CHAPTER 2. WORKING TOOLS AND SHORT-CUT TECHNIQUES

INTRODUCTION

There have been some indications that the mass routing procedures presented in Chapter 1 require considerable time in structure planning. This is especially true where the procedure is used infrequently and formats for recording must be developed. As a result, Form TX-204 has been prepared to record data for each step of the principal spillway design procedure.

FORMS TO RECORD DATA

The following forms were developed for recording data for each step of the structure design:

1. TX-204-1 - Low Head Drop Inlet Design
2. TX-204-1a - Drop Inlet Structure Design Work Sheet
3. TX-204-1b - Table 1-1, Mass Rainfall and Runoff
4. TX-204-1c - Table 1-2, Storage-Indication Routing
5. TX-204-1d - Table 1-2A or B, Storage-Indication Routing
6. TX-204-1e - Table 1-2C, Storage-Indication Routing Curve Tabulation

STEP PROCEDURE

Each step, as presented in the technical note and provided for on Form TX-204-1a, is discussed.

Step 1: The desired level of emergency spillway protection must be determined. Emergency spillway capacity should be provided to carry the outflow from the 25-year frequency flood at the safe velocity for the site. Thus, detention storage capacity will vary depending upon emergency spillway conditions and the need for downstream flood protection. Conditions at each site should be studied to determine the detention capacity needed to protect downstream areas or to keep frequency and depth of emergency spillway flow within limits which will not cause significant damage. The detention capacity may vary from a small amount to that required to detain a 25-year frequency flood.

Step 2: Space is provided on Form TX-204-1a to compute hydrologic soil cover complex curve number.

Step 3: Space is provided on Form TX-204-1a to record determinations. The Time of Concentration is used in flood-routing runoff from its origin to the site location. For small drainage areas (less than .5 square mile), the routing does not modify the mass runoff curve for runoff at its origin appreciably. Routings from the origin of runoff to the site may be omitted on these small watersheds. If it is omitted, Step 3 is not needed.

Steps 4 & 5: Form TX-204-1b provides space to record this data. Generally, rainfall frequencies do not vary greatly within a county or field office area, thus the rainfall volumes recorded on this form may be applicable to a field office or larger area. Also, the soil cover complex curve numbers often do not vary much within certain areas. Thus, in a field office area there may be need for but a limited number of mass rainfall and runoff computations.

Step 6: In this step the mass runoff tabulated in Step 5 is plotted with respect to duration. In an area where only one rainfall tabulation is applicable for each frequency and the soil cover complex curve numbers are limited to a narrow range, a mass runoff curve for each applicable curve number can be reproduced. This would save some time where a large number of structures are being planned.

Also, Step 6 includes a storage indication routing operation which modifies mass runoff at its origin to inflow at the site. As previously stated, this can be omitted on small watersheds. If the routing is made, Form TX-204-1c should be used. Slightly larger volumes of detention storage will be required if the step is omitted. In some areas where large numbers of structures are planned, routings have been made using two or more different routing time increments. The resulting mass runoff was plotted. Then the mass runoff curve for a particular time of concentration is interpolated from these curves.

Step 7: Form TX-204-1a provides space to compute and tabulate the mass outflow through the principal spillway conduit for a 12-hour period.

The remainder of Form TX-204-1a provides space to record pertinent data concerning the structure and permits an analysis of the structure to determine if flood routings through the structure are needed. Listed items are discussed.

1. Design Head ____ Feet - This is the difference in elevation of the outlet end of the conduit and the emergency spillway.

2. Discharge ____ CFS - Determined for design head using figure 1-5 in Chapter 1 of this note.

3. Riser ____ Inches Diameter - Determined for above discharge on figure 1-5A.

4. Minimum Head on Riser____Feet - This is the head over the riser required to make the structure function at capacity and is read at the intercept of the pipe discharge and line of transition on figure 1-5A. For the structure to discharge most of the flood hydrograph at structure capacity, the difference in elevation of the top of the riser and the emergency spillway crest should be at least three times this amount.

5. Available Head at Barrel Entrance____Feet - This is the difference in elevations of the barrel entrance and the emergency spillway.

6. Required Head at Barrel Entrance for Design Discharge____ Feet - This is read from figure 1-5A on the appropriate orifice flow line for the pipe discharge determined above. It is the minimum difference in elevation of the barrel entrance and the crest of the riser plus the minimum head on riser required to make the structure function at capacity.

7. Emergency Spillway Size: Width____Feet; Depth____Feet; - The width must be at least 30 feet and depth at least 2 feet.

8. Detention Storage____Inches - This is read from the stage capacity curve for the structure site. It is storage capacity from the crest of the riser to the crest of the emergency spillway.

9. Detention Storage Plus Emergency Spillway Storage____Inches - This is the capacity from the crest of the principal spillway to the top of the dam.

10. Storage Required to Contain 25-Year Flood____Inches - This is taken from figure 1-1 for the structure. It is the maximum difference between the mass discharge curve and the 25-year frequency mass inflow curve, or the mass runoff curve if routing is omitted.

11. Emergency Spillway Routing (Not Required) (Attached) - On figure 1-2 for the structure determine the elevation required to contain the 25-year flood. This elevation would be reached if emergency spillway discharge is not considered. If the 25-year flood is stored, the elevation would be that of the emergency spillway. However, if less than the 25-year flood is stored, this elevation would include some depth of flow in the emergency spillway. When the 25-year flood is stored in the detention pool or the depth of flow determined above does not cause erosive velocities in the emergency spillway, the emergency spillway routing is not required. When discharge in the emergency spillway is erosive, a flood routing through the structure is required. Form TX-204-1d should be used to record this routing.

12. Storage Required to Contain 100-Year Flood____Inches - This is obtained from figure 1-1 for the structure. It is the maximum difference between the mass discharge curve and the 100-year frequency mass inflow curve, or the mass runoff curve if routing is omitted.

13. Freeboard Routing (Not Required) (Attached) - on figure 1-2 for the structure determine the elevation required to contain the 100-year flood. This elevation would be reached if emergency spillway discharge is not considered. When the minimum elevation of the top of the dam (emergency spillway crest plus 2 feet) is not exceeded, freeboard routings are not required. When the minimum top of the dam elevation is exceeded, flood routing through the structure is required. Form TX-204-1d should be used to record this routing.

Design and cost estimate data should be recorded on Form TX-204-1. Storage indication routing curve tabulations should be recorded on Form TX-204-1e.

U. S. Department of Agriculture
Soil Conservation Service

TX-ENG-204-1
Rev. 2/83

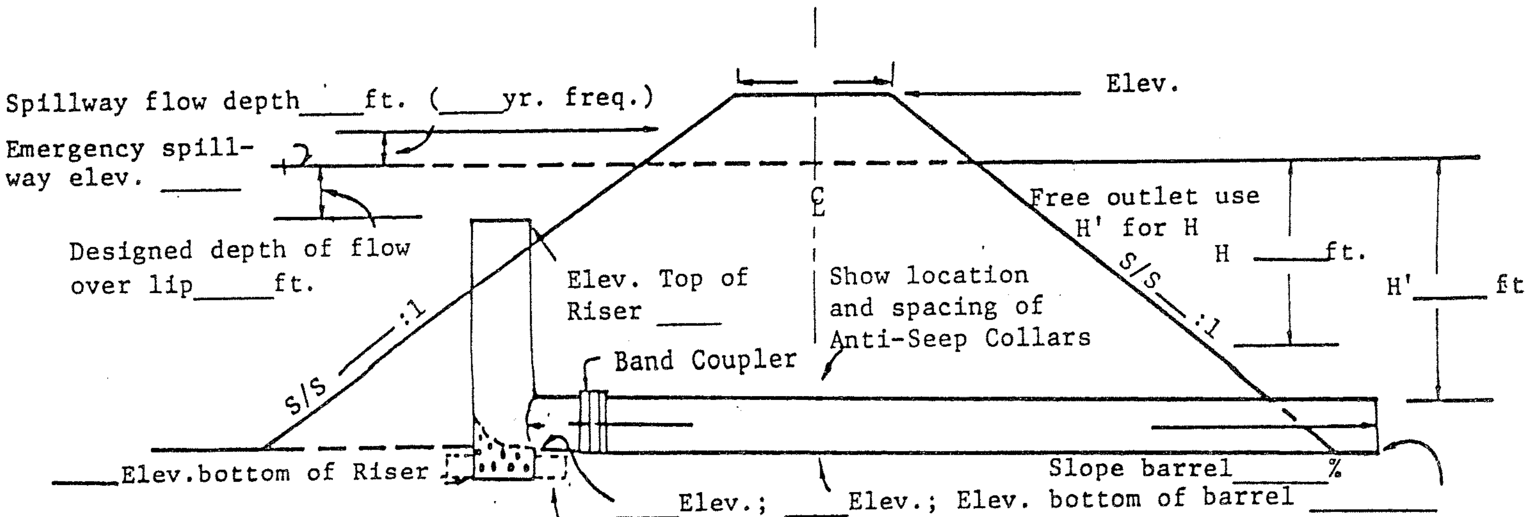
LOW HEAD DROP INLET DESIGN

SWCD ____________ Field Office ____________

Cooperator ____________ Location ____________

____________ County, Texas: Field No. ____________ Structure No. ____________

Designed By ____________ Date ______ Approved By ____________ Date ____________



Footing Dimensions ____ Ft. X ____ Ft. X ____ Ft.

Final Check

Elev. bottom barrel at outlet ____________

Elev. top riser at inlet ____________

Material and component parts meet the plan and design.

Signature ____________

Title ____________ Date ____________

Remarks ____________

____

__________ Anti-seep collars spaced as shown above.

Design drop inlet capacity __________ cfs

Design spillway capacity __________ cfs

(Survey Notes, Computations, and Design Data Attached)

Barrel - Diameter ______ Gage ______
Total Length ______ Ft.

Riser - Diameter ______ Gage ______
Total Length ______ Ft.

(Over)

TX-204-1 Back

COST ESTIMATE OF DROP INLET

Item	Quantity	Unit	Unit Cost	Total
EXCAVATION AND EMBANKMENT:				
Core Trench ______		Cu. Yds.		
Fill ______		Cu. Yds.		
Spillway ______		Cu. Yds.		
SODDING:				
Fill ______		Sq. Yds.		
Spillway ______		Sq. Yds.		
MATERIAL:				
Pipe Barrel, Type ______ Dia. ______		Lin. Ft.		
Pipe Riser, Type ______ Dia. ______		Lin. Ft.		
Cut & Weld "T" Stub ______ Length ______		No.		
Band Couplers 1/ (Standard) (Watertight) ______		No.		
Gauge ______ Width ______ Dia. ______		Lin. Ft.		
Encircling Rods ______ Rods per set ______		Sets		
Anti-Seep Collars ______				
Gauge ______ Projection ______ Dia. ______		No.		
Debris or Safety Guard, Type ______		No.		
Concrete ______		Cu. Yds.		
Installation Cost ______	- - - -	- - - -		
Anti-Vortex Device (if not included w/above guard) ______		No.		

CHECKED BY ______ ESTIMATED TOTAL COST ______

1/ Water tight band couplers recommended.

TX-204-1a (5-67)
Temple, Texas

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

SHEET_____OF_____

DROP INLET STRUCTURE DESIGN WORK SHEET

COOPERATOR: ____

____

TYPE STRUCTURE: ____________________ FIELD NO.: ______ STRUCTURE NO.: ________

LOCATION: ____________________ COUNTY, TEXAS

RESOURCE AREA: ____

DRAINAGE AREA: ______________ ACRES: ______________ SQUARE MILES

PRINCIPAL SPILLWAY DESIGN:

STEP 1:

Desired Level of Emergency Spillway Protection ______________ Years

STEP 2:

Hydrologic Soil Cover Complex Curve Number-Moisture Condition II

COVER	SOIL GROUP	ACRES	%/100	CURVE NO.	(%/100) x (NO.)
TOTALS					

Use Curve Number ______________

STEP 3:

Length of Longest Water Course: ______________ Feet

Total Fall: ______________ Feet

Time of Concentration, T_c: ______________ Hours

STEPS 4 & 5:

See Table 1-1, Sheet ______________

USDA-SCS-FORT WORTH, TEX. 1967

(over)

TX-204-1a Back

STEPS 6 & 7:

See Figures 1-1, Sheet__________ .

$$\frac{(CFS) \times (0.018595)}{D.A. \_\_\_\_\_\_ SQ.\ MI.} = \text{Discharge in inches per 12 hours}$$

(CFS) x (________) = Discharge in inches per 12 hours

PIPE SIZE (Inches Diameter)	LENGTH ______ Ft. HEAD ON PIPE (Feet)	DISCHARGE (CFS)	DISCHARGE (Inches/12 Hours)	DETENTION STORAGE NEEDED (INCHES RUNOFF) Frequency ____Year	25 Year	100 Year

Use__________inch diameter pipe; Design Head________Feet; Discharge__________CFS

Riser: __________ Inches Diameter; Minimum Head on Riser: __________ Feet

Available Head at Barrel Entrance __________ Feet

Required Head at Barrel Entrance for Design Discharge __________ Feet

Emergency Spillway Size: Width: _______ Feet: Depth: __________ Feet

Detention Storage __________ Inches

Detention Storage Plus Emergency Spillway Storage __________ Inches

Storage Required to Contain 25 Year Flood __________ Inches

Emergency Spillway Routing (Not Required) (Attached)

Storage Required to Contain 100 Year Flood __________ Inches

Freeboard Routing (Not Required) (Attached)

DESIGNED BY______________________________DATE ________________

DESIGN CHECKED BY__________________________DATE ________________

DESIGN APPROVED BY_________________________DATE ________________

TX-ENG-204-1b
(Rev. 9/80)

U. S. Department of Agriculture
Soil Conservation Service

Sheet ____ of ____

COOPERATOR ________________ STRUCTURE NO. ________

TABLE 1-1

MASS RAINFALL AND RUNOFF (TYPE II DISTRIBUTION)

COUNTY ________________ CURVE NO. ________

DURATION HOURS	____ YEAR FREQUENCY		____ YEAR FREQUENCY		25 YEAR FREQUENCY		100 YEAR FREQUENCY	
	RAINFALL INCHES	RUNOFF INCHES	RAINFALL INCHES	RUNOFF INCHES	RAINFALL INCHES	RUNOFF INCHES	RAINFALL INCHES	RUNOFF INCHES
0								
4.0								
8.0								
10.0								
11.0								
11.5								
12.0								
12.5								
13.0								
14.0								
16.0								
20.0								
24.0								

U. S. DEPARTMENT OF AGRICULTURE
Soil Conservation Service

TX-ENG-204-1c
Rev. 2/83

Sheet ____ of ____

COOPERATOR ______________________________ STRUCTURE NO. __________

TABLE 1-2

STORAGE-INDICATION ROUTING FOR ____ YEAR FREQUENCY FLOOD

COUNTY ______________________________ CURVE NO. __________

1	2	3	4	5	1	2	3	4	5
Duration	Mass Runoff at Source	Storage Increments Accumulated	Accumulated Storage plus Runoff	Mass Inflow to Site	Duration	Mass Runoff at Source	Storage Increments Accumulated	Accumulated Storage plus Runoff	Mass Inflow to Site
Hours	Inches	Inches	Inches	Inches	Hours	Inches	Inches	Inches	Inches
					Ref. Texas Engr. Tech. Note No. 210-15-TX1				
					Col. 1 = Time increments (Ti)				
					Col. 2 = From Fig. 1-1				
					Col. 3 - Previous Col. 4 minus Col. 5				
					Col. 4 = Col. 2 plus Col. 3				
					Col. 5 = Col.4 x Routing Coeff. (Fig. 1-1A)				

U. S. DEPARTMENT OF AGRICULTURE
Soil Conservation Service

TX-ENG-204-1d
Rev. 2/83

Sheet ____ of ____

COOPERATOR ________________________________ STRUCTURE NO. ______________

TABLE 1-2A or B

STORAGE-INDICATION ROUTING FOR ____ YEAR FREQUENCY FLOOD

COUNTY ________________________________ CURVE NO. ________

1	2	3	4	5	6	7	8	9	10
Duration	Mass Runoff at Source	Storage Increments Accumulated	Storage plus Inflow	Mass Inflow to Site	Incremental Inflow to Site L_v	Storage at beginning of Time Increment S_1	$I_v + S_1$ or $S + 0$	Incremental O_v	Storage at end of Increment S_2
Hours	Inches	Inches	Inches	Inches	Inches	Inches	Inches	Inches	Inches

Ref.: Texas Engineering Technical Note No. 210-15-TX1

Col. 1 = Time increments (Ti)
Col. 2 = From Fig. 1-1
Col. 3 = Prev. Col. 4 - Col. 5
Col. 4 = Col. 2 + Col. 3
Col. 5 = Col. 4 x Routing Coef (Fig 1-1A)
Col. 6 = Difference between values in Col. 5.
Col. 7 = Previous Col. 10
Col. 8 = Col. 6 + Col. 7
Col. 9 = Outflow read from Fig. 1-2A for the S+0 values in Col. 8
Col. 10 = Col. 8 - Col. 9

Sheet ____ of ____

COOPERATOR   STRUCTURE NO.

TABLE 1-2C

STORAGE-INDICATION ROUTING CURVE TABULATION

COUNTY   CURVE NO.

		1	2	3	4	5	6	7
		Stage Feet	Detention Storage S Inches	Discharge Principal Spillway CFS	Discharge Emergency Spillway CFS	Total Discharge CFS	Total Discharge O In/___Hr.	S + O

Ref. Texas Engineering Technical Note No. 210-15-TX1

Col. 1 = From Stage-Capacity Curve ...... Fig. 1-2

Col. 2 = From Stage-Capacity Curve ...... Fig. 1-2

Col. 3, 4 & 5 = From Stage-Capacity Curve ...... Fig. 1-2

Col. 6 = Col. 5 x K. $K = \frac{(0.03719)(Ti)}{(Dr.A.\ Sq.Mi.)(24\ Hrs.)}$

Col. 7 = Col. 2 + Col. 6

Derivation of 0.03719

1 cfs = 1.983 Ac.Ft./24Hrs.
= 23.8 Ac.In./24Hrs.

$\frac{23.8}{640}$ = 0.03719 In./SM/24 Hrs.

CHAPTER 3. ANALYSIS OF SPILLWAY DESIGN

GENERAL

The design of earth spillways for farm ponds and other erosion control structures is covered in Chapter 11 of the Engineering Field Manual for Conservation Practices. Design criteria is based on the assumption that a control section exists, that critical depth and flow occur at the control section, and that the slope in the exit channel is equal to or greater than critical. In many natural spillways, however, these conditions do not exist. This is true where the exit section of the spillway is less than critical and the outflow is permitted to spread on to a well-vegetated area. In most cases, due to the safety factors built into the spillway design criteria, these methods are entirely safe and adequate.

The following is a method of analyzing spillway designs: It should be useful where such an analysis is needed and desirable, especially where the exit channel slope is less than critical. It is not intended that this analysis be used on all spillway designs. It may be used on the larger, more expensive structures, however, where in the opinion of the design engineer, conditions so warrant.

COMPONENT PARTS

In spillways discussed here, there are three component parts - the inlet, the control section, and the exit.

The Exit Channel

The exit channel is that part of the spillway downstream from the control section. In most cases the exit channel will be a natural non-excavated slope. The spillway usually can be selected in such a location that the flow can proceed beyond the control section on to an existing vegetated slope toward the stream's channel below the dam. A short levee, where needed, should be constructed to divert the spillway flow away from the downstream toe of the dam; however, caution should be taken that such a levee is aligned to produce the desired path of the spillway flow. The normal tendency is to exaggerate this diverting and thus "pinch" the cross-section of the spillway.

The Inlet

The inlet is that part of the spillway upstream from the control section. This inlet should be widened to a bottom width of at least fifty percent greater than the designed bottom width at the control section. It should be reasonably short and should be planned for smooth easy curves for alignment. This inlet should have an adverse slope toward the pool of not less than one percent to secure low inlet losses and to insure drainage of the inlet area with a receding lake.

The Control Section

The control section is a section of small length that is level transversely for the entire width of the section. A control section, as such, does not mean physical regulatory metering or control of the discharge, but is a condition of flow in which a definite stage-discharge relationship exists with the discharge occurring at a maximum for that stage.

HYDRAULICS OF THE SPILLWAY

As water flows from a still pond through an inlet and entrance, there is a drop in the water surface that represents an energy or a head. Actually, it is an energy conversion (the potential energy of the stage or standing of the pool converted in part to the kinetic energy of the moving stream). This energy conversion or drop in water surface that is necessary to develop a velocity is appropriately called velocity head (and is always equal to the velocity squared divided by twice the force of gravity, g, as $V^2/2g$). The depth of flow passing a section plus the velocity head is appropriately called the total specific energy at that section as $He = d + V^2/2g$. (This specific energy H_e is frequently, and erroneously, denoted as a depth D).

There are two apparent minimums of the quantity of flows that are entered under this condition. One, when there is no energy conversion there is no velocity head and thus no flow. Secondly, if the entire stage (or depth of the pool above the entrance) were converted to velocity head there would be no depth remaining, as the depth of flow over the entrance and the quantity of flow would be an undefined minimum. Between these two minimum quantities of flow there is a differentiable maximum flow. For a rectangular cross-section of flow, this maximum discharge occurs when 1/3 of the depth above the entrance (the specific energy) is converted to velocity head and 2/3 of this depth is the depth of flow passing the entrance section. This condition of flow that occurs with the discharge at a maximum describes the character of flow called critical flow. The critical flow principle is the basis for the design of all control sections.

Thus, a section selected to be a control section for a certain discharge is not validly designed as a control section until it is determinded whether or not it behaves as a control section. Once flow is underway, resistance to flow is encountered. A section selected to be a control section is not a control section unless the slope in the vicinity below this section is great enough to sustain the flow against friction at a velocity equal to or greater than the velocity accompanying critical flow. This required slope is called critical slope, S_c.

Most earthen control sections are not rectangular, but are trapezoidal in cross-section. However, when the bottom width of a trapezoidal section is great in relation to the depth of flow, the value of the hydraulic radius, r, is very nearly the same depth, d, measured in feet. The assumption of an equivalent rectangular control section with frictionless sidewalls leads to equations that are simplified. Under such assumption the depth of flow, d, and the hydraulic radius, r, are equal.

Design Manipulation

Earth spillways should at all times where possible, be designed for capacity with a section designed and checked as a control section. However, in many areas, natural spillway exit channel slopes are frequently less than critical slope where specific energy values are kept within a desirous 2 or 2-1/2 feet, and the associated flow depths and velocities are reasonably low. It can be reasoned that the essential exit channel slope for desirous hydraulic values can be provided by excavation. However, with the difficulty and planning required for the revegetation of excavated exit channels, it appears under most conditions that further design manipulation with the natural exit slopes is desirable. This means that under some existing conditions, capacity design with a control section may be forsaken in behalf of better design balance. The following nomenclature will apply:

Q - a peak rate of runoff in cfs.
q - a unit discharge in cfs, discharge per foot of width, as Q divided by mean width.
q_c - a unit discharge at critical flow.
d - depth of flow, in feet.
d_c - depth of critical flow, in feet.
V - velocity of flow, in ft/sec.
V_c - velocity of critical flow, in ft/sec.
h - velocity head, also as $V^2/2g$, in feet.
H_e - specific energy, as d + h, in feet.
H_{ec} - specific energy at a control section, as $d_c + 1/2\ d_c$, in feet.
n - Manning's roughness coefficient.
S_e - slope, a natural exit slope.
S_c - critical slope.
α - entrance head loss factor.
L - length of approach channel, in feet.
H_p - a pool head, as depth of flow plus vel. head plus entrance head loss; equal to specific energy plus entrance head loss, as

$$H_p = H_e (1 + \alpha L) \text{ or } H_p = H_{ec} (1 + \alpha L);$$

Figure 3-1 will be used to complete the design of the spillway capacity where critical slope is obtainable and a section of the entrance behaves as a control section. Figure 3-1 and Figure 3-2 will be used where a design as a control section cannot be satisfactorily manipulated.

Example #1.--Conditions given: Q is 700 cfs; a spillway entrance of about 100 feet in width can be satisfactorily cut at one of the embankment abutments. A width greater than this does not appear practical. Solution: For a 100 foot width, the unit q per foot width is 700/100 = 7 cfs. On Figure 3-1 at 7 cfs on the left ordinate extend a straight edge horizontally to the right; at the intersection of d_c curve read downward at base of graph d_c = 1.15'; at the intersection of H_{ec} curve read downward at base of graph H_{ec} = 1.72'; at intersection of V_c

curve read downward at base of graph V_c = 6.1 FPS; with an estimated roughness n = .05, at the intersection of the curve S_c for n = .05 and 7 cfs, read downward at base of graph S_c = 3.5%. Check affirmatively these values, 6.1 FPS velocity is permissible; the exit slope S_e in this vicinity is equal to or greater than 3.5%. If so, this is a valid design and the section is a control section. Now, enter Figure 3-1 at base at 1.72 (this is H_{ec}), proceed vertically and at the intersection of α for n = .05 curve, read horizontally at the right ordinate α = .0052. The estimated length of the approach channel along its centerline is approximately 80 feet. Now $H_p = H_{ec}\ (1 + \alpha L)$ = 1.72 (1 + .0052 x 80) = 1.72 (1 + .416) = 1.72 (1.416) = 2.43', say 2.4'; this is the pool head, the vertical distance from the spillway crest to the pool surface back beyond the influence of spillway flow. This value 2.4' plus freeboard is the design difference in spillway crest and top of embankment.

Example #2.--Conditions given: Permissible velocity at the entrance is about 5 FPS; Q is 600 cfs; n is .04. Solution: At the base of Figure 3-1 at 5 proceed vertically to V_c curve; on a horizontal straight edge at this point proceed to left; at S_c for n = .04 curve read downard S_c = 2.55%; at H_{ec} curve read downward H_{ec} = 1.15'; at d_c curve read downward d_c = .78'; and at the left ordinate read q_c = 3.9 cfs. Spillway width is then Q/q_c or 600/3.9, say 150 feet. S_e must equal or exceed 2.55%. At the base at H_{ec} = 1.15 proceed upward to α for n = .04 curve and read horizontally at right ordinate α = .0057. Now, $H_p = H_{ec}\ (1 + \alpha L)$ and if the approximate length of the approach is 100' , then H_p = 1.15 [1 + (.0057 x 100)] = 1.15 (1.57) = say 1.8 feet; this is the pool head. This value plus freeboard is the design value for the elevation difference in the spillway crest and the embankment topline.

Example #3.--Conditions given: S_e is 2%, Q is 900 cfs, n = .04. Solution: Enter with S_e as being the controlling factor and at 2% slope (S_c) at base of Figure 3-1 proceed vertically to S_c for n = .04 curve and lay a straight edge horizontally at this point. To the right at the intersection of the V_c curve read downward V_c = 7.2 FPS; to the left at the intersection of H_{ec} curve read downward H_{ec} = 2.4' and at the d_c curve read downward d_c = 1.6'; at the left ordinate read q_c = 11.5 cfs. Check these values affirmatively with particular attention to the velocity being permissible. At H_{ec} = 2.4 at base of graph proceed vertically to α for n = .04 curve and horizontally to the right ordinate and read α = .00215. With the estimated approach length, proceed to compute H_p and spillway width as in previous examples.

Example #4.--Condition given: Q = 500 cfs, n = .045, S_e = 1.8%. Solution: A quick glance at Figure 3-1 quickly reveals that some balancing of design is in order. Entering at the base of Figure 3-1 at 1.8% slope as S_c and vertically to S_c for n = .045, interpolated between .04 and .05, quickly shows that in order to have a control section with such a slope and roughness, the unit discharge q is going to have to be in the neighborhood of 60 cfs, the critical depth around 4.8', the specific energy H_{ec} about 7.2' and the critical velocity about 12.5 FPS. Now this, it is readily seen, is a condition where design must be changed. The conditions given are not uncommon. The common mistake is to forget about the exit slope and design a control section with conservative values and never check to see if it behaves as a control section, such procedure has no design validity whatsoever. Proceeding with this solution, the maximum practical width that an entrance can be cut should be considered at this point as an important feature. Suppose, for given values, this is 100 feet. The unit discharge then would be 500/100 or 5 cfs. On Figure 3-1 at 5 cfs (as in previous examples) find d_c = .92'; H_{ec} = 1.38; V_c = 5.4 FPS; and at n = .045, S_c = 3.1%. But S_e, the existing exit slope is only 1.8%, so the above values do not perform as a control section. Obviously, a discharge of 5 cfs per foot width through this 100 ft. width will discharge 500 cfs, but the flow will not be critical, rather subcritical. If this be the case, the designer should have an indication of what alterations occur in depth, velocity, and energy, since the section is not a control section. The ratio of the exit slope existing, S_e, to what would be critical slope, S_c, for the above values is S_e/S_c or 1.8/3.1 which is .62. On Figure 3-2 at the base ordinate are values of S_e/S_c. At the value .62 at the base of this Figure 3-2 proceed vertically and at the intersection of the velocity curve read on the left ordinate .87; at the depth curve read at the left .15; and at the energy curve read .02. This means, that with the flow passing at subcritical, the velocity is .87 V_c; the depth is 1.15 d_c; and the specific energy is 1.02 H_{ec}. These are V = 4.7 FPS; d = 1.06'; and H_e = 1.41'. What has happened is that the depth has increased above the critical depth, the velocity has decreased below the critical velocity, the velocity head has decreased also, and the specific energy H_e has increased above the specific energy as a control section H_{ec}, but only slightly in this example. The velocity head curve has no direct application in solution, it is plotted simply for the reason of showing its relationship to velocity, depth, and specific energy. A study of Figure 3-2 will reveal that discharge that does not reach the critical because of a slope limitation has a definite alteration pattern from the critical, for <u>any one discharge</u>, <u>roughness and section</u>. Discharges at subcritical but in the vicinity of the critical have appreciable changes in depth and velocity head with only minor changes in total specific energy. To complete this solution for the pool head, enter Figure 3-1 with H_e at 1.41 (just as though it were H_{ec}) and solve for alpha as in previous examples; $H_p = H_e (1 + \alpha L)$.

Example #5.--Condition given: Q = 600 cfs, n = .04, S_e = 1.8%. Solution: On Figure 3-1 entering at S_c = 1.8 and going vertically to S_c for n = .04 curve and horizontally to V_c curve shows that the minimum velocity for a control section on this slope with this roughness is 8.4 FPS. Consider now a permissible velocity at the entrance as being an important feature, say this is 5 FPS. It then follows that if the flow is going to be passed on this 1.8% slope with velocity held to a maximum of 5 FPS, the flow is going to have to be subcritical. The procedure now is to select a trial discharge and check it out. Try 6 cfs on Figure 3-1; V_c is 5.8 FPS and S_c is 2.3%. S_e/S_c is 1.8/2.3 or .78. On Figure 3-2 at S_e/S_c = .78 and at the velocity curve read to left .93. Now .93 times 5.8 FPS is 5.4 FPS, hardly down to 5 FPS which was set up as permissible. Try a lower unit discharge, say 5 cfs on Figure 3-1. V_c is 5.4 and S_c is 2.4%. S_e/S_c is 1.8/2.4 = .75. At the base of Figure 3-2 at .75 to the velocity curve and at the left read .92. Now .92 x 5.4 is 4.95 FPS; less than 5 FPS which is permissible. Thus, a unit discharge of 5 cfs will satisfy the given conditions. While bracketing this unit discharge, attention should be given to what is happening at the energy curve on Figure 3-2, very little change in H_e over H_{ec} in the vicinity of S_e/S_c of .78 or .75. In this example the H_p may be solved as in previous examples using a unit discharge of about 5.0 or 5.2.

Summary of Spillway Design

Spillway capacities should be designed with a control section, if at all practical (using only Figure 3-1). Where design values become excessively great, because of exit slope limitations or other reasons, consideration should be given to passing the flow at subcritical. Flow at or in the near vicinity of critical is unstable flow. Consequently, for design of channels where flow is of long duration (such as irrigation or drainage works), the flow should not be designed to operate at or very close to critical; however, on such things as pond spillways (designed on a peak of short duration) this unstableness is not considered so important.

Realistic estimate of roughness and permissible velocities should be made (Reference to SCS-TP-61, or good general texts).

On Figure 3-1, other values of n on the S_c or alpha curves may be interpolated or extrapolated horizontally (logarithmically) on the graph. Additional lines would serve only to clutter the appearance of the graph.

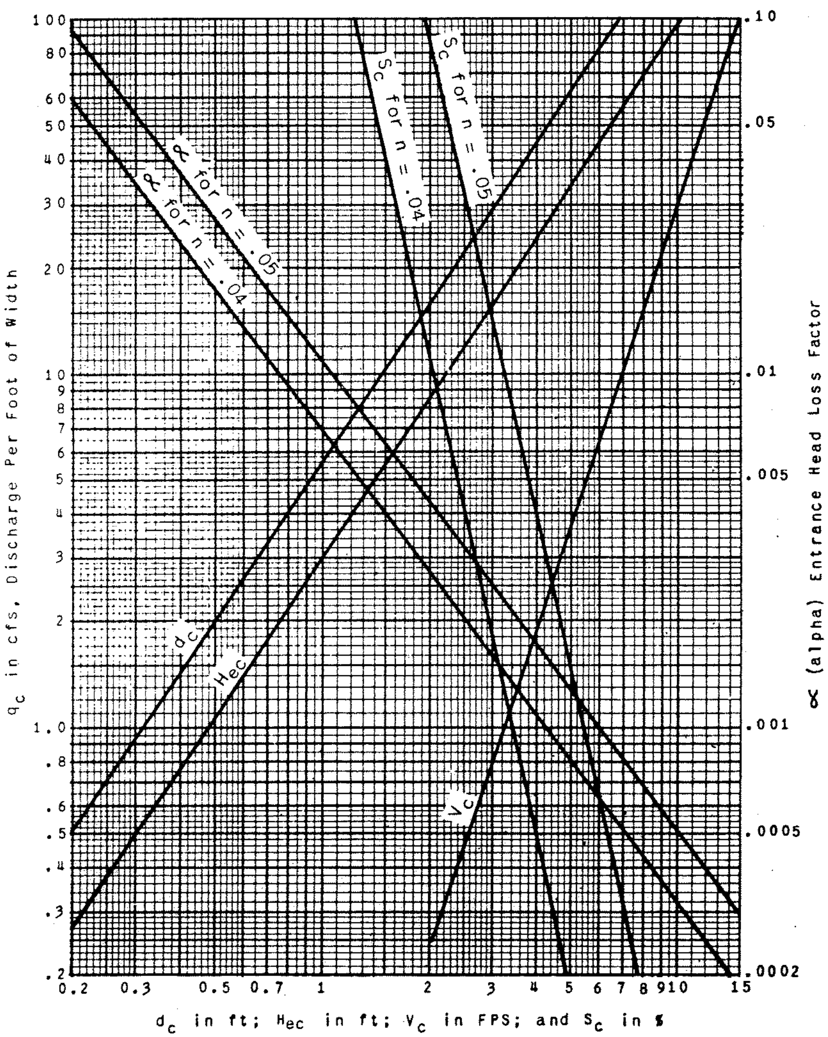
How long a distance from a control section down the exit channel should critical slope exist or be exceeded in order to have a control section? This is a good question. It should be remembered that on natural unexcavated exit slopes there is no lateral confinement of the cross-section of flow beyond the entrance and there is, thus, a natural flow transition or "spread" particular to each site. However, at present, it is considered wise to consider the critical slope beyond the control section for a minimum distance of at least 50 to 100 feet.

The curves on Figure 3-2 are not extended to their limits (zero for vel. head and velocity curves and infinity for depth and energy curves) because the line slopes become so great in this range that values cannot be accurately selected from the curves. Too, when S_e/S_c values for any one section become less than about .10, the discharge of such a section is so far from its maximum (critical) that consideration should be given to selecting an alternate spillway site.

CHART FOR ESTIMATING HYDRAULIC DIMENSIONS AND VALUES OF FARM POND SPILLWAYS

Figure 3-1

Plot of: q_c versus d_c;
q_c versus H_{ec}; q_c versus V_c;
q_c versus S_c; and α versus H_{ec}



U.S. Department of Agriculture, Soil Conservation Service
4-16178 REV. 3-68

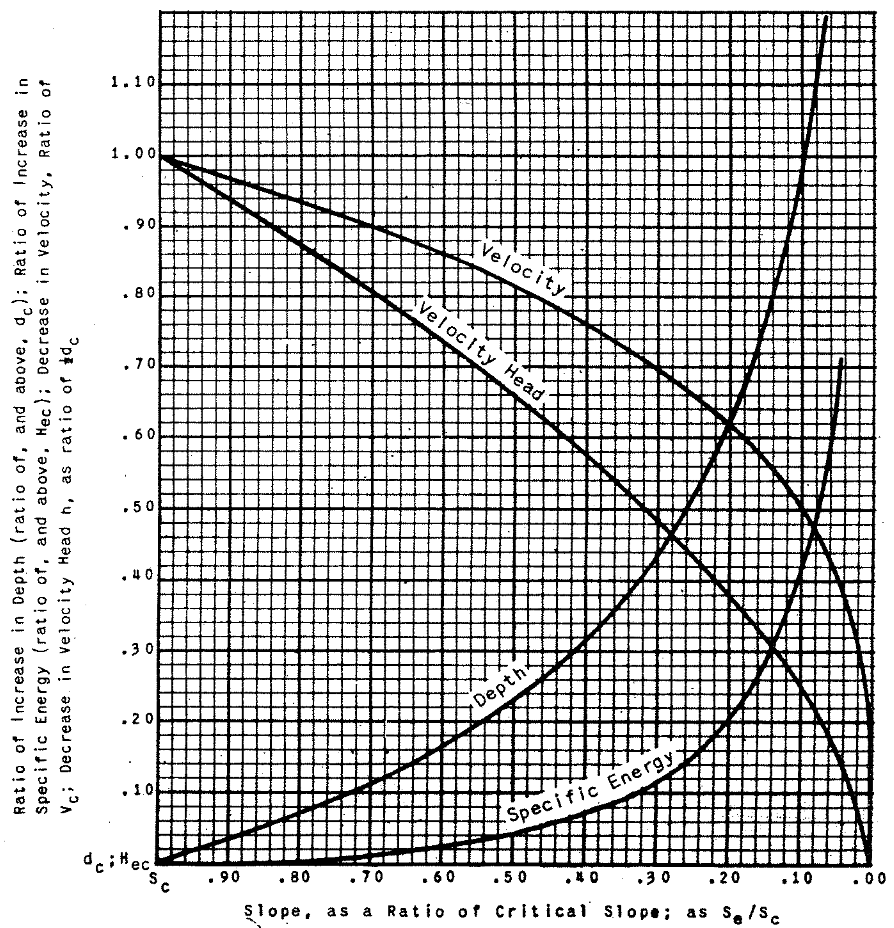
Figure 3-2

For any one unit discharge q (discharge per foot of width) and its associated critical depth d_c, critical Velocity V_c, specific energy H_{ec}, roughness n and critical Slope S_c;

THIS IS A PLOT OF:

d/d_c Vs. S_e/S_c (Depth Curve) V/V_c Vs. S_e/S_c (Velocity Curve)
H_e/H_{ec} Vs. S_e/S_c (Specific Energy Curve) $h/\frac{1}{2}d_c$ Vs. S_e/S_c (Vel. Head Curve)

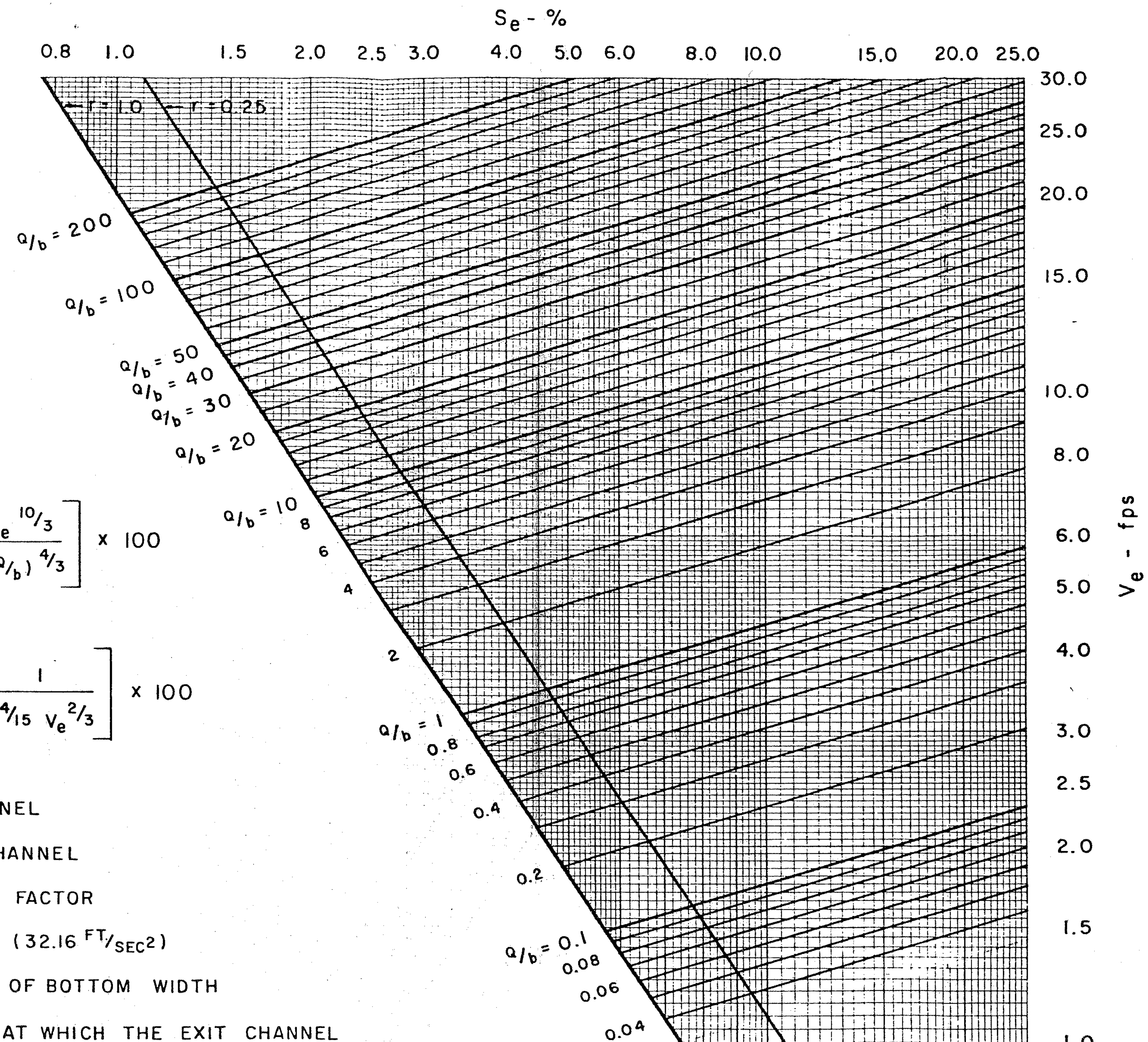
WHERE: S_e is the available natural spillway slope; d is depth, subcritical; H_e is specific energy, subcritical, as d + h; V is velocity, subcritical; h is Velocity head, subcritical; $\frac{1}{2}d_c$ is Velocity head, critical.



Revised 10-23-58 3-29-58 4-N-12,263

UD METHOD: EMERGENCY SPILLWAY HYDRAULICS, VELOCITY CHARTS

n = 0.040



$$\% S_e = \left[\frac{n^2}{1.486^2}\right] \times \left[\frac{V_e^{10/3}}{(Q/b)^{4/3}}\right] \times 100$$

$$\% S_e = \left[\frac{g^{4/3}\, n^2}{1.486^2}\right] \times \left[\frac{1}{r^{4/15}\, V_e^{2/3}}\right] \times 100$$

S_e = SLOPE OF EXIT CHANNEL

V_e = VELOCITY IN EXIT CHANNEL

n = MANNING'S FRICTION FACTOR

g = GRAVITY CONSTANT ($32.16\ ^{FT}/_{SEC^2}$)

Q/b = DISCHARGE PER FOOT OF BOTTOM WIDTH

r = RATIO OF DISCHARGE AT WHICH THE EXIT CHANNEL SLOPE IS EQUAL TO THE CRITICAL SLOPE.

REFERENCE

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
Prepared By
REGIONAL TECHNICAL SERVICE CENTER

STANDARD DWG. NO.
ES - 600
SHEET 1 OF 2
DATE 1-66