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3.0 OPEN CHANNEL FLOW

3.1 GENERAL

This section summarizes the practical considerations, technical principles, and criteria necessary for proper design of open channels. The analysis of open channel flow also aids in determining other flow-related concerns, such as, culvert tailwater depths, time of concentration calculations (travel times), and flood elevations.

In a major drainage system, open channels offer significant advantages over closed conduits in regard to cost, flow capacity, flood storage, recreation, and aesthetics. However, open channels require considerable right-of-way and maintenance. However, open channels require considerable right-of-way and maintenance. Careful consideration must be given in the design process to insure that disadvantages are minimized and the benefits maximized. When a design approach not covered in the manual is to be used, it should be reviewed and discussed with the Fort Bend County Drainage District Engineer prior to commencing significant portions of the design effort.

All open channel hydraulic computations are to be computed in HEC-RAS version 3.1.3 (or newer) for newly developed models. Versions of HEC-RAS must be consistent throughout each project. Additional models (other than HEC-RAS) may be used, such as SWMM (or SWMM variants like XP-SWMM), FLO-2D, MIKE 11/MIKE FLOOD, Quick 2.1.0, WSPRO, or others.

Prior approval from the Fort Bend County Drainage District Engineer is required to use any hydraulic model other than HEC-RAS. Modeling which will require a FEMA submittal must use a FEMA approved model.

3.2 OPEN CHANNEL HYDRAULICS – AN OVERVIEW

Flow conditions in an open channel are characterized as steady or unsteady, uniform or varied, subcritical or supercritical.

3.2.1 Steady or Unsteady Flow

Steady flow occurs when the velocity of successive fluid particles at a particular location is the same for successive periods of time. Therefore, the velocity is constant with respect to time ($\frac{dv}{dt}=0$) although it may vary at different locations in the channel. This statement implies that the flow rate Q must also be constant with respect to time. For unsteady flow, the velocity at a particular point is not constant with respect to time. The unsteady option in HEC-RAS can be used when the following situations are present:

1. Rapid changes in discharge and elevation
2. Channel network has slopes less than 5 feet/mile
3. Varying tailwater or backwater effects dominate
4. Flood forecasting for major rivers
5. Large and complex river systems

Prior to using unsteady flow modeling, a coordination meeting should be held with FBCDD staff.

3.2.2 Uniform Flow

Uniform flow occurs when the magnitude and direction of the velocity are not changing ($\frac{dv}{dx}=0$) from location to location in the channel. This statement implies that the depth of flow is also not changing with respect to distance along the channel.

A true state of uniform flow is difficult to obtain under most conditions. Nevertheless, when a channel is sufficiently long and sufficiently unchanging such that the flow depth is not changing (i.e. the channel resistance and gravity forces can be considered to be balanced), the flow may be assumed to be uniform for design purposes.

3.2.3 Varied Flow

When the physical configuration, slope, or surface roughness of a channel changes, or when a disturbance such as a weir or bridge embankment is introduced in the channel, the depth and velocity of the flow will vary along the channel in the vicinity of the disturbance. If the degree of change is small enough that a hydrostatic pressure distribution can be assumed in the flow, then the flow is considered to be gradually varied. If the degree of change is so large that the pressure distribution is no longer hydrostatic at the point of change, then the flow profile is rapidly varied and must be analyzed on a site- specific basis.

3.2.4 Subcritical or Supercritical Flow

The celerity of small gravity waves in a shallow channel is given by the term $(gy)^{1/2}$ where g is the acceleration due to gravity and y is the depth. When the velocity of flow in a channel exceeds this value, the flow is supercritical. When it is less than this value, the flow is subcritical. Hence, the ratio of velocity of flow to celerity $(v/(gy)^{1/2})$, known as Froude Number, is less than 1 for subcritical flow and more than 1 for supercritical flow. Supercritical flow is generally characterized by high velocities and shallow depths, while subcritical flow is characterized by slower velocities and greater depths. The most important distinction between these two states of flow is that the effect of a disturbance in the channel, such as a bridge constriction, cannot be propagated upstream in supercritical flow as it can in subcritical flow. Therefore, subcritical flow is controlled by downstream channel conditions while supercritical flow is controlled by upstream channel conditions.

3.2.5 Critical Depth

When the velocity of flow in a channel is equivalent to the velocity of a gravity wave $(gy)^{1/2}$, critical flow at critical depth exists. Hence, for critical flow, the value of the Froude Number is 1. Flow at or around critical is characterized by instability and should be avoided in channel design except at specific flow transition points such as weirs and sluice gates. Near critical flow, small changes in hydraulic conditions will cause exaggerated changes in depth and velocity.

The critical depth for a given channel configuration and flow rate can be determined using the following procedure:

From open channel hydraulics theory it is given that specific energy ($E = y + v^2/2g$) is at a minimum when the depth is critical. By differentiating the expression for specific energy and further manipulating the resulting equation, the depth (y) becomes critical depth (y_c) and the following expression is obtained for application to a trapezoidal channel:

$$Q/\,(g)^{1/2} = \frac{(b(y_c)+z(y_c)^2)^{3/2}}{(b+2zy_c)^{1/2}} \qquad (3\text{-}1)$$

where b = channel bottom width (ft)

g = acceleration of gravity (32.2 ft/sec^2)

y_c = critical depth (ft)

Q = discharge (cfs)

z = channel side slope where z equals the horizontal displacement for one unit of vertical displacement.

Thus if Q, z, and b are known, the critical depth can be determined by solving Equation 3-1 to find y_c by trial.

3.2.6 Manning's Equation

Manning's equation is an empirical equation which related friction slope, flow depth, channel roughness, and channel cross-sectional shape to flow rate. The friction slope is a measure of the rate at which energy is being lost in the flow to channel resistance. When the channel slope and the friction slope are equal ($S_f = S_o$) the flow is uniform and Manning's equation may be used to determine the depth for uniform flow (normal depth).

Manning's equation is as follows:

$$V = \frac{1.49}{n} R^{2/3} S_f^{1/2} \qquad (3\text{-}2)$$

Or

$$Q = \frac{1.49}{n} AR^{2/3} S_f^{1/2} \qquad (3\text{-}3)$$

where
Q = total discharge (cfs)
V = velocity of flow (ft/sec)
n = Manning's coefficient of roughness
A = cross-sectional area of the flow (ft^2)
R = hydraulic radius of the channel (ft) (flow area/wetted perimeter)
S_f = friction slope, the rate at which energy is lost due to channel resistance

Figure 3-1 provides a nomograph for the solution of Equation 3-2.

Normal depth may be determined by using Equation 3-3. The area (A) and the hydraulic radius (R) are written in terms of the depth (y_o). Knowing the discharge (Q), Manning's "n" value, and the channel slope (S_o), Equation 3-3 can be solved by trial to find normal depth (y_o). Figure 3-2 provides a nomograph for the solution of Equation 3-3 for trapezoidal channels.

3.2.6.1 Manning's "n" Value

Manning's "n" value is an experimentally derived constant which represents the effect of channel roughness in the Manning's equation. Considerable care must be given to the selection of an appropriate "n" value for a given channel due to its significant effect on the character of the flow. Table 3-1 provides a listing of "n" values for various channel conditions. Table 3-2 presents a method to compute a roughness coefficient based on various channel characteristics.

3.3 CHANNEL DESIGN

The proper hydraulic design of a channel is of primary importance to insure that nuisance drainage conditions, flooding, sedimentation and erosion problems to not occur. The following general criteria should be utilized in the design of open channels.

3.3.1 Design Frequency

Open channels shall be designed to contain the runoff from the 100-year frequency 24-hour duration storm within the channel banks while providing one foot of freeboard. In those cases where channel modifications are necessary to control increased flows from proposed development, there should be no increase in water surface elevations in the hydraulic model upstream or downstream of the proposed project for the design frequencies noted in Section 3.3.2. In addition, the channel must be designed to have sufficient freeboard to provide for adequate drainage of lateral storm sewers during the 25-year storm. If the capacity of the existing channel downstream of the project is less than the 100-year design discharge, consideration shall be given for more frequent events to ensure that the frequency of downstream flooding is not increased.

3.3.2 Required Analyses

In order to ensure that the design is adequate, analyses must be performed for the 10-year, 25-year, and 100-year storm events. Lesser events may be required by the Drainage District, depending on local conditions.

The following information must be submitted to the Fort Bend County Drainage District Engineer for the design of open channels.

1. A vicinity map of the site and subject reach. The subject reach is defined as the stretch of channel necessary for any altered flow profile to match the upstream and downstream existing profiles.

2. A detailed map of the area and subject reach with all pertinent physiographic information.

3. A watershed map showing the existing and proposed drainage area boundary along with all subarea delineations and all areas of existing or proposed development.

4. Discharge calculations specifying methodology and key assumptions used including discharges at key locations.

5. Hydraulic calculations specifying methodology used. All assumptions and values of the design parameters must be clearly stated.

6. A profile of the subject reach which includes the following:

   a. All pertinent water surface profiles. This will minimally include the 10-, 25- and 100-year frequency floods for both existing and proposed channel conditions.
   b. All existing and proposed bridge, culvert and pipeline crossings.
   c. The location of all tributary and drainage confluences.
   d. The location of all hydraulic structures (e.g. dams, weirs, drop structures, etc.)

7. A map delineating existing and proposed rights-of-way.

8. Benchmark, elevation, datum and year of adjustment.

9. Typical existing and proposed cross-sections.

10. A soils report which addresses erosion and slope stability.

In zones specified by FEMA as Special Flood Hazard Areas (SFHAs), including Zones A, AE, AO, and AH, additional hydraulic analyses may be required. A floodway analysis is required when changes are proposed to regulatory floodways. No development or other encroachment including fill is allowed in a regulatory floodway which will result in an increase in Base Flood Elevations (BFEs). Any fill in the SFHA that reduces the conveyance capacity of the flooding source must be offset with a hydraulically equivalent mitigation volume to maintain or improve the existing conveyance capacity. Floodplain storage mitigation shall be required for all creeks and streams except the Brazos River. In areas of the SFHA noted as approximate study (Zone A), a 1:1 ratio of cut to fill is required for any fill within that portion of the SFHA noted as Zone A.

3.3.3 Design Considerations

The path taken by an existing, naturally-carved channel often represents the most logical general pathway of flow. For runoff rates associated with undeveloped conditions, the natural channel is largely stable against erosion and is topographically efficient in draining adjacent land. In light of this, it is logical that the engineer should consider taking advantage of naturally carved drainageways when locating and designing open channels.

Although there are numerous channel designs available to the engineer, a judicious design must conform to certain hydraulic, aesthetic, and safety-related standards. In situations where the use of a natural drainage course is infeasible, the engineer must choose between an earthen channel and a lined channel. Grassed channels generally produce lower flow velocities and greater channel storage. They are, in most cases, aesthetically and economically superior to concrete lined waterways. However, grass-lined channels require more right-of-way, are vulnerable to erosion, and must be continually maintained. They can also have problems with side slope stability and/or sediment deposition.

In areas where land values are extremely high, or right-of-way is limited, concrete lined channels may be the design of choice. However, concrete channels can be significantly more expensive. In addition, they tend to move water faster and store less water possibly resulting in higher peak discharges downstream.

3.3.3.1 Optimal Design Flow Characteristics

When designing a channel, the following flow considerations should be addressed:

Velocity – Excessive velocities can cause erosion and may pose a threat to safety. Velocities which are too low may allow sediment deposition and subsequent channel clogging. Table 3-3 provides average and maximum allowable velocities based on the

25-year flow. Minimum velocities are those produced by a channel invert slope of 0.05 percent.

Flow Depth – Deep channels are generally difficult to maintain and can be hazardous. Therefore, design depths should be as shallow as practical while allowing enough depth to accommodate future storm sewer systems.

Freeboard – Since there is no universally accepted rule governing the amount of freeboard required for a channel, selection of a safe amount should be based on confidence in the design discharge estimates, stability of the flow profile and the expected damage from water overflowing channel banks. A minimum value of one foot is required to provide the needed safety. The necessity for additional freeboard shall be considered on the outside channel edge along curves.

3.3.3.2 Optimal Channel Configuration Characteristics

When designing a channel, the following guidelines for the physical configuration of the channel should be observed:

Invert Slope – Slope of the channel invert is generally governed by topography and the energy head required for flow. Since invert slope directly affects channel velocities, channels should have sufficient grade to prevent significant siltation but grades should not be so large as to create erosion problems. In Fort Bend County, the minimum recommended channel invert slope shall be 0.05 percent. Topographic conditions may necessitate a flatter slope in certain areas and prior discussion with the Fort Bend County Drainage District Engineer is suggested. The maximum channel invert shall be limited by maximum flow velocities as given in Table 3-3. Appropriate channel drop structures may be used to limit channel invert slope in steep areas.

Side Slope – In grass-lined channels, normal maximum slope is 4 (horizontal):1 (vertical), which is also the practical limit for mowing equipment. In some areas, side slopes flatter than 4:1 may be necessary due to local soil conditions.

Bottom Width – In grass-lined channels the minimum channel bottom width should be six feet. In concrete-lined channels the minimum bottom width should be eight feet.

Curvature – In general, centerline curves should be as gradual as possible and not have a radius of less than three times the design flow top width unless erosion protection is provided and not less than 100 feet. The maximum curvature for any man-made channel should be 90º.

Manning's "n" Value – The following values of the Manning's roughness coefficient should be used in man-made channels. Alternative values should be discussed with the Fort Bend County Drainage District Engineer.

Channel Cover	"n" Value
Grass-lined	0.04
Concrete-lined	0.015

Confluences – The angle of intersection between the tributary and main channels should be between 15º and 45º. Angles in excess of 45º are permissible but are discouraged. Angles in excess of 90º are not permitted. If the ditch or channel is enlarged, deepened, or new, the Fort Bend County Drainage District Engineer will require the addition of an adequately sized and designed access path across the ditch or channel to allow for access of maintenance equipment. This may include the requirement for pipe(s), stabilized access location or concrete lining. Coordinate with the Fort Bend County Drainage District Engineer for each location.

Transitions – Expansions and contractions should be designed to create minimal flow disturbance and thus minimal energy loss. Transition angles should be less than 12 degrees. When connecting rectangular to trapezoidal channels, a warped or wedge-type transition is recommended.

Location – Channels should be located a sufficient distance away from existing and proposed roads, buildings, and other infrastructure to protect the stability of those items. Where channels cross roadways, adequate slope stabilization and erosion control measures shall be provided.

3.3.4 Minimum Requirements for Channel Design

The minimum requirements for the design of various type channels applicable to Fort Bend County are listed below. Requirements for grass-lined and concrete-lined channels are listed in the following sections.

3.3.4.1 Grass-Lined Channels

The following are minimum requirements to be used in the design of all grass-lined channels:

1. Maximum side slopes shall be 4:1. Slopes flatter than 4:1 may be necessary in some areas due to local soil conditions.

2. Minimum bottom width is six (6) feet.

3. A minimum maintenance berm is required on both sides of the channel of between 15 and 30 feet, depending upon channel size. For top widths of 30 feet or less, 15-foot berms are acceptable, for top widths between 30 and 60 feet, 20-foot berms are required, and for top widths of 60 feet or greater, 30-foot berms are required along both sides of the channel. The elevation of the top of the berm should be at natural ground along the channel reach. See Table 3-4.

4. Backslope interceptor structures are necessary at a maximum of 800 foot intervals to prevent sheet flow over the ditch side slopes.

5. Channel slopes must be revegetated with a perennial grass cover (typically Common Bermuda) immediately after construction to minimize erosion.

6. Flow from roadside ditches must be conveyed to the channel through a roadside ditch interceptor structure and pipe. See ditch interceptor structure and pipe detail, Figure 5-5.

7. Unless waived by the Fort Bend County Drainage District Engineer, a geotechnical investigation and report must be provided.

3.3.4.2 Concrete-Lined Trapezoidal Channels

All partially or fully concrete-lined trapezoidal channels must meet or exceed the following minimum design requirements:

1. All concrete shall be Class A concrete unless noted otherwise.

2. Fully lined cross-sections shall have a minimum bottom width of eight (8) feet.

3. Concrete slope protection placed on 3:1 side slopes shall have a minimum thickness of 5 inches and minimum 6 x 6 x W2.9 x W2.9 welded wire fabric or equivalent reinforcing.

4. Concrete slope protection placed on 2:1 side slopes shall have a minimum thickness of 5-inches and minimum 6 x 6 x W4.0 x W4.0 welded wire fabric or equivalent reinforcing.

5. The minimum side slopes for any concrete lined areas shall be 2:1 and ensure that the escape stairways are included as per Sec. 3.3.4.3 (6).

6. All slope paving shall include a minimum 18-inch toe wall at the top and sides and a 24-inch toe wall across or along the channel bottom for clay soils. In sandy soils, a 36-inch toe wall is recommended across the channel bottom.

7. In instances where the channel is fully lined, backslope drainage structures may not be required. Partially lined channels will require backslope drainage structures.

8. Weep holes shall be used to relieve hydrostatic head behind lined channel sections. The specific type, spacing and construction method for the weep holes will be based on the recommendations of the geotechnical report.

9. Where construction is to take place under conditions of mud and/or standing water, a seal slab of Class C concrete shall be placed in channel bottom prior to placement of concrete slope paving.

10. Control joints shall be provided at approximately twenty-five feet on center. The use of a sealing agent shall be utilized to prevent moisture infiltration.

3.3.4.3 Rectangular Concrete Pilot Channels

In areas where it is necessary to use a vertical-walled rectangular section, the following minimum requirements are to be addressed:

1. All concrete shall be Class A concrete unless noted otherwise.

2. The structural steel design should be based on ASTM A 615, Grade 60 steel.

3. Minimum bottom width shall be eight (8) feet.

4. For bottom widths twelve (12) feet or greater, the channel bottom shall be graded at 1% toward the channel center line.

5. Minimum height of vertical walls shall be four (4) feet. Heights above this shall be in two (2) foot increments. Exceptions will be considered on a case-by-case basis.

6. Escape stairways shall be located at the upstream side of all street crossings, but not to exceed 1,400 feet intervals.

7. For rectangular concrete pilot channels with grass side slopes the top of the vertical wall should be constructed to allow for future placement of concrete slope paving.

8. Weep holes should be used to relieve hydrostatic pressures. The specific type, spacing and construction method for the weep holes will be based on the recommendations of the geotechnical report.

9. Where construction is to take place under conditions of mud and/or standing water, a seal slab of Class C concrete should be placed in channel bottom prior to placement of concrete slope paving.

10. Concrete pilot channels may be used in combination with slope paving or a maintenance shelf. Horizontal paving sections should be analyzed as one way paving capable of supporting maintenance equipment having a concentrated wheel load of up to 1,350 lbs.

11. Control joints shall be provided at approximately twenty-five feet on center. The use of a sealing agent shall be utilized to prevent moisture infiltration.

3.4 EROSION

Erosion protection is necessary to insure that channels maintain their capacity and stability and to avoid excessive transport and deposition of eroded material. The three main parameters which affect erosion are vegetation, soil type and the magnitude of flow velocities and turbulence. In general, silty and sandy soils are the most vulnerable to erosion.

The necessity for erosion protection should be anticipated in the following settings:

1. Areas of channel curvature, especially where the radius of the curve is less than three times the design flow top width.

2. Around bridges where channel transitions create increased flow velocities.

3. When the channel invert is steep enough to cause excessive flow velocities.

4. Along grassed channel side slopes where significant sheet flow enters the channel laterally.
5. At channel confluences.

6. In areas where the soil is particularly prone to erosion.

Sound engineering judgment and experience should be used in locating areas requiring erosion protection. It is often prudent to analyze potential erosion sites following a significant flow event to pinpoint areas of concern.

3.4.1 Minimum Erosion Protection Requirements

Minimum requirements for Fort Bend County are as follows:

Confluences – Figure 3-3 presents the minimum requirements for determining when erosion protection or channel lining are necessary given the angle of the confluence. A healthy cover of grass must also be established above the top edge of the lining extending to the top of the bank. The top edge of the lining shall extend to the 25-year water surface elevation.

Bends – When required, erosion protection must extend along the outside bank of the bend and at least 20 feet downstream. Additional protection on the channel bottom and inside bank, or beyond 20 feet downstream, will be required if maximum allowable velocities are exceeded. See Table 3-3.

Culverts – In areas where outlet velocities exceed five feet per second on to a grass-lined channel, channel lining or an energy dissipation structure will be required.

Outfalls – Erosion protection will be necessary in areas of high turbulence or velocity as typically found at the outfall of backslope drains, roadside ditches, and storm sewers into the main channel. See Figures 3-4, 3-5, 3-6 and 3-7 for typical pipe and storm sewer outfall details. Toe walls at edges of slope paving shall be a minimum of 18” deep.

3.4.2 Structural Erosion Controls

When flow velocities exceed those allowed in Table 3-3 or when soils are deemed excessively erosive by a geotechnical engineer, acceptable structural erosion control shall be provided. The slope protection must extend up the channel bank at least to the elevation of the 25-year flood level.

3.4.2.1 Riprap

The use of riprap is an allowable erosion control measure only in those locations where concrete slope paving is not feasible. Riprap is defined as broken concrete rubble or well-rounded stone. A discussion of riprap design can be found in Hydraulic Design of Flood Control Channels, EM 1110-2-1601, U.S. Department of the Army, Corps of Engineers, July 1970 (or latest version).

3.4.2.2 Concrete Slope Paving

Minimum requirements for partially or fully concrete-lined channels are presented in Section 3.3.

3.4.2.3 Backslope Drainage Systems

The use of backslope drains and swales is required in Fort Bend County. These systems collect overland flow from channel overbanks and other areas not draining to the storm sewer collection system. Their purpose is to prevent excessive overland flow from eroding grass-lined channel side slopes as it enters the channel. Subject to County approval, back-slope drains may not be required in undeveloped or sparsely developed areas.

The design engineer should carefully consider the drainage area to be intercepted by such systems, particularly when the channel passes through large areas of undeveloped acreage where large quantities of naturally occurring sheet flow could overload the backslope swale and drainage system. In these areas, drain spacing and backslope drainage pipe requirements may have to be modified to account for the conditions. Refer to Figure 3-4 for backslope drainage design.

Documentation of drainage area for each backslope drain system, as well as hydraulic pipe and swale sizing calculations, must be provided by the engineer.

General requirements for backslope drains and swales are as follows:

1. Minimum backslope drain pipe shall be 24” in diameter.

2. Maximum spacing is 800 feet (or 400 feet to the swale high point).

3. The drain structure and swale centerline should be six feet inside the channel right-of-way line.

4. Minimum design depth in swale is 0.5 feet.

5. Maximum design depth in swale is 2.0 feet.

6. Minimum gradient for swale invert is 0.2%.

7. Swale should have a maximum side slope of 3:1.

3.4.2.4 Sloped Drops

Sloped drop structures are recommended when the required drop elevation is small, generally 1-4 feet. They tend to be the most economical and topographically versatile means to accomplish a drop. Slope drops should be no steeper than 3:1 and no flatter than 4:1.

Slopes drops shall be constructed of concrete slope paving or of cellular concrete articulated mats. Riprap or an appropriate alternate erosion protection shall be provided upstream and downstream of the drop.

When subcritical flow approaches a drop, depth decreases and velocity increases as the flow nears critical. Accordingly, appropriate erosion protection must be provided sufficiently upstream such that flow velocities are not excessive in any unprotected reach of channel. The minimum recommended distance is 20 feet.

Downstream of the drop, the required length for protection is dependent on the length of the hydraulic jump. As a rough estimate the jump length may be assumed equal to q/2, one-half of the design flow per unit width of channel. The use of riprap or a combination of riprap and concrete slope paving is recommended downstream of the drop to force the jump closer to the drop. A minimum of 20 feet of riprap is required downstream of any slope paving used at a drop structure to help reduce velocities and protect the concrete toe. The minimum recommended apron length is 40 feet.

3.4.2.5 Baffled Chutes

Baffled chutes are used in drainageways when a relatively large change in elevation is necessary. The baffle blocks prevent undue acceleration of the flow as it passes down the chute. Baffled chutes are generally laid out on a 2:1 slope (no steeper) and can be designed to discharge up to 60 cfs per foot of channel width. The lower end of the chute is constructed to below streambed level and backfilled as necessary thereby minimizing degradation or scour of the streambed. No tailwater or stilling basin is required as velocities will remain moderate.

The following simplified step-by-step procedure taken from the U.S. Department of the Interior Bureau of Reclamation publication, Progress Report V – Research Study on Stilling Basins, Energy Dissipators, and Associated Apurtenances, Section 9, Hydraulic Laboratory Report No. Hyd-445, April 28, 1961 (or latest version) and is recommended for the design of baffled chutes. For a more detailed discussion, the engineer is referred to U.S. Department of the Interior Bureau of Reclamation publication, Hydraulic Design of Stilling Basins and Energy Dissipators, Engineering Monograph No. 25.

Step-By-Step Design Procedure:

1. The baffled apron should be designed for the 100-year discharge, Q.

2. The unit discharge q = Q/W may be as high as 60 cubic feet per second per foot of chute width, W. Less severe flow conditions at the base of the chute exist for 35 cubic feet per second and a relatively mild condition occurs for unit discharges of 20 cubic feet per second and less.

3. Entrance velocity, V_1, should be as low as practical. Ideal conditions exist when $V_1 = (gq)^{1/3} - 5$ (See Curve D, Figure 3-8). Flow conditions are not acceptable when $V_1 = (gq)^{1/3}$ (See Curve C, Figure 3-8).

4. The vertical offset between the approach channel floor and the chute is used to create a stilling pool or desirable V_1 and will vary in individual installations; Figure 3-9 shows a typical approach pool. Use a short radius curve to provide a crest on the 2:1 sloping chute. Place the first row of baffle piers close to the top of the chute no more than 12 inches in elevation below the crest.

5. The baffle pier height, H, should be about $0.8D_c$ (see Curve B, Figure 3-8). The critical depth on the rectangular chute is $D_c = (q^2/g)^{1/3}$ (see Curve A, Figure 3-8). Baffle pier height is not a critical dimension but should not be less than recommended. The height may be increased to $0.9\ D_c$.

6. Baffle pier widths and spaces should be equal, preferably about 3/2 H, but not less than H. Other baffle pier dimensions are not critical; suggested cross section is shown in Figure 3-9. Partial blocks, width 1/3 H to 2/3 H, should be placed against the training walls in Rows 1, 3, 5, 7, etc., alternating with spaces of the same width in Rows 2,4, 6, etc.

7. The slope distance between rows of baffle piers should be 2 H, twice the baffle height H. When the baffle height is less than 3 feet, the row spacing may be greater than 2 H but should not exceed 6 feet.

8. The baffle piers are usually constructed with their upstream faces normal to the chute surface; however, piers with vertical faces may be used. Vertical face piers tend to produce more splash and less bed scour, but differences are not significant.

9. Four rows of baffle piers are required to establish full control of the flow, although fewer rows have operated successfully. Additional rows beyond the fourth maintain the control established above, and as many rows may be constructed as is necessary.

The chute should be extended to below the normal downstream channel elevation. At least one row of baffles should be buried in the backfill.

10. The chute training walls should be three times as high as the baffle piers (measured normal to the chute floor) to contain the main flow of water and splash. It is impractical to increase the wall heights to contain all the splash.

11. Erosion protection measures should be placed at the downstream ends of the training walls to prevent eddies from undermining the walls.

3.5 WATER SURFACE PROFILES

The state of flow in a channel is at all times either uniform, gradually varied, or rapidly varied. A different method for determining water surface profiles is applicable to each of these conditions of flow.

3.5.1 Uniform Flow

When a section of channel is sufficiently long and unchanging such that the flow depth is not changing (i.e. the forces of gravity and channel resistance can be considered balanced), then the flow profile can be analyzed, assuming uniform flow. Under these circumstances, the depth, which is constant, can be determined with Manning's equation (see Section 3.2.6).

3.5.2 Gradually Varied Flow

In the majority of channel flow situations, the state of flow is gradually varied. In other words, the depth is gradually changing with longitudinal distance along the channel due to an imbalance between the forces of gravity and channel resistance.

The recommended means for determining flow profiles under these conditions is with the standard step method. The standard step method is an iterative process in which the one-dimensional energy equation is solved to find the water surface elevation at a cross-section. Manning's equation is utilized to determine channel losses due to friction. Losses due to channel non-uniformities are usually calculated with empirical coefficients.

A widely accepted computer model for calculating gradually varied flow profiles is the U.S. Army Corps of Engineers' program HEC-RAS, River Analysis System. The use of the older HEC-2 software program could be considered on a case by case basis. Discuss your hydraulic modeling approach with the Fort Bend County Drainage Engineer prior to beginning the hydraulic analysis.

The HEC-RAS model can readily accommodate modifications in channel design and losses at bridges, culverts, drop structures, and transitions. The program begins computation at a cross-section of known or estimated water surface elevation and proceeds upstream for subcritical flow, and downstream for supercritical flow.

The following general guidelines should be followed with the use of the HEC-RAS program:

1. Cross-sections should be spaced such that the channel configuration between them is largely uniform. In areas where channel properties are rapidly changing, the distance between cross-sections should be appropriately less.

2. The accuracy of the flow profile is largely dependent on a correct determination of the starting water surface elevation, especially in the vicinity of the first cross-section. The best method of determining starting water surface elevation is with a known rating curve or from past backwater studies. The least favorable is the slope-area method, which determines normal depth given the friction slope and discharge. It is important to begin water surface profile analyses a significant distance downstream of the point(s) of interest for subcritical flow and upstream of the point(s) of interest for supercritical flow.

3. Errors can occur with the improper handling of energy losses, thus loss coefficients should be chosen carefully. The engineer should carefully select a particular bridge routing and understand its operation. If the independent hand calculation of a head loss can be accomplished more accurately, it should be input to the program. Proper care should be taken to ascertain that computed losses are reasonable.

3.5.3 Rapidly Varied Flow

When depth changes abruptly over a short distance the flow profile is rapidly varied. Rapidly varied flow is a local phenomenon which occurs in such areas as the contraction beneath a sluice gate, where the channel slope changes from mild to steep, where the flow passes over a weir, and in a hydraulic jump. Determination of the change of the flow profile at such locations must be carried out on a site-specific basis by the engineer.

3.5.4 Energy Losses

Analysis of flow profiles in open channels must include proper consideration of energy losses due to local disturbances such as bridges, drop structures, transitions and confluences. In many cases, such head losses are adequately handled with empirical coefficients. When specific site conditions warrant a more careful analysis, or when a particular program cannot handle local losses, hand calculated losses may be utilized in the flow profile. The following guidelines should be followed for typical sources of non-frictional energy loss.

3.5.4.1 Expansions and Contractions

Losses at transitions are generally expressed in terms of the absolute change in velocity head between downstream and upstream of the transition. The head loss is given by:

$$h_1 = C \frac{(V_2^2 - V_1^2)}{2g} \qquad (3\text{-}4)$$

where

h_1	=	head loss across the transition (ft)
C	=	empirical expansion or contraction coefficient
V_2, V_1	=	average channel velocity (fps) of the downstream and upstream sections, respectively
g	=	acceleration of gravity (32.2 ft/sec^2)

Typical transition loss coefficients for subcritical flow are as follows:

Transition	Coefficient	
	Contraction	Expansion
Gradual or warped	0.1	0.3
Bridge Sections, wedge, Straight-lined	0.3	0.5
Abrupt or square-edged	0.6	0.8

Source: HEC-RAS User Manual.

The above transition loss coefficients are also adequate for general design with supercritical flow; however, the effects of standing waves and other considerations make exact determination of losses in supercritical flow difficult. Therefore, with important transitions, a more detailed analysis may be necessary (see Section 3.6).

3.5.4.2 Bends

The HEC-RAS program does not make allowances for energy losses due to significant bends in the channel. In most cases, losses in channel bends are negligible. However, when the radius of a bend is less than three times the design top width of flow, energy losses due to the bend should be specifically included in the backwater analysis. Any bend loss analysis should be clearly documented in the submitted analysis. Such losses are expressed in terms of the velocity head multiplied by a loss coefficient and may be input to a computer run and can be expressed as:

$$h_L = C_F \frac{V^2}{2g}$$

where	h_L	=	head loss (ft)
	C_F	=	coefficient of resistance
	V	=	average channel velocity (feet per second)
	g	=	acceleration of gravity (32.2 ft/sec^2)

3.5.4.3 Bridges

There are various methods available to compute losses associated with flow through a bridge. Sources of energy loss in bridges include flow resistance, channel transitions, and direct obstructions to the flow such as piers. Each bridge should be examined individually to determine the best approach. The bridge routines found in HEC-RAS are recommended for their versatility and flexibility. Additional information on HEC-RAS analysis of bridges and culverts can be found in the HEC-RAS manuals which are available on-line.

The use of alternative means for computing bridge-related losses is encouraged when the engineer is properly aware of how and why such a strategy is appropriate and its results are reasonable.

3.6 SUPERCRITICAL TRANSITIONS

The design engineer should be aware that if flow through a transition is supercritical, standing waves will be generated and additional freeboard may be necessary to safely contain the flow. For a discussion of the analysis of supercritical flow in transitions, the engineer is referred to the U.S. Army Corps of Engineer's publication Hydraulic Design of Flood Control Channels, EM 1110-2-1601, July, 1970 (or latest version).

3.7 RIGHT-OF-WAY

All new drainage facilities must take into consideration the existing drainage upstream. In addition, new development must provide the ultimate planned right-of-way width based on fully developed watershed conditions. Fully developed conditions mean undetained flows from 100 percent development of the watershed with future impervious conditions being typical to existing development patterns within the area.

The amount of right-of-way required for open channels shall be based on full development of the watershed and is dependent on channel top width and channel type (earthen or lined) as required to accommodate the discharge resulting from the 100-year, 24-hour rainfall event. Adequate area must be set aside for both the channel itself and the adjacent berm required for channel maintenance. Minimum right-of-way requirements for Fort Bend County include the channel from bank to bank plus the maintenance berm areas on both sides and shall be dedicated at the time of platting of the adjacent property. However, if additional right-of-way is required to serve upstream development prior to downstream platting, sufficient right-of-way must be dedicated to accommodate the improved channel and provide adequate maintenance berms. See Table 3-4.

3.8 UTILITY LINE CROSSINGS

Prior to design, the Fort Bend County Drainage District Engineer should be contacted for information pertaining to the ultimate channel cross-section and right-of-way. In addition, County approval must be obtained for all future utility lines crossing Fort Bend County flood control facilities. All manholes required for the utility conduit shall be located outside of the ultimate Fort Bend County right-of-way.

TABLE 3-1

VALUES OF THE MANNING ROUGHNESS COEFFICIENT – “n”

Type of Channel and Description	Minimum	Normal	Maximum
A. Lined or Built-up Channels			
A1. Metal			
a. Smooth steel surface			
1. Unpainted	0.011	0.012	0.014
2. Painted	0.012	0.013	0.017
b. Corrugated	0.021	0.025	0.030
A2. Nonmetal			
a. Cement			
1. Neat, surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
b. Wood			
1. Planed, untreated	0.010	0.012	0.014
2. Planed, creosoted	0.011	0.012	0.015
3. Unplaned	0.011	0.013	0.015
4. Plank with battens	0.012	0.015	0.018
5. Lined with roofing paper	0.010	0.014	0.017
c. Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Finished, with gravel on bottom	0.015	0.017	0.020
4. Unfinished	0.014	0.017	0.020
5. Gunite, good section	0.016	0.019	0.023
6. Gunite, wavy section	0.018	0.022	0.025
7. On good excavated rock	0.017	0.020	--
8. On irregular excavated rock	0.022	0.027	--
d. Concrete bottom float finished with sides of			
1. Dressed stone in mortar	0.015	0.017	0.020
2. Random stone in mortar	0.017	0.020	0.024
3. Cement rubble masonry, plastered	0.016	0.020	0.024
4. Cement rubble masonry	0.020	0.025	0.030
5. Dry rubble or riprap	0.020	0.030	0.035
e. Gravel bottom with sides of			
1. Formed concrete	0.017	0.020	0.025
2. Random stone in mortar	0.020	0.023	0.026
3. Dry rubble or riprap	0.023	0.033	0.036
f. Brick			
1. Glazed	0.011	0.013	0.015
2. In cement mortar	0.012	0.015	0.018
g. Masonry			
1. Cemented rubble	0.017	0.025	0.030
2. Dry rubble	0.023	0.032	0.035
h. Dressed ashlar	0.013	0.015	0.017

TABLE 3-1 (Cont'd)

Type of Channel and Description	Minimum	Normal	Maximum
i. Asphalt			
1. Smooth	0.013	0.013	--
2. Rough	0.016	0.016	--
j. Vegetal lining	0.030	--	0.050
B. Excavated or Dredged			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stone bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline – excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush or banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140
C. Natural Streams			
C1. Minor streams (top width at flood stage <100 ft)			
a. Streams on plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			

TABLE 3-1(Concluded)

Type of Channel and Description	Minimum	Normal	Maximum
1. Bottom: gravels, cobbles, and few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070
C2. Floodplains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated areas			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees, in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Dense willows, summer, straight	0.110	0.150	0.200
2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of sprouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160
C3. Major streams (top width at flood stage >100 ft). The "n" value is less than that for minor streams of similar description because banks offer less effective resistance			
a. Regular section with no boulders or brush	0.025	--	0.060
b. Irregular and rough section	0.035	--	0.100

Source: Open-Channel Hydraulics by Ven Te Chow (1959).

TABLE 3-2

COMPUTATION OF COMPOSITE ROUGHNESS COEFFICIENT

FOR EXCAVATED AND NATURAL CHANNELS

$$N = (n_0 + n_1 + n_2 + n_3 + n_4)\ m$$

	Channel Conditions	Value
Material Involved	Earth	0.020
n_0	Rockcut	0.025
	Fine Gravel	0.024
	Coarse Gravel	0.028
Degree of Irregularity	Smooth	0.000
	Minor	0.005
n_1	Moderate	0.010
	Severe	0.020
Variation of Channel Cross-Section	Gradual	0.000
	Alternating Occasionally	0.005
n_2	Alternating Frequently	0.010-0.015
Relative Effect of Obstructions	Negligible	0.000
	Minor	0.010-0.015
n_3	Appreciable	0.020-0.030
	Severe	0.040-0.060
Vegetation	Low	0.005-0.010
n_4	Medium	0.010-0.025
	High	0.025-0.050
	Very High	0.050-0.100
Degree of Meandering	Minor	1.000
	Appreciable	1.150
m	Severe	1.300

Source: Open-Channel Hydraulics by Ven Te Chow (1959).

TABLE 3-3

ALLOWABLE 25-YEAR VELOCITIES FOR CHANNEL DESIGN

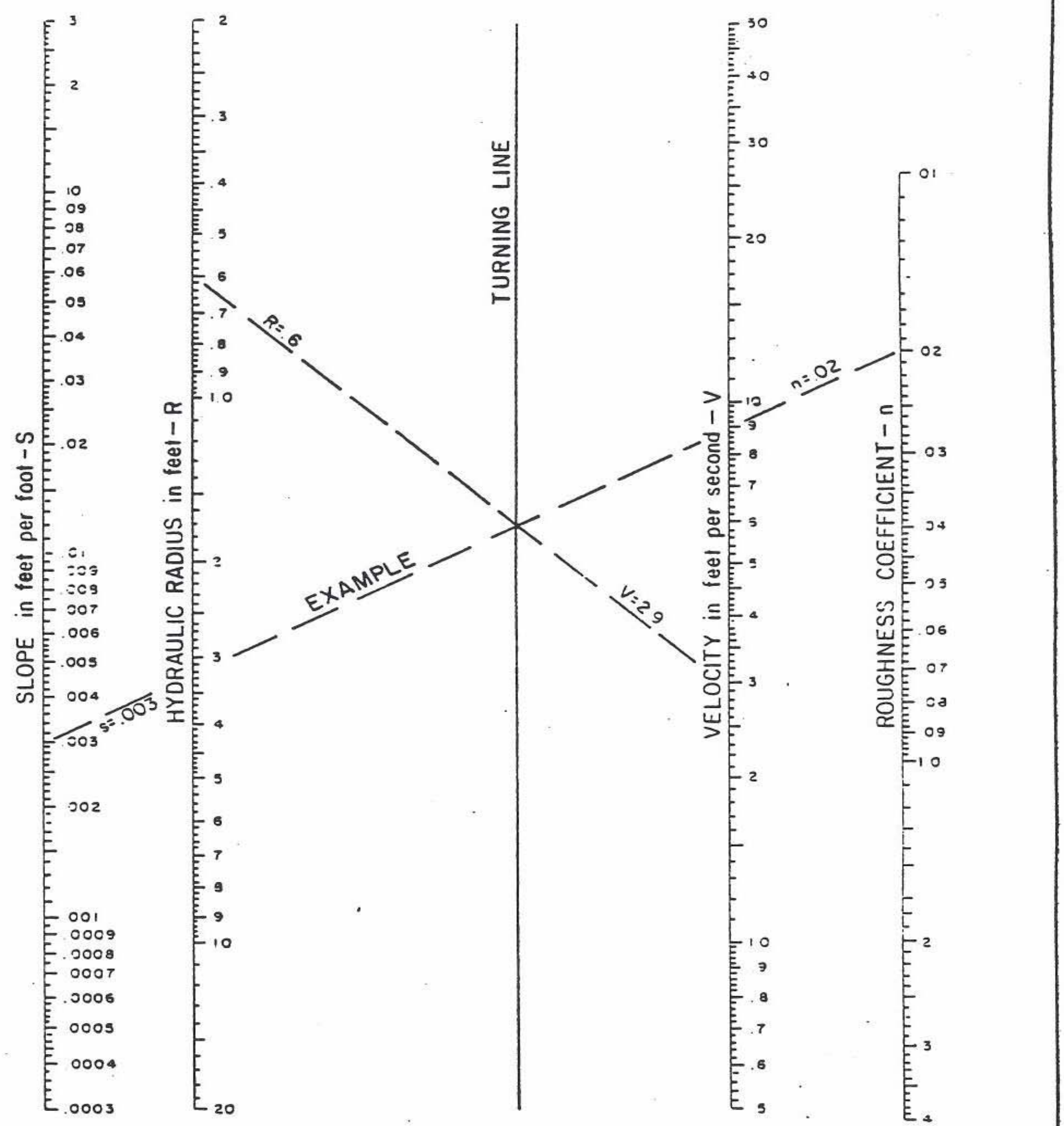
Channel Description	Average Velocity (Feet Per Second)	Maximum Velocity (Feet Per Second)
Grass Lined:		
Predominantly Clay Soil	3.0	5.0
Predominantly Sand Soil	2.0	4.0
Concrete Lined	6.0	10.0

Derived from the Criteria Manual for the Design of Flood Control and Drainage Facilities in Harris County, Texas, February 1984.

TABLE 3-4

RIGHT-OF-WAY REQUIREMENTS FOR FORT BEND COUNTY, TEXAS

Channel Type	Top Width	Maintenance Berm Width Necessary on Both Sides of Channel
All	TW ≤ 30 feet	15 feet
	30 feet < TW < 60 feet	20 feet
	TW ≥ 60 feet	30 feet



NOMOGRAPH FOR SOLUTION OF MANNING'S EQUATION

Source: Illinois Department of Transportation

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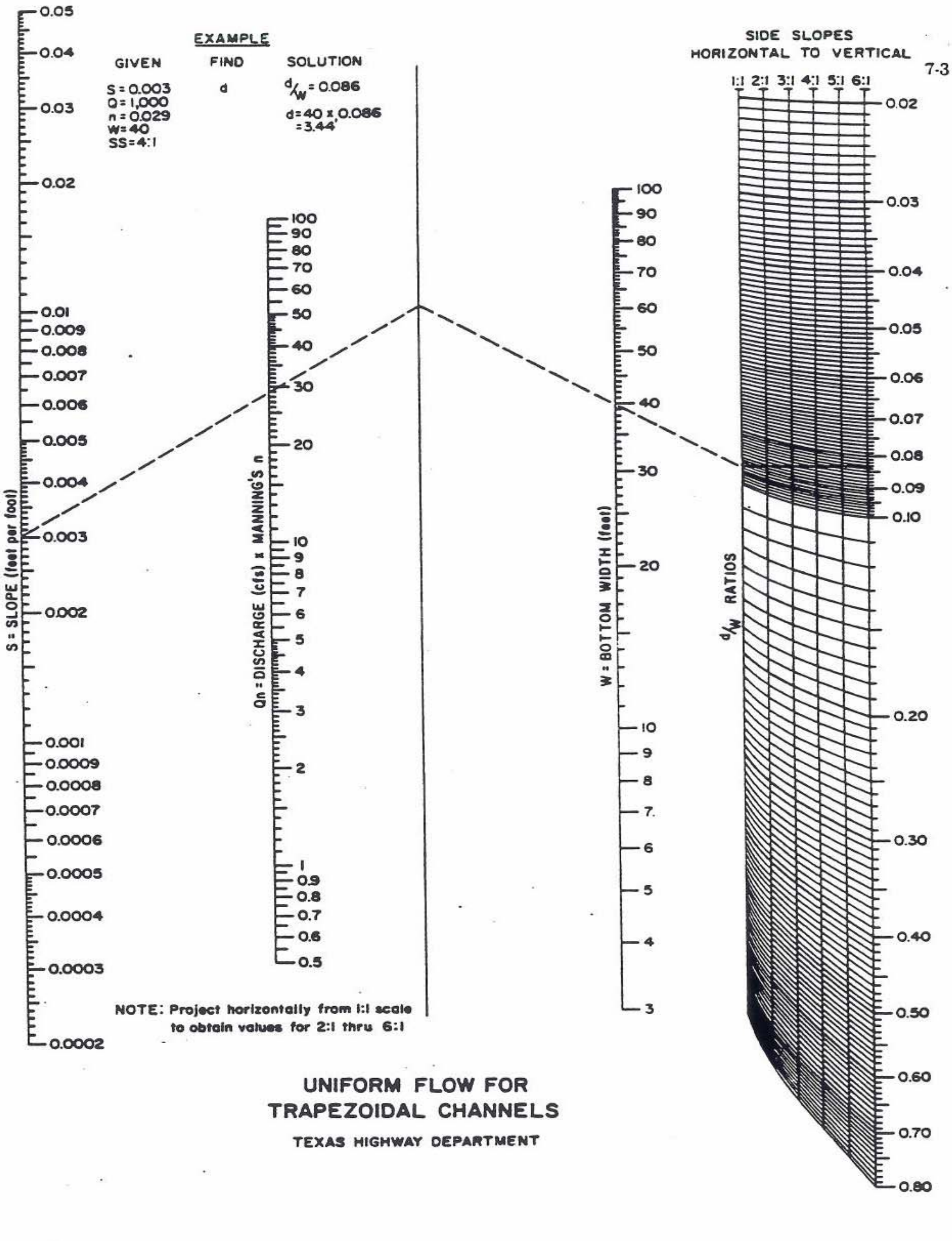
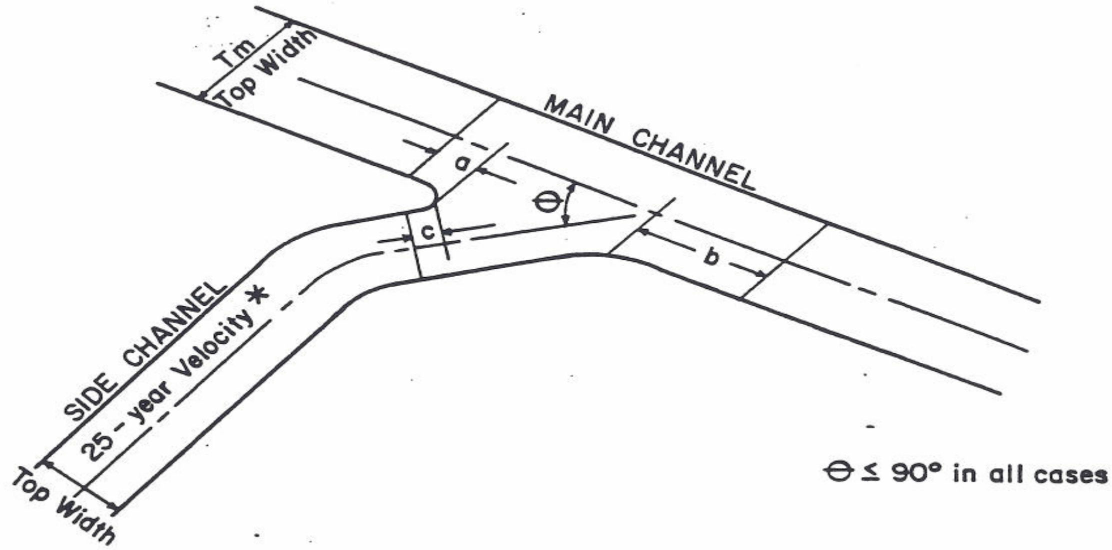
FIGURE 3-1



UNIFORM FLOW FOR
TRAPEZOIDAL CHANNELS
TEXAS HIGHWAY DEPARTMENT

August 1986	FIGURE 3- 2



MINIMUM EXTENT OF EROSION PROTECTION

Location	Distance (ft.)
a	20
b	longer of 50' or 0.75 x Tm ÷ Tan Θ
c	20'

25-year Velocity * in Side Channel (feet per second)	ANGLE OF INTERSECTION Θ: 15°—45°	ANGLE OF INTERSECTION Θ: 45°—90°
4 or more	Protection	Protection
2 – 4	No Protection	Protection
2 or less	No Protection	No Protection

* Note: 25–year velocity in side channel assuming no backwater from main channel

Note: Erosion protection must be provided to the level of the 25–year water surface elevation .

SOURCE : Criteria Manual For Design of Flood Control and Drainage Facilities in Harris County, TX Feb., 1984.

REQUIRED EROSION PROTECTION AT CHANNEL CONFLUENCE	
August 1986	3–3

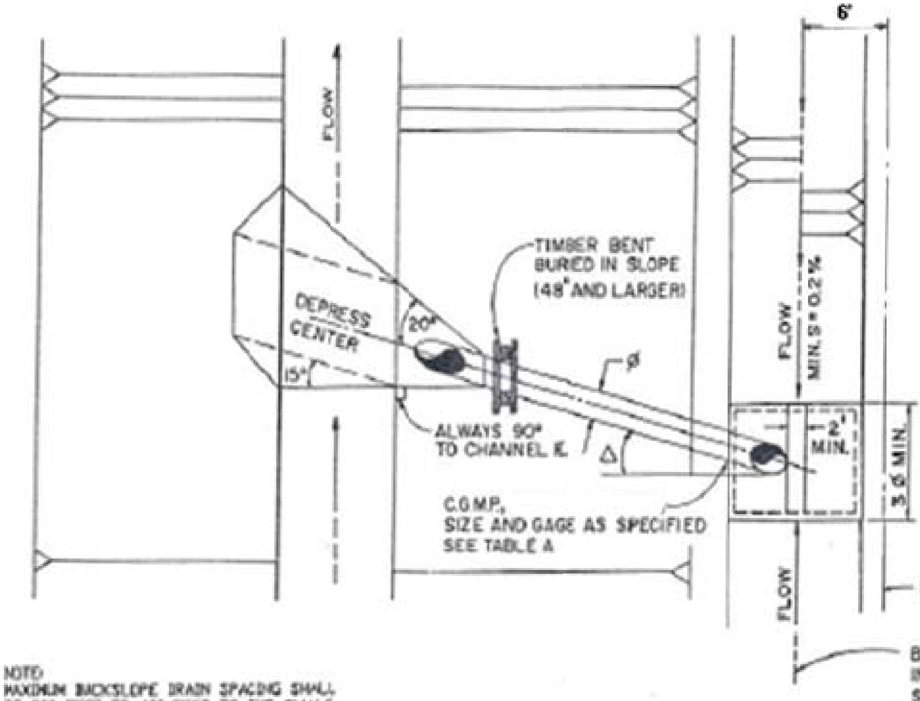


TABLE A

SIZE 2 2/3" X 1/2" CORRUGATION	PIPE GAUGE	BAND COUPLER GAUGE	SIZE 3"X1" & 5"X1" CORRUGATION	PIPE GAUGE	BAND COUPLER GAUGE
24"	16	16			
30"	16	16			
36"	16	16			
42"	14	16			
48"	14	16	48"	16	18
54"	12	14	54"	16	18
60"	12	14	60"	16	18
66"	10	12	66"	16	18
72"	10	12	72"	16	18
78"	8	10	78"	14	16
84"	8	10	84"	14	16

TABLE B

PIPE DIA.	SLOPE	VELOCITY
24"	0.6%	3.25 f.p.s.
36"	0.3%	3.00 f.p.s.
42"	0.2%	2.75 f.p.s.
48"	0.2%	3.00 f.p.s.
54"	0.2%	3.25 f.p.s.

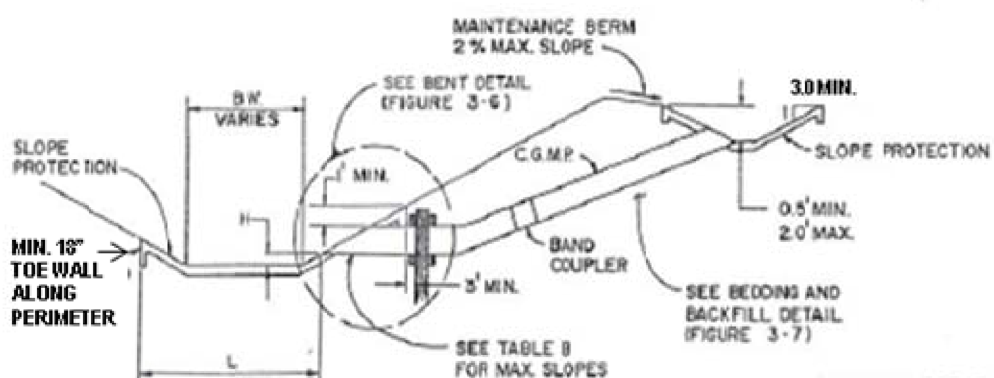
NOTE:
MAXIMUM BACKSLOPE DRAIN SPACING SHALL BE 800 FEET OR 400 FEET TO THE SWALE HIGH POINT.

Δ PROP. 24" TO 42" Δ = 15°
PROP. 48" AND LARGER Δ = 30°

H : FOR PIPE SIZES 24" TO 42" H=3' MAX. AND 1' MIN.
FOR PIPE SIZES 48" AND LARGER H=1' MAX. AND MIN.

L : $\frac{B.V.}{PIPE\ DIA.} \leq$ 7' - 6" ⇒ L WILL EXTEND ONE PIPE DIA. ABOVE FLOWLINE ON OPPOSITE BANK (OBD)

$\frac{B.V.}{PIPE\ DIA.} >$ 7' - 6" ⇒ L = 6 DIA OR MIN. 1'-6" INTO B.V. WHICHEVER IS GREATER



NOTE: CONCRETE SLOPE PAVING SHALL HAVE A MINIMUM THICKNESS OF 4". MINIMUM REINFORCING STEEL SHALL BE #3 REBAR AT 18" O.C. OR 6 x 6 x W4.0 x W4.0 WELDED WIRE FABRIC

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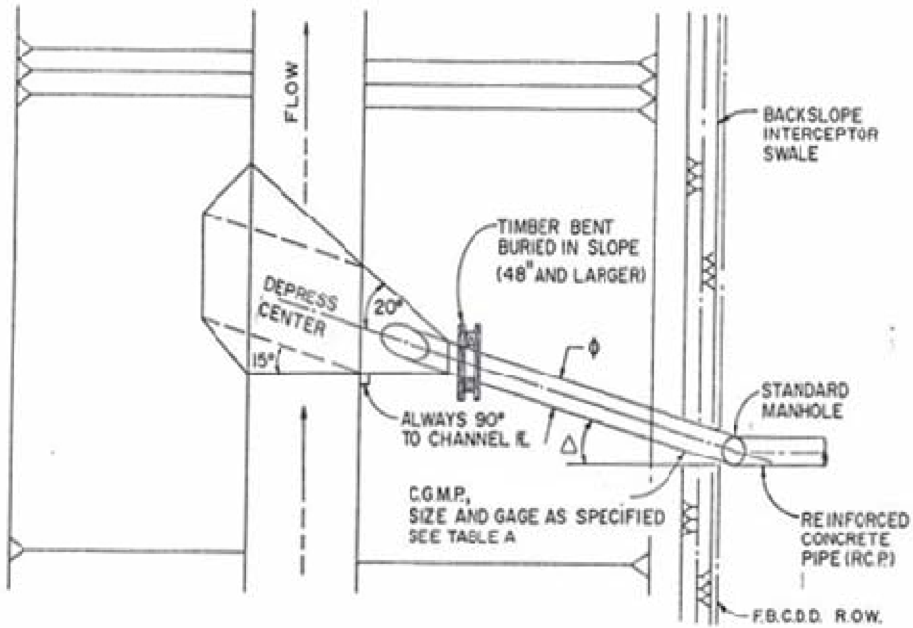
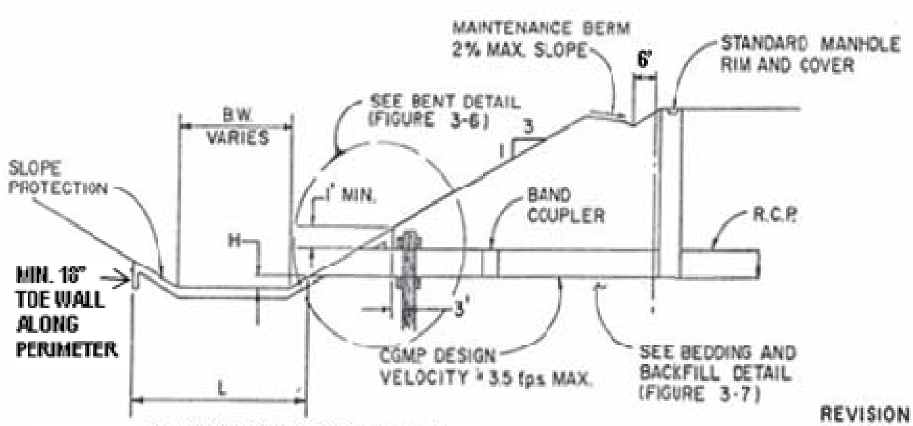
TYPICAL BACKSLOPE DRAIN DETAIL FOR FORT BEND COUNTY, TEXAS	
August 1986	FIGURE 3-4





NOTE: CONCRETE SLOPE PAVING SHALL HAVE A MINIMUM THICKNESS OF 4". MINIMUM REINFORCING STEEL SHALL BE #3 REBAR AT 18" O.C. OR 6 x 6 x W4.0 x W4.0 WELDED WIRE FABRIC

TABLE A

SIZE 2 2/3" X 1/2" CORRUGATION	PIPE GAUGE	BAND COUPLER GAUGE	SIZE 3"X1" & 5"X1" CORRUGATION	PIPE GAUGE	BAND COUPLER GAUGE
24"	16	16			
30"	16	16			
36"	16	16			
42"	14	16			
48"	14	16	48"	16	18
54"	12	14	54"	16	18
60"	12	14	60"	16	18
66"	10	12	66"	16	18
72"	10	12	72"	16	18
78"	8	10	78"	14	16
84"	8	10	84"	14	16

H: FOR PIPE SIZES 24" TO 42"
H=3' MAX. AND 1' MIN.

FOR PIPE SIZES 48" AND LARGER
H=1' MAX. AND MIN.

L: $\frac{B.W.}{PIPE\ Ø} \leq$ 7'-6" ⇒ L WILL EXTEND ONE PIPE Ø ABOVE ℄ ON OPPOSITE BANK (MIN.)

$\frac{B.W.}{PIPE\ Ø} >$ 7'-6" ⇒ L = 6 Ø OR MIN. 1'-6" INTO B.W. WHICHEVER IS GREATER

Δ: PROP. 24" TO 42" Δ = 15°
PROP. 48" AND LARGER Δ = 30°

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2-5-20-97
3-12-01-10

TYPICAL STORM SEWER OUTFALL
DETAIL FOR
FORT BEND COUNTY, TEXAS

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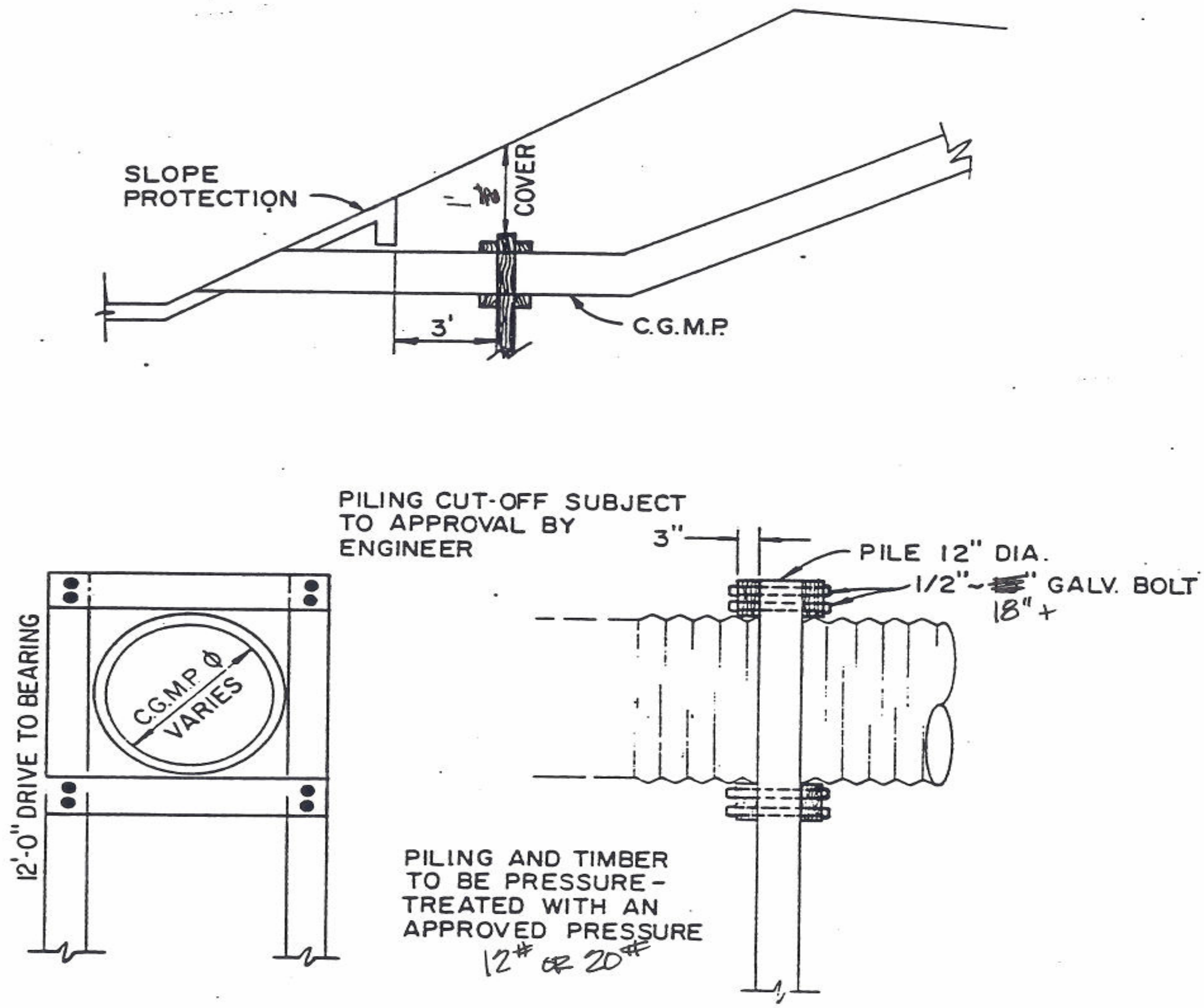
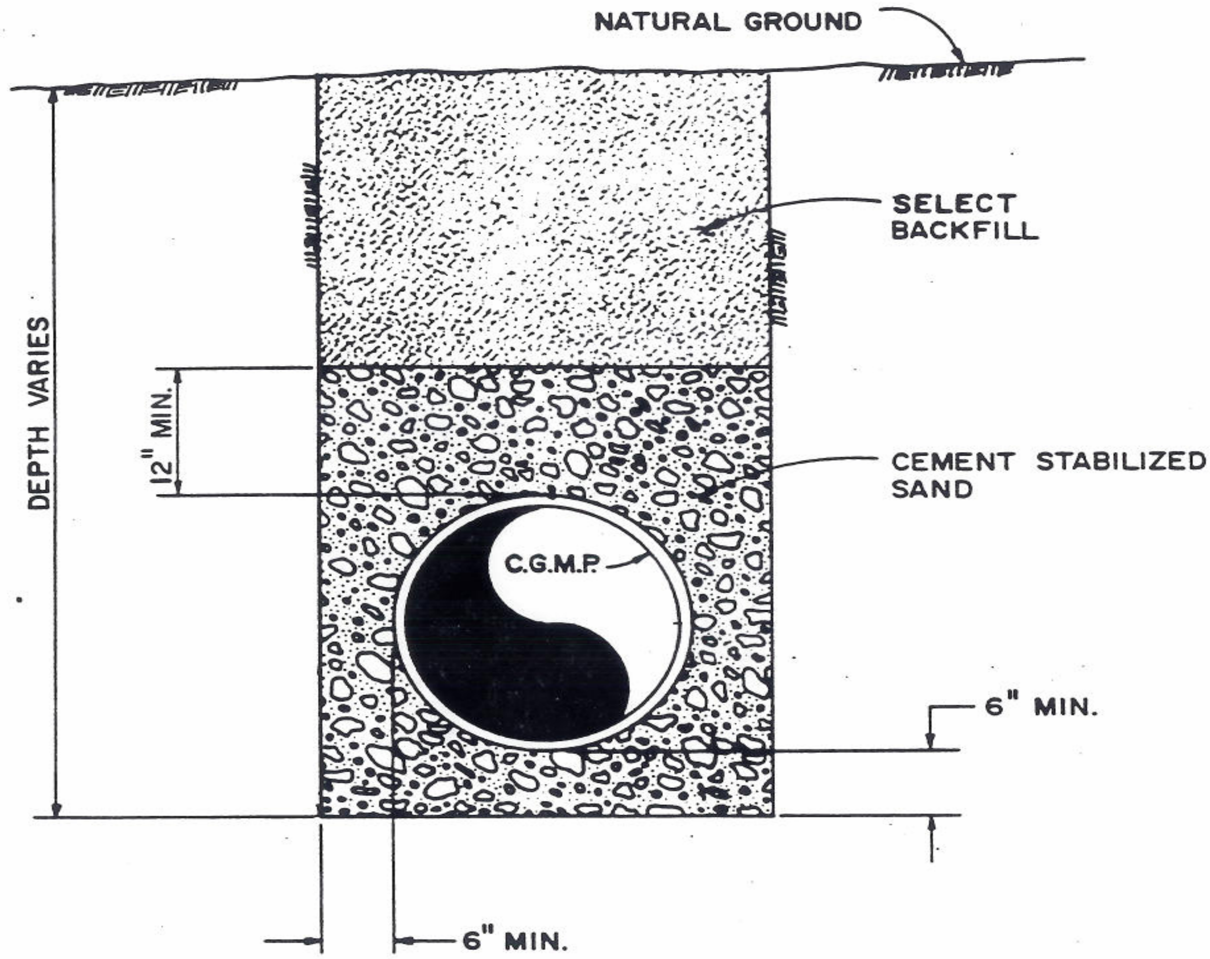


BENT FOR CORRUGATED METAL PIPE OUTFALL

48-INCH AND LARGER

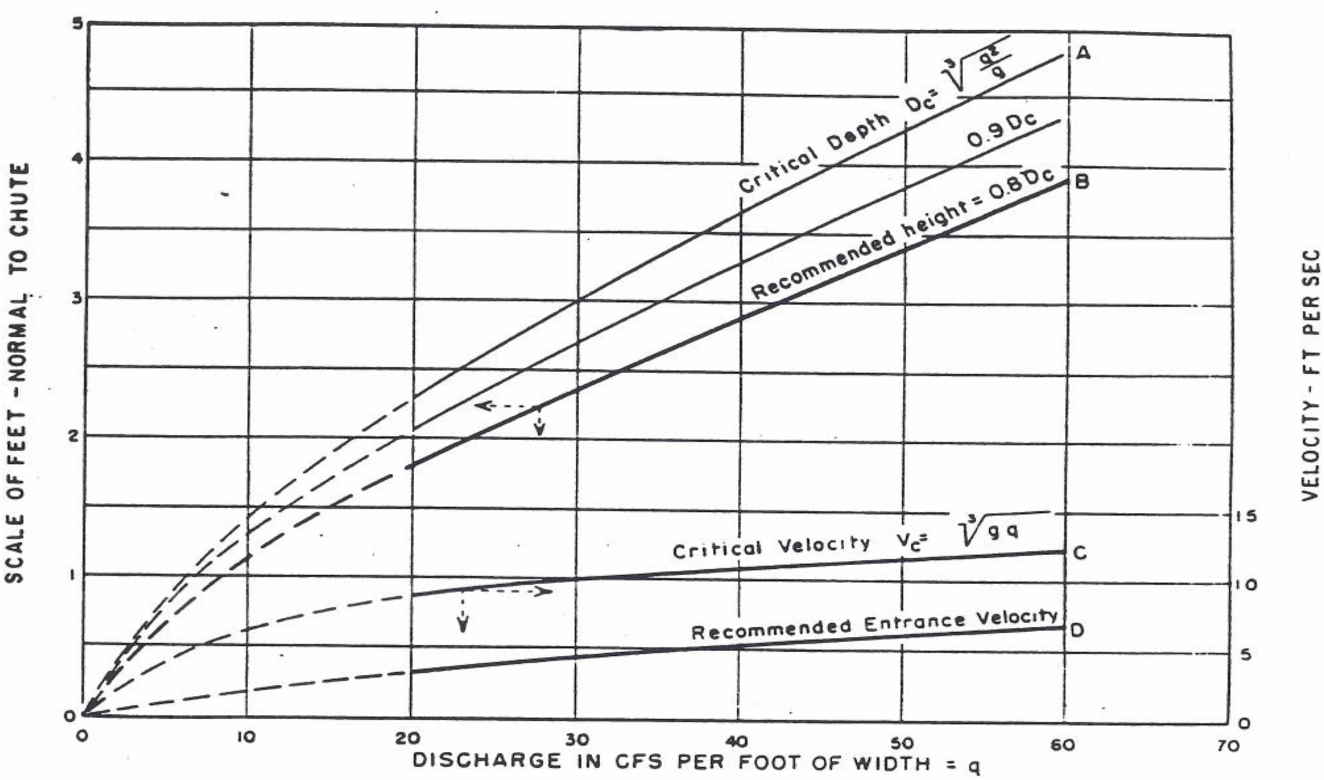
TYPICAL BENT DETAIL FOR
C.G.M.P. OUTFALL FOR
FORT BEND COUNTY, TEXAS

August 1986 | FIGURE 3-6



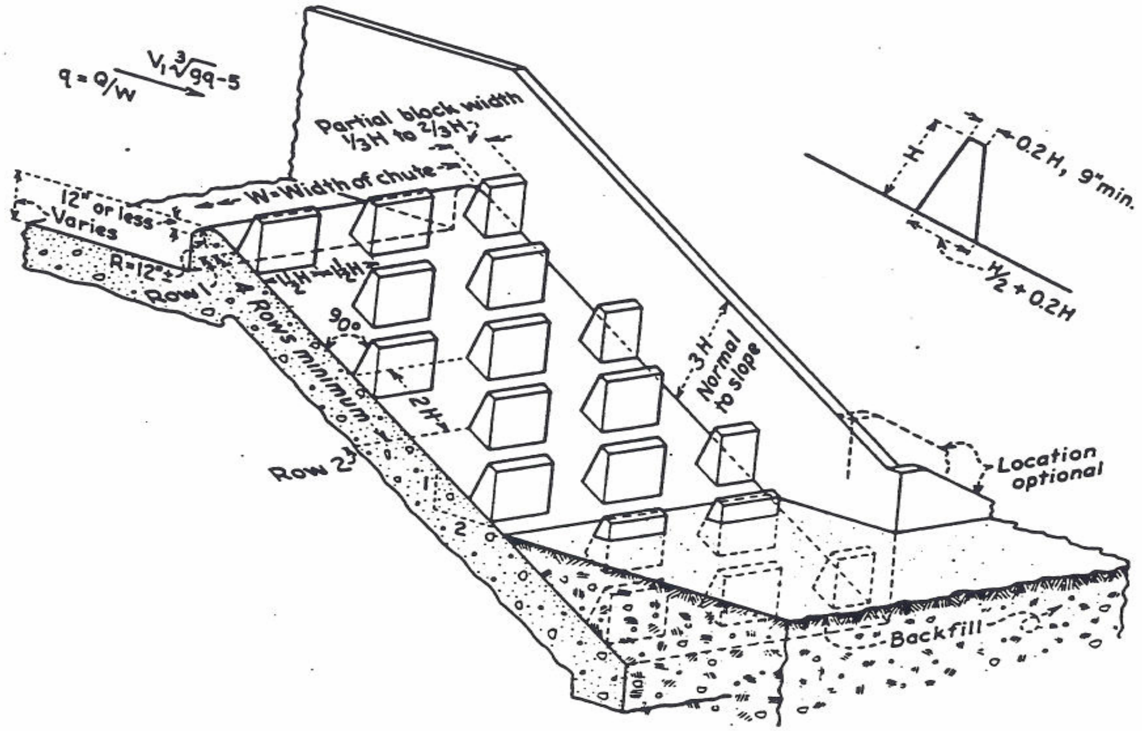
BEDDING AND BACKFILL DETAIL

TYPICAL BEDDING AND BACKFILL DETAIL FOR C.G.M.P. OUTFALL FOR FORT BEND COUNTY, TEXAS	
August 1986	FIGURE 3-7



SOURCE: **Progress Report V– Research Study on Stilling Basins, Energy Dissipators and Associated Appurtenances, Hyd-445, Bureau of Reclamation, April, 1961.**

RECOMMENDED BAFFLE PIER HEIGHTS AND ALLOWABLE VELOCITIES	
August 1986	FIGURE 3-8



SOURCE: Progress Report V- Research Study on Stilling Basins, Energy Dissipators and Associated Appurtenances, Hyd-445, Bureau of Reclamation, April, 1961.

BAFFLED CHUTE
TYPICAL DETAIL

August 1986

FIGURE 3-9