

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.

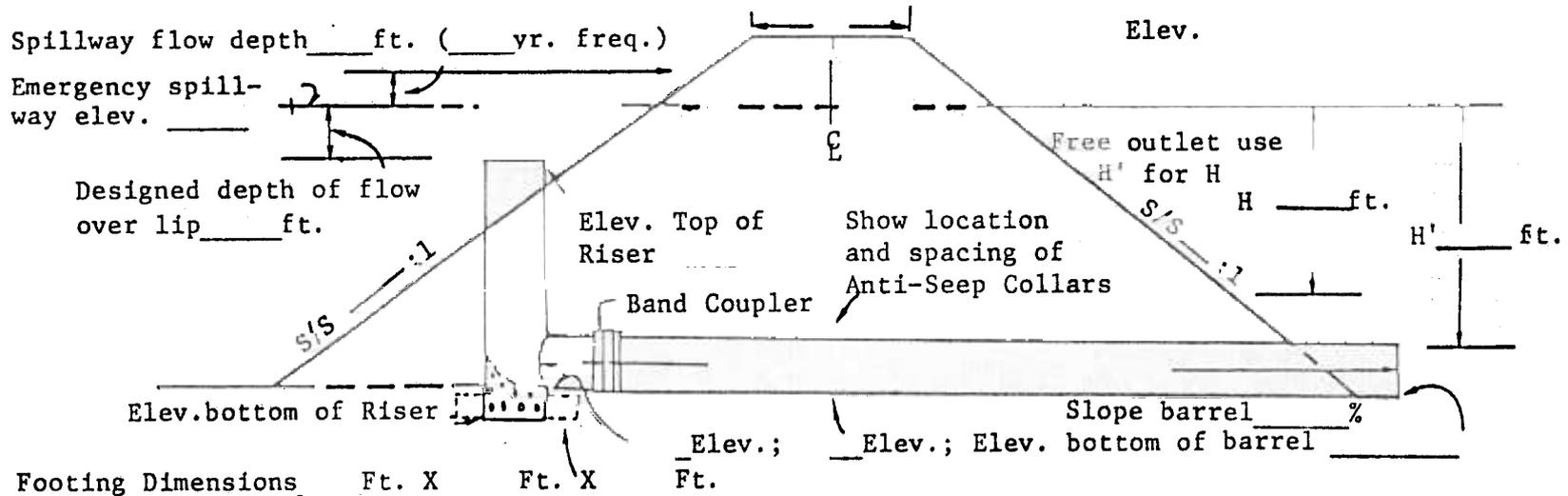
4. Minimum Head on Riser \_\_\_\_\_ Feet - This is the head over the riser required to make the structure function at capacity and is read at the intercept of the pipe discharge and line of transition on figure 1-5A. For the structure to discharge most of the flood hydrograph at structure capacity, the 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.

### LOW HEAD DROP INLET DESIGN

SWCD \_\_\_\_\_ Field Office \_\_\_\_\_  
 Cooperator \_\_\_\_\_ Location \_\_\_\_\_  
 \_\_\_\_\_ County, Texas: Field No. \_\_\_\_\_ Structure No. \_\_\_\_\_  
 Designed By \_\_\_\_\_ Date \_\_\_\_\_ Approved By \_\_\_\_\_ Date \_\_\_\_\_



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)

COST ESTIMATE OF DROP INLET

Item	Quantity	Unit	Unit Cost	Total
<b>EXCAVATION AND EMBANKMENT:</b>				
Core Trench _____		Cu. Yds.		
FI _____		Cu. Yds.		
Spillway _____		Cu. Yds.		
<b>SODDING:</b>				
Fill _____		Sq. Yds.		
Spillway _____		Sq. Yds.		
<b>MATERIAL:</b>				
Pipe Barrel, Type _____ Dia. _____		Lin. Ft.		
Pipe Riser, Type _____ Dia. _____		Lin. Ft.		
Cut & Weld "T" Stub _____ Length _____		No.		
Band Couplers <sup>1</sup> (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 \_\_\_\_\_

<sup>1</sup>/ Water tight band couplers recommended.

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 I

<u>COVER</u>	<u>SOIL GROUP</u>	<u>ACRES</u>	<u>%/100</u>	<u>CURVE NO.</u>	<u>(%/100) x (NO.)</u>
_____	_____	_____	_____	_____	_____
_____	_____	_____	_____	_____	_____
_____	_____	_____	_____	_____	_____

TOTALS

Use Curve Number

STEP 3:

Length of Longest Water Course \_\_\_\_\_ Feet

Total Fall \_\_\_\_\_ Feet

Time of Concentration, T, \_\_\_\_\_ Hours

STEPS 4 & 5

See Table Sheet \_\_\_\_\_

STEPS 6 & 7:

See Figures Sheet \_\_\_\_\_

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

$$(CFS) \times \text{LENGTH} = \text{Discharge in inches per 12 hours}$$

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

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Use \_\_\_\_\_ inch diameter pipe Design Head \_\_\_\_\_ Feet; Discharge \_\_\_\_\_ CFS

Riser \_\_\_\_\_ Inches Diameter; Minimum Head on Riser \_\_\_\_\_

Available Head at Barrel Entrance \_\_\_\_\_

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 \_\_\_\_\_









## 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  $H_e = d + v^2/2g$ . (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 determined 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 side-walls 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 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.  
 $\alpha$  - 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

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

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  $\omega$  for  $n = .05$  curve, read horizontally at the right ordinate  $\omega = .0052$ . The estimated length of the approach channel along its centerline is approximately 80 feet. Now  $H_p = H_{ec} (1 + \omega L) = 1.72 (1 + .0052 \times 80) = 1.72 (1 + .416) = 1.72 (1.416) = 2.43'$ , 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 downward  $S_C = 2.55\%$ ; 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  $\omega$  for  $n = .04$  curve and read horizontally at right ordinate  $\omega = .0057$ . Now,  $H_p = H_{ec} (1 + \omega L)$  and if the approximate length of the approach is 100', then  $H_p = 1.15 [1 + (.0057 \times 100)] = 1.15 (1.57) =$  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  $\omega$  for  $n = .04$  curve and horizontally to the right ordinate and read  $\omega = .00215$ . 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 any one discharge, roughness and section. 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