the adequacy of the spillway, particular note should be made of these two items during the site investigation.

A careful study of each site should be made to determine:

1. The volume of storm runoff, erosion and sediment yield, and conservation measures that might be applied to improve the watershed conditions.

2. Possible dam sites, the suitability of foundation conditions, availability of suitable fill materials, available storage capacity, and spillway conditions.

This information will indicate whether or not an earth dam erosion control structure can be economically installed to meet the requirements of the individual job.

The location of an earth dam, designed to protect against head erosion, must be such that the dam will have influence far enough upstream to submerge the overfall(s). To obtain best results, the crest of the principal spillway should be placed level with the crest of the overfall to be protected. Sufficient detention storage or principal spillway capacity should be provided to permit passage of the 25-year frequency flood at a safe velocity in the emergency spillway. Under such conditions the reservoir fills with water and the overfall is submerged, controlling erosion at this point. When it is not practical to set the crest of the principal spillway level with the overfall, the projected grade from the crest of the principal spillway to the crest of the overfall should not exceed the silting grade. The silting grade is dependent on soil conditions and can be determined best by investigating the grades of existing gullies in the area that appear to be stable. As a general rule, the silting grade should be kept below 0.5% to insure that the overfall eventually will disappear.

A satisfactory emergency spillway must be available at the site to assure stability of the structure. Unstable spillways are the chief cause of failure of this type of erosion control structure. The main factors causing vegetated spillway failures are too frequent operation and continual flow of a small stream of water through the spillway. The latter causes the soil to become saturated and the vegetative cover loses its binding qualities. Scouring occurs and small channels are formed which increase in size whenever subjected to either trickle or flash flow. Therefore, in selecting sites for this type of structure, every effort should be made to obtain the best spillway possible and to consider providing sufficient detention storage or principal spillway capacity, based on the quality of the spillway, to control frequency of operation and to eliminate prolonged trickle flow.

Use should be made of any existing natural depression adequate in size to pass the designed outflow into an adjacent drainageway, and which is protected against erosion by native vegetation. It is sometimes possible to take the emergency spillway discharge away from the gully, or problem area, by spilling it over a small divide or ridge onto a well vegetated area. In this case the spill area should be thoroughly examined to make sure that the additional water will not create other erosion problems, or be diverted onto neighboring property. Natural vegetated spillways and natural exit channels are desirable, especially in semi-arid areas where difficulty is encountered in establishing adequate protective vegetation. Natural spillways should have a broad and flat cross section so as to distribute the flow over as wide as an area as possible.

Some sites will require all parts of the emergency spillway(s) to be excavated. In such cases the spillway should be so located as to hold excavation to a minimum, and constructed to designed dimensions and grades which provide for uniform distribution of flow throughout its entire length. The grade of the exit channel should merge with the stable downstream grade of the draw or gully and there should be no abrupt turns. The excavated spillway should be vegetated with the most erosion-resistant grass adapted to the site, unless it is constructed in a nonerodible material.

In selecting sites for this type of structure, consideration also should be given to the stability of the draw or gully below the dam. Since the spillway discharge usually is returned to the draw or gully immediately downstream from the dam, the channel gradient must be stable if the structure is to be effective. Stability of grade is dependent on soil type, extent of vegetation, velocity of flow, and frequency and extent of flow. As a general rule the velocity is the best criterion for ascertaining the stability of grade. Examination of existing channels in the area also will help to determine stable grades. SCS TP-61, "Handbook of Channel Design," gives information on safe velocities for earth and vegetal lined channels. It should be kept in mind that slightly higher velocities are permissible for intermittent spillway flows than those permitted for continuous or prolonged flows. If the grade is not stable at the structure site, the principal spillway outlet can be recessed below the gully floor to the intersection of the projection of a stable grade upstream from the point where a stable grade exists. Under such a plan, erosion will occur in the channel to a point where a stable grade is reached. The other alternative is to construct one or more structures downstream to bring the grade within limits.

Foundation conditions should be suitable for embankment construction and suitable fill material should be available within an economical range of the site. Investigation and selection of foundation and fill material, as a minimum requirement, should be in accordance with existing procedures given in the Engineering Field Manual for Conservation Practices. Reservoir seepage is not a major concern in this type of structure except where stock water storage is involved, or where the seepage might affect the stability of the dam.

Selection of a site for an earth dam erosion control structure essentially involves checking the site to determine if the structure will meet the purpose of the practice and be more economical than other types. If it meets these criteria, it should be designed to meet the particular needs.

Design---The adequate design of an earth dam erosion control structure requires the gathering and use of specific data. The scope and detail of this data depends upon the size and cost of the structure involved. Some of the data may be secured from aerial photos, published contour maps, or by reconnaissance survey. Some of the information will have to be secured by engineering surveys.

1. Design surveys---The following data should be secured and recorded:

   a. The watershed boundary should be shown on a map together with the following information:

      (1) The size in acres.
      (2) The percentage of each land use, amounts of applied land treatment or practices and the condition of the vegetative cover. This information is needed to determine the proper hydrologic soil-cover complex curve number.
      (3) The predominant soil types, to determine the hydrologic soil groupings.
      (4) The length of the longest watercourse from the proposed site to the ridge in feet.
      (5) The difference in elevation between the proposed site and the farthermost point, in feet, but omit appreciable drops due to gully overfalls, waterfalls, etc.

   b. Profile and cross sections of the gully or draw and topography of the proposed site. The following details should be obtained:

      (1) Profile upstream from the probable dam site far enough to include the entire area of the gully to be influenced by the dam and below the proposed site to the point where a stable grade is reached. The profile should be plotted on cross section paper using a horizontal scale of not more than 200 feet to the inch and a vertical scale of not more than 5 feet to the inch. The profile is needed to determine the elevation of the dam components, to make any needed changes in the proposed dam location, and to help determine stability of channel below dam, etc.
      (2) A profile of the proposed centerline of the dam. Extend the elevations well above the anticipated height of the dam. Plot on cross section paper showing elevations of most desirable spillway sites. Where there is a question as to the stability of grade below the structure, cross sections should be taken every 200 to 400 feet as far as necessary to determine the safe allowable grade below the structure based on velocity calculations.

(3) A topographic map of the proposed site, including the entire reservoir and spillway area. The contours should be plotted on two foot intervals, and they should extend up to or beyond the anticipated height of the dam. The contour map is needed to determine the elevations of the dam, emergency spillway, and principal spillway needed to provide the necessary storage, etc. The contour map may be made with a telescopic alidade and plane table or by taking cross sections between distinguishable points on an aerial photo.

(4) All profiles, cross sections, and topographic information should be referenced to a common benchmark.

After the location of the spillway has been definitely established, profile and cross sections of the spillway should be made for design and construction purposes.

2. Design criteria---When the basic data has been secured, this type structure should be designed in accordance with the requirements of the appropriate practice standard and specification (NHCP) and the following criteria:

a. Detention Storage - The volume of detention storage will be contingent upon the level of protection needed and the capacity of the principal spillway. The detention storage capacity and the size of the principal spillway should be proportioned so that the least expensive combination which will provide the desired level of protection can be determined.

When the purpose of the structure is to stabilize a watercourse or gully, the required detention capacity may vary from that required to produce flow in the spillways to that required to store the 25-year flood event. Where little detention storage is required, particular care must be taken to see that the emergency spillway is so located and protected that excess runoff will be discharged into the channel below the dam without the possibility of the development of an overfall which may work up the spillway and destroy the dam; or, the spillway should be extended to empty into a smooth, well protected area or into a nearby stabilized drain.

When the purpose of the structure is to protect downstream land or structures from sediment or debris or to stabilize the area below, detention capacity for no less than a 10-year frequency flood should be provided.

Earth dams with "trickle tubes" installed for the purpose of protecting the emergency spillway from prolonged base flow usually would not have detention storage or principal spillway capacity enough to effect significant reduction in the emergency spillway discharge. These should be designed in accordance with existing criteria for ponds.

b. Sediment Storage - Adequate sediment storage capacity should be provided in this type structure to insure a reasonable useful life for the structure. When the purpose of the structure is to protect downstream land or structures from sediment or debris, capacity to store no less than the 10-year accumulation of sediment should be provided. For the larger, more expensive structures, or where the land or structures being protected below have a high economic value, the period may need to be increased to a 50-year maximum.

   Where the structure is for grade stabilization only and has a low detention storage-drainage area relationship, sediment storage ordinarily will not be considered.

   Sediment rates for the various resource areas based on the farm pond survey may serve as a guide in determining the sediment rates to use for design of the sediment storage capacity. (See Table 1-4, Weighted Average Estimated Gross Erosion Rates.)

c. Principal Spillway - This type structure usually should be equipped with a principal spillway to release the water from the detention storage area. At sites where the detention storage will cover sizeable areas of grass or cropland, the minimum release rate should be based on the period of time the plants can withstand inundation. Where spring flow and/or prolonged surface flow can be expected the release rate should be increased to include both these flows and the detention flow.

   The average release rate needed to drain the detention storage within the interval of the time selected can be determined by the following formula:

$$Q \text{ in cfs} = \frac{\text{Storage in Acre Feet}}{\text{Removal time in days}} \times 0.50417$$

   The average discharge used shall be 80 percent of the capacity of the principal spillway with the head assumed to be at the emergency spillway crest elevation. Since small diameter tubes are particularly susceptible to clogging with trash, no pipe less than 6 inches in diameter should be used. A minimum of 8 inch diameter is preferable.

   The crest of the principal spillway of grade stabilization structures should be set level with the top of the overfall being treated. If it is necessary to set the crest below the elevation of the overfall, the projected grade to the overfall never should exceed the silting grade of the channel. The detention pool will be between the lip of the principal spillway and the crest of the emergency spillway. Where stock water is involved, the necessary storage to provide the livestock needs is provided between the sediment pool and the detention pool.

Pipe, such as asbestos cement, plastic, steel, concrete and corrugated metal, may be used for principal spillway installations. Steel or corrugated metal pipe usually is used for this purpose because of the simplicity of installation.

Principal spillways may be of the drop inlet, hood inlet or inclined tube type. In some cases it may need to be only a horizontal straight pipe. In such cases both ends of the pipe usually are submerged and it functions as a culvert. Discharge capacities of culverts can be determined by the formula $Q = CA \sqrt{2\,gH}$, where H is the difference in elevation between the water surfaces at the inlet and outlet of the pipe. The proper coefficients (c) are given in King's Handbook of Hydraulics, based on the various sizes and lengths of pipe. Table 1-5 contains pertinent data for solving the formula.

The adaptability and design of hood inlets is given in USDA, SCS, Engineering Division, Technical Release Number 3 - "Hood Inlets for Culvert Spillways".

The drop inlet type principal spillway consists of pipe under the embankment with a larger diameter riser at its upper end. The cross sectional area of the riser pipe usually should be at least 1.5 times the cross-sectional area of the barrel in order for the barrel to flow full. The drop inlet type principal spillway should be designed by using the following formulae:

<u>Weir Flow</u> - $Q_W = 3.5\ L\ H_W^{\frac{3}{2}}$

Where L = Circumference of the vertical riser pipe in feet.

H_W - Head on the weir in feet

Orifice Flow - $Q_O = 0.6A\ \sqrt{2gH_O}$

Where A = Cross-sectional area of orifice in sq. ft.

H_O = Head on orifice in ft.

Pipe Flow - $Q_p = A. \sqrt{\frac{2gH_p}{2+f\frac{L}{D}}}$

Where A = Cross-sectional area of barrel in sq. ft.

H_p = Total head on barrel in feet taken from the top of barrel at its outlet (or the water surface if outlet is submerged) to the design point in the detention pool.

f = Friction factor

L = Length of barrel in ft.

D = Diameter of barrel in ft.

FRICTION FACTORS (f) FOR USE WITH PIPE FLOW

Barrel Diameter	Corrug. Metal Pipe	Concrete	Steel/Smooth Pipe
6"	0.1456	.0394	0.0336
8"	0.1323	.0358	0.0305
10"	0.1229	.0332	0.0283
12"	0.1156	.0313	0.0266
15"	0.1074	.0290	
18"	0.101	.0273	
24"	0.092	.0248	
30"	0.085	.0230	
36"	0.080	.0217	
42"	0.076	.0206	
48"	0.073	.0197	
54"	0.070	.0189	
60"	0.0676	.0183	

A number of different flow patterns may exist in a drop inlet structure of this type, depending on the head discharge relationships of the various component parts. In other words, that part of the drop inlet controlling the flow pattern may be a weir at the lip of the riser, an orifice at the lip of the riser, or at the barrel entrance, a section of the riser acting as a short tube, or the horizontal barrel flowing full and controlling the flow pattern. For all practical purposes it is necessary to consider only the flow patterns as outlined in the example on Figure 1-5A. Orifice control of flow at the riser lip is undesirable. A drop inlet so designed as to permit weir, orifice, and pipe flow at the same head also is undesirable. The ideal condition is to design the structure so that weir flow over the riser lip will control the flow until the head discharge relationship reaches a point where pipe flow in the barrel will assume control of the discharge. The transition between these two types of flow is smooth for both increasing and decreasing heads. This can be done by designing the drop inlet so that pipe flow in the barrel will control the flow at the higher discharge stages, using a vertical riser sufficiently larger than the barrel to maintain weir flow at lower discharge stages. Select also a height of riser which will provide a head adequate at the entrance of the barrel for orifice discharge equal to or greater than the barrel's designed capacity.

The inclined tube type of principal spillway also consists of a pipe under the embankment with a larger diameter riser at its upper end. However, the riser usually is shorter than the drop inlet type riser and the barrel usually is angled down the natural valley slope on a grade greater than critical. Discharge of the principal spillway under this condition is limited by the amount of water that can enter the pipe. By proportioning the riser, the discharge can be controlled by orifice flow at the entrance of the barrel and can be determined by the formula $Q = 0.6A \sqrt{2gH}$,

where H is the head measured from the center of the barrel entrance to the design elevation in the detention pool. The cross-sectional area of the riser should be not less than 1.5 times that of the barrel, and the height of the riser should be proportioned to meet site conditions. Because barrel diameters in this type principal spillway are usually less than 12 inches, one 3 or 4 foot section of concrete pipe makes an acceptable riser. The minimum head above the lip of the riser to produce the desired discharge can be determined by using the above orifice formula and the weir

formula $Q = 3.5\ L\ H^{\frac{3}{2}}$ where H is the head in feet above the lip of the riser, L is the length of the weir in feet, and A (in the orifice formula) is the area of the riser in square feet. The minimum head is that head above the riser lip which will produce a Q, calculated by either the orifice or weir formula, which slightly exceeds the designed Q of the barrel.

d. Emergency Spillway - Where detention storage is provided and considered in the design, vegetated spillways for this type of erosion control structure should be designed to handle the expected 100-year frequency peak discharge. Existing methods, procedures and specifications are given in the Engineering Field Manual for Conservation Practices. USDA-SCS-Engineering Division, Technical Release Number 2 - "Earth Spillways" and Chapter 3 of this note also may be used in the design of these spillways. TR-39 -"Hydraulics of Broad Crested Spillways" provides tools for their design.

e. Embankment - The embankment for this type structure should be designed in accordance with existing procedures and specifications for Ponds.

3. Design Procedure. (Example)

a. Principal Spillway

Step 1: Determine the desired level of protection to be afforded the emergency spillway. This will depend on site conditions and may vary from frequent flows to flows no more frequent than once in 25 years. In this example a 10-year level of protection was desired.

Step 2: Compute hydrologic soil cover complex curve number (Procedure given in National Engineering Handbook (NEH), Hydrology, Section 4.) For this example, Curve No. 88 is used.

Step 3: Determine Time of Concentration "T_c" for drainage area:

(1) Determine 24-hour design storm (Texas Engineering Technical Note No. 210-18-TX5). For example, .6 square mile drainage area with 2% average slope in Houston County area is used. 10-year frequency is 7.50 inches.

(2) Determine runoff for 24-hour storm (6.07 inches).

(3) Read peak discharge for design storm (210-18-TX5) (720 cfs).

(4) Adjust peak discharge for slope. (Reference 210-18-TX5). (720) (1.16) = 835 cfs.

(5) Determine peak discharge per inch runoff per square mile drainage area.

$$\frac{(835 \text{ cfs})}{(0.6 \text{ sq. mi.})\ (6.07 \text{ inches})} = 229 \text{ csm/inch}$$

(6) Read T_c from Figure 1-6 (1.50 hrs.)

Step 4: Obtain 24-hour rainfall for 10-, 25-, and 100-year frequency from 210-18-TX5 and tabulate Type II storm distribution from Table 1-1. (See Table 1-1A - for example Houston County Area used.)

Step 5: Tabulate runoff from rainfall using rainfall-runoff tables on NEH-4 Standard Drawing No. ES-1001, on Table 1-1A.

Step 6: Plot tabulated mass runoff vs. duration (Figure 1-1). This represents runoff at the original source. A storage indication routing operation will be necessary to obtain the inflow to the site location. On small watersheds with short time of concentration the mass runoff curve will not change appreciably. The use of time of concentration as a parameter for flood routing will introduce some adjustment to the hydrograph for watershed size and shape. A simple storage-indication routing can be accomplished by selecting the proper routing coefficient and time increment (T_i). The time increment should be short enough to approximate the runoff mass curve. Generally it should not be more than 1. hour. Figure 1-1A shows the relationship of the routing coefficient to the ratio of T_i to T_c. In the example, Table 1-2, the T_c is 1.5 and T_i is 0.75.

The ratio is 0.5, and the routing coefficient is 0.33 or 1/3. This routing procedure is illustrated in Table 1-2. It is accomplished thus:

(1) List duration as the accumulation of time increments.

(2) Tabulate mass runoff at source for each entry of time from Figure 1-1.

(3) Add mass runoff at source (Col. 2) and storage (Col. 3) to obtain Column 4. Column 3 is Column 4 minus Column 5 for the previous time.

(4) Multiply Column 4 by routing coefficient and tabulate in Column 5.

(5) Plot Column 5 on Figure 1-1. This is the mass inflow to the structure site.

These computations can be programmed on a programmable calculator. The equation is $I_2 = C\ (Q - I_1) + I_1$.

Where I_2 is inflow for current time

I_1 is inflow for previous time

C is routing coefficient from Figure 1-1A

Q is runoff for current time

Step 7: Compute mass outflow through the principal spillway conduit for a 6-, 12- or 24-hour period using head at the principal spillway crest elevation. A range of pipe sizes should be computed to use in proportioning the size of the principal spillway and detention storage capacity. It is recognized that some storage will occur before pipe flow occurs in the principal spillway, but offsetting this is the increased flow caused by the added head from the principal to the emergency spillway. These will balance satisfactorily if the inlet is designed to exclude orifice flow in the principal spillway. Discharges for corrugated metal pipe may be obtained from Figure 1-5. For this example a pipe length of 80 feet and head of 16 feet is used to obtain the discharge for 48", 42", 36", 30", 24", and 18" pipes. These discharges are then expressed in terms of inches of runoff from the .60 square mile drainage area.

$$\frac{(\text{cfs} \times .03719)}{.60 \text{ sq. mi.}} = \text{Discharge in inches per day.}$$

Pipe Size (In. Diameter)	Discharge cfs	Discharge Inches/day
18	21	1.30
24	42	2.60
30	73	4.53
36	112	6.95
42	160	9.93
48	216	13.40

Step 8: Plot mass discharge computed in Step 7 on Figure 1-1. Mass discharge is shown as a straight line tangent to the mass inflow curve. The maximum difference in the 10-year mass inflow and outflow is the storage required to provide a 10-year level of protection to the emergency spillway. In the example the following storage and pipe size combinations are required to provide 10-year protection to the emergency spillway.

Pipe Diameter (Inches)	Detention Storage (Inches Runoff)
18	4.90
24	4.20
30	3.25
36	2.60
42	2.00
48	1.45

The duration of inflow in excess of outflow varies with the size of the pipe. Inflow and outflow curves need not be extended for more than 24 hours, if the principal spillway will discharge the detention storage capacity within 10 days.

Preliminary cost estimates involving comparative cost of different combinations of principal spillway and storage will indicate the cheapest structure that will provide the desired level of protection. The 36" pipe and 2.60 inches of detention storage are used in this example.

- Emergency Spillway

The emergency spillway or spillways shall be designed to carry the structure outflow from the 100-year frequency storm but must have a minimum width of 30 feet and minimum depth of 2 feet. Also, it must pass the 25-year flood at the safe velocity determined for the site. The design can be accomplished by the use of mass routing procedures. Effective storm duration in the design of this type of structure varies with detention storage and spillway capacity. The design through mass routing procedures considers effective storm duration.

The procedure may be accomplished by the use of two storage indication routings. The first is to obtain the 25- and 100-year mass inflow curves to the site, as accomplished for the 10-year flood in the principal spillway design procedure, except the 25- and 100-year frequency storms are used. (Tables 1-2A and 1-2B, Cols. 1-5.)

Storage indication routing through the structure for the 25- and 100-year flood is accomplished in the same manner. The 25-year frequency flood routing is explained.

<u>Step 1</u>: Prepare Stage-Detention Storage curve for site - Figure 1-2.

<u>Step 2</u>: Prepare Stage-Discharge curve for the site. This should include discharge through the principal and emergency spillway. The size of the principal spillway conduit has been determined, but the size of the riser necessary to exclude orifice flow and the rating curve for the spillway must be determined. This can be done by computing the head-discharge relationships for pipe, weir, and orifice flow (Table 1-3). The flow characteristics can be analyzed by plotting the head discharge curves for the various component parts as shown in Figure 1-3 and 1-4.

In examining the performance of the 54" riser it will be noted that the weir flow curve will control the flow to the point where the orifice flow curve crosses it. This occurs at about elevation 99.9'. From this point orifice flow exercises control to about elevation 100.8', where control will jump to pipe flow. This is an undesirable condition, so the 54" riser is too small.

In examining the performance of the 60" riser it will be noted that the weir curve controls flow up to the elevation at which pipe flow assumes control. This is the flow condition desired, so that the 60" riser will be used.

The section of the orifice curve below the weir curve never determines the head-discharge relationship because there is insufficient flow to fill the orifice.

The portion of the pipe flow curve below the weir curve cannot control the flow because there is insufficient water to fill the pipe.

Those portions of the weir and orifice curves to the right of the pipe curve cannot control the flow because they are flooded out by the pipe flow control.

The size of the riser and the stage-discharge relationship for corrugated metal pipe drop inlets can be obtained from Figures 1-5, 1-5A and 1-5B. The stage discharge for pipe flow is plotted from Figure 1-5 or 1-5B. The size of the riser and the part of the stage-discharge curve controlled by weir flow can be taken from Figure 1-5A.

For the structures with barrel diameter less than 12 inches, excluding orifice flow in the design of the drop inlet type principal spillway may not be necessary, but for the structures with larger diameter barrels, the method of analysis as set forth herein should be used to determine adequacy of design.

The width of the emergency spillway must be assumed. It can be rated by procedures given in Chapter 3 of this technical note. For this example a 30-foot wide spillway was used. It is convenient to plot this curve on the same sheet with the Stage-Detention Storage curve, Figure 1-2.

<u>Step 3</u>: Prepare Storage Indication Routing curve tabulations (Table 1-2C). Columns 1 and 2 are taken from the Stage-Capacity curve (Figure 1-2) for the site. Columns 3, 4 and 5 are taken from the Stage-Discharge curve (Figure 1-2) for the site. Column 6 is Column 5 converted to inches for a particular time interval. The time interval should be short enough that the Mass Inflow curve can be reproduced with the points selected. It is convenient to use the same interval as was used to determine the Mass Inflow. For this example 0.75 hour was used:

$$\text{Col. 6} = \frac{\text{Col. 5 x 0.03719 x 0.75 hr.}}{\text{0.60 sq. mi. x 24 hrs.}}$$

Col. 7 is Col. 2 + Col. 6

Step 4: Plot Storage Indication curve from Table 1-2C, Cols. 6 and 7, Figures 1-2A.

Step 5: Storage Indication Routing, Table 1-2A, Cols. 1 to 5 are the routing of runoff to obtain inflow to the site from the 25-year frequency flood. This is discussed under principal spillway design procedure.

Column 6 is the incremental inflow to the site for each time interval and is the difference between values tabulated in Column 5.

Column 7 is the storage at the beginning of the time increment and is the same as Column 10 for the previous time increment.

Column 8 is the sum of Column 6 and Column 7.

Column 9 is the Outflow read from Figure 1-2A for the S + O value in Column 8.

Column 10 is the Storage at the end of the time increment and is obtained by subtracting Column 9 from Column 8.

This routing needs to be carried past the point of maximum storage only. In this example the maximum storage of 3.220 inches occurred at 16.75 hours.

Step 6: Determine depth of flow and emergency spillway peak discharge. From the Stage-Detention Storage curve (Figure 1-2) 3.220 inches of storage occurs at elevation 104.25 or an emergency spillway flow depth of 0.75 feet.

The total peak discharge through the spillways from the Stage-Discharge curve (Figure 1-2) is 163 cfs. Principal Spillway Discharge is 131 cfs and Emergency Spillway Discharge is 32 cfs.

The emergency spillway must pass the 32 cfs at a non-erosive velocity.

When the 100-year flood is routed through the site, the maximum storage of 4.179 inches occurs at 16.00 hours (Table 1-2B). The maximum storage occurs at elevation

105.10 or 1.60 feet depth of flow in the emergency spillway. An emergency spillway 30 feet wide and 1.60 feet deep is required to pass the runoff from the 100-year frequency storm. The emergency spillway should be no less than 30 feet wide and 2 feet deep.

Standard Drawing No. ES-600 is included in Chapter 3 of this technical note and can be used to estimate the exit channel velocity.

Other Types

Other types of erosion control structures include formless concrete chutes, reinforced concrete chutes and drop spillways, masonry drop spillways and rubble masonry overfall dams.

The discharge these structures are expected to convey is determined from hydrologic data and possibly reservoir routings and economic considerations. Generally, storage is not significant and the design capacity is based on peak discharges for a particular frequency flood. When detention capacity is significant it should be considered and the flood routing procedures given for earth dams used to determine the required capacity of the structure.

The flood frequency for which these structures are designed will depend upon the cost of the structure and the effects of flows in excess of design capacity. These types of structures and appurtenant emergency spillways should be designed to pass no less than a 50-year frequency flood without damage to the structure.

Formless concrete chutes should be designed in accordance with Standard Plan #4-E-15030.

Reinforced concrete chute spillways should be designed in accordance with Section 14 - "Chute Spillways" of the National Engineering Handbook.

Drop spillway type structures should be designed in accordance with Section 11 - "Drop Spillways" of the National Engineering Handbook.

Rubble masonry overfall dams should be designed in accordance with Figure 1-6A -"Small Masonry Gravity Dam - for Rock Foundation".

Corrugated metal toewall drop structures should be designed in accordance with Standard Plan #TX-EN-0064.