Disposal Site
 Job No. 0874097110
 Calculations by: Gini Perkins
 Date Calcs: 5-Aug-97
 Date Checked: _____
 Checked by: _____

TIME OF CONCENTRATION CALCULATIONS

Sheet Flow: (t_{sl})

1. Surface description (Table 6-8)
2. Manning's roughness coef., n (Table 6-1)
3. Flow length, L ($L \leq 300'$)
4. 2-yr, 24-hr rainfall, P2 (table 6-3 of Miss. ESS)
5. Land slope, S (ft/ft) (Grading Plan)
6. $t_{sl} = \text{Eqn 3.1 in notes}$ (Hours)
7. Total T_{sl} :

Segment ID:

D-1	D-2	D-3
Woods	Woods	Woods
0.8	0.8	0.8
250	300	250
4.2	4.2	4.2
0.12	0.2	0.04
0.5528724	0.5214713	0.8579728
0.55	0.52	0.86

Shallow Concentrated Flow, t_{sc} (overland too)

1. Surface description, paved/unpaved
3. Flow length, L ($L > 300'$)
4. Average Velocity (figure 3-1)
5. Land slope, S (ft/ft) (Grading Plan)
6. $T_{sc} = \text{Eqn 3.2 in notes}$
7. Total T_{sc} :

Segment ID:

D-1	D-2	D-3
Woods	Woods	Woods
0	0	0
5.5	5.5	1
0.12	0.114	0
0	0	0
0.00	0.00	0.00

Segment ID:

D-1	D-2	D-3
N/A	N/A	N/A
1650	2130	1450
3.1	2.7	4
0.0364	0.028	0.062
0.14785	0.21914	0.10069
0.15	0.22	0.10

Total time, Tc = Sum all t_{sl} , t_{sc} , t_{oc} (hours)

0.70	0.74	0.96
-------------	-------------	-------------

KEY:

- RUN-ON
 D-1 Cell I Inner Diversion Dikes - Temporary - upper portion
 D-2 Cell I Inner Diversion Dike - Temporary - lower portion
 D-3 Cell I Perimeter Diversion Dike - Permanent

APPENDIX A

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8/5/97



10-Year Grading Plan - Swale Watershed flow's
 Red Hills Generation Facility - Ash Disposal Site
 Job No. 2899001300
 Calculations Gini Perkins
 Date Calcs: 5-Aug-97
 Date Checked: _____
 Checked by: _____

GRAPHICAL PEAK DISCHARGE CALCULATIONS

Segment ID

1. Data:

Drainage area, A_m

Runoff Curve No., CN

T_c

Rainfall Distribution type:

Pond/Swamp area distribution:

D-1	D-2	D-3
0.08209	0.129	0.009407
77	77	77
0.70	0.74	0.96
II	II	II
0%	0%	0%

mi²

(Table 6-6B)

(T_c worksheet)

(Figure 6-4)

(From Sequential Filling Plan)

2. Rainfall Frequency Event (yr):

3. 24-hour rainfall, P

4. Initial Abstraction, I_a :

5. I_a/P

6. Unit Peak Discharge, q_u

7. Runoff, Pe

8. Pond/Swamp Adjusmt Factor:

Peak Discharge, $Q_p = q_u A_m P e F$

25	25	25
7	7	7
0.597	0.597	0.597
0.09	0.09	0.09
450	425	370
4.4	4.4	4.4
1	1	1
162.54	241.23	15.32

Year (Table 6-3)

Inches (Table 6-3)

Inches (Use CN on Table 6-7)

csm/in (Use T_c , I_a/P , & rainfall distrib. with Figure 6-3)

inches (Figure 6-5)

Table 6-2

cfs

Total Flow:

403.77

APPENDIX A

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(B)

Stability

Choctow Generation Facility - Ash Disposal Site - 10 Year Grading Plan
 Subsidiary of Red Hills Generation Facility
 Job No. 0874097110
 Date Calc 5-Aug-97

V_p = permissible velocity based on Miss. SCS 4 fps, low resistance
 S = Slope of swale = 0.003 ft/ft (Assumed value)

Swale Number: D-1

Trial No.	Mannings n	VR _{Table}	R (VR _{Table} /V _p)	VR _{Computed}	Delta VR
1	0.03	2.25	0.5625	1.04	-116%
2	0.04	0.70	0.175	0.11	-527%
3	0.032	1.70	0.43	0.61	-177%
4	0.0255	5.67 ✓	1.42	5.72 ✓	1% ✓

H₂O Area 40.63 ✓ sf

Wetted Perimeter (P) = 28.67 ft

Using the charts in Appendix B of Chow's "Open Channel Hydraulics", the best configuration for this swale with this flow is 4:1 trapezoidal. This yields in a swale 10 feet wide, 2.15 feet depth and side slopes of 4:1. This configuration is used for capacity calculations (velocity). ✓

Bottom Width: 10 ft ✓
 Side Slopes: 4 :1

V_p = permissible velocity using Enkamat = 8 fps, as reinforcement
 S = Slope of swale = 0.003 ft/ft (Assumed value)

Swale Number: D-2

Trial No.	Mannings n	VR _{Table}	R (VR _{Table} /V _p)	VR _{Computed}	Delta VR
1	0.03	2.25	0.28125	0.33	-585%
2	0.04	0.70	0.0875	0.04	-1889%
3	0.032	1.70	0.21	0.19	-781%
4	0.02	20.00	2.50 ✓	18.79	-6%

H₂O Area 50.47 sf ✓

Wetted Perimeter (P) = 20.19 ft

Using the charts in Appendix B of Chow's "Open Channel Hydraulics", the best configuration for this swale with this flow is 1:1 triangular. However, since we will be constructing the swale with trapezoidal shape, 10 feet wide, and 4:1 side slopes, the slope will be stable enough using Enka-reinforcement. This configuration is used for capacity calculations (velocity).

Bottom Width: 10 ft ✓
 Side Slopes: 4 :1

Stability



V_p = permissible velocity based on Miss. SCS 4 fps, low resistance
 S = Slope of swale = 0.01 ft/ft (Assumed value)

Swale Number: D-3

Trial No.	Mannings n	VR _{Table}	R (VR _{Table} /V _p)	VR _{Computed}	Delta VR
1	0.03	2.25	0.5625	1.90	-18%
2	0.04	0.70	0.175	0.20	-243%
3	0.032	1.70	0.43	1.12	-52%
4	0.028	2.75	0.69	2.85	4%

H₂O Area 3.83 sf

Wetted Perimeter (P) = 5.57 ft

Using the charts in Appendix B of Chow's "Open Channel Hydraulics", the best configuration for this swale with this flow is 1.0:1 triangular. However, since we will be constructing the swale with trapezoidal shape, 2 feet wide, and 4:1 side slopes, the slope will be stable enough using Enka-reinforcement. This configuration is used for capacity calculations (velocity). ✓

Bottom Width: 2 ft ✓
 Side Slopes: 4 : 1

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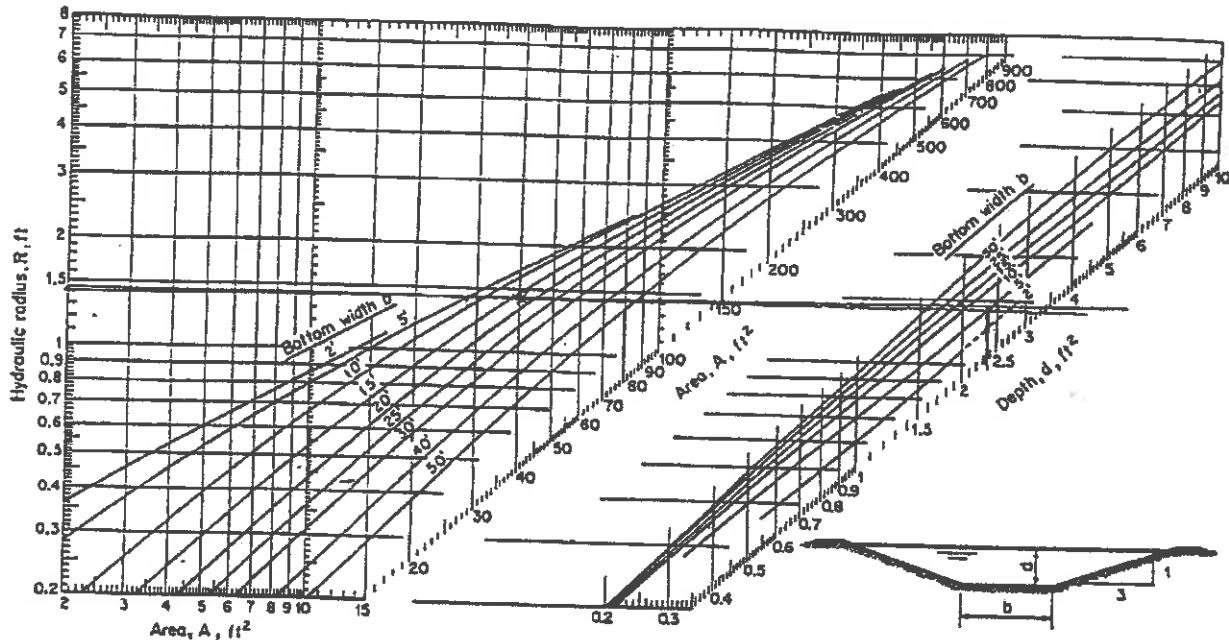


Fig. B-5. Hydraulic elements of trapezoidal channels with 3:1 side slopes.

SWALE 0-1

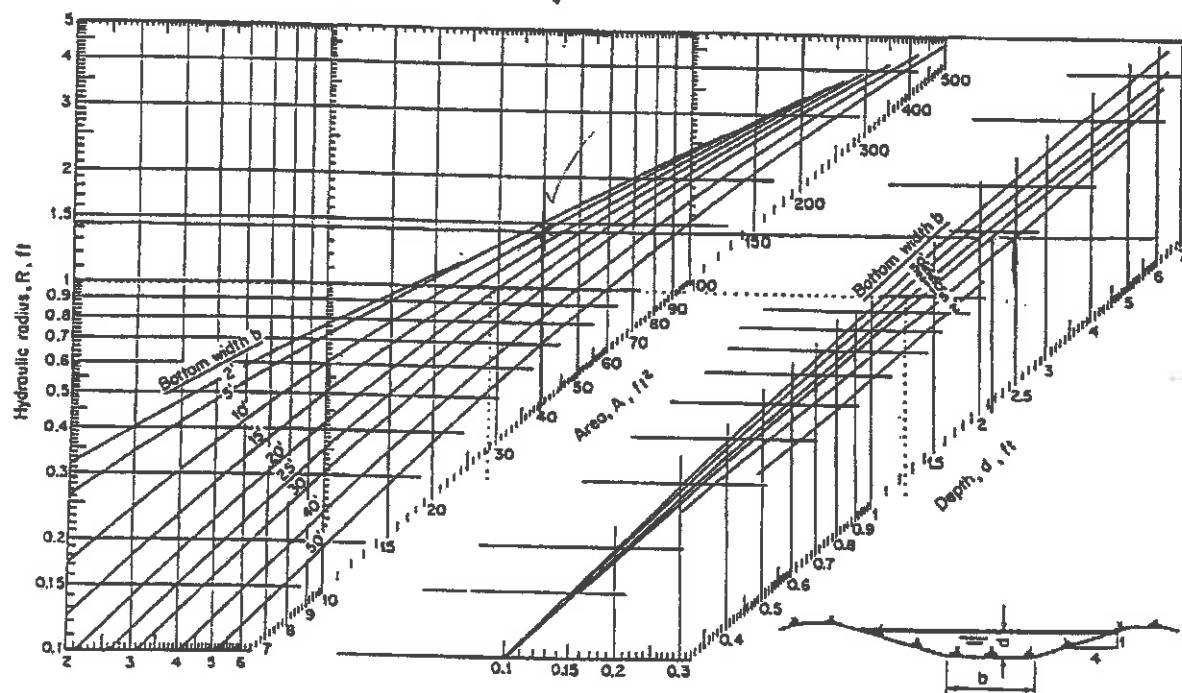


Fig. B-6. Hydraulic elements of trapezoidal channels with 4:1 side slopes.

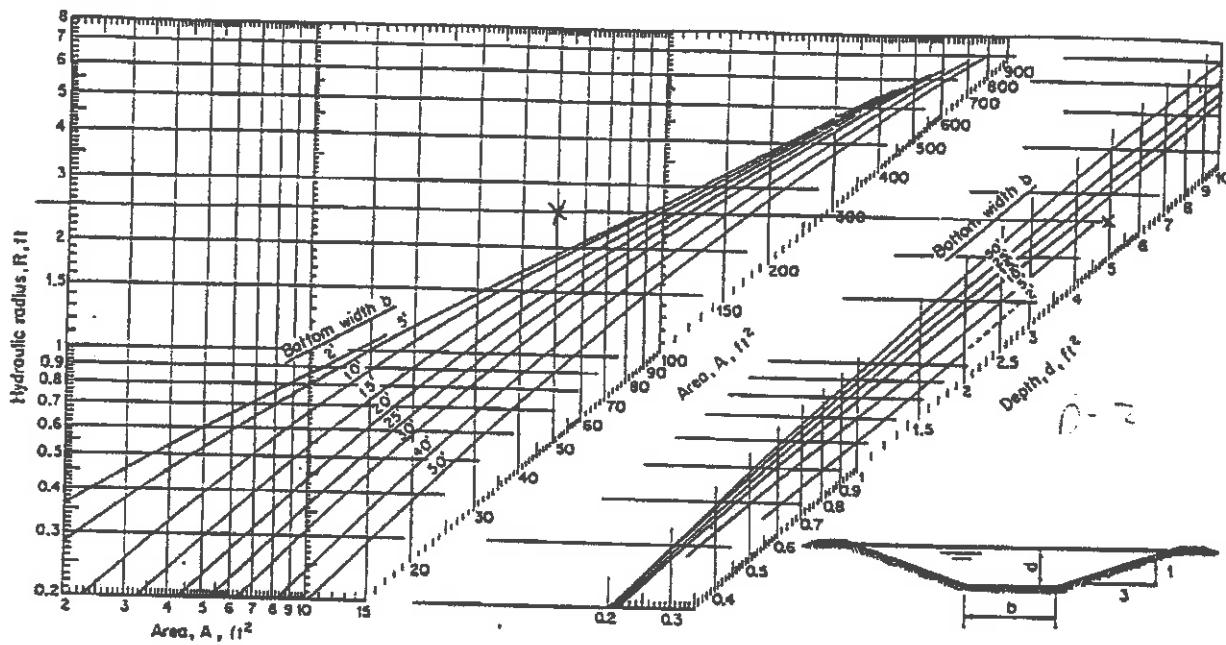


Fig. B-5. Hydraulic elements of trapezoidal channels with 3:1 side slopes.

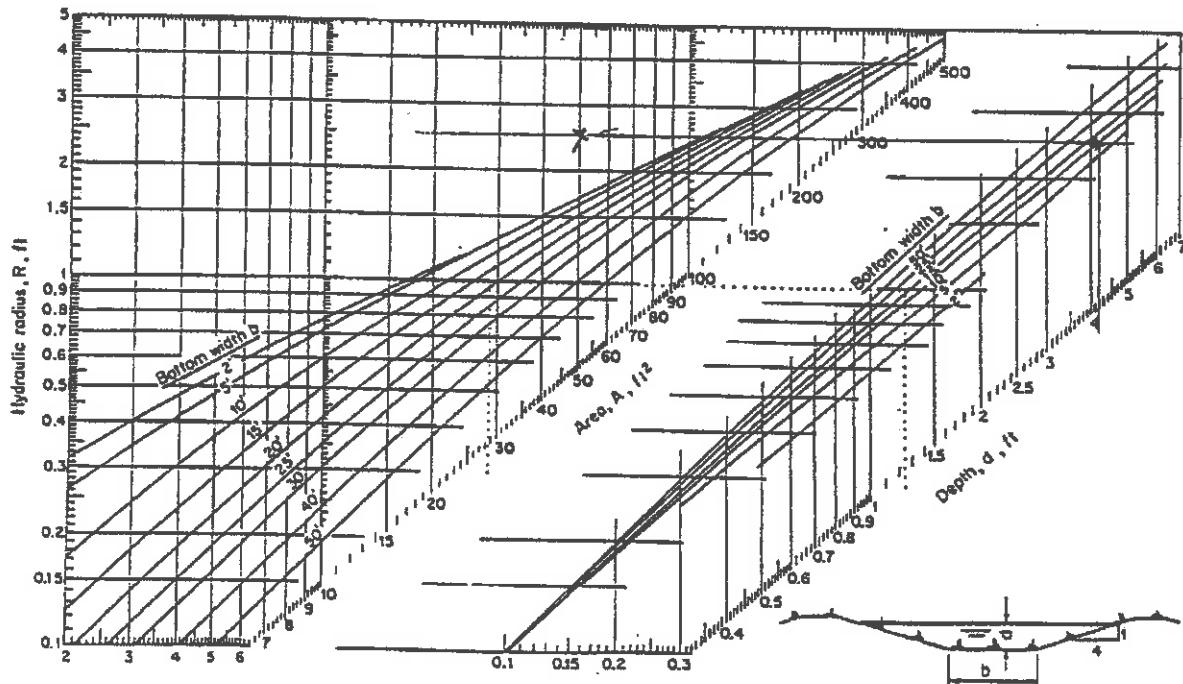


Fig. B-5. Hydraulic elements of trapezoidal channels with 4:1 side slopes.

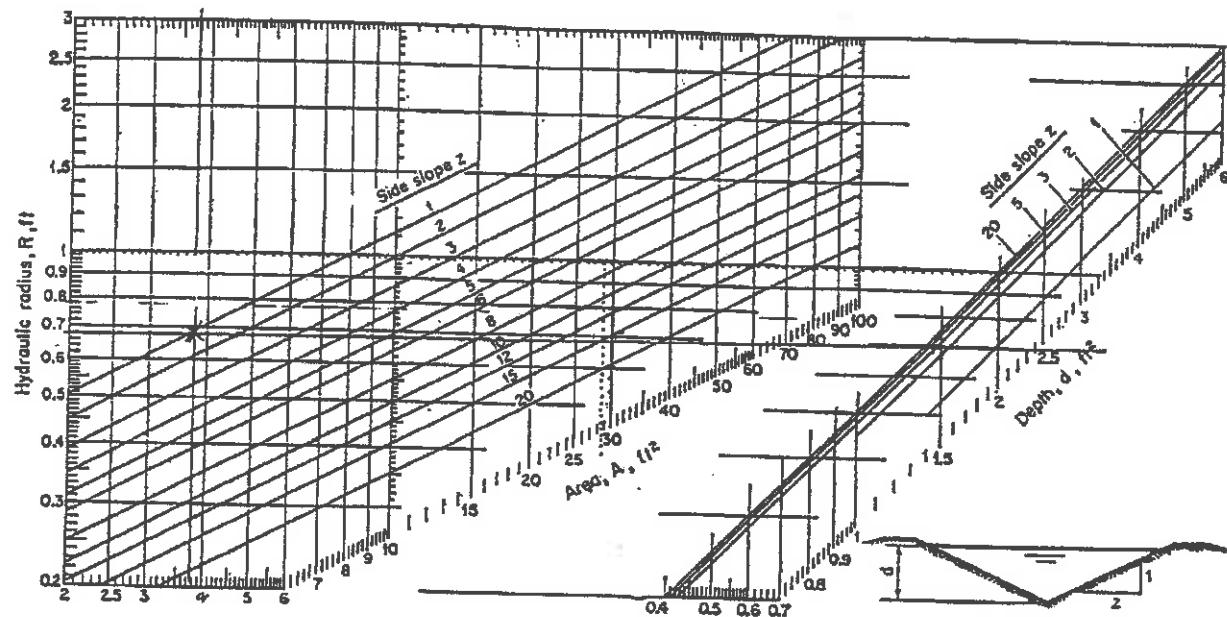


FIG. B-9. Hydraulic elements of triangular channels.

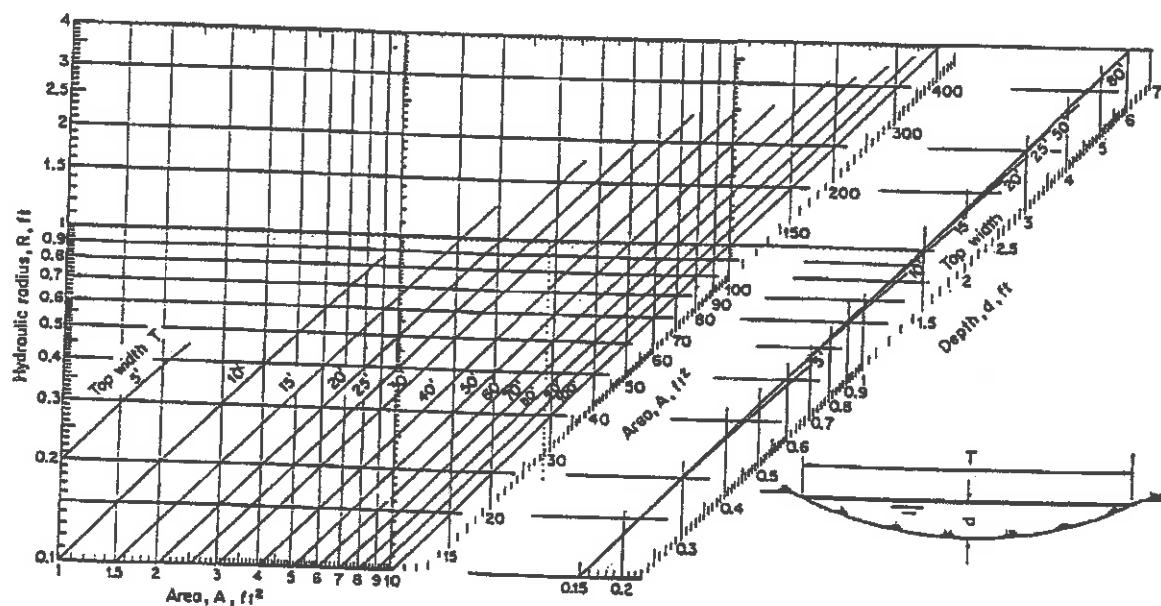


FIG. B-10. Hydraulic elements of parabolic channels.

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APPENDIX A

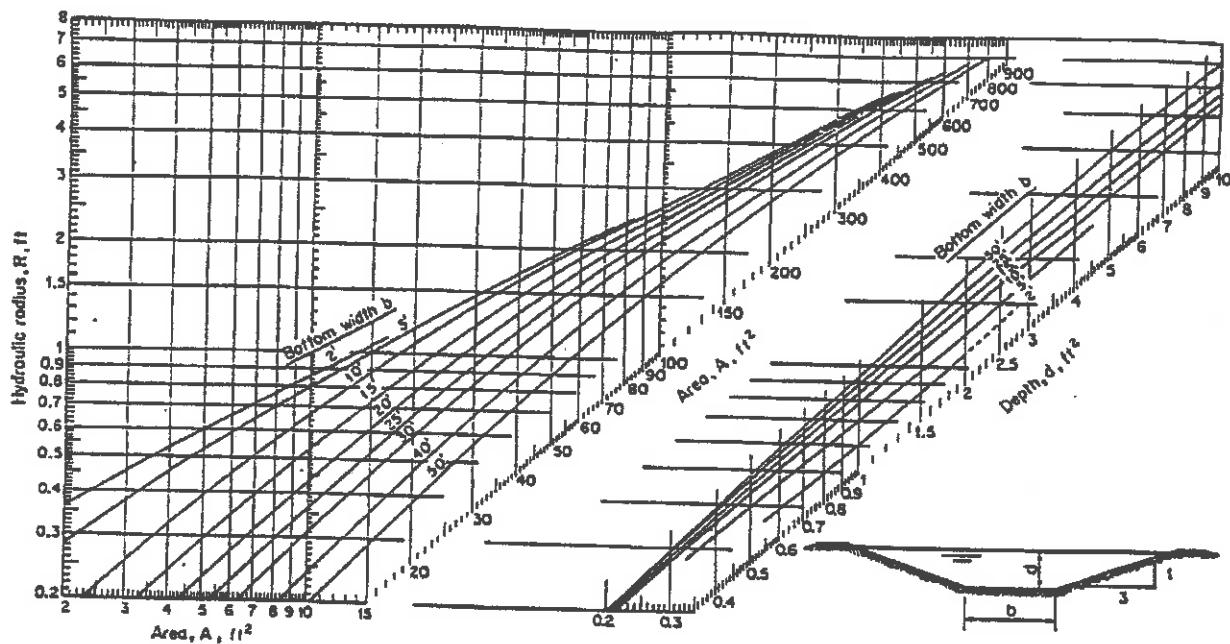


Fig. B-5. Hydraulic elements of trapezoidal channels with 3:1 side slopes.

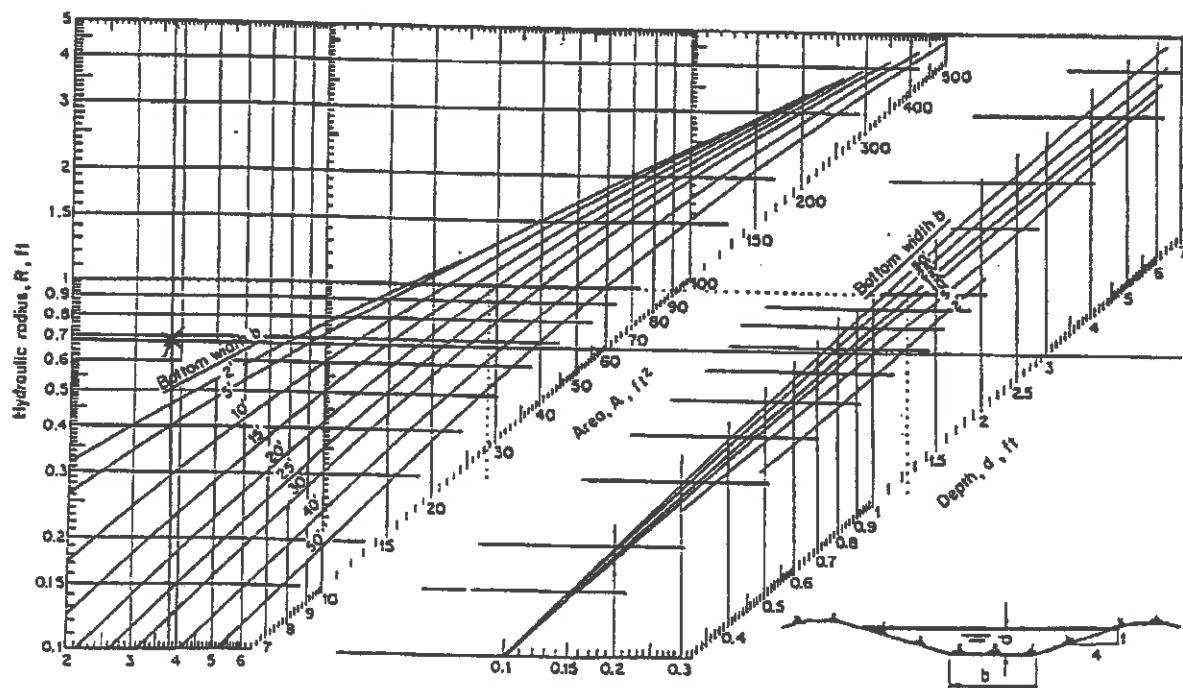


Fig. B-6. Hydraulic elements of trapezoidal channels with 4:1 side slopes.

D-3

B

Capacity

Choctow Generation Facility - Ash Disposal Site - 10 Year Grading Plan
 Subsidiary of Red Hills Generation Facility
 Job No. 0874097110
 Date Calc 5-Aug-97

Width: 10 ft

Swale Number: D-1

Trial No.	y (ft)	Area (sf)	R	Mannings				Delta V
				V _{calc}	VR	n	V _{manning}	
1	2.6	53.04	1.687015	3.06 ✓	5.17	0.038	3.04 ✓	-1%

This results in a Swale with
 of 3.0 fps, a flow of 4 :1 side slopes & depth 2.9 ft at a velocity
 162.54 cfs.

Width: 10 ft

Swale Number: D-2

Trial No.	y (ft)	Area (sf)	R	Mannings				Delta V
				V _{calc}	VR	n	V _{manning}	
1	3.8	95.76	2.316647	4.22 ✓	9.77	0.033	4.33 ✓	3%

This results in a Swale with
 of 4.33 fps, a flow of 4 :1 side slopes & depth 4.1 ✓ ft at a velocity
 403.77 cfs.

Width: 2 ft

Swale Number: D-3

Trial No.	y (ft)	Area (sf)	R	Mannings				Delta V
				V _{calc}	VR	n	V _{manning}	
1	1.3	9.36	0.735845	1.64	1.20	0.0767	1.58	-3%

This results in a Swale with
 of 1.58 fps, a flow of 4 :1 side slopes & depth 1.6 ✓ ft at a velocity
 15.32 cfs.

Area

Summary of Drainage Areas (from Planimeter)
Red Hills Generation Facility - Ash Disposal Site
Job No. 0874097110
Date Calcs: 5-Aug-97

Drainage Area	Average Area	
location	A_m (mi ²)	A_m (acres)
D-1	0.08209	52.5376
D-2	0.129	82.56
D-3	0.009407	6.020672

APPENDIX A

Very Low Vegetal Retardance

Curve for Very Low Vegetal Retardance for Grass-lined Channels (E) for Stability

VR	Manning's n
15	0.0235
20	0.023
10	0.024
8	0.025
5.67	0.0255
4.33	0.026
3	0.027
2.75	0.028
2.5	0.029
2.25	0.03
2.15	0.031
1.7	0.032
1.3	0.034
1.1	0.035
1	0.036
0.9	0.037
0.84	0.039
0.7	0.04
0.58	0.0425
0.55	0.045

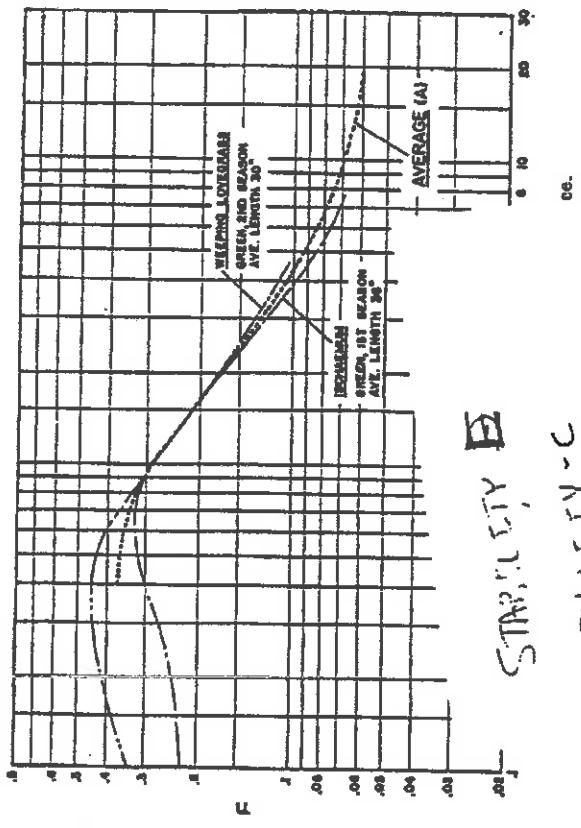


Fig. 7-14. Experimental n - VR curves.
(U.S. Soil Conservation Service.)

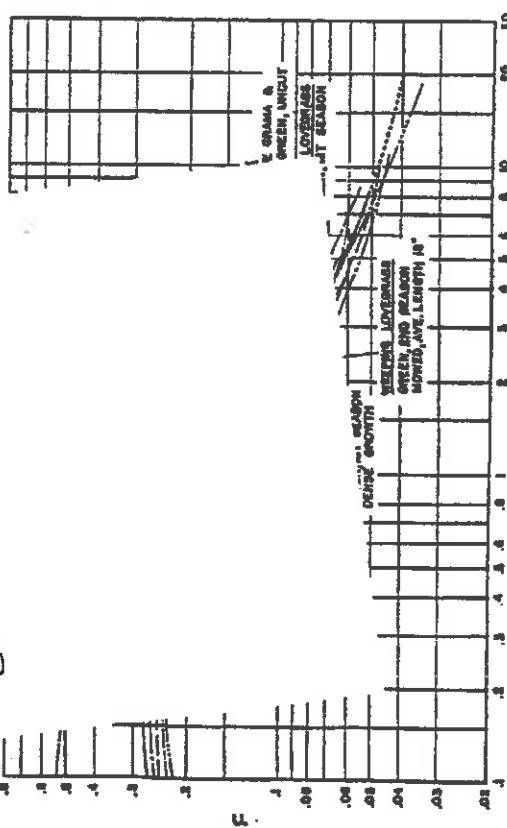


Fig. 7-14. Experimental n - VR curves.
(U.S. Soil Conservation Service.)

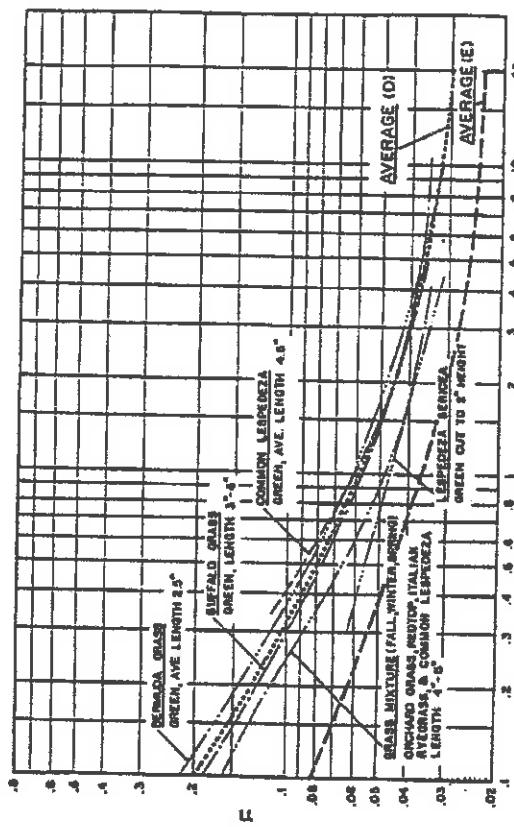


Fig. 7-14 (Continued).

(d) Curves for D or low vegetal retardance, and an average curve for E or very low vegetal retardance.



**Section 2
Hydrograph and Peak Flows
for Final Grade**

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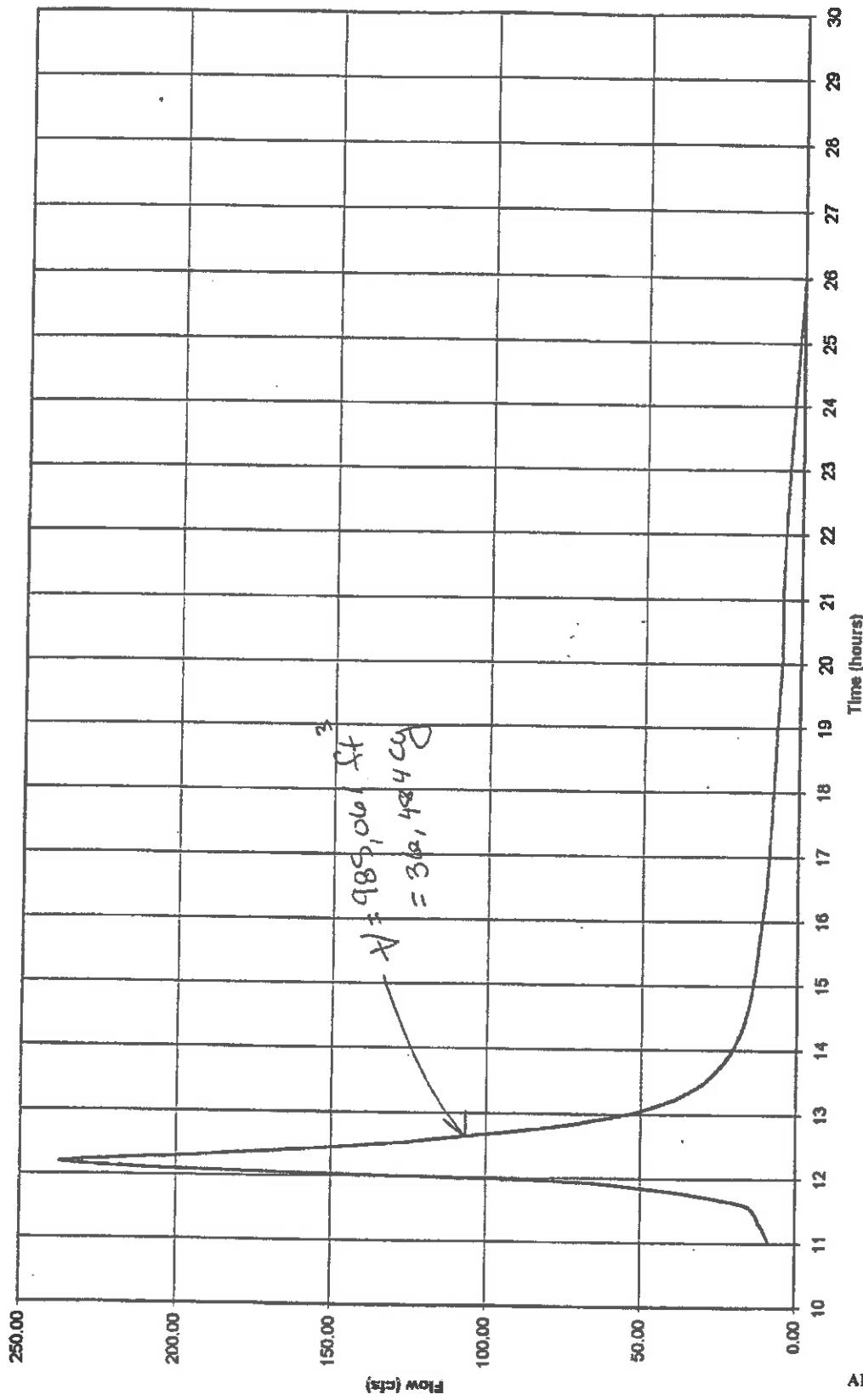
checked 7/3/97
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Curves Chart 6

2.418 0048
 2.418 6048
 2.418 0048

7.418 0048

25-year Storm Event for Sedimentation Basin 10-Year Plan



APPENDIX A

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$\frac{1}{2} = 44,314$

areas

Job Title: Red Hills Generation Facility - Ash Disposal Site - 10-Year Grading Plan

Job No. 0874097110

Calculations by: Gini Perkins

Date Calcs: 0874097110

Date Checked: 1/21/97

Checked by: [Signature]

Drainage Area	Area Miles Sqr	Area inches sqr (planimeter)	ACRES
a	0.021454	14.95236	13.73036
b	0.0186	12.96319	11.90376
c	0.007584	5.285511	4.853545
d	0.005426	3.782008	3.472918
e	0.00281	1.958171	1.798136
f	0.004633	3.229173	2.965265
g	0.003514	2.449005	2.248857
h	0.002016	1.405336	1.290483
i	0.00444	3.094839	2.84191
j	0.003254	2.268171	2.082802
k	0.002283	1.591336	1.461282
l	0.002231	1.55517	1.428071
M	0.003529	2.459338	2.258345
N	0.004589	3.198173	2.936798
O	0.006894	4.80501	4.412314
P	0.015153	10.56069	9.697601
Q	0.01284	8.948685	8.217341
R	0.004973	3.465855	3.182603
S	0.002663	1.855737	1.704074
T	0.003521	2.454172	2.253601
			84.74006

Planned Area in Inches squared:

Trial 1	Trial 2	Trial 3	Average
14.98853	14.92653	14.94203	14.95236
12.88053	13.08203	12.92703	12.96319
5.301011	5.23901	5.316511	5.285511
3.704507	3.797508	3.844008	3.782008
1.922004	1.953004	1.999504	1.958171
3.224006	3.255007	3.208506	3.229173
2.433505	2.418005	2.495505	2.449005
1.410503	1.395003	1.410503	1.405336
3.084506	3.084506	3.115506	3.094839
2.309505	2.294005	2.201004	2.268171
1.581003	1.612003	1.581003	1.591336
1.550003	1.550003	1.565503	1.55517
2.464505	2.480005	2.433505	2.459338
3.146506	3.208506	3.239506	3.198173
4.78951	4.83601	4.78951	4.80501
10.50902	10.57102	10.60202	10.56069
8.897018	8.959018	8.990018	8.948685
3.422551	3.487507	3.487507	3.465855
2.095204	1.674003	1.798004	1.855737
2.480005	2.433505	2.449005	2.454172

APPENDIX A

9

Red Hill Compensation Facility - Anti Disposal Site - 11- Year Ground Plan
0074897110
Gail Peckin
21-Nov-87
10/25/87
Job Title:
do No.
Calculations by:
Date Calculated:
Date Checked:
Checked by:

THE CHINESE COMMUNISTS

Shallow Concentrated Flows: I - (continued)

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APPENDIX

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Hydrograph

Job Title: Red Hills Generation Facility - Ash Disposal Site - 10-Year Grading Plan
 Job No.: 0874997110
 Calculation Gini Perkins
 Date Calc'd: 21-Jul-97
 Date Checked: 7-21-'97
 Checked by: [Signature]

Methodology for determining T_c and T_r :

First, the T_c , which is the total time it takes one droplet to travel from the most distant point in that watershed (end of the swale itself) to the end of the watershed (end of the stream) is found. Then the T_r , the time it takes to travel through the "stream" (swale), is found. T_c and T_r are found by Chapter 2 method of the TR-55. Note that there is a T_r associated with the very first swale watershed but this value is included in the T_c since the T_c is for that watershed. But if there is a downstream watershed, then the T_r in the second watershed is the T_r for the first watershed.

For example, for the hydrograph for Point 1 (see Sheet 3/8 of hand calculations for figure), the hydrograph includes swales S-8 and P-6. P-6 is downstream of S-8 so the T_c for S-8 is that calculated, and the T_r for S-8 is zero, since that is included in the T_c . The T_r for P-6 is the time just to travel through the swale P-6 itself. Then, the TOTAL T_r for swale S-8 is the T_r for P-6 and the TOTAL T_r for P-6 is zero. Note that as one follows watersheds downstream, the T_r becomes smaller and smaller as the number of watersheds downstream decrease. The last one is always zero because that T_c is at the point where the hydrograph is being drawn.

The actual values for T_r are rounded to the nearest tenth (0.1) since that is what is given in the TR-55 chart. If the T_c is less than 0.1, it is rounded up to 0.1 as this is the smallest value given in the TR-55. All other values for T_c are rounded to the nearest tenth.

HYDROGRAPHS FOR 25-YEAR, 24-HOUR STORM EVENT - PROPOSED LANDFILL

Area Name (Insert)	Area A_m (mi ²) (planimeter)	Time T_c (hrs) Ch. 2	Travel Time thru river, T_r , area names of interest	Total travel time thru river			Curve #	Runoff Pe (in)	$A_m P_e$ (calculate)	I_a (in)	I_a/P
				Total travel time thru river	25-yr Rainfall	CN					
a	0.0214537	0.64	0.00	7	0.00	7	79	4.6	0.098887	0.632	0.1
f	0.0046332	0.12	0.00	7	0.00	7	79	4.6	0.021313	0.532	0.1
g	0.0035138	0.18	0.00	7	0.00	7	79	4.6	0.016164	0.532	0.1
h	0.0020184	0.14	0.00	7	0.00	7	79	4.6	0.009275	0.532	0.1
i	0.0044405	0.24	0.00	7	0	7	79	4.6	0.020426	0.532	0.1
j	0.0032544	0.17	0.00	7	0.00	7	79	4.6	0.01497	0.532	0.1
o	0.00868942	0.39	0.00	S, T	0.15	7	79	4.6	0.03714	0.532	0.1
s	0.0026628	0.09	0.06	T	0.09	7	79	4.6	0.012248	0.532	0.1
t	0.0035213	0.14	0.09	---	0.00	7	79	4.6	0.016198	0.532	0.1

Hydrography

Hydrograph for Point 1:

$$^1q = qt^*(A_{\Omega}Pe)$$

Hydrograph for Point 2:

Hydrograph	Area Name (Insert)	A_m (mi^2) T_c (hrs) thru river, T_r area names	Time Travel Time Ch. 2	Down River (hrs)	Total travel time thru river to pt of interest	Rainfall P (in)	Curve # Fig. B-3 > 8	CN	Runoff P_e (in)	A_{m_P} Table 2-1 (calculate)	ia (in)	la/P Table 4-1 (calculate)
R	0.0949728	0.28	0.00	—	0.00	7	79	2-2	7	4.6	0.022875	0.532

$$^1q = qt^*(A_{\Omega}Pe)$$

Hydrograph for Point					
	12	12	12.1	12.1	12.2
q (csm/in)	q (cfs) ¹	q (csm/in)	q (cfs) ¹	q (csm/in)	q (csm/in)
235	5.28	447	10.23	878	10.23

Hydrograph (hrs) - See Exhibits 5-1, 5-II, or 5-III						
$A_n Pe$	11	11	11.3	11.6	11.6	1
q (csm/in)	q (cfs)	q (csm/in)	q (cfs) ¹	q (csm/in)	q (cfs) ¹	q (csm/in)
0.022875	20	0.46	28	0.64	41	0.94
Total Flow (cfs):		0.48		0.64		0.94

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APPENDIX A

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Hydrograph

Hydrograph for Point 1:

$q \text{ (cfs)}^1$	12.2	12.3	12.4	12.5	12.6	12.7	12.8	12.9	13	13.2	13.4
$q \text{ (csm/in)}$	11.35	19.4	19.15	29.01	q (cfs) ¹	q (csm/in)	q (cfs) ¹	q (csm/in)	q (cfs) ¹	q (csm/in)	q (cfs) ¹
13.38	217	4.62	147	3.13	123	2.62	104	2.22	86	1.83	76
12.93	481	7.77	250	4.04	166	2.68	128	2.07	102	1.65	86
5.82	217	2.01	147	1.36	123	1.14	104	0.96	86	0.80	76
16.34	481	9.83	250	5.11	166	3.39	128	2.61	102	2.00	86
11.98	481	7.20	250	3.74	166	2.49	128	1.92	102	1.53	86
12.21	523	16.59	557	17.66	473	15.00	357	11.32	263	8.34	196
10.37	701	8.59	378	4.63	224	2.74	157	1.92	122	1.49	98
10.17	217	3.51	147	2.38	123	1.99	104	1.68	86	1.39	76
71.81	50.58	46.40	49.82	51.62	46.35	49.82	51.62	46.35	43.17	30.50	21.86

Hydrograph for Point 2:

$q \text{ (cfs)}^1$	12.2	12.3	12.4	12.5	12.6	12.7	12.8	12.9	13	13.2	13.4
$q \text{ (csm/in)}$	15.46	676	15.46	459	q (csm/in)	q (cfs) ¹	q (csm/in)	q (cfs) ¹	q (csm/in)	q (cfs) ¹	q (csm/in)
15.46	15.46	15.46	10.50	10.50	283	0.47	196	4.48	146	3.34	114
15.46	15.46	15.46	10.50	10.50	6.47	4.48	4.48	3.34	2.61	2.61	2.61

Hydrograph

Hydrograph for Point 1:

$q \text{ (cfs)}^1$	$q \text{ (csm/min)}^1$																						
13.4	13.6	13.6	13.8	13.8	14	14	14.3	14.3	14.6	14.6	15	15	15.5	15.5	16	16	16	16	16.5	16.5	16	16.5	
12.14	9.18	74	7.30	6.02	49	4.84	41	4.05	35	3.45	31	3.06	27	2.66	24								
1.09	0.98	42	0.90	0.81	38	0.72	32	0.69	29	0.62	28	0.53	23	0.49	21								
0.87	0.79	44	0.71	0.65	35	0.57	33	0.53	30	0.48	27	0.44	24	0.39	21								
0.47	0.43	46	0.42	0.39	38	0.35	34	0.32	32	0.30	29	0.27	26	0.24	23	0.21							
1.10	1.00	49	0.80	0.82	40	0.71	35	0.71	33	0.67	30	0.61	27	0.55	24	0.49	21						
0.81	0.73	49	0.66	0.60	40	0.60	35	0.52	33	0.49	30	0.45	27	0.40	24	0.36	21						
2.12	57	51	1.81	1.62	46	1.46	39	1.24	35	1.11	32	1.01	29	0.92	25	0.79	22						
0.69	0.61	50	0.61	0.55	41	0.50	36	0.44	33	0.40	30	0.37	27	0.33	24	0.29	21						
0.63	0.63	46	0.75	0.68	38	0.62	34	0.55	32	0.52	29	0.47	26	0.42	23	0.37	21						
18.48	13.11		10.85	9.24			7.88	8.73			5.89		6.25										

Hydrograph for Point 2:

$q \text{ (cfs)}^1$	$q \text{ (csm/min)}^1$																						
13.4	13.6	13.6	13.8	13.8	14	14	14.3	14.3	14.6	14.6	15	15	15.5	15.5	16	16	16	16	16.5	16.5	16	16.5	
1.30	1.30	51	1.17	1.17	46	1.05	42	0.96	37	0.85	33	0.76	31	0.71	28	0.64	24	0.55	22	0.55			

Hydrograph for Point 1:

Hydrograph for Point 1:

Hydrograph for Point 2:

t (hrs)	q (cfs)
0	0.50
1	0.46
2	0.46
3	0.43
4	0.43
5	0.41
6	0.41
7	0.41
8	0.41
9	0.37
10	0.37
11	0.30
12	0.27
13	0.27
14	0.30
15	0.37
16	0.41
17	0.43
18	0.43
19	0.41
20	0.41
21	0.46
22	0.46
23	0.50

Hinduism for Dummies

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Hydrograph for Point 1:

Hydrograph for Point 2:

Hydrograph

Hydrograph for Point 3:

Hydrograph Name (Inset)	Area A _m (mi ²) (planimeter)	Time T _e (hrs) Ch. 2	Travel Time Down river thru river, T, area names river to pt of interest	Total Travel		Rainfall P (in)	Curve # Fig. B-3->B Table 2-2 CN	Runoff Pe (in)	A _m Pe (in)	Ia (in)	Ia/P
				Time	Down river						
b	0.01859986	0.42	0	0.00	0.00	7	79	4.6	0.085558	0.532	0.1
c	0.00756337	0.15	0.00	0.00	0.00	7	79	4.6	0.034885	0.532	0.1
d	0.0054284	0.22	0.00	0.00	0.00	7	79	4.6	0.024962	0.532	0.1
e	0.0028096	0.18	0.00	0.00	0	7	79	4.6	0.012924	0.532	0.1
K	0.0022833	0.19	0.00	0.00	0	7	79	4.6	0.010503	0.532	0.1
L	0.0022334	0.15	0.00	0.00	0	7	79	4.6	0.010284	0.532	0.1

Hydrograph for Point M: See Exhibits 5-I, 5-II, or 5-III

Hydrograph Name (Inset)	Area A _m (mi ²) (planimeter)	Time T _e (hrs) Ch. 2	Travel Time Down river thru river, T, area names river to pt of interest	Total Travel		Rainfall P (in)	Curve # Fig. B-3->B Table 2-2 CN	Runoff Pe (in)	A _m Pe (in)	Ia (in)	Ia/P
				Time	Down river						
b	0.4	0.00	0.1	0.085558	0.18	1.54	25	2.14	36	3.08	77
c	0.2	0.00	0.1	0.0348849	0.23	0.80	31	1.08	47	1.84	209
d	0.2	0.00	0.1	0.0249616	0.23	0.57	31	0.77	47	1.17	209
e	0.2	0.00	0.1	0.0129241	0.23	0.30	31	0.40	47	0.61	209
K	0.2	0.00	0.1	0.010503	0.23	0.24	31	0.33	47	0.49	209
L	0.1	0.00	0.1	0.0102643	0.24	0.25	34	0.35	53	0.54	334
			Total Flow (cfs):		2.92	3.99		5.89	19.10	6.64	36.18
										12.1	67.41

Hydrograph for Point M:

Hydrograph Name (Inset)	Area A _m (mi ²) (planimeter)	Time T _e (hrs) Ch. 2	Travel Time Down river thru river, T, area names river to pt of interest	Total Travel		Rainfall P (in)	Curve # Fig. B-3->B Table 2-2 CN	Runoff Pe (in)	A _m Pe (in)	Ia (in)	Ia/P
				Time	Down river						
M	0.0033287	0.2	0	—	0	7	79	4.6	0.016232	0.532	0.1

Hydrograph Name (Inset)	Area A _m (mi ²) (planimeter)	Time T _e (hrs) Ch. 2	Travel Time Down river thru river, T, area names river to pt of interest	Total Travel		Rainfall P (in)	Curve # Fig. B-3->B Table 2-2 CN	Runoff Pe (in)	A _m Pe (in)	Ia (in)	Ia/P
				Time	Down river						
M	0.2	0.0	0.1	0.0162319	0.37	0.50	47	0.76	209	3.39	403
			Total Flow (cfs):		0.50	0.50		0.76	209	6.54	736
										12.1	12.00
										12.1	12.00

Hydrograph

Hydrograph for Point 3:											
	q (cfs) ¹	q (cfs) ¹	q (cfs) ¹	q (csm/in)	q (cfs) ¹						
12.2	12.3	12.3	12.4	12.4	12.5	12.6	12.6	12.7	12.7	12.8	12.8
40.04	59.2	50.65	57.4	49.11	43.1	36.88	29.8	25.50	21.8	18.48	16.3
27.91	48.1	16.78	25.0	8.72	18.6	5.79	12.8	4.47	10.2	3.56	8.90
19.97	48.1	12.01	25.0	6.24	16.6	4.14	12.8	3.20	10.2	2.55	3.00
10.34	48.1	6.22	25.0	3.23	16.6	2.15	12.8	1.85	10.2	1.32	2.15
8.40	48.1	5.05	25.0	2.63	16.6	1.74	12.8	1.34	10.2	1.07	1.11
6.45	21.7	2.23	14.7	1.51	12.3	1.26	10.4	1.07	8.6	0.90	0.90
87.92					64.07	46.81	33.16	24.58		76	76
									19.09		13.09
										10.24	

Hydrograph for Point M:											
	q (cfs) ¹	q (cfs) ¹	q (cfs) ¹	q (csm/in)	q (cfs) ¹						
12.2	12.3	12.3	12.4	12.4	12.5	12.6	12.6	12.7	12.7	12.8	12.8
12.99	48.1	7.81	250	4.08	186	2.69	128	2.08	102	1.66	1.40
12.99					2.69					1.66	
										1.40	1.14
											0.99

Hydrograph

Hydrograph for Point 3:

$q \text{ (cfs)}^1$	13.4	13.6	13.8	14.	14.	14.3	14.3	14.	14.6	14.6	15.	15.	15.5	15.5	16.	16.	16.5
$q \text{ (cfs/min)}^1$	5.39	55	49	4.19	44	3.78	38	3.25	34	2.91	31	2.65	28	2.40	25	2.14	22
$q \text{ (cfs)}^1$	1.88	49	44	1.71	44	1.53	40	1.40	35	1.22	33	1.15	30	1.05	27	0.94	24
$q \text{ (cfs)}^1$	1.35	49	44	1.22	44	1.10	40	1.00	35	0.87	33	0.82	30	0.75	27	0.67	24
$q \text{ (cfs)}^1$	0.70	49	44	0.63	44	0.57	40	0.52	35	0.45	33	0.43	30	0.39	27	0.36	24
$q \text{ (cfs)}^1$	0.57	49	44	0.51	44	0.46	40	0.42	35	0.37	33	0.35	30	0.32	27	0.28	24
$q \text{ (cfs)}^1$	0.52	46	42	0.47	42	0.43	38	0.39	34	0.35	32	0.33	29	0.30	26	0.27	23
$q \text{ (cfs)}^1$	0.62	8.62	7.64	6.83	6.16	5.35	4.88	4.45	4.01	3.58							

Hydrograph for Point M:

$q \text{ (cfs)}^1$	13.4	13.6	13.8	14.	14.	14.3	14.3	14.	14.6	14.6	15.	15.	15.5	15.5	16.	16.	16.5
$q \text{ (cfs/min)}^1$	0.68	49	44	0.80	40	0.71	40	0.65	35	0.57	33	0.54	30	0.49	27	0.44	24
$q \text{ (cfs)}^1$	0.68	0.62	0.60	0.59	0.57	0.55	0.55	0.55	0.57	0.55	0.54	0.54	0.54	0.54	0.59	0.59	0.59

Hydrograph

Hydrograph for Point M:

q (cfs)	17	17	17.5	17.5	18	18	19	19	20	20	22	22	26	26
q (csm/in)														
q (cfs) ¹														
16.5														
1.88	2.1	1.80	2.0	1.71	1.8	1.54	1.6	1.37	1.4	1.20	1.03	0	0.00	
0.73	20	0.70	19	0.86	18	0.63	16	0.56	13	0.45	12	0.42	0	0.00
0.52	20	0.50	19	0.47	18	0.45	16	0.40	13	0.32	12	0.30	0	0.00
0.27	20	0.26	19	0.25	18	0.23	16	0.21	13	0.17	12	0.16	0	0.00
0.22	20	0.21	19	0.20	18	0.19	16	0.17	13	0.14	12	0.13	0	0.00
0.22	20	0.21	19	0.20	18	0.18	15	0.15	13	0.13	12	0.12	0	0.00
3.14		2.99		2.85		2.62		2.33		1.98		1.74		0.00

Hydrograph for Point M:

q (cfs)	17	17	17.5	17.5	18	18	19	19	20	20	22	22	26	26
q (csm/in)														
q (cfs) ¹														
18.5														
0.34	20	0.32	19	0.31	18	0.29	16	0.26	13	0.21	12	0.19	0	0.00
0.34		0.32		0.31		0.29		0.26		0.21		0.19		0.00

Hydrograph

Hydrograph for Pollin N:

Hydrograph	Area Name (Inset)	Time T_c (hrs) (planimeter)	Travel Time Ch. 2 (hrs)	Down river area names of interest	Total travel time thru river to pt of interest	Runoff P (in)	CN	Curve #	Runoff $A_m Pe$ (in)	Ia/P (in)
N	0.0045687	0.2	0	—	0	—	Fig. 8-3->8	Fig. 2-1	(calculate)	Table 4-1 (calculate)

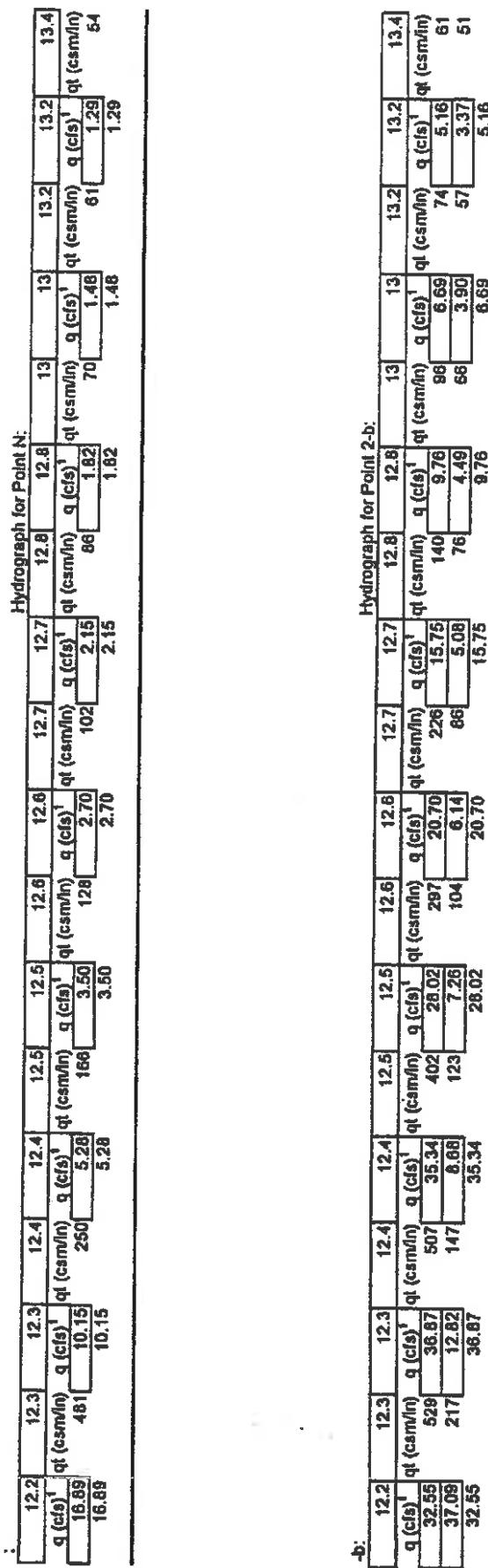
Total travel

Hydrograph	Area Name (inset)	Time T_0 (hrs) (planimetered)	Travel Time thru river, T_1 (hrs)	Down river area names of interest	Rainfall P (in)	Curve # CN	Rundoff Fig. B-3->8 Table 2-2	Ia A_m Pa (in)	Ia (calculated) Table 4-1 Fig. 2-1	Ia/P
P	0.0151525	0.62	0	***	0	7	79	4.6	0.069702	0.532
Q	0.0128398	0.04	0	***	0	7	79	4.6	0.0509052	0.532

1

Name	T _c (hrs)	$\Sigma(T_j)$ (hrs)	lat/P	A _m P _e	q _d (cm ³ /hr)	q _(calulate)	q _(calculate)	q _(cal)	q _(cm³/hr)	q _(cfs)	q _(csm-hr)									
(Insert)	Ch. 2	Ch. 2	(calculate)	(calculate)	q _d (cm ³ /hr)	q _(cfs)	q _(cfs)	q _(csm-hr)	q _(csm-hr)	q _(cfs)	q _(csm-hr)	q _(cfs)	q _(csm-hr)	q _(cfs)	q _(csm-hr)	q _(cfs)	q _(csm-hr)	q _(cfs)	q _(csm-hr)	
P	0.6	0.0	0.1	0.0697015	17	1.18	23	1.60	32	2.23	94	6.55	170	11.85	308	21.47	467			
Q	0.0	0.0	0.1	0.0593621	24	1.42	34	2.01	53	3.13	334	19.73	647	3821	1010	59.65	628			
			Total Flow (cfs):		1.16		1.60		2.23		6.55		11.85		3147					

Hydrograph



Hydrograph

Hydrograph for Point N:

$q \text{ (cfs)}^1$	13.4	13.6	13.8	13.8	14	14	14.3	14.3	14.6	14.6	15	15.5	15.5	16	16	16.5	
$q \text{ (csm/in)}$	1.14	49	1.03	44	0.93	40	0.84	35	0.74	33	0.70	30	0.63	27	0.57	24	0.51
$q \text{ (cfs)}^1$	1.14	49	1.03	44	0.93	40	0.84	35	0.74	33	0.70	30	0.63	27	0.57	24	0.51

Hydrograph for Point 2-b:

$q \text{ (cfs)}^1$	13.4	13.8	13.6	13.8	14	14	14.3	14.3	14.6	14.6	15	15.5	15.5	16	16	16.5
$q \text{ (csm/in)}$	4.25	63	3.69	47	3.28	41	2.86	38	2.51	32	2.23	29	2.02	26	1.81	23
$q \text{ (cfs)}^1$	4.25	63	3.69	47	3.28	41	2.86	38	2.51	32	2.23	29	2.02	26	1.81	23

Hydrograph

Hydrograph for Point N:

q (cfs) ¹	17	17	17.5	17.5	18	18	19	19	20	20	22	22	26	26
q (csm/in)														
q (cfs) ¹														
0.44														
0.42	0.42	0.42	0.40	0.40	0.38	0.38	0.34	0.34	0.27	0.27	0.25	0.25	0.00	0.00
0.44														
0.42														

Hydrograph for Point 2-b:

q (cfs) ¹	17	17	17.5	17.5	18	18	19	19	20	20	22	22	26	26
q (csm/in)														
q (cfs) ¹														
1.60														
1.60	1.60	1.60	1.46	1.46	1.39	1.39	1.32	1.32	1.12	1.12	0.98	0.98	0.00	0.00
1.24														
20	1.18	1.18	1.12	1.12	1.06	1.06	0.89	0.89	0.77	0.77	0.71	0.71	0.00	0.00
1.60														
1.46														
1.39														
1.32														
1.12														
0.98														
0.71														
0.84														
0.00														
0.00														
0.71														
0.98														
0.84														
0.00														
0.00														

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For sheet flow of less than 300 feet, use Manning's kinematic solution to compute T_t :

$$T_t = \frac{0.007 (nL)^{0.8}}{(P_2)^{0.5} s^{0.4}} \quad (\text{Eq.8})$$

where

T_t = travel time (hr),
 n = Manning's roughness coefficient (table 6-1),
 L = flow length (ft),
 P_2 = 2-year, 24-hour rainfall (in), and
 s = slope of hydraulic grade line (land slope, ft/ft).

This simplified form of the Manning's kinematic solution is based on the following: (1) shallow steady uniform flow, (2) constant intensity of rainfall excess (that part of a rain available for runoff), (3) rainfall duration of 24 hours, and (4) minor effect of infiltration on travel time. Rainfall depth can be obtained from table 6-3.

Table 6-1. Roughness coefficients (Manning's n) for sheet flow.

Surface description	n^1
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover < 20%	0.06
Residue cover > 20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses ²	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods: ³	
Light underbrush	0.40
Dense underbrush	0.80

¹The n values are a composite of information compiled by Engman.

²Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.

³When selecting n , consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

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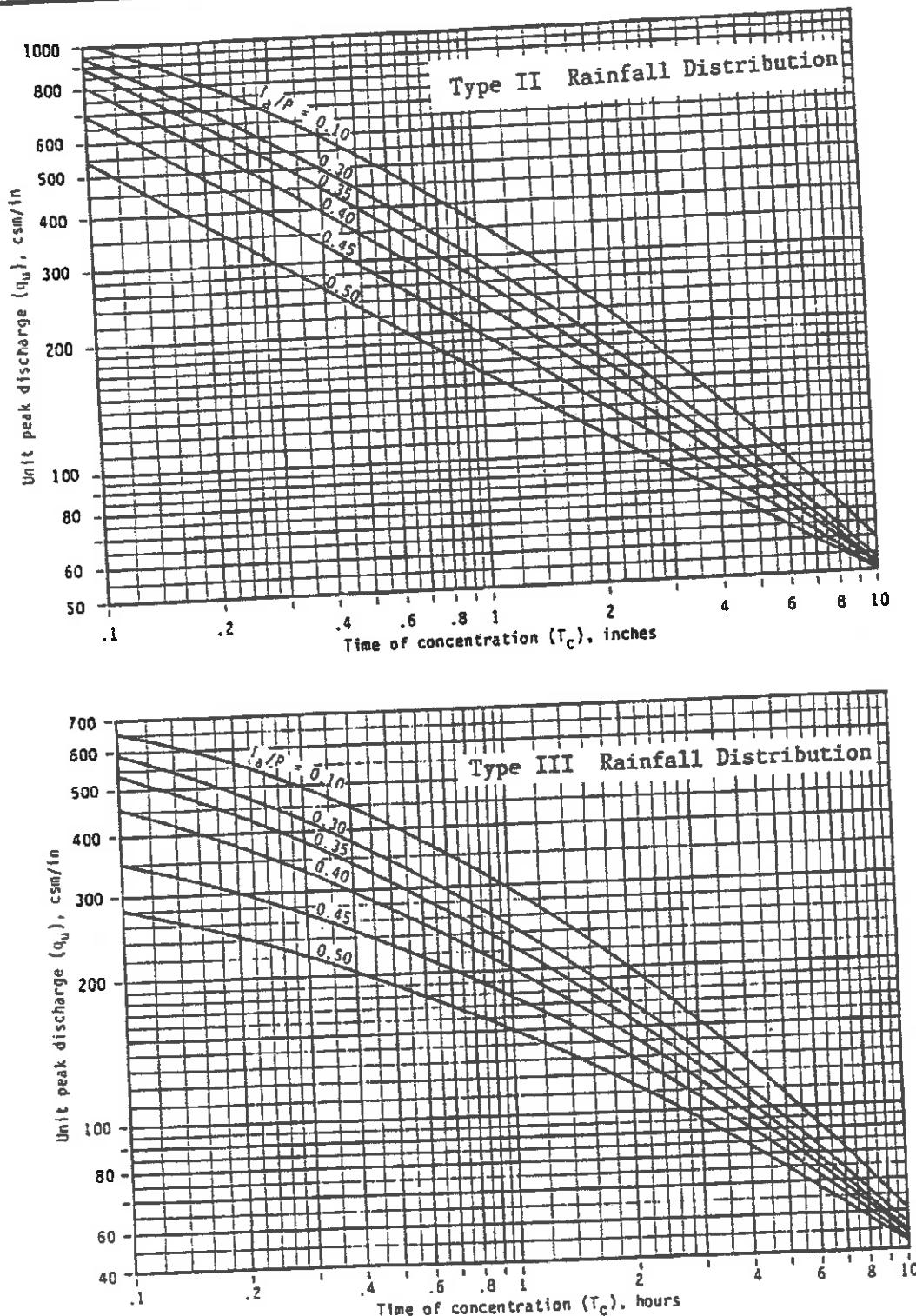
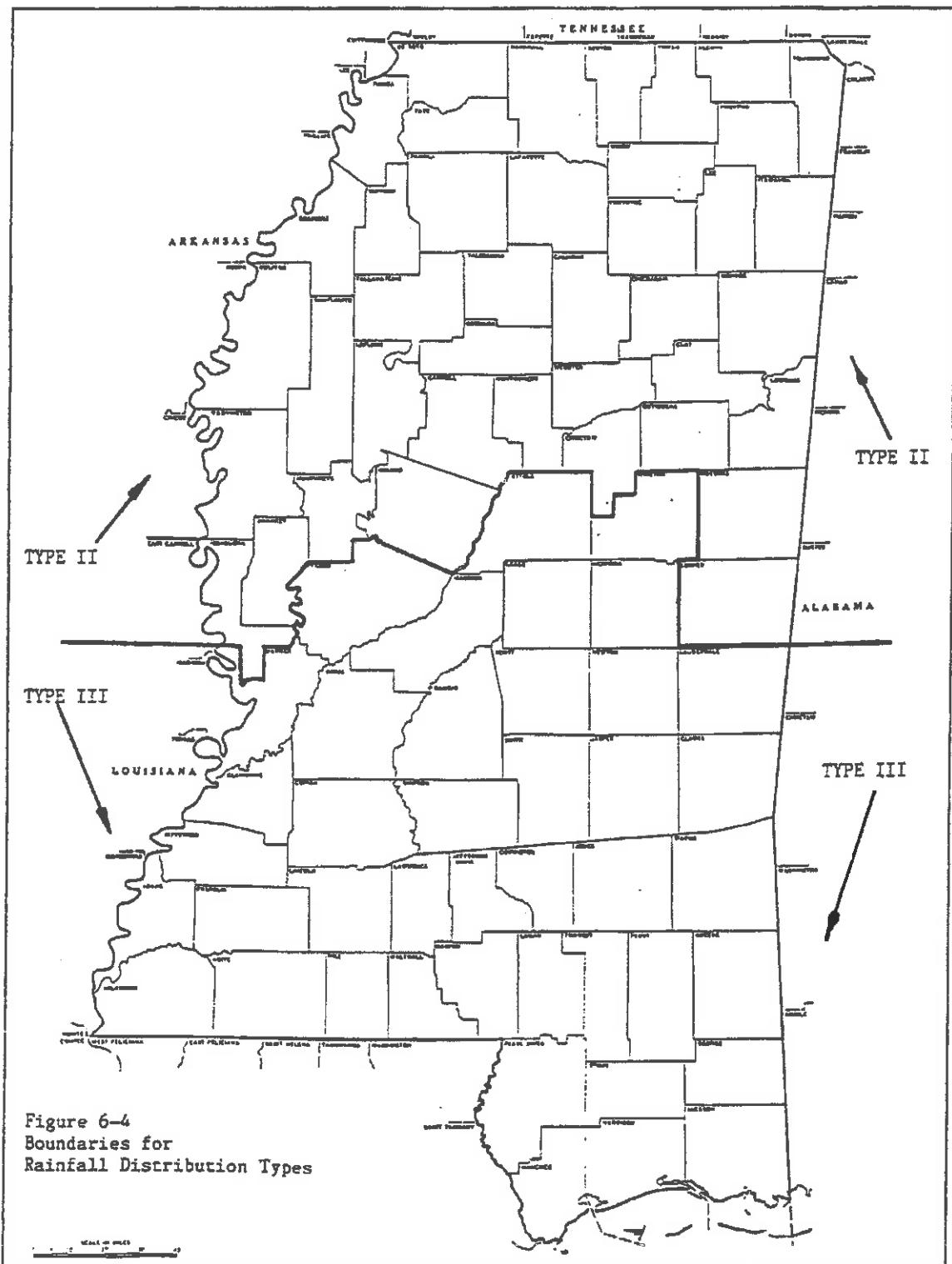


Figure 6-3. Unit peak discharge (q_u).

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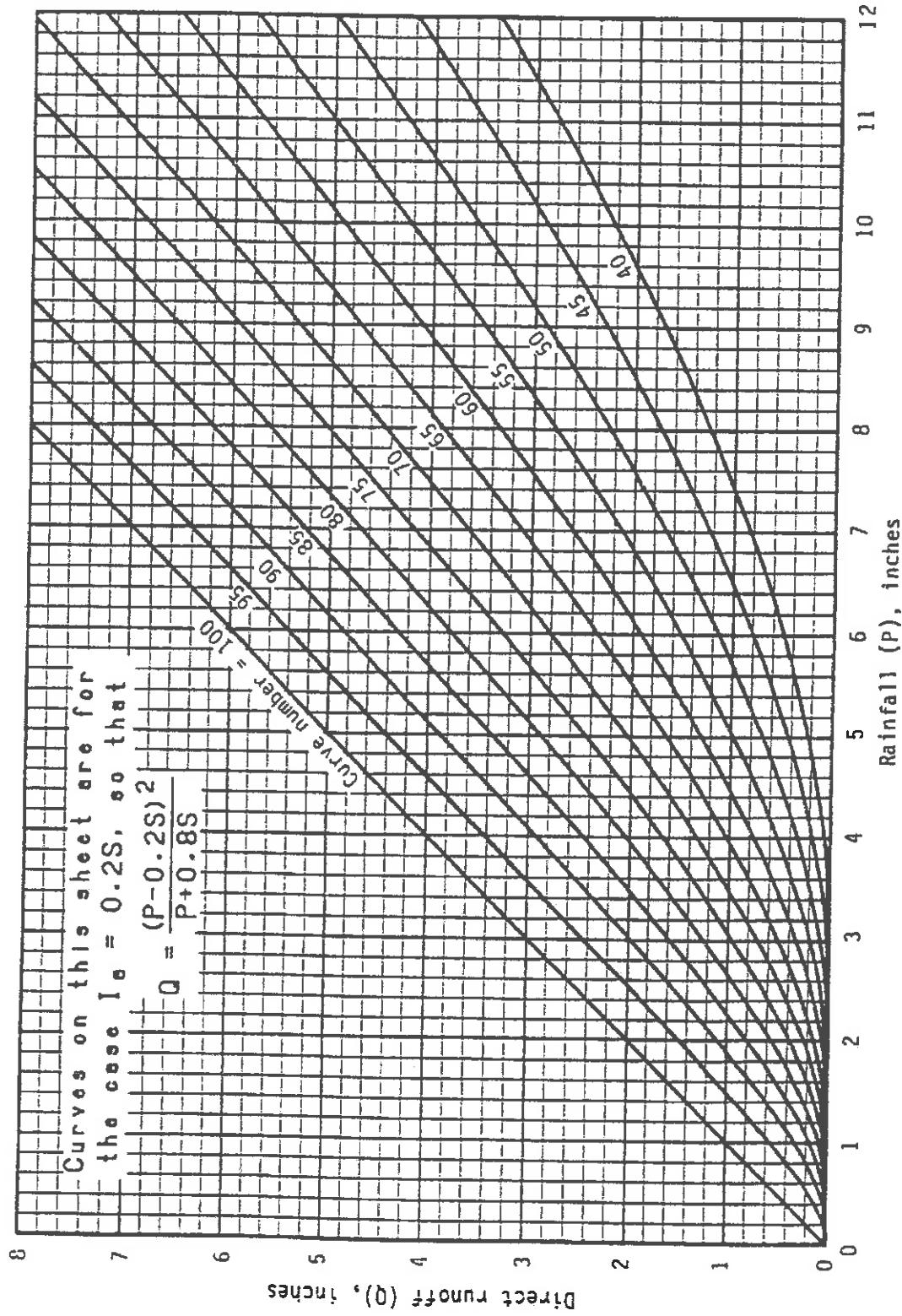


Figure 6-5. Solution for runoff equation.

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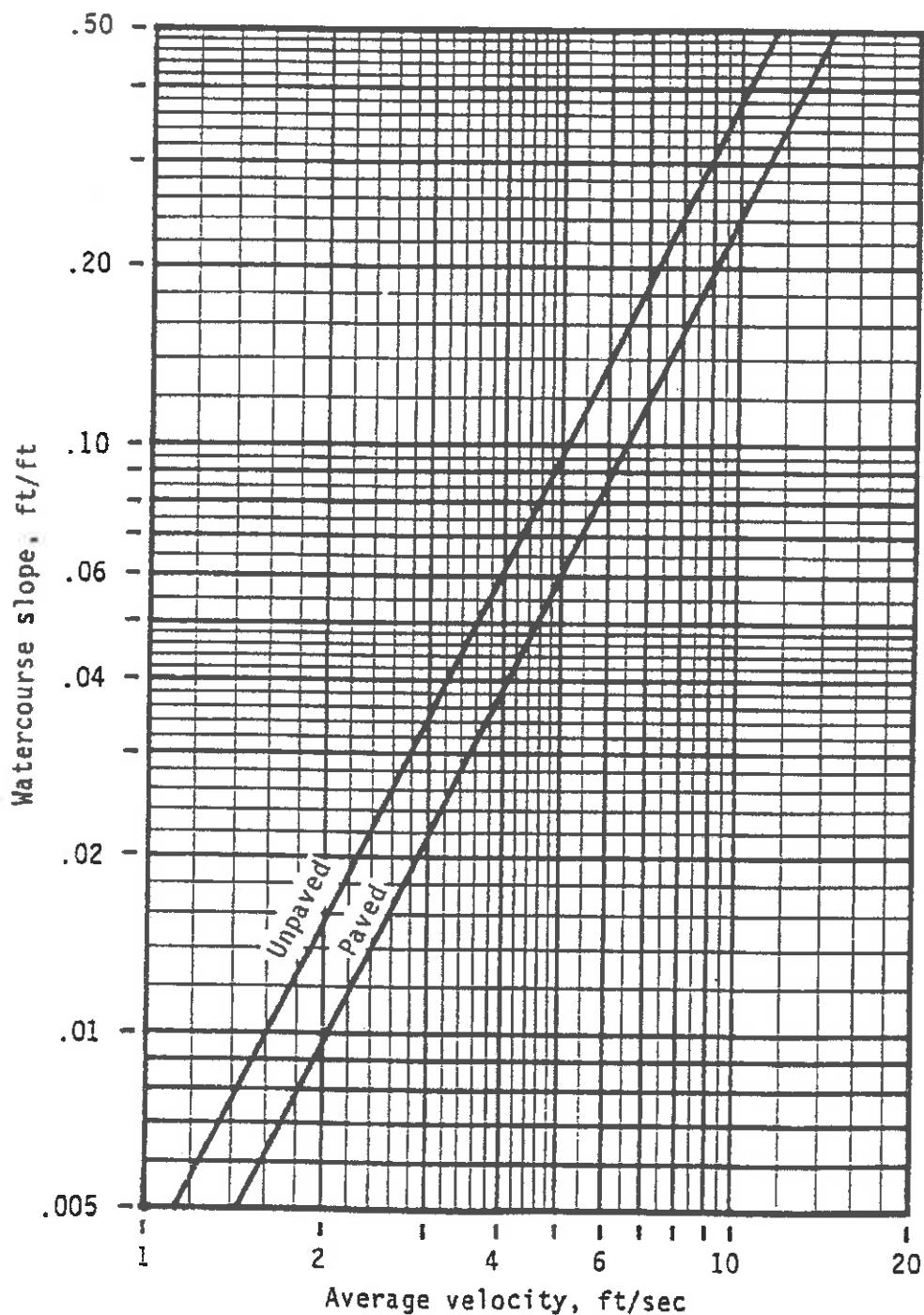


Figure 6-9. Average velocities for estimating travel time for shallow concentrated flow.

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Table 6-3. Rainfall frequency values
(estimated precipitation for 24-hour period expressed in inches).

County	Storm Frequency						
	1 yr.	2 yr.	5 yr.	10 yr.	25 yr.	50 yr.	100 yr.
ADAMS	4.0	4.8	6.3	7.0	8.4	9.1	10.3
ALCORN	3.3	3.9	4.8	5.6	6.5	7.1	7.7
AMITE	4.2	4.8	6.4	7.4	8.6	9.4	10.6
ATTALA	3.7	4.3	5.4	6.3	7.2	8.1	8.8
BENTON	3.4	4.0	4.9	5.7	6.6	7.2	7.8
BOLIVAR	3.6	4.3	5.3	6.2	7.0	7.8	8.6
CALHOUN	3.5	4.1	5.2	5.9	6.9	7.6	8.3
CARROLL	3.6	4.2	5.3	6.2	7.1	6.2	8.6
CHICKASAW	3.5	4.1	5.1	5.9	6.8	7.6	8.3
CHOCTAW	3.6	4.2	5.3	6.2	7.0	7.9	8.6
CLAIBORNE	3.9	4.6	6.0	6.9	7.8	8.8	9.6
CLARKE	3.9	4.7	6.0	6.9	7.9	8.9	9.8
CLAY	3.5	4.1	5.2	6.0	6.9	7.7	8.4
COAHOMA	3.5	4.2	5.2	6.0	6.9	7.7	8.4
COPIAH	3.9	4.6	6.0	6.9	7.9	8.8	9.7
COVINGTON	4.0	4.7	6.2	7.1	8.2	9.2	10.2
DESOTO	4.0	4.1	5.0	5.8	6.7	7.2	8.1
FORREST	4.2	4.9	6.5	7.7	8.8	10.0	11.2
FRANKLIN	4.0	4.8	6.3	7.0	8.3	9.0	10.1
GEORGE	4.4	5.4	7.3	8.5	9.7	11.1	12.3
GREENE	4.2	4.9	6.7	7.8	8.9	10.4	11.5
GRENADA	3.6	4.2	5.2	6.0	6.9	7.7	8.5
HANCOCK	4.7	5.8	7.5	8.7	10.5	11.4	12.5
HARRISON	4.7	5.8	7.5	8.8	10.5	11.4	12.6
HINDS	3.9	4.4	5.8	6.7	7.7	8.6	9.4
HOLMES	3.7	4.3	5.4	6.3	7.2	8.1	8.8
HUMPHREYS	3.7	4.4	5.4	6.4	7.3	8.2	8.8
ISSAQUENA	3.8	4.3	5.6	6.6	7.5	8.4	9.1
ITAWAMBA	3.4	3.9	5.1	5.8	6.6	7.3	8.0
JACKSON	4.7	5.9	7.7	9.0	10.5	11.5	13.0
JASPER	3.9	4.6	6.0	6.8	7.9	8.8	9.7
JEFFERSON	4.0	4.7	6.1	7.0	8.0	8.9	9.9
JEFF DAVIS	4.0	4.6	6.2	7.0	8.2	9.0	10.0
JONES	4.0	4.8	6.2	7.2	8.2	9.3	10.5
KEMPER	3.7	4.4	5.6	6.5	7.4	8.2	9.0
LAFAYETTE	3.5	4.1	5.1	5.8	6.8	7.4	8.2
LAMAR	4.2	4.9	6.5	7.6	8.6	9.8	11.0
LAUDERDALE	3.8	4.6	5.7	6.7	7.6	8.5	9.4
LAWRENCE	4.0	4.6	6.2	7.0	8.2	9.0	10.0
LEAKE	3.8	4.3	5.5	6.5	7.4	8.3	9.0
LEE	3.4	4.0	5.0	5.8	6.7	7.4	8.1
LEFLORE	3.6	4.3	5.3	6.2	7.1	7.9	8.6
LINCOLN	4.0	4.7	6.2	7.0	8.2	9.0	10.0
LOWNES	3.6	4.2	5.3	6.1	7.0	7.8	8.5

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Table 6-6B. Runoff curve numbers for other agricultural lands 1/.

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

¹Average runoff condition.

²Poor: <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.

³Poor: <50% ground cover.

Fair: 50 to 75% ground cover.

Good: >75% ground cover.

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

⁵CN'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.

⁶Poor: Forest, litter, small trees, and brush have been 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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Table 6-6C. Runoff curve numbers for urban areas ^{1/}.

Cover description	Cover type and hydrologic condition	Average percent impervious area ²	Curve numbers for hydrologic soil group			
			A	B	C	D
<i>Fully developed urban areas (vegetation established)</i>						
Open space (lawns, parks, golf courses, cemeteries, etc.) ³ :			68	79	86	89
Poor condition (grass cover < 50%)			49	69	79	84
Fair condition (grass cover 50% to 75%)			39	61	74	80
Good condition (grass cover > 75%)						
<i>Impervious areas:</i>						
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
<i>Western desert urban areas:</i>						
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
<i>Urban districts:</i>						
Commercial and business		85	89	92	94	95
Industrial		72	81	88	91	93
<i>Residential districts by average lot size:</i>						
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
<i>Developing urban areas</i>						
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 6-6A and 6B)						

¹Average runoff condition, and $I_a = 0.25$.

²The 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 6-2 or 6-3.

³CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.

⁴Composite CN's for natural desert landscaping should be computed using figures 6-2 or 6-3 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.

⁵Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 6-2 or 6-3, based on the degree of development (impervious area percentage) and the CN's for the newly graded perVIOUS areas.

Chapter 6 - HYDROLOGY

Table 6-7. -I_a values for runoff curve numbers.

Curve Number	I _a (in)	Curve Number	I _a (in)
40	3.000	68	0.941
41	2.878	69	0.899
42	2.762	70	0.857
43	2.651	71	0.817
44	2.545	72	0.778
45	2.444	73	0.740
46	2.348	74	0.703
47	2.255	75	0.667
47	2.167	76	0.632
49	2.082	77	0.597
50	2.000	78	0.564
51	1.922	79	0.532
52	1.846	80	0.500
53	1.774	81	0.469
54	1.704	82	0.439
55	1.636	83	0.410
56	1.571	84	0.381
57	1.509	85	0.353
58	1.448	86	0.326
59	1.390	87	0.299
60	1.333	88	0.273
61	1.279	89	0.247
62	1.226	90	0.222
63	1.175	91	0.198
64	1.125	92	0.174
65	1.077	93	0.151
66	1.030	94	0.128
67	0.985	95	0.105

APPENDIX A

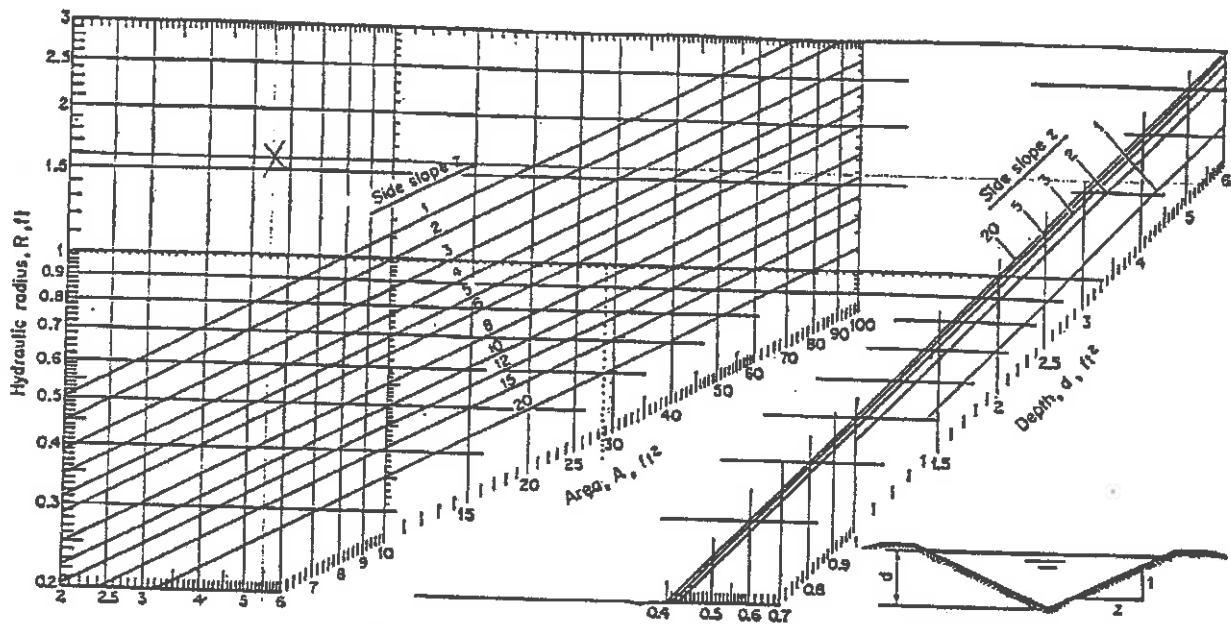


Fig. B-9. Hydraulic elements of triangular channels.

Subscale S

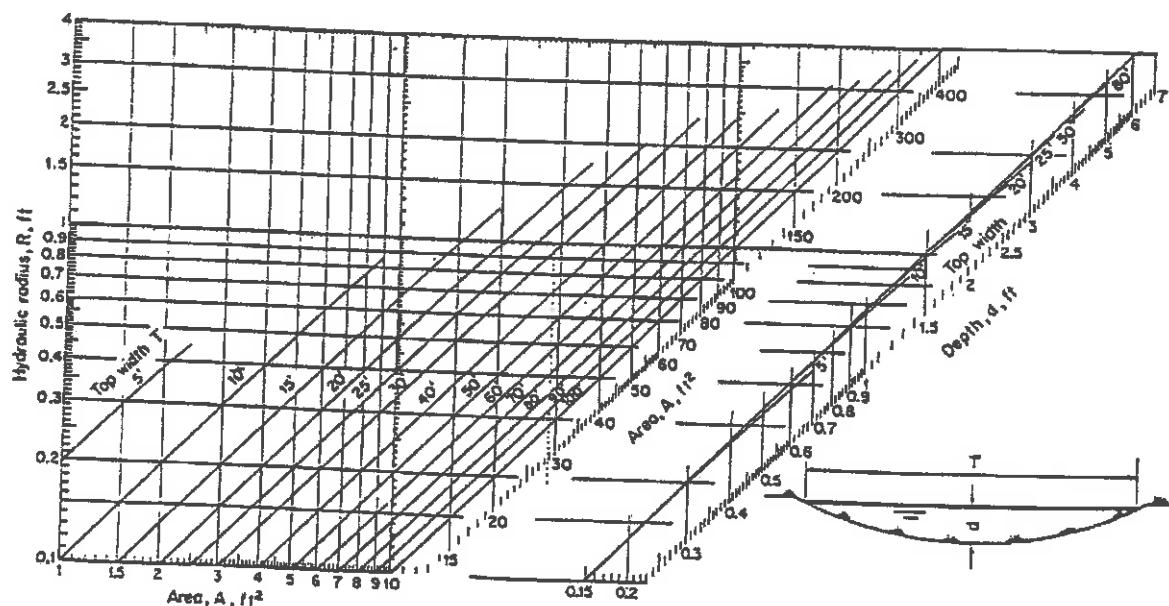


Fig. B-10. Hydraulic elements of parabolic channels.

Section 3
Design of Swales for Final Grade

APPENDIX A

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000103

(S)

Assumptions

10-Year Grading Plan - Design of Diversion Swales
Choctow Generation Facility - Ash Disposal Site
Subsidiary of Red Hills Generation Facility
Job No. 0874097110
Date Calc 5-Aug-97

PROBLEM STATEMENT:

Design diversion swales for the final 10-year grading plan to accommodate the 25-year, 24-hour storm event.

ASSUMPTIONS:

1. Permanent diversions will be designed to be stable under the as-built, bare soil condition, according to the velocities set forth in the P & D Manual: 4 fps for clay, 3 fps for silt, and 2.5 fps for sandy soils. Critical swales, such as perimeter swales, and representative swales on the final grade were chosen to be designed. All other swales will follow the representative and critically designed configurations.
2. All sediment and erosion control features are designed for the 25-year, 24-hour storm event.
3. CN based on Soil Group D (assuming silty clay), for runoff, agricultural lands, woods, good condition was assumed for a CN = 77. For final condition runoff, grass fair condition was assumed, for a CN of 79. See Table 6-6B of Mississippi Erosion and Sediment Control Manual.
4. Design of all swales is based on the proposed sequential filling plans (attached).
5. Drainage areas were determined using the Planix planimeter.
6. Stability calculations based on "Open-Channel Hydraulics" by Ven Te Chow, Section 7-20. The process for designing for stability is the following (attached):
 1. Assume value of n and determine VR from Figure 7-14.
 2. Select permissible velocity from SCS or Table 7-6.
 3. Compute VR using Manning's Formula:
$$VR = (1.49 \cdot R^{5/3} \cdot S^{1/2})/n$$
 4. Check the computed VR against the VR in the n-VR curve. Continue until equal or within approximately 3% error.
 5. Compute the water area by $A = Q/V$.
 6. Determine various sections by using Ra nd A computed, and figures in Appendix B of Chow. This is approximately your depth, width, bottom slope and sideslopes.
7. Perform capacity calculations. This method compares velocity and is usually deeper:
 1. Assume a depth y and compute area A and hydraulic radius R (geometric formula).
 2. Compute velocity V by $V = Q/A$, and VR by $VR = V \cdot R$.
 3. From this VR and n-VR curve (higher vegetal retardance), find n.
 4. Using this n and Manning's formula, compute V.
 5. Compare this V with the V in Step 2. Continue until equal or within 3% error. Note that this value should be equal to or less than the assumed velocity in the stability calculations. If not, then go back to stability and change the bottom or side slopes (or both), and continue through the steps again.
9. Add the proper freeboard, 0.3 ft.

APPENDIX A

(C)

Assumptions

REFERENCES:

1. "Planning & Design Manual for the Control of Erosion, Sediment & Stormwater" by Mississippi Department of Environmental Quality, Mississippi SCS & USSCS.
2. "Open-Channel Hydraulics" by Ven Te Chow, Section 7-20.
3. "Urban Hydrology for Small Watersheds" by the USDA, SCS, Technical Release 55 (TR-55).

APPENDIX A

Summary

Summary of Design Criteria for Diversion Swales

Swale Name	Base Slope	Side slope			Actual Velocity (fps) ²	Recommended Velocity (fps) ³	Reinforcement Required?
		X:1	Depth (ft)	Flow (cfs)			
a	0.01	4.0	2.2	41.84	2.9	8.0	Yes
b	0.01	4.0	2.3	50.65	3.2	8.0	Yes
T	0.015	4.0	2.8	140.64	5.6	8.0	Yes
R	0.009	4.0	2.6	75.12	3.6	8.0	Yes
O	0.010	4.0	2.1	33.42	2.5	8.0	Yes
S	0.010	4.0	2.2	43.79	2.9	8.0	Yes
d	0.010	4.0	1.5	19.97	1.5	4.0	No
P	0.010	4.0	1.9	36.87	1.9	8.0	Yes

¹Includes a minimum 0.3 foot freeboard.

²Actual calculated velocity based on the actual flow and configuration determined in Stability.

³Maximum permissible velocity based on the Mississippi Stormwater Design Manual or a higher value for reinforced soil using Enkamat. Based on stability analysis. If above 4 fps, then reinforcement is necessary to obtain capacity velocities.

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Stability

V_p = permissible velocity using Enkamat = 8 fps, as reinforcement
 S = Slope of swale = 0.01 ft/ft (Assumed value)

Swale Number: a

Trial No.	Mannings n	VR _{Table}	R (VR _{Table} /V _p)	VR _{Computed}	Delta VR
1	0.03	2.25	0.28125	0.60	-275%
2	0.04	0.70	0.0875	0.06	-990%
3	0.032	1.70	0.21	0.35	-382%
4	0.02375	12.50	1.56	13.20	5%

H₂O Area 5.23 sf

Wetted Perimeter (P) = 3.35 ft

Using the charts in Appendix B of Chow's "Open Channel Hydraulics", the best configuration for this swale with this flow is 1:1 triangular. This yields in a swale 4.5 feet depth. However, since we will be building this swale with 4:1 side slopes, the slope will be stable enough using enkamat reinforcement. This configuration is used for capacity calculations (velocity).

Bottom Width: 0 ft

Side Slopes: 4 : 1

V_p = permissible velocity using Enkamat = 8 fps, as reinforcement
 S = Slope of swale = 0.01 ft/ft (Assumed value)

Swale Number: b

Trial No.	Mannings n	VR _{Table}	R (VR _{Table} /V _p)	VR _{Computed}	Delta VR
1	0.03	2.25	0.28125	0.60	-275%
2	0.04	0.70	0.0875	0.06	-990%
3	0.032	1.70	0.21	0.35	-382%
4	0.02375	12.50	1.56	13.20	5%

H₂O Area 6.33 sf

Wetted Perimeter (P) = 4.05 ft

Using the charts in Appendix B of Chow's "Open Channel Hydraulics", the best configuration for this swale with this flow is 1:1 triangular. This yields in a swale 4.5 feet depth. However, since we will be building this swale with 4:1 side slopes, the slope will be stable enough using enkamat reinforcement. This configuration is used for capacity calculations (velocity).

Bottom Width: 0 ft

Side Slopes: 4 : 1



Stability

V_p = permissible velocity using Enkamat = 8 fps, as reinforcement
 S = Slope of swale = 0.015 ft/ft (Assumed value)

Swale Number: T

Trial No.	Mannings n	VR _{Table}	R (VR _{Table} /V _p)	VR _{Computed}	Delta VR
1	0.03	2.25	0.28125	0.73	-206%
2	0.04	0.70	0.0875	0.08	-790%
3	0.032	1.70	0.21	0.43	-294%
4	0.0245	9.00	1.13	9.06	1%

H₂O Area 17.58 sf Flow = 140.64 cfs
 Wetted Perimeter (P) = 15.63 ft

Using the charts in Appendix B of Chow's "Open Channel Hydraulics", the best configuration for this swale with this flow is 3:1 triangular. This yields in a swale 2.4 feet depth. However, since we will be building this swale with 4:1 side slopes, the slope will be stable enough using enkamat reinforcement. This configuration is used for capacity calculations (velocity).

Bottom Width: 0 ft
 Side Slopes: 4:1

V_p = permissible velocity using Enkamat = 8 fps, as reinforcement
 S = Slope of swale = 0.009 ft/ft (Assumed value)
 Q = Flow = 75.12 cfs

Swale Number: R

Trial No.	Mannings n	VR _{Table}	R (VR _{Table} /V _p)	VR _{Computed}	Delta VR
1	0.03	2.25	0.28125	0.56	-302%
2	0.04	0.70	0.0875	0.06	-1068%
3	0.032	1.70	0.21	0.33	-417%
4	0.02375	12.50	1.56	12.31	-2%

H₂O Area 9.39 sf
 Wetted Perimeter (P) = 6.01 ft

Using the charts in Appendix B of Chow's "Open Channel Hydraulics", the best configuration for this swale with this flow is 1:1 triangular. This yields in a swale 4.5 feet depth. However, since we will be building this swale with 4:1 side slopes, the slope will be stable enough.

This configuration is used for capacity calculations (velocity).

Bottom Width: 0 ft
 Side Slopes: 4:1



Stability

V_p = permissible velocity using Enkamat = 8 fps, as reinforcement

S = Slope of swale = 0.01 ft/ft (Assumed value)

Q = Flow = 33.42 cfs

Swale Number: O

Trial No.	Mannings n	VR _{Table}	R (VR _{Table} /V _p)	VR _{Computed}	Delta VR
1	0.03	2.25	0.28125	0.60	-275%
2	0.04	0.70	0.0875	0.06	-990%
3	0.032	1.70	0.21	0.35	-382%
4	0.02375	12.50	1.56	13.20	5%

H₂O Area 4.18 sf

Wetted Perimeter (P) = 2.67 ft

Using the charts in Appendix B of Chow's "Open Channel Hydraulics", the best configuration for this swale with this flow is 1:1 triangular. This yields in a swale 4.5 feet depth. However, since we will be building this swale with 4:1 side slopes, the slope will be stable enough. This configuration is used for capacity calculations (velocity).

Bottom Width: 0 ft

Side Slopes: 4 :1

V_p = permissible velocity using Enkamat = 8 fps, as reinforcement

S = Slope of swale = 0.01 ft/ft (Assumed value)

Q = Flow = 43.79 cfs

Swale Number: S

Trial No.	Mannings n	VR _{Table}	R (VR _{Table} /V _p)	VR _{Computed}	Delta VR
1	0.03	2.25	0.28125	0.60	-275%
2	0.04	0.70	0.0875	0.06	-990%
3	0.032	1.70	0.21	0.35	-382%
4	0.02375	12.50	1.56	13.20	5%

H₂O Area 5.47 sf

Wetted Perimeter (P) = 3.50 ft

Using the charts in Appendix B of Chow's "Open Channel Hydraulics", the best configuration for this swale with this flow is 1:1 triangular. This yields in a swale 4.5 feet depth. However, since we will be building this swale with 4:1 side slopes, the slope will be stable enough.

This configuration is used for capacity calculations (velocity).

Bottom Width: 0 ft

Side Slopes: 4 :1

(8)

Stability

V_p = permissible velocity based on Miss. SCS 4 fps, low resistance

S = Slope of swale = 0.01 ft/ft (Assumed value)

Q = Flow = 19.97 cfs

Swale Number: d

Trial No.	Mannings n	VR _{Table}	R (VR _{Table} /V _p)	VR _{Computed}	Delta VR
1	0.03	2.25	0.5625	1.90	-18%
2	0.04	0.70	0.175	0.20	-243%
3	0.032	1.70	0.43	1.12	-52%
4	0.0285	2.62	0.66	2.58	-1%

H₂O Area 4.99 sf

Wetted Perimeter (P) = 7.62 ft

Using the charts in Appendix B of Chow's "Open Channel Hydraulics", the best configuration for this swale with this flow is 2.5:1 triangular. This yields in a swale 1.4 feet depth. However, since we will be building this swale with 4:1 side slopes, the slope will be stable enough.

This configuration is used for capacity calculations (velocity).

Bottom Width: 0 ft

Side Slopes: 4 :1

V_p = permissible velocity using Enkamat = 8 fps, as reinforcement

S = Slope of swale = 0.01 ft/ft (Assumed value)

Q = Flow = 36.87 cfs

Swale Number: P

Trial No.	Mannings n	VR _{Table}	R (VR _{Table} /V _p)	VR _{Computed}	Delta VR
1	0.03	2.25	0.28125	0.60	-275%
2	0.04	0.70	0.0875	0.06	-990%
3	0.032	1.70	0.21	0.35	-382%
4	0.02375	12.50	1.56	13.20	5%

H₂O Area 4.61 sf

Wetted Perimeter (P) = 2.95 ft

Using the charts in Appendix B of Chow's "Open Channel Hydraulics", the best configuration for this swale with this flow is 1:1 triangular. This yields in a swale 4.5 feet depth. However, since we will be building this swale with 4:1 side slopes, the slope will be stable enough using enkamat reinforcement. This configuration is used for capacity calculations (velocity).

Bottom Width: 0 ft

Side Slopes: 4 :1

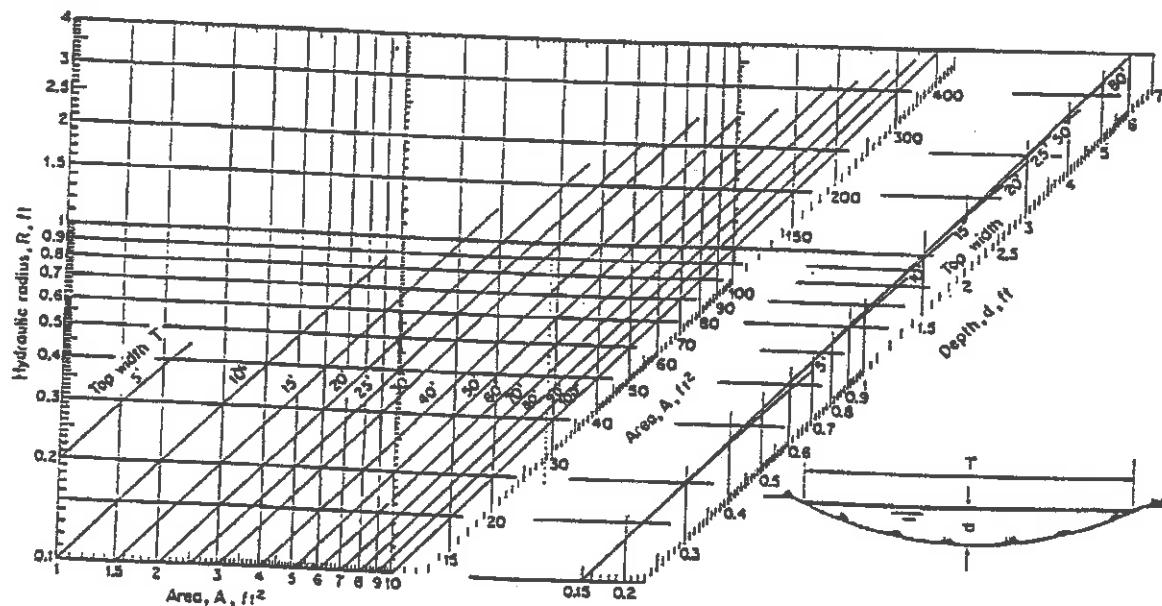
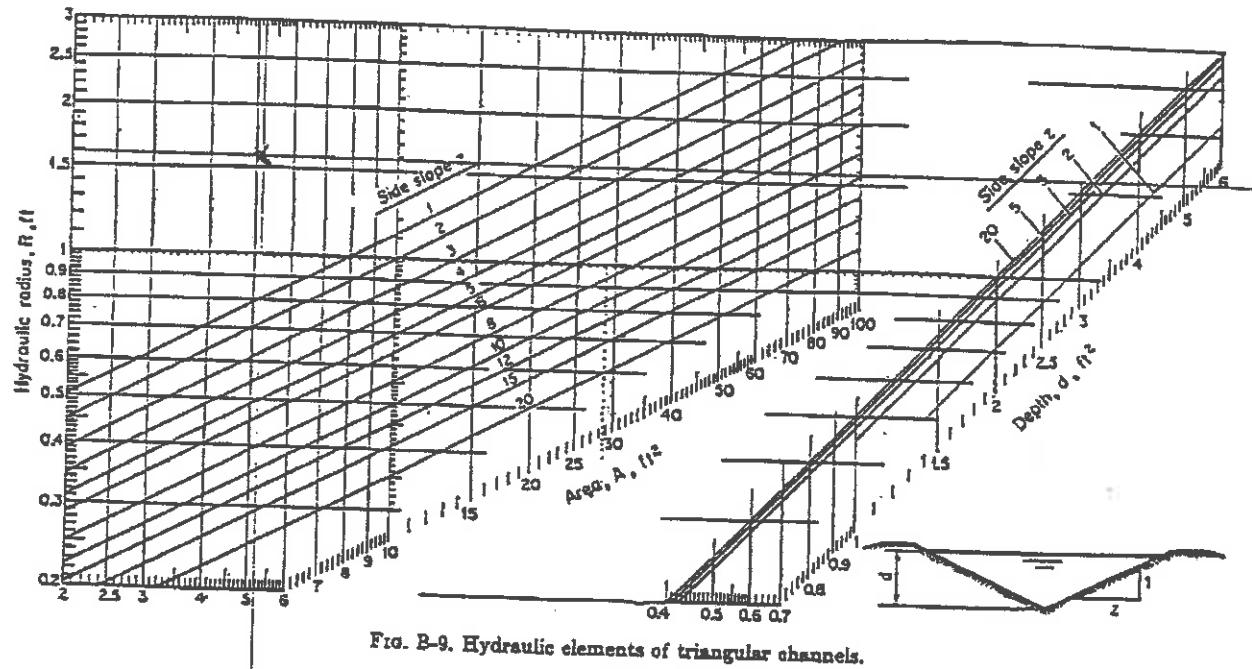
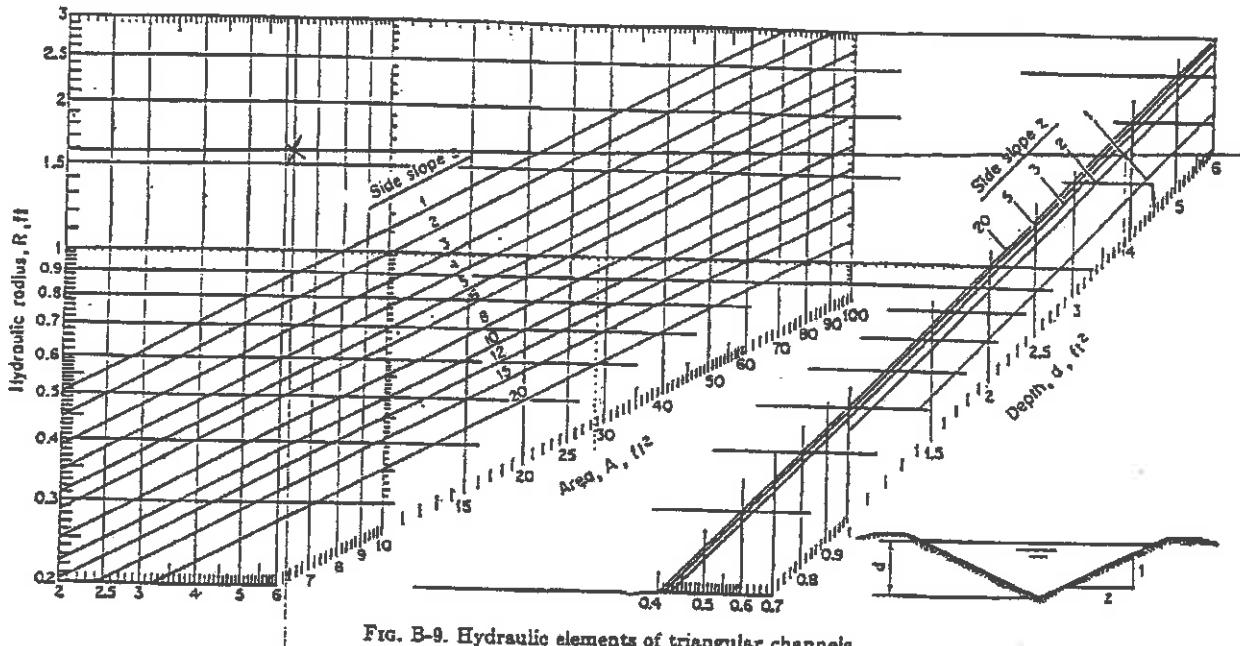


Fig. B-10. Hydraulic elements of parabolic channels.



Swale b

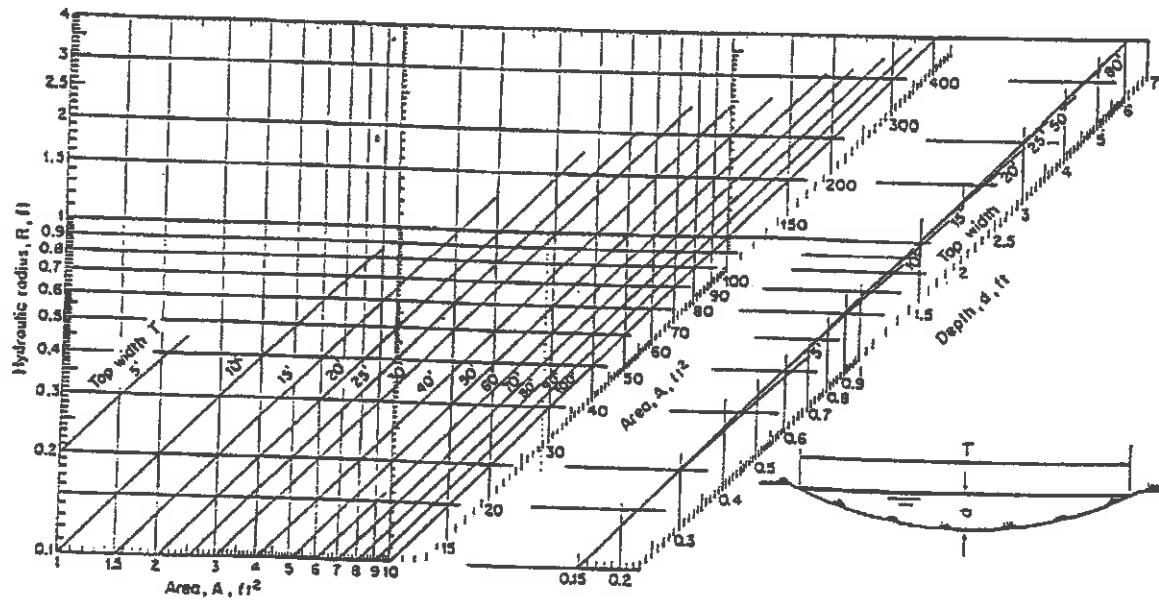


FIG. B-10. Hydraulic elements of parabolic channels.

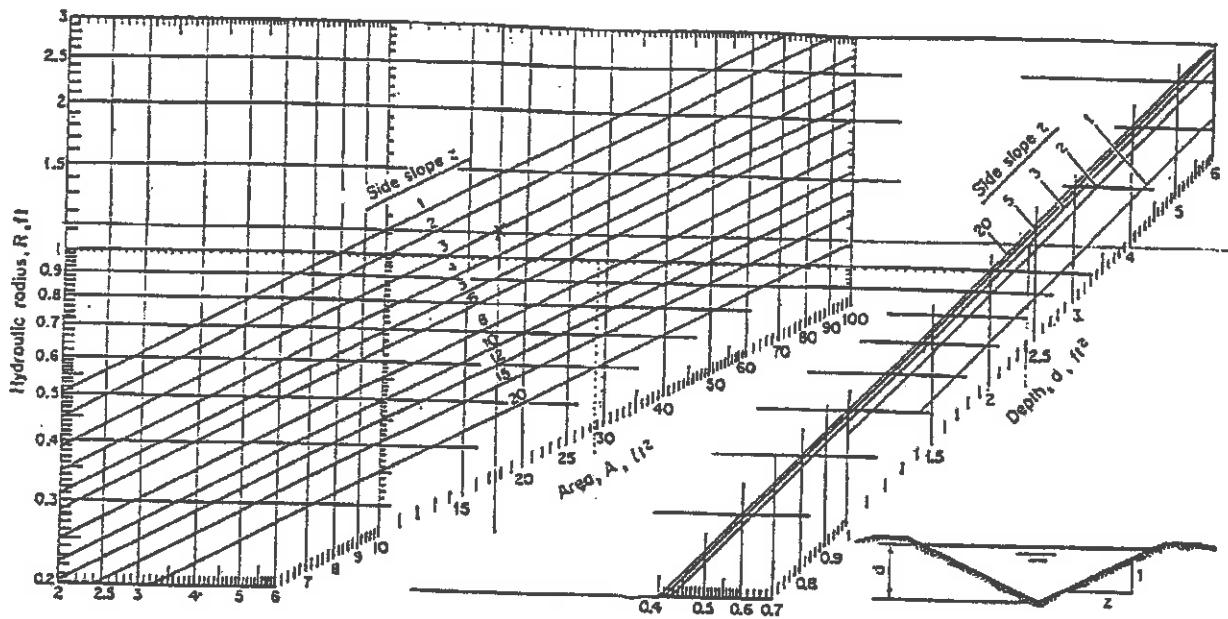


FIG. B-9. Hydraulic elements of triangular channels.

Swale T

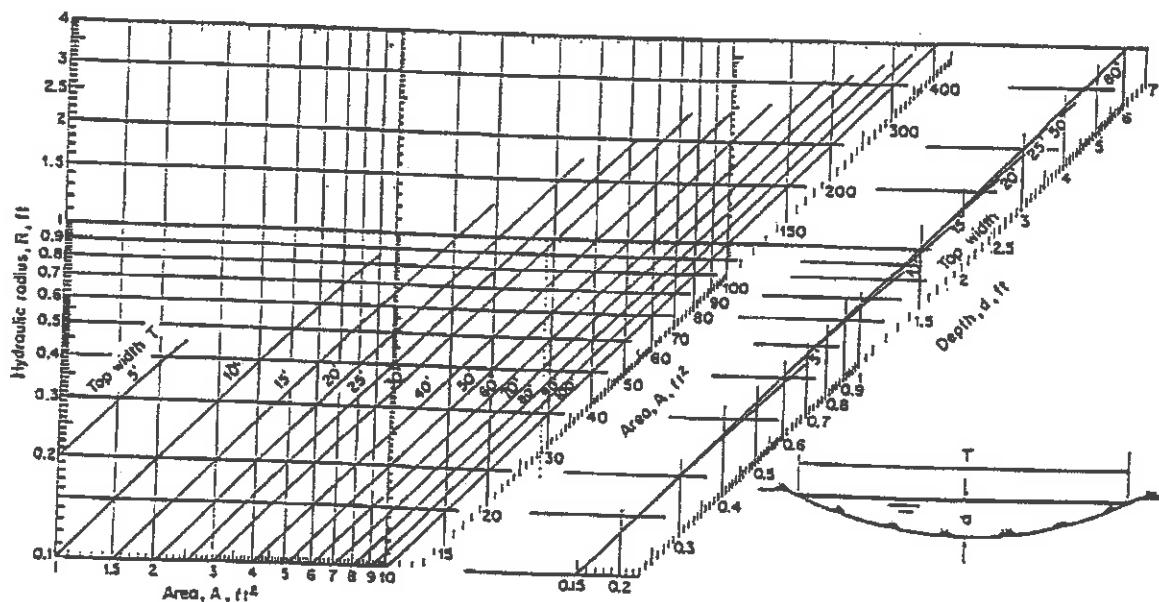


FIG. B-10. Hydraulic elements of parabolic channels.

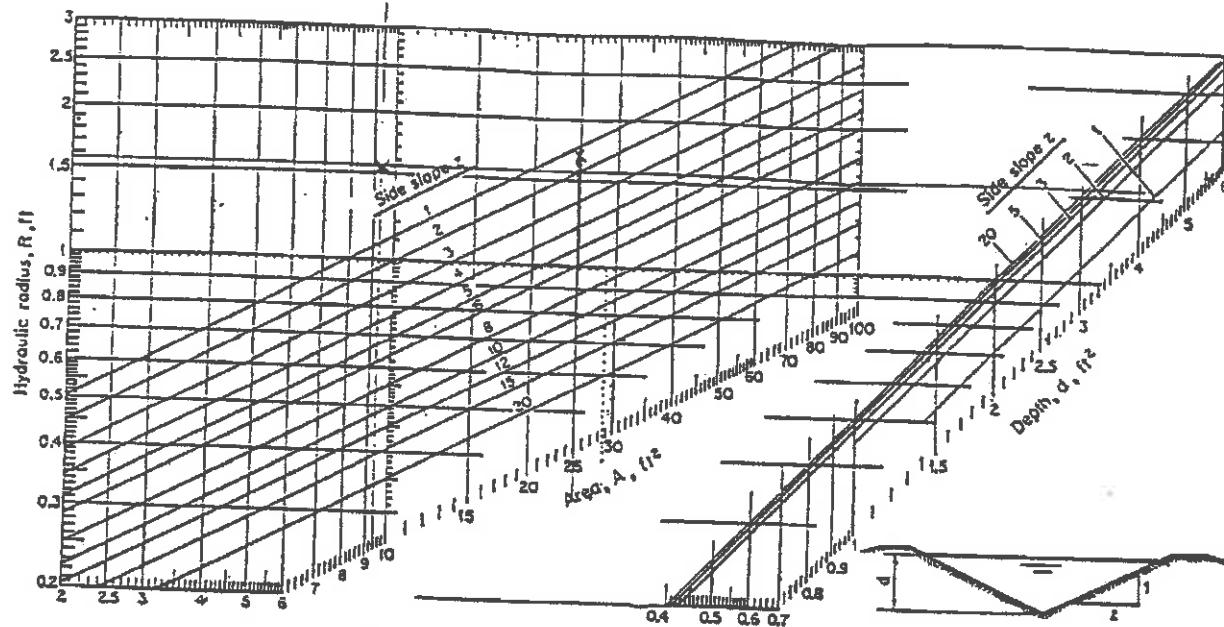


FIG. B-9. Hydraulic elements of triangular channels.

Swal R

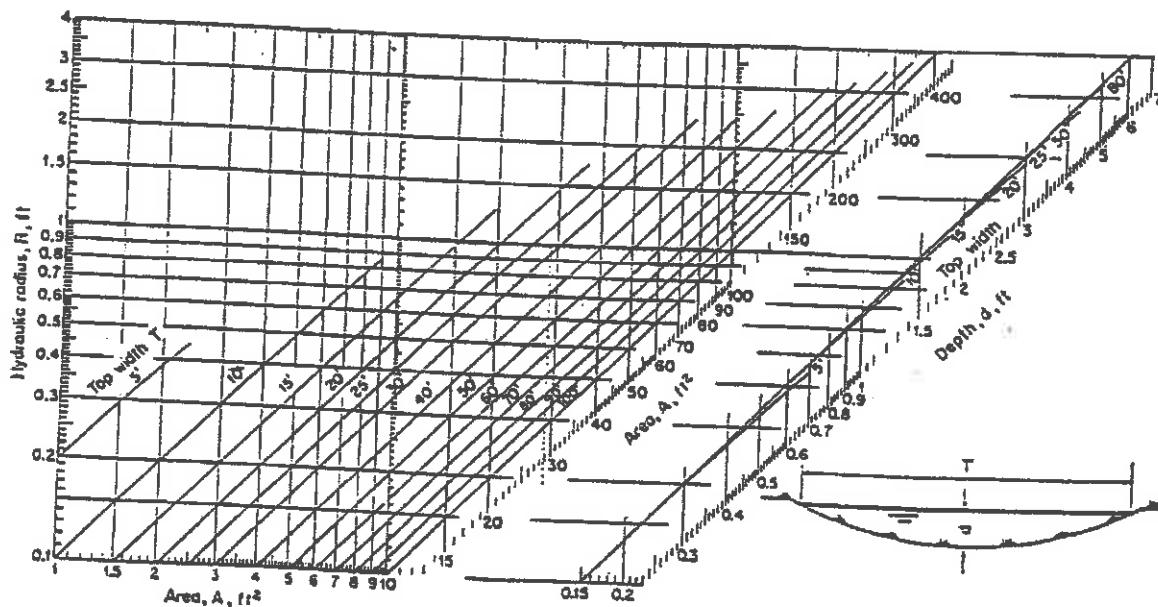


FIG. B-10. Hydraulic elements of parabolic channels.

APPENDIX A

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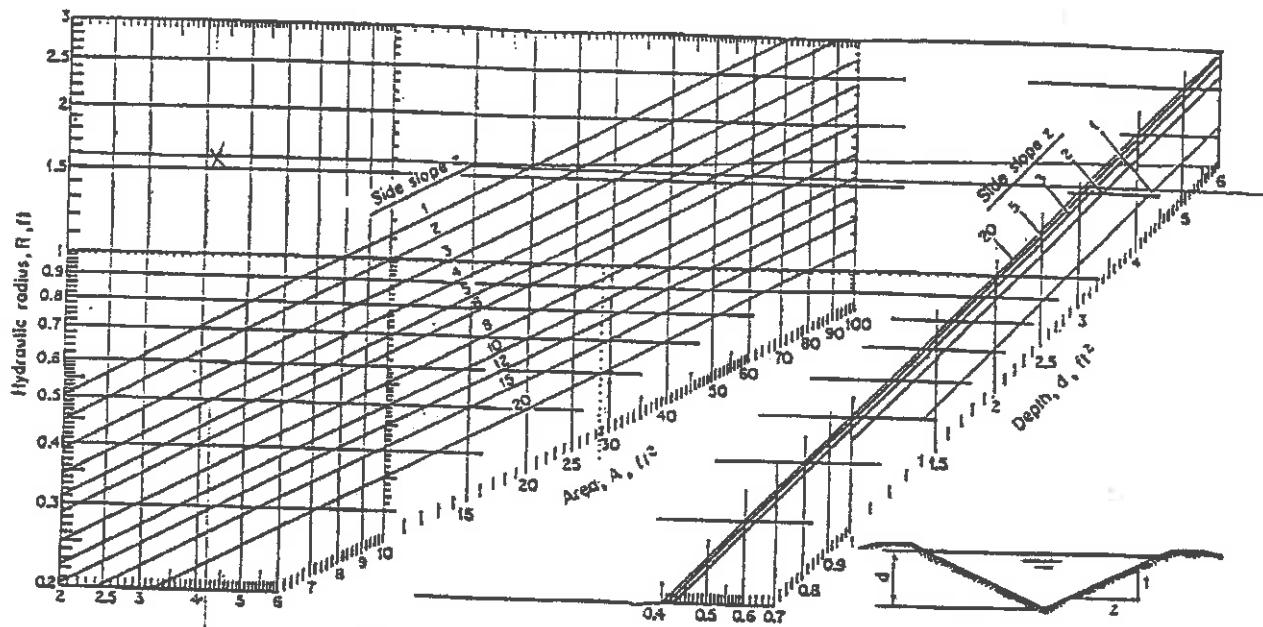


Fig. B-9. Hydraulic elements of triangular channels.

Swele O

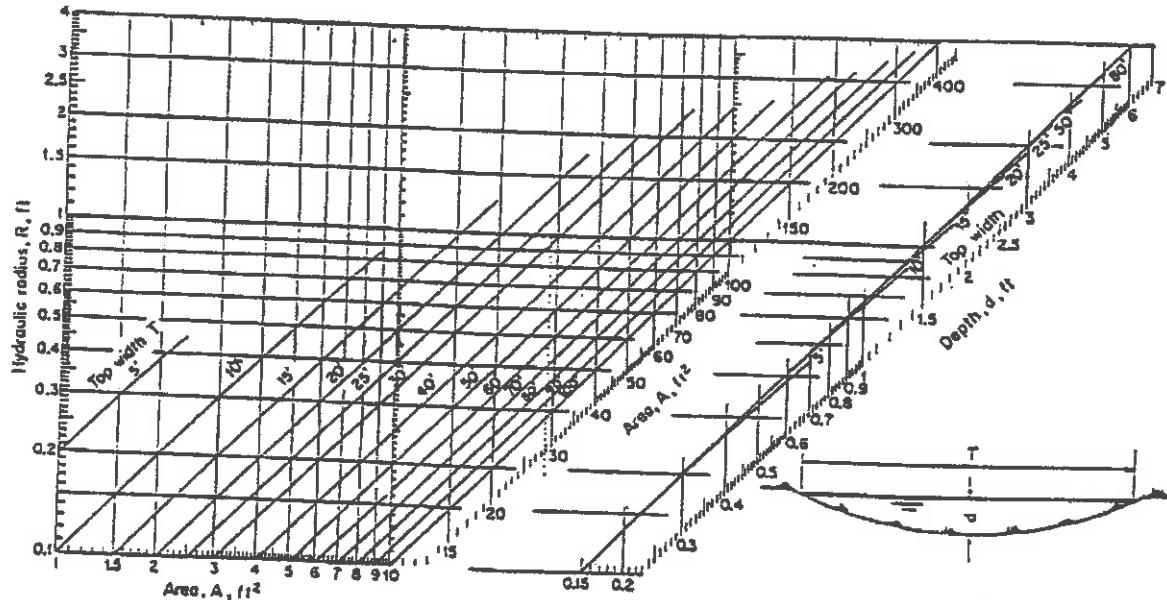


Fig. B-10. Hydraulic elements of parabolic channels.

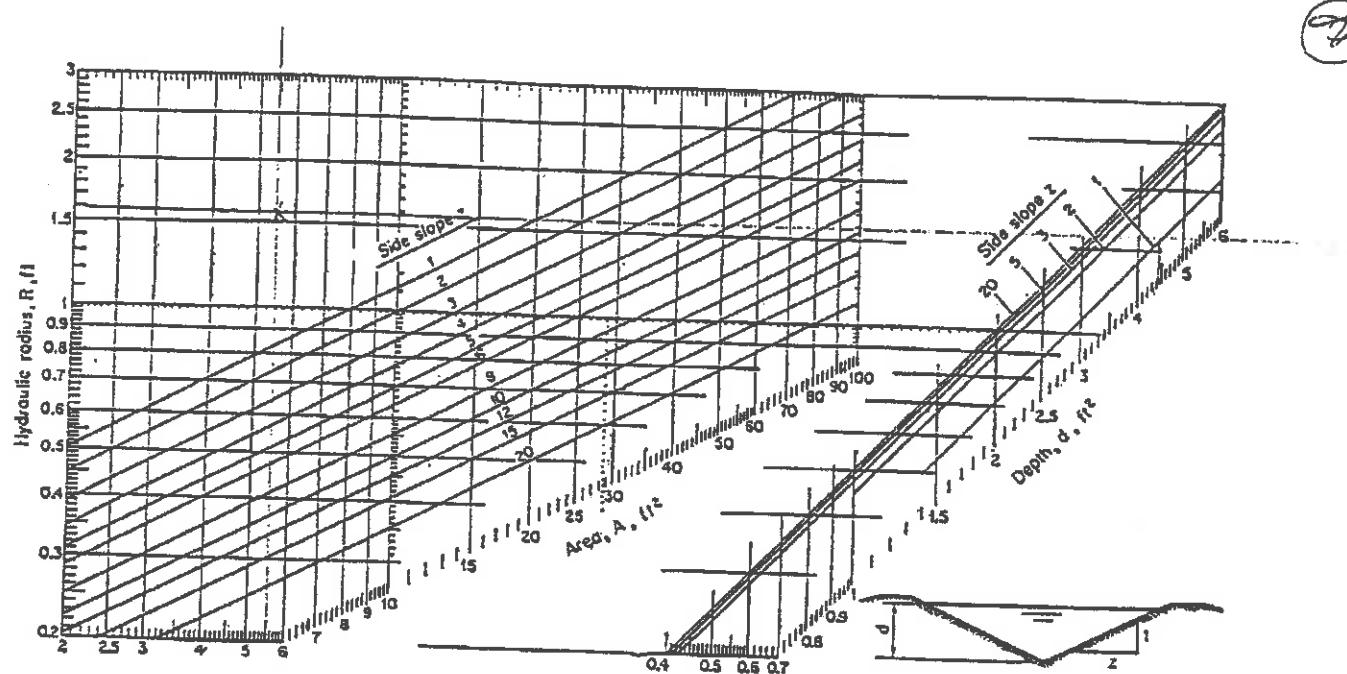


FIG. B-9. Hydraulic elements of triangular channels.

Scale S

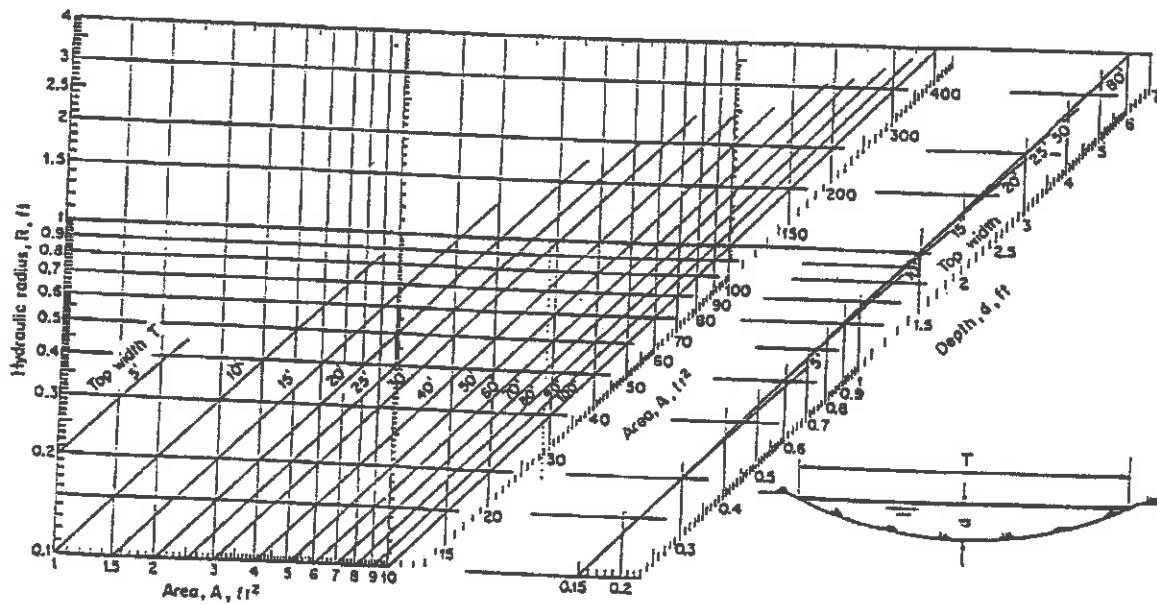


FIG. B-10. Hydraulic elements of parabolic channels.

APPENDIX A

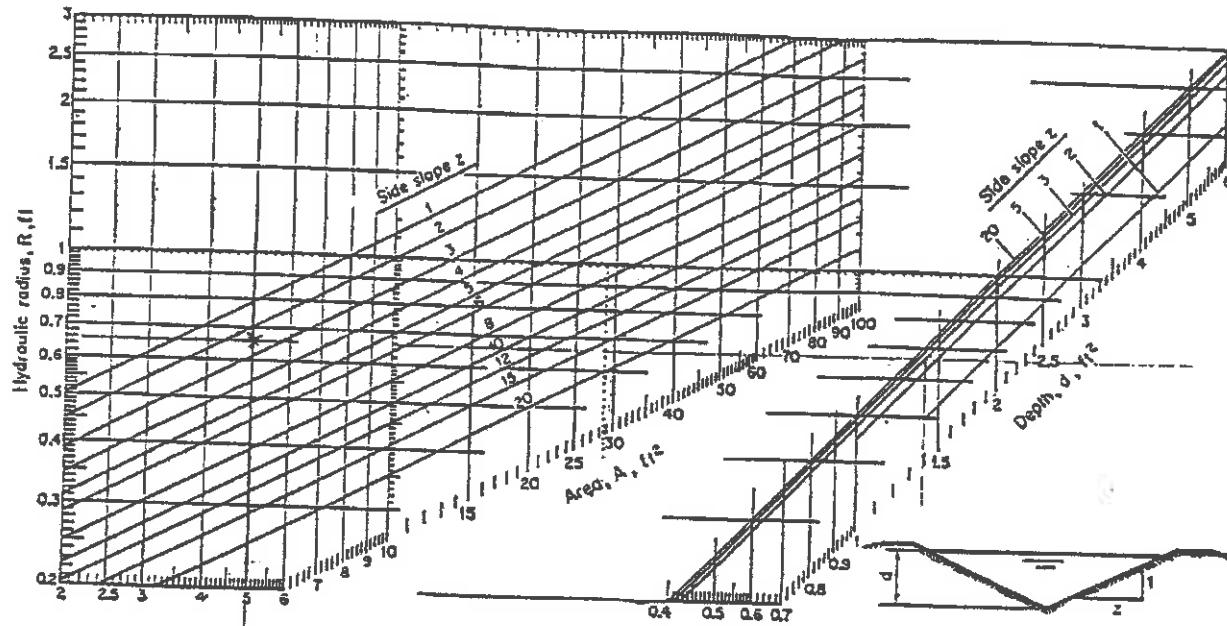


FIG. B-9. Hydraulic elements of triangular channels.

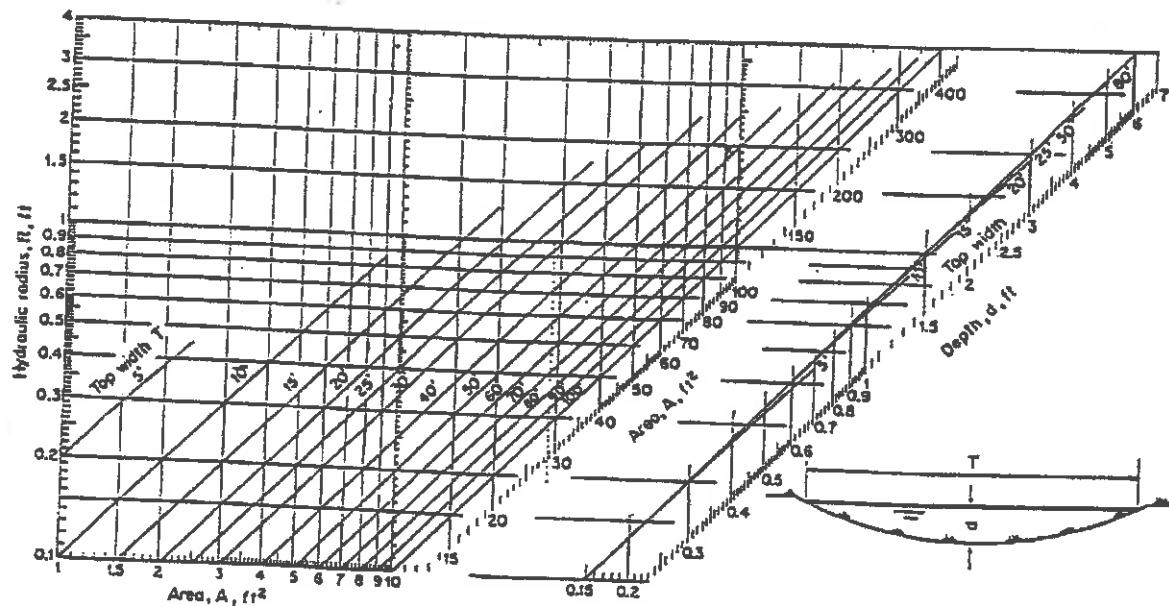


FIG. B-10. Hydraulic elements of parabolic channels.

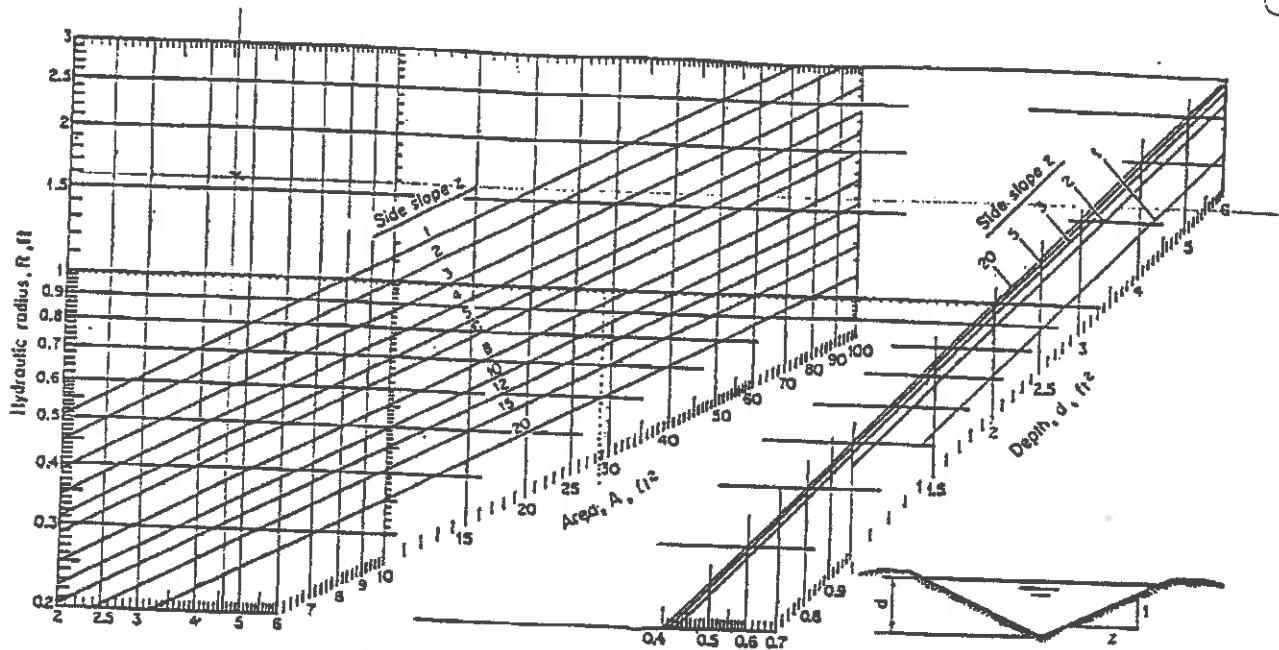


FIG. B-9. Hydraulic elements of triangular channels.

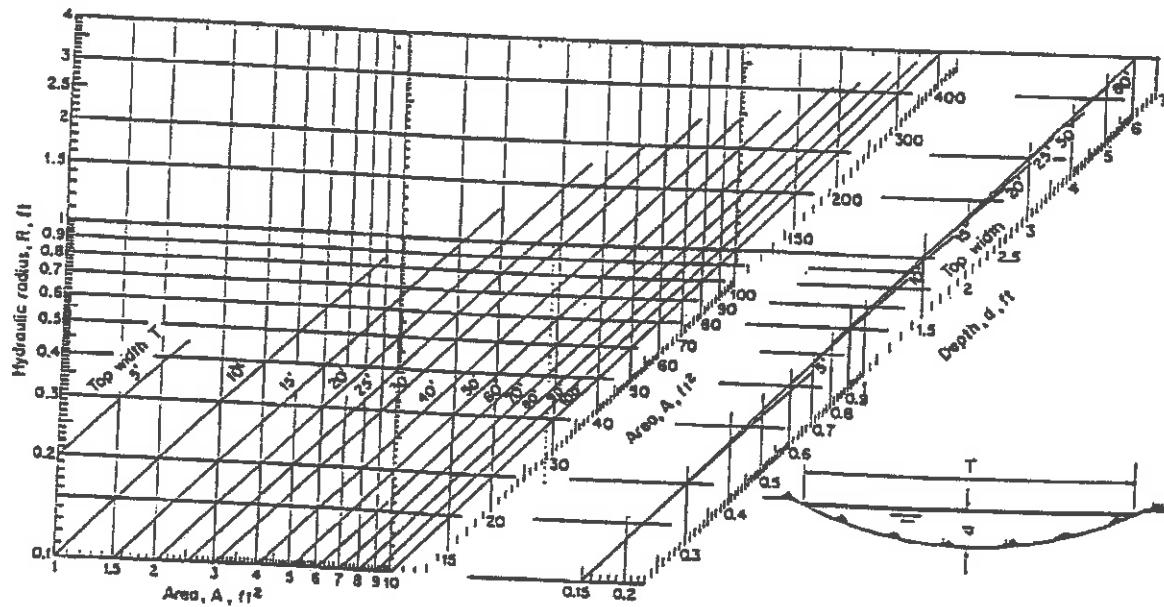


FIG. B-10. Hydraulic elements of parabolic channels.

APPENDIX A

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(B)

Capacity

Red Hills Generation Facility - Design of Swales - 10-year Grading Plan

Width: 0 ft

Swale Number:		a	Mannings					
Trial No.	y (ft)	Area (sf)	R	V _{calc}	VR	n	V _{manning}	Delta V
1	1.9	14.44	0.921635	2.80	2.58	0.0495	2.85	2%

This results in a Swale with
of 2.9 fps, a flow of 4 :1 side slopes & depth 41.84 cfs. 2.2 ft at a velocity

Width: 0 ft

Swale Number:		b	Mannings					
Trial No.	y (ft)	Area (sf)	R	V _{calc}	VR	n	V _{manning}	Delta V
1	2	16.00	0.970143	3.17	3.07	0.046	3.17	0%

This results in a Swale with
of 3.17 fps, a flow of 4 :1 side slopes & depth 50.65 cfs. 2.3 ft at a velocity

Width: 0 ft

Swale Number:		T	Mannings					
Trial No.	y (ft)	Area (sf)	R	V _{calc}	VR	n	V _{manning}	Delta V
1	2.5	25.00	1.212578	5.63	6.82	0.037	5.61	0%

This results in a Swale with
of 5.61 fps, a flow of 4 :1 side slopes & depth 140.64 cfs. 2.8 ft at a velocity

Width: 0 ft

Swale Number:		R	Mannings					
Trial No.	y (ft)	Area (sf)	R	V _{calc}	VR	n	V _{manning}	Delta V
1	2.3	21.16	1.115664	3.55	3.96	0.042	3.56	0%

This results in a Swale with
of 3.56 fps, a flow of 4 :1 side slopes & depth 75.12 cfs. 2.6 ft at a velocity

Width: 0 ft

Swale Number:		O	Mannings					
Trial No.	y (ft)	Area (sf)	R	V _{calc}	VR	n	V _{manning}	Delta V
1	1.8	12.96	0.873128	2.58	2.25	0.054	2.52	-2%

This results in a Swale with
of 2.52 fps, a flow of 4.00 :1 side slopes & depth 33.42 cfs. 2.1 ft at a velocity

(B)

Capacity

Width: 0 ft

Swale Number:

S

Trial No.	y (ft)	Area (sf)	R	Mannings				Delta V
				V _{calc}	VR	n	V _{manning}	
1	1.9	14.44	0.921635	3.03	2.79	0.049	2.88	-5%

This results in a Swale with
of 2.88 fps, a flow of 43.79 cfs.
4.00 :1 side slopes & depth 2.2 ft at a velocity

Width: 0 ft

Swale Number:

d

Trial No.	y (ft)	Area (sf)	R	Mannings				Delta V
				V _{calc}	VR	n	V _{manning}	
1	1.2	5.76	0.582086	0.61	0.35	0.17	0.61	0%

This results in a Swale with
of 0.61 fps, a flow of 19.97 cfs.
4.00 :1 side slopes & depth 1.5 ft at a velocity

Width: 0 ft

Swale Number:

P

Trial No.	y (ft)	Area (sf)	R	Mannings				Delta V
				V _{calc}	VR	n	V _{manning}	
1	1.63	10.63	0.790666	1.88	1.49	0.067	1.90	1%

This results in a Swale with
of 1.90 fps, a flow of 36.87 cfs.
4.00 :1 side slopes & depth 1.9 ft at a velocity

VeryLowVegetalRetardance

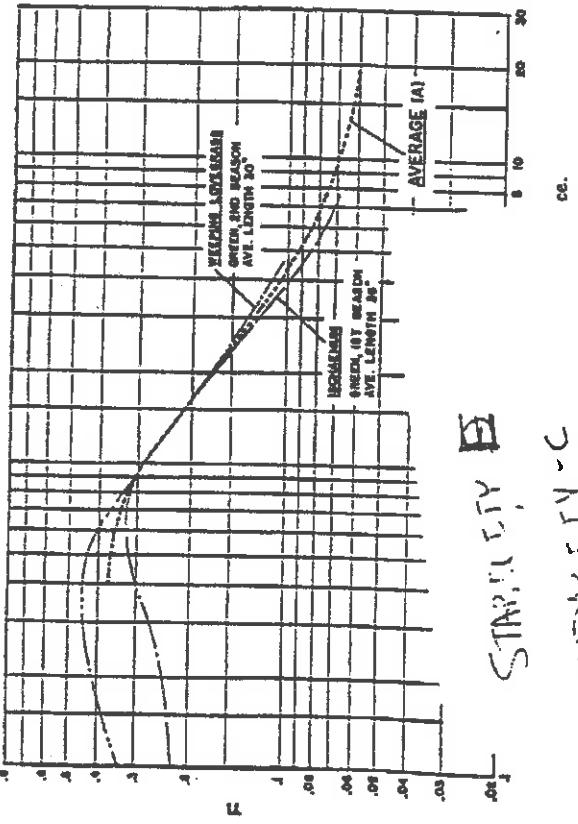
Curve for Very Low Vegetal Retardance for Grass-lined Channels (E) for Stability

VR	Manning's n
15	0.0235
20	0.023
10	0.024
8	0.025
5.67	0.0255
4.33	0.026
3	0.027
2.75	0.028
2.5	0.029
2.25	0.03
2.15	0.031
1.7	0.032
1.3	0.034
1.1	0.035
1	0.036
0.9	0.037
0.84	0.039
0.7	0.04
0.58	0.0425
0.55	0.045

APPENDIX A

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(b) Curves for B or high vegetal retardance.

ce.

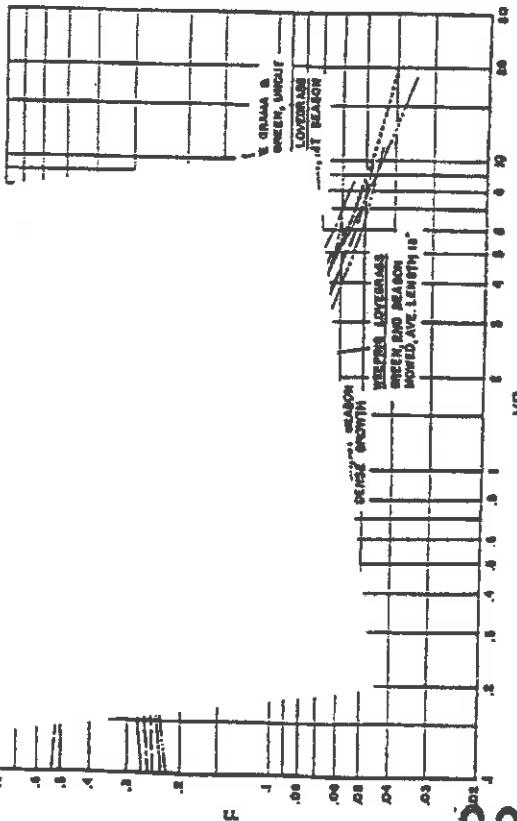
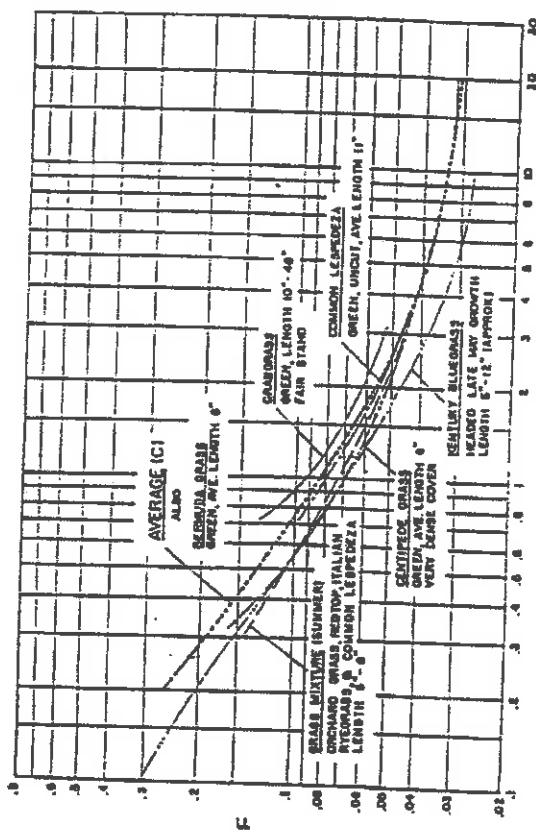
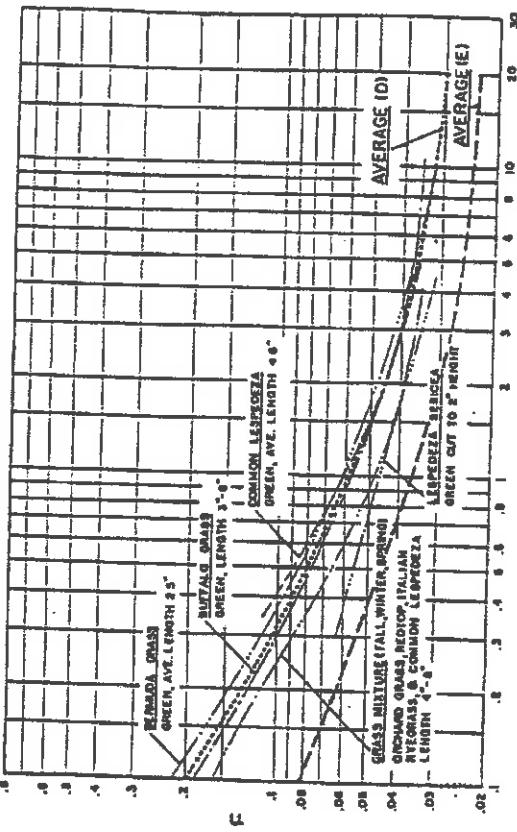


Fig. 7-14. Experimental n-VR curves. (U.S. Soil Conservation Service.)



(c) Curves for C or moderate vegetal retardance.



(d) Curves for D or low vegetal retardance, and an average curve for E or very low vegetal retardance.

Fig. 7-14 (Continued).

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Section 4
Design of Downchutes

APPENDIX A

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Chet L. Safford

000123

(R)

Assumptions

Design of Downchutes - 10-Year Grading Plan
Choctaw Generation Facility - Ash Disposal Site
Subsidiary of Red Hills Generation Facility
Job No. 0874097110
Date Calc 5-Aug-97

PROBLEM STATEMENT:

Design diversion swales for the final 10-year grading plan to accommodate the 25-year, 24-hour storm event.

ASSUMPTIONS:

Assumptions include:

1. Following "Planning & Design Manual for the Control of Erosion, Sediment & Stormwater" by Mississippi Department of Environmental Quality, Mississippi SCS & USSCS.
2. All sediment and erosion control features are designed for the 25-year, 24-hour storm event.
3. Downchute design is based on flows from final grade swales.
4. Design of all downchutes is based on the proposed sequential filling plans (attached).
5. Stability calculations not necessary. Capacity calculations based on "Open-Channel Hydraulics" by Ven Te Chow, Section 7-20. This method compares velocity and is more conservative than depth calculated for stability.
The process for designing for capacity is the following:
 1. Assume a depth y and compute area A and hydraulic radius R (geometric formula).
 2. Compute velocity V by $V = Q/A$, and VR by $VR = V^*R$.
 3. Assume an n of 0.03 for gabions.
 4. Using this n and Manning's formula, compute V .
 5. Compare this V with the V in Step 2. Continue until equal or within 3% error.

6. Add the proper freeboard. In Mississippi, minimum depth at bottom should be 2 ft.

REFERENCES:

1. "Planning & Design Manual for the Control of Erosion, Sediment & Stormwater" by Mississippi Department of Environmental Quality, Mississippi SCS & USSCS.
2. "Open-Channel Hydraulics" by Ven Te Chow, Section 7-20.
3. "Urban Hydrology for Small Watersheds" by the USDA, SCS, Technical Release 55 (TR-55).

APPENDIX A

Capacity



Design of Downchutes - 10-Year Grading Plan

Job No. 0874097110

Calculations by: Gini Perkins

Date Calcs: 18-Jul-96

Date Checked: _____

Checked by: _____

Design for Capacity, assuming the channel is trapezoidal in shape.

Flow for Downchute 1 = flow from swales: b,c,d,e,k,L

Flow:	127.54	cfs
Elev. Delta:	150.00	ft
Length:	600.00	ft
Bottom slope:	0.25	ft/ft
Trap. base width:	6.00	ft
Side slopes (h:v):	3.5	

Trial No.	Downchute Number		DC-1					
	y (ft)	Area (sf)	R	V _{calc}	VR	Mannings n	V _{manning}	Delta V
1	1	9.50	0.715356	13.44	9.61	0.03	19.86	32.4%
2	0.5	3.88	0.401969	32.94	13.24	0.03	13.52	-143.6%
3	0.9	8.24	0.656066	15.50	10.17	0.03	18.75	17.3%
4	0.88	7.99	0.644405	15.97	10.29	0.03	18.52	13.7%
5	0.814	7.20	0.603981	17.72	10.70	0.03	17.74	0.1%

This results in a Swale with 3.5 :1 slope, base of 6.00 ft, depth of 1.2 ft at a velocity of 18.00 fps and a flow of 127.54 cfs. Based on the above velocity, the gabion will be designed at a thick ness of 1.5 feet, or 18 inches.

Flow for Downchute 2 = flow from swales: a,j,f,g,h

Flow:	112.60	cfs
Elev. Delta:	100.00	ft
Length:	400.00	ft
Bottom slope:	0.25	ft/ft
Trap. base width:	6.00	ft
Side slopes (h:v):	3.5	

Trial No.	Downchute Number		DC-2					
	y (ft)	Area (sf)	R	V _{calc}	VR	Mannings n	V _{manning}	Delta V
1	0.25	1.72	0.219788	65.51	14.40	0.03	9.04	-624.7%
2	0.3	2.12	0.25843	53.24	13.76	0.03	10.07	-428.7%
3	0.5	3.88	0.401969	29.06	11.68	0.03	13.52	-114.9%
4	1	9.50	0.715356	11.85	8.48	0.03	19.86	40.3%
5	0.761	6.59	0.571302	17.08	9.76	0.03	17.09	0.1%

This results in a Swale with 3.5 :1 slope, base of 6.00 ft, depth of 1.1 ft at a velocity of 18.00 fps and a flow of 112.60 cfs. Based on the above velocity, the gabion will be designed at a thick ness of 1.5 feet, or 18 inches.

APPENDIX A

(Signature)

Gab-thick

Max Velocity (fps)	Gabion Thickness (feet)
5.9	0.5
11.8	0.75
14.8	1
17.7	1.5

APPENDIX A

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APPENDIX B

CCR LANDFILL WEEKLY INSPECTION CHECKLIST

CCR LANDFILL WEEKLY INSPECTION CHECKLIST

I. GENERAL INFORMATION			
DATE OF INSPECTION	CCR UNIT NAME		
	Plant Red Hills Ash Management Unit		
OWNER	COUNTY	PERMIT NUMBER (If applicable)	
Choctaw Generation Limited Partnership, LLLP	Choctaw	SW0100040462	
MAILING ADDRESS	CITY	STATE	ZIP CODE
2391 Pensacola Road	Ackerman	MS	39735
NAME OF INSPECTOR	TEMPERATURE/WEATHER		
II. INSPECTION CHECKLIST – Check all sections S = Satisfactory, U = Unsatisfactory, or N = Not Applicable			
<u>S U N</u>		<u>Observations</u>	
1.	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/>	Site buffers being maintained	
2.	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/>	Run-on / run-off controls being maintained	
3.	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/>	Groundwater monitoring wells clearly visible and access being maintained	
4.	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/>	CCR being uniformly spread and compacted as required by the operating plan	
5.	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/>	Intermediate cover being maintained	
6.	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/>	Dust being controlled according to the procedures of the Fugitive Dust Control Plan	
7.	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/>	No visual signs of geometry change, ground movement, or slope distress since previous weekly inspection	
III. REMARKS – All blocks marked U require a written explanation and recommended corrective action. Attach additional sheets as necessary			
SIGNATURE OF INSPECTOR		OFFICE	
		Environmental Compliance	