CHAPTER 13

HYDRAULIC ANALYSIS AND DESIGN

SECTION 1

OPEN CHANNELS

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1.1 INTRODUCTION

Presented in this section are the technical criteria and design standards for hydraulic evaluation and design of natural and artificial open channels. Discussions and hydraulic standards are provided for various channel types anticipated to be encountered or used in the State of Colorado.

The information presented in this section should be considered to be the minimum hydraulic standards upon which channel evaluation and design should be based. Additional analyses may be necessary for unique or unusual channel and site conditions. The users of this manual are encouraged to review the related textbooks and other technical literatures on the subject for more in-depth discussions. The following is a short list of some of the related publications.

- Chow, V. T., Open Channel Hydraulics, McGraw-Hill, 1970
- Brater and King, Handbook of Hydraulics, McGraw-Hill Book Co., 6th Ed., 1976.
- Dave Rosgen, illustrated by Hilton Lee Silvey, Applied River Morphology, 1996
- US Army Corps of Engineers, HEC-2 User Manual, Version 3.0, January 2001
- US Army Corps of Engineers, HEC-RAS User Manual, Version 4.6, February 1991
- US Army Corps of Engineers, Hydraulic Design of Flood Control Channels, EM 1110-2-1601, July 1991
- US Army Corps of Engineers, River Hydraulics, EM 1110-2-1416, October 1993

1.2 CHANNEL TYPES

Open channels can be categorized as either natural or artificial. Natural channels include all watercourses that are carved and shaped naturally by the erosion and sediment transport process. Artificial channels are those constructed or developed by human efforts. Essentially, open channels in Colorado can be separated into the following six (6) different types:

1.2.1 NATURAL CHANNELS

A natural channel is a watercourse formed naturally by the erosion and sediment transport process. In general, a natural channel continually changes its position and shape as a result of hydraulic forces acting on its bed and banks. If feasible, natural channels should be kept undisturbed and new developments should be placed sufficiently away from the channel banks.

1.2.2 GRASS-LINED CHANNELS

Grass-lined channels may be considered to be the most desirable artificial channels from an aesthetic viewpoint. The channel storage, lower velocities, and the sociological benefits create significant advantages over other types of channels. The grass cover can stabilize the channel side slopes, check erosion of the channel surface, and control the movement of soil particles along the channel bottom. Low flow areas may need to be concrete or rock lined to minimize erosion and maintenance problems.

1.2.3 CONCRETE-LINED CHANNELS

Concrete-lined channels are defined as rectangular or trapezoidal channels in which reinforced concrete is used for lining of the channel banks and bottom. Concrete-lined channels will be permitted only where ROW restrictions due to existing developments prohibit use of other channel types and will be approved on a case-by-case basis only. Special attentions should be taken to provide safety measures (i.e. fence) around the concrete channels.

1.2.4 RIPRAP-LINED CHANNELS

Riprap-lined channels are defined as channels in which riprap is used for lining of the channel banks and the channel bottom. Riprap is a popular choice for erosion protection because the initial installation costs are often less than alternative methods for preventing erosion. However, the designer needs to bear in mind that there are additional costs associated with riprap erosion protection since riprap installations require periodic inspection and maintenance.

Riprap-lined channels will be permitted in areas of existing development where ROW is limited and such limitation prohibits the use of bio-engineered channels. Situations for which riprap lining might be appropriate are: 1) where major flows, such as the 100-year flood are found to produce channel velocities in excess of allowable non-eroding values; 2) where channel side slopes should be steeper than 3:1; 3) for low flow channels, and 4) where rapid changes in channel geometry occur such as channel bends and transitions.

1.2.5 OTHER CHANNEL LINERS

A variety of artificial channel liners are on the market, all intended to protect the channel from erosion at higher velocities. These include gabion, interlocked concrete blocks, concrete revetment mats formed by injecting concrete into double layer fabric forms, and various types of synthetic fiber liners. As with rock and concrete liners, all of these types are best considered for helping to solve existing urban flooding problems and are not recommended for new developments. Each type of liner has to be scrutinized for its merits, applicability, how it meets other community needs, its long term integrity, and maintenance needs and costs.

1.2.6 WETLAND VEGETATION BOTTOM CHANNELS

This type of channel is a subset of "grass-lined" channels, designed to encourage the development of wetlands or certain types of riparian vegetation in the channel bottom. The potential benefits associated with a wetland bottom channel include habitat for aquatic, terrestrial, and avian wildlife and possible water quality enhancement as the base flows move through the marshy vegetation.

1.3 NATURAL CHANNEL SYSTEMS

When a natural channel is modified locally, the change frequently causes alteration in channel characteristics both upstream and downstream.

Natural and human-induced changes in natural channels frequently set in motion responses that can be propagated for long distances.

In general, a natural channel system continually changes its position and shape as a result of hydraulic forces acting on its bed and banks. These changes may be slow or rapid and may result from natural environmental changes or from changes caused by human activities. When a natural channel is modified locally, the change frequently causes alteration in channel characteristics both upstream and downstream. The response of a natural channel to human-induced changes often occurs in spite of attempts to control the natural channel environment.

Natural and human-induced changes in natural channels frequently set in motion responses that can be propagated for long distances. In spite of the complexity of these responses, all natural channels are governed by the same basic forces but to varying degrees. It is necessary that a natural channel system design be based on adequate knowledge of: (1) geologic factors, including soil conditions; (2) hydrologic factors, including possible changes in flow and runoff, and the hydrologic effects of changes in land use; (3) geometric characteristics of the stream, including the probable geometric alterations that developments will impose on the channel; (4) hydraulic characteristics such as depth, slope, velocity of streams, sediment transport, and the changes that may be expected in these characteristics over space and time; and (5) ecological/biological changes that will result from physical changes that may in turn induce or modify physical changes.

Effects of development in natural channels, flood control measures, and constructed channel structures have proven the need for considering the immediate, delayed, and far-reaching effects of alterations imposed on natural channel systems.

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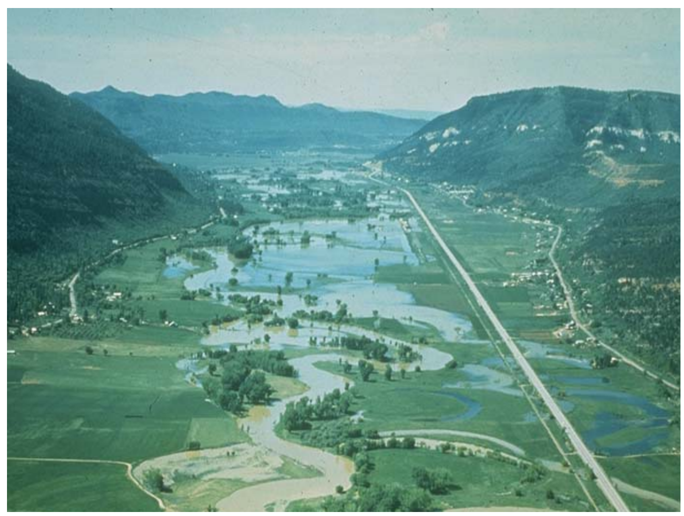
Variables affecting natural channels are numerous and interrelated. Their nature is such that, unlike rigid-boundary hydraulic problems, it is not possible to isolate and study the role of each individual variable. Because of the complexity of the processes occurring in natural flows that influence the erosion and deposition of material, a detached analytical approach to the problem may be difficult and time consuming. Most relationships describing natural channel processes have been derived empirically. The major factors affecting natural channel geometry are: (1) stream discharge; (2) sediment load; (3) longitudinal slope; (4) characteristics of bed and bank material; (5) bank and bed resistance to flow; (6) vegetation or lack there of; (7) geology, including type of sediment; and (8) constructed improvements.

1.3.1 CHANNEL MORPHOLOGY

When seeking to utilize or modify a natural channel, an understanding of the mechanism of its morphology is important. Without incorporating thorough understanding of the geomorphic conditions of the stream and the watershed, alterations to channels or to their watersheds can lead to unexpected instabilities, bring about unwanted erosion or aggradation, and cause significant damage to fluvial systems.



The morphology of a stream is a result of the variables that determine the quantity of water and sediment it carries, including the geology, soils and vegetation of the stream and watershed, the hydrology and dominant discharge of the system, and the slope of the stream. The following is a short discussion of some fundamentals of fluvial geomorphology. The users of this manual are encouraged to review the related textbooks and other technical literatures on the subject for more detailed discussions. The following is a short list of some of the related publications.

- Dave Rosgen, illustrated by Hilton Lee Silvey, Applied River Morphology, 1996
- Lane, E. W., 14957. A study of the shape of channels formed by natural streams flossing in erodible material: M.R. D. Sediment Series No. 9, US. Army Engineer Division, Missouri River, Corps of Engineers, Omaha, NE.
- Ritter, Dale F, 1986. Process Geomorphology. Wm C. Brown Publishers, Dubuque, Iowa.
- Simons Li and Associates, 1982. Engineering Analysis of Fluvial Systems.

There are three general principles governing the geomorphology of a natural stream system. First, riverine systems are dynamic. Erosion and aggradation can occur over a relatively short period of time (as sudden as one storm event) and can result from unstable conditions brought about by changing hydrologic or sediment-supply conditions (either natural or anthropomorphic). However, because all systems are dynamic, normal progression of a stream is not always a result or a symptom of instability. Second, the responses resulting from changes to a channel or its watershed are complex. Morphologic responses can be anticipated but cannot always be quantitatively predicted, even by the most trained engineers. Additionally, short reaches of streams cannot be looked at individually; a change to a short stretch or even to a single area of the stream can cause unwanted or unexpected alterations upstream or downstream of the change. Third, most geomorphic boundaries within a riverine system can be classified as thresholds. Gradual changes to a channel or its watershed will not always bring about gradual responses. Instead, gradual changes may build-up to a threshold so that a small-scale occurrence, such as a moderately large flood, will seemingly cause a catastrophic result. (SLA, 1982)

Natural streams can be classified generally into three prevailing patterns. These patterns, straight, meandering and braided, are characteristics of the responses of a system to its prevailing discharge and load.

Straight and meandering streams are two manifestations of similar dynamics. The thalwegs in both shift from bank to bank and sediment deposition and erosion within the channel bottom establish a series of riffles and pools. Straight channels have relatively straight banks; meandering streams have sinuous banks. Straight channels are fairly rare; most natural channels have some degree of sinuosity. Although meandering and straight streams can be in quasi-equilibrium, their thalwegs, meanders and riffle-pool sequences migrate in predictable patterns if left untouched. Braided systems, unlike meandering and straight, do not have a single trunk; they have a network of branches and series of islands. The single branches usually meander to some degree. Braided channels convey low to medium flows in the series of branches; large flows intermingle into a single floodplain. Meandering and straight systems are generally more stable than braided. Braided channels tend to carve new channels and deposit islands at a relatively fast pace and be horizontally unstable. The divisions between the three classifications are imprecise and relatively indistinct. A given stream can have reaches of each classification, and given reaches can include characteristics of one or more pattern. (Ritter, 1986)

Any change to a variable of a natural stream system, such as the slope or dominant discharge, can change the morphology and/or the existing stream pattern according to the three principles outlined above. These changes can be somewhat predicted; much work has been done to establish relationships between the variables and characteristics of natural streams. Two general relationships for predicting morphological responses to changes in riverine variables are as follows:

$$Q \cong \frac{b, d, \lambda}{S} \qquad \text{(Eq. CH13-100)}$$

and

$$Qs \cong \frac{b, \lambda, S}{d, P} \quad \text{(SLA, 1982)} \qquad \text{(Eq. CH13-101)}$$

Where Q = Average discharge
Qs = Sediment supply
B = Channel width
d = Channel depth
λ = Meander wavelength
S = Bed slope
P = Sinuosity

An increase in mean annual discharge will generally cause an increase in channel depth, width, and meander wavelength and a decrease in bed slope. An increase in sediment supply will generally cause an increase in channel width, meander wavelength and bed slope and a decrease in sinuosity and channel width. Because the average flow rate is usually directly related to sediment supply, these relationships can become complex when both flow and sediment supply increase or flow increases and sediment supply decreases, or vice-versa. Additionally, changes to one or more channel morphology characteristics can cause changes to other characteristics. An increase in slope can cause a decrease in channel depth or a decrease in meander wavelength. Further complicating these relationships are variables such as the average grain-size and type of sediment, the percentage of sediment carried as bed load, and the geology of the valley, all which can affect the responses of the stream and contribute to unexpected or seemingly counter-intuitive results.

A general relationship between slope, mean annual discharge and the tendency of a system to be meandering or braided has been established by Lane (1957). They found that if a stream's $SQ^{1/4} \leq 0.0017$, it tends to be meandering. If $SQ^{1/4} \geq 0.01$, systems tend toward a braided pattern. Streams that have $SQ^{1/4}$ between 0.0017 and 0.01 are in an intermediate zone and can be either braided or meandering with a greater tendency to respond to flow and slope alterations with a change in river pattern. These relationships are complicated and not absolute.

Some specific examples of man-induced changes to the natural stream/river systems that could cause undesired responses by channel morphology are as follows:

1) **Change in Flow:** As demonstrated in the above relationships, a decrease in flow due to diversion or reservoir routing change can cause a decrease in channel width, depth, and sinuosity and an increase in slope; an increase in flow due to development can have

the opposite effect. In addition to these changes, the corresponding decrease or increase of average stage of the main stem of a river can have significant effects on the streams' tributaries. If the average stage decreases, the tributaries' energy slopes will increase, increasing the ability of the tributary to transport sediment, which can cause degradation of its channel, commonly referred to as headcutting. Similarly, an increase in stage in the main stem can lead to aggradation within its tributaries. Both of these scenarios can do serious damage to the tributary channel and increase its horizontal instability. Headcutting can cause bank destabilization and failure. Aggradation can cause increased flooding potential and rerouting of the channel.

2) **Channelization:** The channelization of a natural stream to allow increased conveyance often straightens channels and cuts off meanders causing an increase in slope through the improved stretch. This can increase velocities and degradation through the stretch and then decrease slopes and increase aggradation downstream of the stretch. The increase in slope and average discharge can also cause a meandering system to tend toward a braided configuration that can lead to further horizontal and vertical instabilities. In addition, by lowering the average stage, channelization will affect the stream's tributary channels in the same manner as the first example.

3) **Construction of Dams**: The construction of both large and small-scale dams can have far-reaching effects on a stream system. Without a design-approach that will allow frequent flows to travel through the dam unadulterated, some suspended sediment and most bedload will be deposited upstream of the dam. This will decrease slopes and change channel configuration upstream and release clear water and potentially cause scour and degradation in the downstream reach. This can upset any equilibrium that was established within the system prior the construction and may even potentially cause failure of the dam itself.

4) **Construction of Bridges**: The construction of bridges and culverts, in addition to the well-documented local scour issues, can cause more regional channel morphology problems. An undersized bridge or culvert can decrease velocity and increase average stage upstream of the bridge, causing deposition and affecting the tributaries' channels. Scour around the bridges can cause an increase in sediment supply in the channel, leading to deposition downstream.

There are many additional examples of morphological problems that can be caused by man-made changes on a natural stream system. Any substantial modification to a natural channel system should be evaluated carefully to determine the potential adverse impacts on the stream system both upstream and downstream of the proposed modification.

1.3.2 CHANNEL RESTORATION

Channel/river restoration projects typically involve re-connection of the floodplain back to its channel, establishment of wetland areas around the channel, restoration of meanders, point-bars and riffle-pool sequences, and re-creation of the chemical and biological complexity that exists in the natural channel system.

The practice of channel restoration is becoming more common in Colorado and the rest of the United States as the negative effects of urbanization, channelization, and other hydraulic "improvements" have taken their toll on the sediment balance, channel stability, biological habitats, and the aesthetic and recreational benefits of the impacted rivers and streams.

Although, it may not be feasible to restore the disturbed stream/river systems back to its original conditions, channel restoration projects can help expedite the natural channel recovering process and help to re-create an environment that closely resembles the original configuration of the stream system. Channel/river restoration projects typically involve re-connection of the floodplain back to its channel, establishment of wetland areas around the channel, restoration of meanders, point-bars and riffle-pool sequences, and re-creation of the chemical and biological complexity that exists in the natural channel system. Benching, allowing for a low-flow meandering channel with terraced banks above the low-flow channel, is a popular technique that allows for expansive riparian plant and wildlife habitat, recreation opportunities, and unique flood control options. Channel restoration usually involves a significant degree of both planting and seeding native, wetland, and self-sustainable vegetations within the channel and along the banks.

A design team comprised of hydraulic engineers, fluvial geomorphologists, biologists and botanists who are highly knowledgeable of the system should be involved in the channel restoration design process. Furthermore, due to the advantage of irregular alignments and channel cross sections, the construction phase should be carefully managed and overseen to ensure that the design is fully incorporated into the final improvement.

1.4 OPEN-CHANNEL HYDRAULICS

An open channel is a conduit in which water flows with a free surface (non-pressurized flow). The hydraulics of an open channel can be very complex, encompassing many different flow conditions from steady-state uniform flow to unsteady, rapidly varying flow. Most of the problems in storm water drainage involve uniform, gradually varying or rapidly varying flow states. Examples of these flow conditions are illustrated in Figure CH13-F101. The calculations for uniform and gradually varying flow are relatively straight forward and are based upon similar assumptions (i.e., parallel streamlines). Rapidly varying flow computations (i.e., hydraulic jumps and flow over spillways), however, can be very complex, and the solutions are generally empirical in nature.

Presented in this section are the basic equations and computational procedures for uniform, gradually varying and rapidly varying flow. The user is encouraged to review the many hydraulics textbooks available for more detailed discussions.

1.4.1 UNIFORM FLOW

Open-channel flow is said to be uniform if the depth of flow is the same at every section of the channel. For a given channel geometry, roughness, discharge and slope, the only possible depth for maintaining uniform flow is the normal depth. For uniform flow in a prismatic channel (i.e., uniform cross section), the water surface will be parallel to the channel bottom.

Uniform flow rarely occurs in nature and is difficult to achieve in a laboratory, because not all of the parameters remain exactly the same. However, channels are designed assuming uniform flow as an approximation, which is adequate for planning and design purposes.

The computation of uniform flow and normal depth should be based upon the Manning formula as follows:

$$Q = \left(\frac{1.49}{n}\right) A R^{2/3} S^{1/2} \qquad \text{(Eq. CH13-102)}$$

Where Q = Flow rate (cubic feet per second (cfs))
n = Roughness coefficient
A = Area (square feet (sf))
P = Wetted perimeter (feet)
R = A/P = Hydraulic radius (feet)
S = Slope of the energy grade line (feet/feet)

For prismatic channels, the energy gradeline (EGL) slope, hydraulic gradeline (HGL) slope, and the bottom slope are assumed to be the same for uniform, normal depth flow conditions.

Presented in Table CH13-T101 are equations for calculating many of the parameters required for hydraulic analysis of different channel sections. Table CH13-T102 provides a list of Manning roughness coefficient values for many types of conditions that may occur in the State of Colorado. These parameters and the Manning equation may also be readily computed using hand-held calculators and personal computers.

1.4.2 UNIFORM CRITICAL FLOW ANALYSIS

The critical state of uniform flow through a channel is characterized by several important conditions.

1. The specific energy is a minimum for a given discharge.
2. The discharge is a maximum for a given specific energy.
3. The specific force is a minimum for a given discharge.
4. The velocity head is equal to half the hydraulic depth in a channel of small slope.
5. The Froude Number is equal to 1.0.

If the critical state of uniform flow exists throughout an entire reach, the channel flow is critical and the channel slope is at critical slope, S_c. A slope less than S_c will cause sub-critical flow. A slope greater than S_c will cause super-critical flow. A flow at or near the critical state is unstable. Factors creating minor changes in specific energy, such as channel debris, will cause a major change in depth.

A flow at or near the critical state is unstable. Factors creating minor changes in specific energy, such as channel debris, will cause a major change in depth.

The criteria of minimum specific energy for critical flow results in the definition of the Froude Number (F_r) as follows:

$$F_r = \frac{V}{(gD)^{0.5}} \qquad \text{(Eq. CH13-103)}$$

Where F_r = Froude Number
V = Velocity (feet per second (fps))
g = Acceleration of gravity (feet per second squared)
A = Channel flow area (square feet)
T = Top width of flow area (feet)
D = A/T = Hydraulic depth (feet)

The Froude Number for a given channel section and flow can be easily computed using the above equation. The critical depth in a given trapezoidal channel section with a known flow rate can be determined using the following methodology. First, the section factor, Z, is computed.

$$Z = \frac{Q}{g^{0.5}} \qquad \text{(Eq. CH13-104)}$$

Where Z = Section factor
Q = Flow rate (cfs)
g = Acceleration of gravity (feet per second squared)

Utilizing values for Z, the channel bottom width, b, and the side slope, z, the critical depth in the channel, y, can be determined from Figure CH13-F102. For other prismatic channel shapes, Equation CH13-104 above can be used with the section factors provided in Table CH13-T101 to determine the critical depth.

Since flows at or near critical depth are unstable, all channels should be designed with Froude Numbers and flow depths as follows:

Flow Condition	Froude Number (F_r)	Flow Depth
Sub-Critical	<0.8	>1.1d_c
Super-Critical	>1.13	<0.9d_c

Where d_c = critical depth

All channel design submittals should include the calculated Froude Number and critical depth for each unique reach of channel to identify the flow state and verify compliance with the MANUAL.

1.4.3 GRADUALLY VARYING FLOW

The most common occurrence of gradually varying flow in storm drainage is the backwater created by culverts, storm sewer inlets, or channel constrictions. For these conditions, the flow depth will be greater than normal depth in the channel, and the water surface profile should be computed using backwater techniques.

Backwater computations can be made using the methods presented in Chow, 1959. Many computer programs are available for computation of backwater curves. The most general and widely used programs are US Army Corps of Engineers' HEC-2 and HEC-RAS. These programs are recommended for floodwater profile computations for channel and floodplain analyses.

For prismatic channels, the backwater calculation can be computed manually using the Direct Step Method as described in Chow, 1959. The Direct Step Method is also available in many hand-held and personal computer software programs. For an irregular non-uniform channel, the Standard Step Method is used which is a more tedious and iterative process. For these channels, the use of HEC-2 or HEC-RAS is recommended.

1.4.4 RAPIDLY VARYING FLOW

Rapidly varying flow is characterized by very pronounced curvature of the flow streamlines. The change in curvature may become so abrupt that the flow profile is virtually broken, resulting in a state of high turbulence. There are mathematical solutions to some specific cases of rapidly varying flow, but empirical solutions are generally relied on for most rapidly varying flow problems.

The most common occurrence of rapidly varying flow in storm drainage applications involves weirs, orifices, hydraulic jumps, non-prismatic channel sections (transitions, culverts and bridges), and non-linear channel alignments (bends). Each of these flow conditions require extensive and detailed calculations to properly identify the flow capacities and depths of flow in the given section. The designer should be cognizant of the design requirements for each of the above conditions and should include all necessary calculations as part of the design submittal documents. The designer is referred to the many hydraulic references for the proper calculation methods to use in the design of rapidly varying flow facilities.

1.4.5 TRANSITIONS

Channel transitions occur in open channel design whenever there is a change in channel slope or shape and at junctions with other open channels or storm sewers. The goal of a good transition design is to minimize the loss of energy as well as minimize surface disturbances from cross-waves and turbulence. Special cases of transitions where excess energy is dissipated by design are drop structures and hydraulic jumps. Channel drop structures are discussed in Chapter 13, Section 6.

Transitions in open channels are generally designed for the following four flow conditions:

1. Sub-critical flow to sub-critical flow.
2. Sub-critical flow to super-critical flow.
3. Super-critical flow to sub-critical flow (Hydraulic Jump).
4. Super-critical flow to super-critical flow.

For definition purposes, conditions 1 and 2 will be considered as sub-critical transitions and are later discussed in Section 1.8.1. Conditions 3 and 4 will be considered as super-critical transitions and are later discussed in Section 1.8.2.

1.5 OPEN CHANNEL DESIGN

Adequate drainage facilities in developed areas are essential to preserve and promote the general health, welfare, and economic well being of the region. All new open channels should be designed, as a minimum, to safely confine and convey the estimated 100-year flood flows.

The design standards for major and minor drainageways are included in this section. A major drainageway is defined as a channel/drainageway with a contributing tributary area of 160 acres or more. The design standards presented in this chapter are minimum standards, and the channel designer is reminded that the ultimate responsibility for a safe and stable channel design lies solely with the engineer responsible for the design. Thus, the execution of this responsibility may require additional analysis and stricter standards than are presented in this chapter. In addition, unique or unusual site conditions may require additional design analysis be performed to verify the suitability of the proposed channel design for the project site.

All new open channels should be designed, as a minimum, to safely confine and convey the estimated 100-year flood flows. A major drainageway is defined as a channel/ drainageway with a contributing tributary area of 160 acres or more.

1.5.1 CHANNEL TYPE SELECTION

As discussed previously in Section 1.2, open channels can be generally separated into the following six (6) different channel types.

- Natural Channels
- Grass-lined Channels
- Concrete-lined Channels
- Riprap-lined Channels
- Wetland Vegetation Bottom Channels
- Other Channel Liners

The selection of a channel type appropriate for the conditions that exist at the project site should be based on the following multi-disciplinary factors including hydraulic, structural, environmental, sociological, maintenance, and regulatory factors. In general, the use of concrete-lined and riprap-lined channels is discouraged.

Hydraulic Factors

1. Slope of thalweg
2. Right-of-way
3. Capacity needed
4. Basin sediment yield
5. Topography
6. Ability to drain adjacent lands

Structural Factors

1. Cost
2. Availability of material
3. Areas for wasting excess excavated material
4. Seepage and uplift forces
5. Shear stresses
6. Pressures and pressure fluctuations
7. Momentum transfer

Environmental Factors

1. Neighborhood character
2. Neighborhood aesthetic requirements
3. Need for new green areas
4. Street and traffic patterns
5. Municipal or county policies
6. Wetland mitigation
7. Wildlife habitat
8. Water quality enhancement

Sociological Factors

1. Neighborhood social patterns
2. Neighborhood children population
3. Pedestrian traffic
4. Recreational needs

Maintenance Factors

1. Life expectancy
2. Repair and reconstruction needs
3. Maintainability
4. Proven performance
5. Accessibility

Regulatory Factors

1. Federal Regulations
2. State Regulations
3. Local Regulations

1.5.2 MAXIMUM PERMISSIBLE VELOCITIES

The design of open channels should be based on maximum permissible velocities. Presented in Table CH13-T103 are the maximum permissible velocities for natural, improved, unlined, and lined channels.

The design of open channels should be based on maximum permissible velocities. This method of design assumes that a given channel section will remain stable up to the stated maximum permissible velocity provided that the channel is designed in accordance with the provisions of this MANUAL. Presented in Table CH13-T103 are the maximum permissible velocities for natural, improved, unlined, and lined channels. These values should be used for all channel designs in the State of Colorado. If a higher velocity is desired, the design engineer should demonstrate that the higher velocity would not endanger the health or safety of the public and would not increase maintenance of the channel section. For natural and improved unlined channels, a geotechnical report should be submitted identifying the existing and/or proposed soil material classification used for the maximum permissible velocity determination. Additional analysis may be required for natural channels or improved unlined channels to verify that the channel will remain stable based on the stated maximum permissible velocities.

The stated maximum permissible velocities are based on flow studies conducted by various governmental agencies and private individuals using non-clear water conditions. The application of these velocities to actual site conditions are subject to proper design and competent construction of the channel sections. The design engineer should be responsible for designing the channel section so it will remain stable at the final design flow rate and velocity. For channels constructed in part or in whole from fill materials, the design engineer should be responsible for designing the channel based upon the characteristics of the fill material.

1.6 NATURAL CHANNEL DESIGN

Presented in this section are the typical natural open channel sections that are encountered in Colorado. A graphical illustration of the typical design sections is presented in Figure CH13-F103. The selection of a design section for a natural channel is generally dependent on the value of developable land versus the cost to remove the land from a floodplain. The costs for the removal depend on the rate of flow, slope, alignment, and depth of the channel as well as material and fill costs for construction of the encroachment. The design sections discussed herein vary from no encroachment to the level of encroachment at which point an improved channel (unlined or lined) becomes more economical or is required to adequately protect the proposed development. The design standards of natural channels are the same for both major and minor drainageways.

For natural channel sections, the engineer should identify through stable channel (normal depth) calculations the stability or instability of the channel to contain the major (100-year) storm flows. If this analysis demonstrates that either bank erosion outside of the designated flow path (easement and/or right-of-way) or channel degradation is likely to occur, then an analysis of the magnitude and extent of the erosion may be necessary. In such a condition, the design engineer should meet with the local official to determine: a) what additional analysis should be prepared to estimate the potential extent of lateral and vertical channel movement, b) what is the potential risk to the proposed development from channel degradation and/or bank failure, c) what solutions and/or remedies are available which can mitigate the potential risk to the proposed development, and d) what improvements and/or reduction in encroachment in or adjacent to the subject channel will be required to allow approval of the subject development.

Some general design considerations and evaluation techniques for natural channels are as follows:

1. The channel and overbank areas should have adequate conveyance capacity for the major (100-year) storm runoff.

2. Natural channel segments with a calculated flow velocity greater than the allowable flow velocity should be analyzed for erosion potential with a suitable methodology using standard engineering practice. Additional erosion protection may be required.

3. The water surface profiles should be defined so that the 100-year floodplain can be delineated.

4. Filling of the floodplain fringe may reduce valuable storage capacity and may increase downstream runoff peaks.

5. Erosion control structures, such as drop structures or check dams, may be required to control flow velocities for both the minor storm and major storm events.

6. Plan and profile information (i.e., HEC-2 output) for both existing and proposed floodplain site conditions should be prepared.

7. The engineer should verify, through stable channel (normal depth) calculations, the suitability of the floodplain to contain the flows. If this analysis demonstrates erosion outside of the designated flow path (easement and/or ROW), an analysis of the equilibrium slope and degradation or aggregation depths is required and suitable improvements identified.

With many natural channels, erosion control structures may need to be constructed at regular intervals to decrease the thalweg slope and to minimize erosion. However, these channels should be left in as near a natural state as possible. For that reason, extensive modifications should not be pursued unless they are found to be necessary to avoid excessive erosion with substantial deposition downstream.

The usual rules of freeboard depth, curvature, and other rules, which are applicable to artificial channels, do not apply for natural channels. There are significant advantages that occur if the designer incorporates into his planning the overtopping of the channel and localized flooding of adjacent areas, which remain undeveloped for the purpose of being inundated during the major runoff peak.

If a natural channel is to be modified or encroached upon for a development, then the applicant should meet with the agencies with jurisdiction over the channel to discuss the design concept and to obtain the requirements for planning, design analysis, and documentation.

1.6.1 NATURAL UNENCROACHED CHANNELS

Natural unencroached channels are defined as channels where overlot grading from the development process does not encroach into the 100-year floodplain of a given channel. Although the development does not alter the flow carrying capacity of the floodplain, it is necessary to ensure that the development is protected from movement of the floodplain boundaries due to erosion and scour. Therefore, the designer needs to identify the locations susceptible to erosion and scour and provide a design that reinforces these locations to minimize potential damage to the proposed development. For natural channels with velocities that exceed stable velocities, erosion protection may include the construction of buried grade control/check structures to minimize head-cutting and subsequent bank failures.

1.6.2 NATURAL ENCROACHED CHANNELS

Natural encroached channels are defined as channels where the development process has encroached into the 100-year floodplain fringe. This definition includes both excavation and/or fill in the floodplain fringe. The designer should prepare a design that will minimize damage to the development from movement of the floodplain boundaries due to erosion and scour. Consideration of erosion protection is similar to that for unencroached channels with emphasis on protection of the fill embankment.

1.6.3 BANK-LINED CHANNELS

Bank-lined channels are channels where the banks will be lined but the channel bottom will remain in a natural state with minimal regrading. The concerns with bank-lined channels are to minimize scour of the channel bottom at the bank lining interface as well as maintaining a stable natural channel. The designer should prepare a design that addresses scour depths at the lining interface to assure that the lining extends below this depth to avoid undermining of the lining.

1.6.4 PARTIALLY LINED CHANNELS

Partially lined channels are defined as channels in which half of the channel is lined and other half is left in a natural or unimproved condition. The concerns with partially lined channels are twofold. First, the improvement and lining of one side of the channel will cause changes to the hydraulic parameters of the unlined section which could increase erosion and scour in the unlined section. Second, floods which occur during the temporary condition may damage the improved channel section and require avoidable costly repairs.

Partially lined channels will only be allowed if:

a) The bottom paving is bonded, or there is another mechanism in place to pay for the bottom paving once the channel is completed.

b) Erosion in the unlined section is addressed to the satisfaction of the local official.

c) Scour below the lining is addressed to the satisfaction of the local official.

The analysis and design should show that the proposed temporary channel does not adversely impact the hydraulic parameters and stability of the unlined section in a significant way.

1.7 DESIGN STANDARDS FOR MAJOR ARTIFICIAL DRAINAGEWAYS

Presented in this section are the typical improved channel design sections. A graphical illustration of the typical design sections is presented in Figure CH13-F104. The selection of a channel section and lining type is generally dependent on physical and economic channel restrictions (i.e. value of developable land), the slope of the proposed channel alignment, the rate of flow to be conveyed by the channel, and the comparative costs of the lining materials. The channel sections and linings discussed herein provide a range of options from which an appropriate channel may be selected. Specific hydraulic design standards that are applicable to all improved channels (i.e. transition, freeboard, etc.) are presented later in Section 1.8.

Within this section, six types of improved channels will be discussed: unlined channels, grass-lined channels, wetland bottom channels, riprap-lined channels, concrete-lined channels, and channels with other types of linings.

1.7.1 PERMANENT UNLINED CHANNELS

Permanent unlined channels are improved channels, which are constructed to the shape of vegetation-lined channels but are not re-vegetated. The cost of construction of these channels is relatively low for areas with flat slopes and where the design flow rates and velocities are small. The designer should adequately address potential erosion problem areas (i.e. bends, transitions, structures) as well as the overall stability of the unlined channel and the effect that possible future natural re-vegetation may have on the channel hydraulics. The stability of the channel should be analyzed as if the channel was a natural channel using the design standards in Section 1.6 of this chapter.

1.7.2 GRASS-LINED CHANNELS

Grass-lined channels may be considered to be the most desirable artificial channels from an aesthetic viewpoint.

The channel storage, lower velocities, and the sociological benefits create significant advantages over other types of channels. The designer should give full consideration to flow hydraulics for which calculations should be submitted for review and approval by the local official.

The satisfactory performance of a grass-lined channel depends on constructing the channel with the proper shape and preparing the area in a manner to provide conditions favorable to vegetative growth. Between the time of seeding and the actual establishments of the grass, the channel is unprotected and subject to considerable damage unless special protection is provided. Channels subject to constant or prolonged flows require special supplemental treatment, such as grade control structures, stone centers, or subsurface drainage capable of carrying such flows. After establishment, the protective vegetative cover should be maintained.

A maintenance agreement and/or bond may be required to cover maintenance of grass-lined channels. In addition, the grass-lined channels may not be allowed on project sites where insufficient precipitation exists to maintain grass lining without irrigation.

1.7.2.1 LONGITUDINAL CHANNEL SLOPES

Grass-lined channel slopes are dictated by maximum permissible velocity requirements. Where the natural topography is steeper then desirable, drop structures may be utilized to maintain design velocities.

1.7.2.2 ROUGHNESS COEFFICIENT

The Manning's roughness coefficient used in the channel design should be obtained from Figure CH13-F105 assuming a mature channel (i.e., substantial vegetation with minimal maintenance).

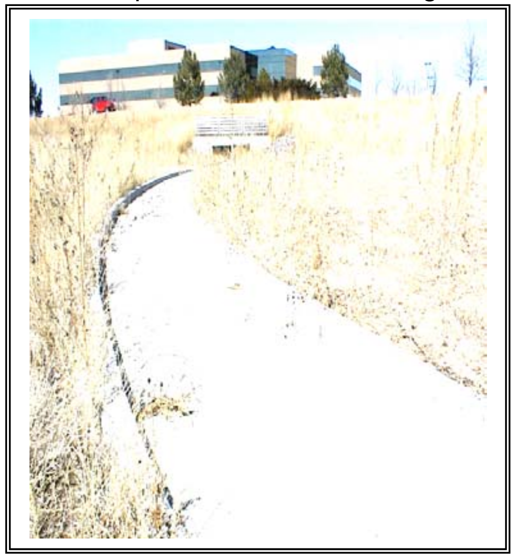
1.7.2.3 LOW FLOW AND TRICKLE CHANNELS

Low flows or base flows, from urban areas should be given specific attention. Waterways that are normally dry prior to urbanization will often have a continuous flow after urbanization because of lawn irrigation return flows, both overland and from ground water in-flow. Since continuous flow over grass will destroy a grass stand and may cause the channel profile to degrade, trickle channels or low flow channels are required on all urban grass-lined channels. Concrete trickle channels are preferred because of their ease of maintenance. Other types are acceptable if they are properly designed. Trickle channels may not be practical on larger major drainageways, streams and rivers, or in channels located on sandy soils where a low flow channel may be the more appropriate choice.

a) Trickle Channels

Trickle channels are used for channels with a 100-year design flow less than or equal to 200 cfs. The trickle channel's capacity should be a minimum of 5.0 percent of the 100-year design flow rate or 5 cfs, whichever is greater. The flow capacity of the main channel should be determined without considering the flow capacity of the trickle channel. Care should be taken to ensure that low flows enter the trickle channel without flow paralleling the trickle channel or bypassing the inlets.

i) Concrete Trickle Channel: To prevent erosion, silting, and excessive plant growth, concrete trickle channels are preferred. The concrete trickle channel should have a minimum depth of 6 inches. A Manning's roughness coefficient value of 0.015 will be used to design the concrete trickle channel. The trickle channel should be a minimum 6-inches thick with, as a minimum, #4 reinforcement at 12-inches each direction. Figure CH13-F106 shows a typical cross-section of a concrete trickle channel.



ii) Riprap Trickle Channel: The riprap trickle channel should have a minimum depth of 12 inches. Manning's roughness coefficient will be determined by Equation CH6-106. Figure CH13-F107 is a typical cross-section of a riprap trickle channel.

b) Low Flow Channels

Low-flow channels will be used in channels with a 100-year flow greater than 200 cfs. The low-flow channel will have the capacity to carry the 2-year flow event with no freeboard. Low-flow channels are used to contain relatively frequently occurring flows within a recognizable channel section. The flow capacity of the main channel should include the flow in the low flow channel. Figure CH13-F108 illustrates an example of a low-flow channel.

Low-flow channels should have a minimum depth of 12 inches. The riprap-lined side slopes of the low-flow channel will be 2.5:1 to 3:1. The main channel depth limitation does not apply to the low-flow channel area of the total channel cross-section.

1.7.2.4 BOTTOM WIDTH

The following design factors should be considered in selecting an appropriate channel bottom width.

- Constructability
- Channel stability and maintenance
- Multi-use purpose
- Trickle/low flow channel width

1.7.2.5 FLOW DEPTH

Typically, the maximum design depth of flow (outside the low flow channel area) should not exceed 5.0 feet for a 100-year flow of 1,500 cfs or less. For greater flows, excessive depths should be avoided to minimize high velocities and for public safely considerations.

1.7.2.6 SIDE SLOPES

Side slopes should not be designed steeper than 3 horizontal to 1 vertical. The use of 4 horizontal to 1 vertical side slope is recommended.

1.7.2.7 GRASS LINING

The grass lining for channels should be seeded or sodded with a grass species adapted to the local climate and will flourish without irrigation. Flowering plants (i.e. Honeysuckle) and weeds should not be used for grass-lined channels.

1.7.2.8 ESTABLISHING VEGETATION

Channel vegetation is usually established by seeding. In the more critical sections of some channels, it may be desirable to provide immediate protection by transplanting a complete sod cover.

Jute, plastic, paper mesh, hay mulch may be used to protect the entire width and side slopes of a waterway until the vegetation becomes established. All seeding, planting, and sodding should conform to local agronomic recommendations.

1.7.2.9 CHANNEL BEND PROTECTION

The potential for erosion increases along the outside bank of a channel bend due to the acceleration of flow velocities on the outside part of the bend. Thus, it is often necessary to provide erosion protection in natural or grass-lined channels which otherwise would not need protection.

In erosion resistant soils, no extra protection is required along bends where the radius is greater than 2 times the top width of the 100-year water surface, but in no case less than 100 feet. Channel bends with radii smaller than stated above require erosion protection. If erosion protection is provided, the minimum radius is 1.2 times the top width and in no case less than 50 feet. Erosion protection should extend downstream from the end of the bend a distance that is equal to the length of the bend measured along the channel centerline.

1.7.3 WETLAND BOTTOM CHANNEL

Under certain circumstances, such as when existing wetland areas are affected or natural channels are modified, the Corps of Engineers Section 404 permitting process may mandate the use of channels with wetland vegetation in their bottoms. In other cases, a wetland bottom channel may better suit individual site needs if used to mitigate wetland damages somewhere else or if used to enhance urban runoff quality. These types of channels are in essence grass-lined channels; with the exception that wetland type vegetation is encouraged to grow in their bottom. The easiest way to achieve this is to eliminate the concrete lined trickle/low-flow channel from the channel bottom and to limit the channel longitudinal slope so that low flows have low velocities.

There are potential benefits associated with a wetland bottom channel. These include habitat for aquatic, terrestrial, and avian wildlife and possible water quality enhancement as the base flows move through the marshy vegetation.

The down side of this practice is that the channel bottom is "boggy" and can become overgrown. This more abundant bottom vegetation traps sediments, thereby reducing channel flow carrying capacity as the bottom fills with sediments. Depending on the sediment loads being carried by the flows, the channel bottom will eventually have to be dredged to restore its flood carrying capacity or the channel section should be over-designed to compensate for the sediment deposition within the channel. Wetland bottom channels can provide habitat for mosquito breeding, and because the abundant vegetation can dislodge during a flood, an increased potential exists for blockage of roadway crossing structures.

The design of channels with wetland bottoms can be a complicated and iterative process. In order to simplify the design procedure for this manual, assumptions have been made concerning how the flow depth in a channel interacts with the wetland vegetation and affects the channel roughness and the rate of sediment deposition on the bottom.

1.7.3.1 LONGITUDINAL CHANNEL SLOPE

The longitudinal channel slope should be set so the maximum permissible velocity criteria provided in Table CH13-T103 is not violated. To prevent channel degradation, the channel slope should be determined assuming there is no wetland vegetation on the bottom (i.e., "New Channel"). In addition to the velocity requirements, the Froude Number for the New Channel condition should be less than 0.7.

1.7.3.2 ROUGHNESS COEFFICIENTS

The channel should be designed for two flow roughness conditions. As previously mentioned, a Manning's roughness coefficient assuming there is no growth in the channel bottom is used to set the channel slope. This is referred to as the New Channel condition. The Mature Channel condition assumes that wetland vegetation in the channel bottom has been established. The required channel depth including freeboard is determined assuming Mature Channel conditions.

A composite Manning's roughness coefficient should be used for the New Channel condition design and the Mature Channel condition design. The composite Manning's roughness coefficient is determined by the following equation (Chow, 1959):

$$n_c = \frac{(n_0^2 P_0 + n_w^2 P_w)^{0.5}}{(P_o + P_w)} \qquad \text{(Eq. CH13-105)}$$

Where n_c = Manning's roughness coefficient for the composite channel (Dimensionless)
n_o = Manning's roughness coefficient for areas above the wetland area (Dimensionless)
n_w = Manning's roughness coefficient for the wetland area (Dimensionless)
P_o = Wetland perimeter of channel cross-section above the wetland area (feet)
P_w = Wetland perimeter of the wetland channel bottom (feet)

For grass-lined areas above the wetland area, use a Manning's roughness coefficient, n_o, of 0.035. Manning's roughness coefficients for the wetland area (N_w) can be obtained from Figure CH13-F109.

1.7.3.3 LOW-FLOW CHANNEL

Trickle channels are not permitted in wetland bottom channels. Low-flow channels may be used when the 100-year flow exceeds 1,000 cfs. The design of the low flow channel should be according to Section 1.7.2.3 of this Chapter.

1.7.3.4 BOTTOM WIDTH

The following design factors should be considered in selecting an appropriate channel bottom width.

- Wetland mitigation requirements
- Constructability
- Channel stability and maintenance
- Multi-use purpose
- Low flow channel width

1.7.3.5 FLOW DEPTH

Typically, the maximum design depth of flow (outside the low flow channel area) should not exceed 5.0 feet for a 100-year flow of 1,500 cfs or less. For greater flows, excessive depths should be avoided to minimize high velocities and for public safely considerations.

1.7.3.6 SIDE SLOPES

Side slopes should not be designed steeper than 3 horizontal to 1 vertical.

1.7.3.7 GRASS LINING

The side slopes may be grass-lined according to the guidelines provided previously in Sections 1.7.2.7 and 1.7.2.8.

1.7.3.8 CHANNEL BEND PROTECTION

Channel bends should be designed according to the criteria discussed previously in Section 1.7.2.9.

1.7.3.9 CHANNEL CROSSINGS

Whenever a wetland bottom channel is crossed by a road, railroad or a trail requiring a culvert or a bridge, a drop structure should be provided immediately downstream of such a crossing. This will help reduce the silting-in of the crossing with sediments. A 1-foot to 2-foot drop is recommended. The designer should determine the hydraulics of the crossing and the drop structure and design the structures to ensure the stability of the channel.

1.7.3.10 LIFE EXPECTANCY

Wetland vegetation bottom channels are expected to fill with sediment over time. This occurs because the bottom vegetation traps some of the sediments carried by the flow. The life expectancy of such a channel will depend primarily on the land use of the tributary watershed and could range anywhere from 20 to 40 years before major channel dredging is needed. However, life expectancy can be dramatically reduced, to as little as two to five years, if land erosion in the tributary watershed is not controlled. Therefore, land erosion practices need to be strictly controlled during new construction within the watershed and all facilities need to be built to minimize soil erosion in the watershed to maintain a reasonable economic life of a wetland bottom channel.

1.7.4 RIPRAP-LINED CHANNELS

Riprap-lined channels are defined as channels in which riprap is used for lining of the channel banks and the channel bottom, if required. Riprap used for erosion protection at transitions and bends is also considered as a riprap-lined channel and those portions should be designed in accordance with the riprap-lined channel and transition design standards. The design standards presented in this section are the minimum hydraulic design parameters.

Riprap has proven to be an effective means to deter erosion along channel banks, in channel beds, upstream and downstream from hydraulic structures, at bends, at bridges, and in other areas where erosive tendencies exist. Riprap is a popular choice for erosion protection because the initial installation costs are often less than alternative methods for preventing erosion. However, the designer needs to bear in mind that there are additional costs associated with riprap erosion protection since riprap installations require periodic inspection and maintenance.

Channel linings constructed from loose riprap or grouted riprap to control channel erosion have been found to be cost effective where channel reaches are relatively short (less than 3 miles). Situations for which riprap lining might be appropriate are: 1) where major flows, such as the 100-year flood are found to produce channel velocities in excess of allowable non-eroding values; 2) where channel side slopes should be steeper than 3:1; 3) for low flow channels, and 4) where rapid changes in channel geometry occur such as channel bends and transitions. Design criteria applicable to these situations are presented in the following sections.

1.7.4.1 LONGITUDINAL CHANNEL SLOPE

Riprap-lined channel slopes are dictated by the maximum permissible velocity requirements (Table CH13-T103). Where topography is steeper than desirable, drop structures could be utilized to maintain design velocities.

1.7.4.2 ROUGHNESS COEFFICIENTS

The Manning's roughness coefficient, n, for hydraulic computations may be estimated for loose riprap using the following equation.

$$n = .0395\,(d_{50})^{1/6} \qquad \text{(Eq. CH13-106)}$$

Where d_{50} = mean stone size (feet)

This equation (Anderson, 1968) does not apply to grouted riprap (n= .023 to .030) or to very shallow flow (hydraulic radius is less than or equal to 2 times the maximum rock size) where the roughness coefficient will be greater than indicated by the formula.

1.7.4.3 LOW FLOW CHANNEL

The design criteria for the low-flow channel are discussed in the previous Section 1.7.2.3.

1.7.4.4 BOTTOM WIDTH

The following design factors should be considered in selecting an appropriate channel bottom width.

- Constructability
- Channel stability and maintenance
- Multi-use purpose
- Trickle/low flow channel width

1.7.4.5 FLOW DEPTH

As preliminary criteria, the design depth of flow for the major (100-year) storm runoff flow should not exceed 7.0 feet in areas of the channel cross-section outside the low-flow or trickle channel.

1.7.4.6 SIDE SLOPES

Due to stability, safety, and maintenance considerations, riprap-lined side slopes should be 2 horizontal to 1 vertical or flatter.

1.7.4.7 TOE PROTECTION

Where only the channel sides are to be lined, additional riprap is needed to provide for long-term stability of the lining. In this case, the riprap blanket should extend a minimum of 3 feet below the proposed channel bed, and the thickness of the blanket below the proposed channel bed should be increased to a minimum of 3 times d_{50} to accommodate possible channel scour during floods. If the velocity exceeds the permissible velocity requirements of the soil comprising the channel bottom, a scour analysis should be performed to determine if the toe requires additional protection.

1.7.4.8 BEGINNING AND END OF RIPRAP-LINED CHANNEL

At the upstream and downstream termination of a riprap lining, the thickness should be increased 50 percent for at least 3 feet to prevent undercutting. Depending on the site-specific conditions, concrete cutoff walls at both ends may be necessary.

1.7.4.9 LOOSE RIPRAP LINING

Loose riprap, or simply riprap, refers to a protective blanket of large loose angular stones that are usually placed by machine to achieve a desired configuration. The term loose riprap has been introduced to differentiate loose stones from grouted riprap.

Rock having a minimum specific gravity of 2.65 is preferred; however, in no case should the specific gravity of the individual stones be less than 2.50.

Classification and gradation for riprap are shown in Table CH13-T104 and are based on a minimum specific gravity of 2.50 for the rock.

Many factors govern the size of the rock necessary to resist the forces tending to move the riprap. For the riprap itself, this includes the size and weight of the individual rock, the shape of stones, the gradation of the particles, the blanket thickness, the type of bedding under the riprap, and slope of the riprap layer. Hydraulic factors affecting riprap include the velocity, current direction, eddy action, and waves. Figure CH13-F110 provides typical cross-sections for riprap-lined channels.

Experience has shown that riprap failures generally result from undersized individual rocks in the maximum size range, improper gradation of the rock which reduces the interlocking of individual particles and improper bedding for the riprap which allows leaching of channel particles through the riprap blanket.

a) Riprap Material

Rock used for loose riprap, grouted riprap, or wire enclosed riprap should be hard, durable, angular in shape, and free from cracks, overburden, shale and organic matter. Neither breadth nor thickness of a single stone should be less than 1/3 of its length and rounded stone should be avoided. Rock having a minimum specific gravity of 2.65 is preferred; however, in no case should the specific gravity of the individual stones be less than 2.50.

Classification and gradation for riprap are shown in Table CH13-T104 and are based on a minimum specific gravity of 2.50 for the rock. Because of its relatively small size and weight, riprap Class 150 should be buried with native topsoil and revegetated to protect the rock from vandalism.

Riprap lining requirements for a stable channel lining are based on the following relationship which resulted from model studies by Smith and Murray (Smith, 1965)

$$d_{50} = \frac{0.05 \ V^2 \ S^{0.34}}{(S_s - 1)^{1.332}} \qquad \text{(Eq. CH13-107)}$$

Where d_{50} = Rock size for which 50 percent of riprap by weight is smaller (feet)
V = Mean channel velocity (fps)
S = Longitudinal channel slope (feet/feet)
S_s = Specific gravity of rock (minimum S_s = 2.50) (dimensionless)

The riprap blanket thickness should be at least 2.0 times d_{50} and should extend up the side slopes to an elevation of the design water surface plus the calculated freeboard and superelevation.

b) Bedding Requirements

Long term stability of riprap erosion protection is strongly influenced by proper bedding conditions. A large percentage of all riprap failures is directly attributable to bedding failures.

Properly designed bedding provides a buffer of intermediate sized material between the channel bed and the riprap to prevent movement of soil particles through the voids in the riprap. Three types of bedding are in common use: a generic single-layer granular bedding, a granular bedding based on the T-V methodology, and filter fabric.

1) Granular Bedding - Generic Design

The gradation of a single layer bedding specification is based on the assumption that said bedding will generally protect the underlying soil from displacement during a flood event. The single layer bedding design does not require any soil information, but in order to be effective covering a wide range of soil types and sizes, this method requires a greater thickness than the T-V method.

A single 12-inch layer of said granular bedding can be used except at drop structures. At drop structures, filter fabric should be added below the 12-inch layer of granular bedding.

2) Granular Bedding - T-V Design

The T-V (Terzughi-Vicksburg) design establishes an optimum granular bedding gradation for a specific channel soil. Since this method designs the granular bedding for a particular soil, the allowable granular bedding thickness may be much less than the generic design.

The specifications for the T-V reverse filter method relate the gradation of the protective layer (filter) to that of the bed material (base) by the following inequalities:

$$D_{15(filter)} \leq 5d_{85(base)} \quad \text{(Eq. CH13-108)}$$

$$4d_{15(base)} \leq D_{15(filter)} \leq 20d_{15(base)} \quad \text{(Eq. CH13-109)}$$

$$D_{50(filter)} < 25d_{50(base)} \quad \text{(Eq. CH13-110)}$$

Where the capital "D" refers to the filter grain size and the lower case "d" to the base grain size. The subscripts refer to the percent by weight which is finer than the grain size denoted by either "D" or "d". For example, 15 percent of the filter material is finer than $D_{15(filter)}$ and 85 percent of the base material is finer than $d_{85(base)}$.

When the T-V method is used, the thickness of the resulting layer of granular bedding may be reduced to six inches. However, if a gradation analysis of the existing soils shows that a single layer of T-V Method designed granular bedding can not bridge the gap between the riprap specification and the existing soils, then two or more layers of granular bedding should be used. The design of the bedding layer closest to the existing soils should be based on the existing soil gradation. The design of the upper bedding layer should be based on the gradation of the lower bedding layer. The thickness of each of the two or more layers should be four inches.

3) Filter Fabrics

Filter fabric is not a complete substitute for granular bedding. Filter fabric provides filtering action only perpendicular to the fabric and has only a single equivalent pore opening between the channel bed and the riprap. Filter fabric has a relatively smooth surface which provides less resistance to stone movement. As a result, it is recommended that the use of filter fabric in place of granular bedding be restricted to slopes no steeper than 2.5 horizontal to 1 vertical, and that such filter fabric only replace the bottom layer in a multi-layer T-V Method granular bedding design. The granular bedding should be placed on top of the filter fabric to act as a cushion when placing the riprap. Tears in the fabric greatly reduce its effectiveness so that direct dumping of riprap on the filter fabric is not allowed and due care should be exercised during construction. Nonetheless, filter fabric has proven to be an adequate replacement for granular bedding in many instances. Filter fabric provides adequate bedding for channel linings along uniform mild sloping channels where leaching forces are primarily perpendicular to the fabric.

At drop structures and sloped channel drops, where seepage forces may run parallel with the fabric and cause piping along the bottom surface of the fabric, special care is required in the use of filter fabric. Seepage parallel with the fabric may be reduced by folding the edge of the fabric vertically downward about 2 feet (similar to a cutoff wall) at 12-foot intervals along the installation, particularly at the entrance and exit of the channel reach. Filter fabric has to be lapped a minimum of 12 inches at roll edges with upstream fabric being placed on top of downstream fabric at the lap.

Fine silt and clay has been found to clog the openings in filter fabric. This prevents free drainage which increases failure potential due to uplift. For this reason, a granular filter is often a more appropriate bedding for fine silt and clay channel beds.

1.7.4.10 GROUTED RIPRAP LINING

Grouted riprap provides a relatively impervious channel lining which is less subject to vandalism than loose riprap. Grouted riprap requires less routine maintenance by reducing silt and trash accumulation and is particularly useful for lining low-flow channels and steep banks. The appearance of grouted riprap is enhanced by exposing the tops of individual stones and by cleaning excess grout from the projecting rock with a wet broom prior to curing. Figure CH13-F111 provides a typical cross-section for a grouted riprap lining.

a) Riprap Material

The rock used for grouted riprap is different from the standard gradation of riprap in that the smaller rock has been reduced to allow greater penetration by the grout. The riprap specifications are shown on Table CH13-T106. Riprap smaller than Class 400 should not be grouted.

b) Bedding Material

The bedding material will be the same as for loose riprap.

c) Cutoff Trench

As the riprap layer is placed, a cutoff trench should be excavated around the rock section at the top of the slope and at the upstream and downstream edges. The trench should be, at a minimum, the full depth of the riprap and bedding layer and at least 1-foot wide. This trench is filled with grout to prevent water from undermining the grouted rock mass.

d) Grout

After the riprap has been placed to the required thickness and the trench excavated, the rock is sprayed with clean water which cleans the rock and allows better adherence by the grout. The rock is then grouted using a low pressure (less than 10 psi) grout pump with a 2" maximum diameter hose. Using a low pressure grout pump allows the work crew time to move the hose and vibrate the grout. Vibrating the grout with a pencil vibrator assures complete penetration and filling of the voids. After the grout has been placed and vibrated, a small hand broom or gloved hand is used to smooth the grout and remove any excess grout from the rock. The finished surface is sealed with a curing compound.

The grout should consist of 6 sacks (564 pounds) of cement per cubic yard, and the aggregate should consist of 30% of 3/8-inch coarse gravel and 70% natural sand. The grout should contain 7.5% +/- 1.5% air entrainment, have a 28-day compressive strength of at least 2,000 p.s.i., and have a slump of 7 inches +/- 2 inches. Fiber reinforcement should be used such as 1.5 pounds per cubic yard of Fibermesh or an approved equivalent amount. A maximum of 25% flyash maybe substituted for the cementations material.

1.7.4.11 CHANNEL BEND PROTECTION

When riprap protection is required for a straight channel, increase the rock size by one category (e.g., Class 300 to Class 400) through bends. The minimum radius for a riprap-lined bend is 1.2 times the top width and in no case less than 50 feet. Riprap protection should extend downstream from the end of the bend a distance that is equal to the length of the bend measured along the channel centerline.

1.7.4.12 TRANSITION PROTECTION

Scour potential is amplified by turbulent eddies in the vicinity of rapid changes in channel geometry such at transitions and bridges. For these locations, the riprap lining thickness should be increased by one size category.

Protection should extend upstream from the transition entrance at least 5 feet and extend downstream from the transition exit at least 10 feet. See Section 1.8 for further discussions on transitions.

1.7.4.13 CONCRETE CUTOFF WALLS

Transverse concrete cutoff walls may be required for riprap lined channels where a resulting failure of the riprap lining could seriously affect the health and safety of the public. The designer should consult with the local officials prior to design of riprap lined-channels to determine if concrete cutoff walls are required as well as their sizing and spacing, if required.

1.7.4.14 RIPRAP-LINED CHANNELS ON STEEP SLOPES

Achieving channel stability on steep slopes usually requires some type of channel lining. The only exception is a channel constructed in durable bedrock.

On mild slopes, the water velocity is slow enough and the depth of flow is large enough (relative to the riprap size) that a reasonable estimate of the resistance to flow can be made. On steep channels, the riprap size required to stabilize the channel is on the same order of magnitude or greater than the flow depth, which invalidates the Manning's relation. Since the resistance to flow is now unknown, an estimate of the velocity needed for the design of the riprap cannot be accurately estimated.

A graphically based methodology was developed for the U.S. Department of Interior, Office of Surface Mining Reclamation and Enforcement (SIMONS, 1989) to design riprap-lined channels on steep slopes (supercritical flow). This methodology was based on a study by BATHURST, 1979 that analyzed the hydraulics of mountain rivers where roughness elements are on the same order of magnitude as the depth of flow. Using the resistance equation developed by Bathurst, the velocity can be estimated for a given riprap size. The velocity is then used to predict the stability of the riprap.

This procedure should be used for all riprap lined channels whose depth of flow is equal to or less than d_{50} as computed initially using Equation CH13-107.

a) Rock Size

Five sets of design curves (Figures CH13-F112 through CH13-F116) have been developed from Bathurst's relationship to simplify riprap design for steep channels. The design curves were developed for channels with 2 to 1 side slopes and bottom widths of 0 feet, 6 feet, 10 feet, 14 feet, and 20 feet. The curves were terminated at the point where flow velocity exceeded 15 fps. A median rock diameter could be determined that would be stable at higher flows and velocities; however, rock durability at velocities greater than 15 fps becomes of greater concern.

For a given flow, channel slope, and channel width, Figures CH13-F112 through CH13-F116 will provide the median riprap size. When the channel slope is not provided by one of the design curves, linear interpolation is used to determine the riprap size. This is done by extending a horizontal line at the given flow through the curves with slopes bracketing the design slope. A curve at the design slope is then estimated by visual interpolation. The design D_{50} size is then chosen at the point that the flow intercepts the estimated design curve. Linear interpolation can also be used to estimate the D_{50} size for bottom widths other than those supplied in the figures.

For practical engineering purposes, the D_{50} size specified for the design should be given in 0.25-foot increments. The final minimum design size is determined using Table CH13-T107.

b) Riprap Gradation For Steep Slopes

Lack of proper riprap gradation is one of the most common causes of riprap failure. With the proper rock gradation, the voids formed by large stones are filled with smaller sizes in an interlocking fashion that prevents jets of water from contacting the underlying soil and ultimately eroding the soil supporting the riprap layer.

Table CH13-T108 provides ratios used to determine the D_{10}, D_{20}, and D_{max} rock sizes from the D_{50} rock size determined in the previous section. It is important to establish a smooth gradation from the largest to the smallest sizes to prevent large voids between rocks.

c) Riprap Thickness For Steep Slopes

For riprap linings on steep slopes, a thickness of 1.25 times the median rock size is recommended. The maximum resistance to the erosive forces of flowing water occurs when all rock is contained within the riprap layer thickness. Oversize rocks that protrude above the riprap layer reduce channel capacity and reduce riprap stability.

d) Riprap Placement On Steep Slopes

Improper placement is another major cause of failure in riprap-lined channels. To prevent segregation of rock sizes, riprap should never be placed by dropping it down the slope in a chute or pushing it down with a bulldozer. Rock can be dumped directly from trucks from the top of the embankment, and draglines with orange peel buckets, backhoes, and other power equipment can also be used to place riprap with minimal handwork.

e) Freeboard

Figures CH13-F112 through CH13-F116 also provide the depth of flow for a given flowrate, channel slope, and channel dimensions. The required freeboard is given by Equation CH13-115 for subcritical flow or CH13-122 for supercritical flow. The velocity can be estimated by dividing the flow rate by the area of flow.

f) Bedding Requirements on Steep Slopes

Either a granular bedding material or filter fabric may be used on steep slopes according to the requirements previously specified in Section 1.7.4.9.

1.7.5 CONCRETE-LINED CHANNELS

Concrete-lined channels are defined as rectangular or trapezoidal channels in which reinforced concrete is used for lining of the channel banks and channel bottom. The cost of concrete channels generally can be more economical than other lining types in an urban environment due to their greater flow carrying capacity resulting in less land area requirements. Special attentions should be taken to provide safety measures

(i.e. fence) around the concrete channels (Section 1.10.2, Chapter 13).

The following sections present design parameters for concrete-lined channels. The design parameters presented do not relieve the designer of performing appropriate engineering analyses.

1.7.5.1 LONGITUDINAL CHANNEL SLOPE

The maximum slope of concrete-lined channels is determined by the maximum permissible velocity requirements (Table CH13-T103). Concrete-lined channels have the ability to accommodate super-critical flow conditions and thus can be constructed to almost any naturally occurring slope.

1.7.5.2 ROUGHNESS COEFFICIENTS

The Manning's roughness coefficient for concrete-lined channels is as shown in Table CH13-T102. For concrete-lined channels with subcritical flow, check the Froude Number using a roughness coefficient of 0.011.

1.7.5.3 LOW FLOW CHANNEL

The bottom of the concrete channel should be constructed with a defined low flow channel but should be adequately sloped to confine the low flows to the middle or one side of the channel. Low flows are defined in Section 1.7.2.3, Chapter 13.

1.7.5.4 BOTTOM WIDTH

There are no bottom width requirements for concrete-lined channels.

1.7.5.5 FLOW DEPTH

There are no flow depth requirements for concrete-lined channels.

1.7.5.6 SIDE SLOPES

Concrete-lined channels may have side slopes that are vertical or flatter.

1.7.5.7 CONCRETE LINING SECTION

a) Thickness

All concrete lining should have a minimum thickness of 6 inches for flow velocities less than 30 fps and a minimum thickness of 7 inches for flow velocities of 30 fps and greater.

b) Concrete Joints

The following design standards, found to work in similar conditions, are suggested. Alternatives will be considered on a case-by-case basis.

- Channels should be continuously reinforced without transverse joints. Expansion/ contraction joints (without continuous reinforcement) should only be installed where the new concrete lining is connected to a rigid structure or to an existing concrete lining which is not continuously reinforced. The design of the expansion joint should be coordinated with the local officials.

- Longitudinal joints, where required, should be constructed on the sidewalls at least one foot vertically above the channel invert.

- All joints should be designed to prevent differential movement.

- Construction joints are required for all cold joints and where the lining thickness changes. Reinforcement should be continuous through the joint and the concrete lining should be thickened at the joint.

c) Concrete Finish

The surface of the concrete lining should be provided with a wood float finish unless the design requires additional finishing treatment. Excessive working or wetting of the finish should be avoided if additional finishing is required.

d) Concrete Curing

It is suggested that concrete-lined channels be cured by the application of a liquid membrane-forming curing compound (white pigmented) upon completion of the concrete finish. All curing should be completed in accordance with the standard specifications of the local government agency.

e) Reinforcement Steel

- Steel reinforcement should be a minimum grade - 40 deformed bars. Wire mesh should not be used.

- Ratio of longitudinal steel area to concrete cross sectional area should be greater than .0905 but not less than a #4 rebar placed at a 12-inch spacing. The longitudinal steel should be placed on top of the transverse steel.

- Ratio of transverse steel area to concrete cross sectional area should be greater than .0025 but not less than a #4 rebar placed at a 12-inch spacing.

- Reinforcing steel should be placed near the center of the section with a minimum clear cover of three inches adjacent to the earth.

- Additional steel should be added as needed. If a retaining wall structure is used, the structure should be designed by a registered structural engineer with structural design calculations submitted for review and approval.

f) Earthwork

As a minimum, the following areas should be compacted to at least 90 percent of maximum density as determined by ASTM 1557 (Modified Proctor). Additional requirements may be required by the geotechnical report.

- The 12 inches of subgrade immediately beneath concrete lining (both channel bottom and side slopes).

- Top 12 inches of maintenance road.

- Top 12 inches of earth surface within 10 feet of concrete channel lip.

- All fill material.

g) Bedding

A geotechnical report should be submitted which addresses the required bedding necessary for the specific concrete section under consideration.

h) Underdrain and Weepholes

The necessity for longitudinal underdrains and weepholes should be addressed in a geotechnical report submitted for the specific concrete channel section under consideration.

i) Concrete Cutoffs

A transverse concrete cutoff should be installed at the beginning and end of the concrete-lined section of channel and at a maximum spacing of 90 feet. The concrete cutoffs should extend a minimum of three feet below the bottom of the concrete slab and across the entire width of the channel lining. Longitudinal cutoffs, a minimum of 3 feet in depth, at top lining are required to ensure integrity of the concrete lining.

If the channel is continuously reinforced without transverse joints then a concrete cutoff is required to be incorporated into the expansion/concrete joint.

1.7.5.8 SPECIAL CONSIDERATION FOR SUPERCRITICAL FLOW

Supercritical flow in an open channel in an urbanized area creates hazards which the designer should take into consideration. Careful attention should be taken to insure against excessive waves which may extend down the entire length of the channel from only minor obstructions. Imperfections at joints may rapidly cause a deterioration of the joints, in which case a complete failure of the channel can readily occur. In addition, high velocity flow entering cracks or joints creates an uplift force by the conversion of velocity head to pressure head which can damage the channel lining.

Generally, there should not be a drastic reduction in cross section shape and diligent care should be taken to minimize the change in wetted area of the cross-section at bridges and culverts. Bridges and other structures crossing the channel should be anchored satisfactorily to withstand the full dynamic load which might be imposed upon the structure in the event of major debris plugging.

The concrete lining should be protected from hydrostatic uplift forces, which are often created by a high-water table or momentary inflow behind the lining from localized flooding. Generally, an underdrain will be required under and/or adjacent to the lining. The underdrain should be designed to be free draining. With supercritical flows, minor downstream obstructions do not create any backwater effect. Backwater computation methods are applicable for computing the water-surface profile or the energy gradient in channels having a supercritical flow; however, the computations should proceed in a downstream direction. The designer should take care to insure against the possibility of unanticipated hydraulic jumps forming in the channel.

1.7.6 OTHER CHANNEL LININGS

Other channel linings include all channel linings that are not discussed in the previous sections. These include composite-lined channels, which are channels in which two or more different lining materials are used (i.e. riprap bottom with concrete side slope lining). They also include gabions, soil cement linings, synthetic fabric and geotextile linings, preformed block linings, reinforced soil linings, and floodwalls (vertical walls constructed on both sides of an existing floodplain). The wide range of composite combinations and other lining types does not allow a discussion of all potential linings in this Manual. For those linings not discussed in this Manual, supporting documentation will be required to support the use of the desired lining. A guideline of some of the items which should be addressed in the supporting documentation is as follows:

a. Structural integrity of the proposed lining.

b. Interfacing between different linings.

c. The maximum velocity under which the lining will remain stable.

d. Potential erosion and scour problems.

e. Access for operations and maintenance.

f. Long term durability of the product under the extreme meteorological and soil conditions.

g. Ease of repair of damaged section.

h. Past case history (if available) of the lining system in other arid areas.

i. Potential groundwater mitigation issues (i.e. weepholes, underdrains, etc.)

These linings will be allowed on a case by case basis. The local community and/or the CWCB may reject the proposed lining system in the interests of operation, maintenance, and protecting the public safety.

1.8 ADDITIONAL HYDRAULIC DESIGN STANDARDS

Presented in this section are the hydraulic design standards for design of improved channels. The standards included herein are those standards that are the same for all improved channels. Standards which are specific to a lining type are included in the discussion for the specific lining under consideration.

1.8.1 SUBCRITICAL FLOW DESIGN STANDARDS

The following design standards are to be used when the design runoff in the channel is flowing in a Subcritical condition (F_r<0.8). Furthermore, all subcritical channels (F_r<0.8) should be designed with the limits as stated in Section 1.4.2, Chapter 13.

1.8.1.1 TRANSITIONS

For the purposes of this manual, subcritical transitions occur when transitioning one sub-critical channel section to another subcritical channel section (expansion or contraction) or when a subcritical channel section is steepened to create a super critical flow condition downstream (i.e. sloping spillway entrance). Several typical subcritical transition sections are presented in Figures CH13-F117 and CH13-F118. The warped transition section, although most efficient, should only be used in extreme cases where minimum loss of energy is required since the section is very difficult and costly to construct. Conversely, the square-ended transition should only be used when either a straight-line transition or a cylinder-quadrant transition cannot be used due to topographic constraints or utility conflicts.

a) Transition Energy Loss

The energy loss created by a contracting section may be calculated using the following equation:

$$H_t = K_{tc}\left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right) \quad \text{(Eq. CH13-111)}$$

Where H_t = Energy loss (feet)
K_{tc}= Transition coefficient - contraction
V_1 = Upstream velocity (feet per second)
V_2 = Downstream velocity (feet per second)
g = Acceleration of gravity (feet per second squared)

K_{tc} values for the typical transition sections are presented in Figure CH13-F118.

Similarly, the energy loss created by an expanding transition section may be calculated using the following equation:

$$H_t = K_{te}\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) \quad \text{(Eq. CH13-112)}$$

Where H_t = Energy loss (feet)
K_{te}= Transition coefficient - expansion
V_1 = Upstream velocity (feet per second)
V_2 = Downstream velocity (feet per second)
g = Acceleration of gravity (feet per second squared)

K_{te} values for the typical transition sections are also presented in Figure CH13-F118.

The energy loss in a contracting transition for straight-line or warped transitions is allowed to be partially or totally accommodated by sloping the transition channel bottom from the transition entrance to the exit.

b) Transition Length

The length of the transition section should be long enough to keep the streamlines smooth and nearly parallel throughout the expanding (contracting) section. Experimental data and performance of existing structures have been used to estimate the minimum transition length necessary to maintain the stated flow conditions. Based on this information, the minimum length of the transition section should be as follows:

$$L_t \geq 0.5 L_c (\Delta T_w) \quad \text{(Eq. CH13-113)}$$

Where L_t = Minimum transition length (feet)
L_c = Length coefficient (dimensionless)
ΔT_w = Difference in the top width of the normal water surface upstream and downstream of the transition (feet)

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For an approach flow velocity less than 12 feet per second, L_c = 4.5. This represents a 4.5 (length) to 1.0 (width) wall expansion or contraction with the angle of expansion or contraction of 12.5 degrees from the channel centerline. For an approach flow velocity equal to or greater than 12 feet per second, L_c = 10.0. This represents a 10.0 (length) to 1.0 (width) expansion or contraction with the angle of expansion or contraction of about 5.75 degrees from the channel centerline.

The transition length equation is not applicable to cylinder-quadrant or square-ended transitions.

1.8.1.2 SUPERELEVATION IN BENDS

Superelevation in bends is estimated from the following equations:

$$S_e = \frac{C V^2 T_w}{rg} \quad \text{(Eq. CH13-114)}$$

Where r = Radius of curvature (feet)
C = Superelevation coefficient (=0.5 for subcritical flow)
S_e = Superelevation water surface increase (feet)
T_w = Top width of the design water surface (feet)
V = Mean design velocity (feet per second)
g = Acceleration of gravity (feet per second squared)

Superelevation should be limited to a maximum of 1.0 feet, and the radius of curvature should conform to the requirements provided in Section 1.7.2.9, Chapter 13.

1.8.1.3 FREEBOARD

All subcritical channels should be constructed with a minimum freeboard determined as follows:

$$F_b = 0.5 + \frac{V^2}{2g} \quad \text{(Eq. CH13-115)}$$

Where F_b = Freeboard height (feet)
V = Mean design velocity (feet per second)
g = Acceleration of gravity (feet per second squared)

In no case should the freeboard be less than 1.0 foot. All channel linings should extend to the freeboard height plus the increase in water surface elevation due to superelevation.

1.8.2 SUPERCRITICAL FLOW DESIGN STANDARDS

The following design standards are to be used when the design runoff in the channel is flowing in a supercritical condition (F_r,>1.13). Furthermore, all supercritical channels should be designed within the limits as situated in Section 1.4.2, Chapter 13.

1.8.2.1 SUPER CRITICAL TRANSITIONS

The design of supercritical flow in a transition is much more complicated and requires more special attention than a subcritical transition design due to the potential damaging effects of the oblique jump which is created by the transition. The oblique jump results in cross waves and higher flow depths which can cause severe damage if not properly accounted for in the design. A simpler design analysis is to force a hydraulic jump (supercritical flow to subcritical flow). However, hydraulic jumps should also be carefully designed to assure the jump will remain where the jump is designed to occur. Hydraulic jumps should not be designed to occur in an erodible channel section but only within energy dissipation or drop structure. The design guidelines of these structures are presented in Chapter 13, Section 6.

a) Contracting Transitions

Presented in Figure CH13-F119 is an example of a supercritical contracting transition. As shown in this figure, the upstream flow is contracted from width b_1 to b_3 with a wall diffraction angle of θ. The oblique jump occurs at the points A and B where the diffraction angles start. Wave fronts generated by the oblique jumps on both sides propagate toward the centerline with a wave angle β_1. Since the flow pattern is symmetric, the centerline acts as if there was a solid wall that causes a subsequent oblique jump and generates a backward wave front toward the wall with another angle β_2. These continuous oblique jumps result in turbulent fluctuations in the water surface.

To minimize the turbulence, the first two wave fronts are designed to meet at the center and then end at the exit of the contraction. Using the contraction geometry, the length of the transition should be as follows:

$$L_t = \frac{b_1 - b_3}{2 \tan\theta} \qquad \text{(Eq. CH13-116)}$$

Where L_t = Transition length (feet)
b_1 = Upstream top width of flow (feet)
b_3 = Downstream top width of flow (feet)
θ = Wall angle as related to the channel centerline (degrees)

Using the continuity principle,

$$\frac{b_1}{b_3} = \left(\frac{Y_3^{1.5}}{Y_1}\right)\left(\frac{F_3}{F_1}\right) \qquad \text{(Eq. CH13-117)}$$

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Where Y_1 = Upstream depth of flow (feet)
Y_3 = Downstream depth of flow (feet)
F_1 = Upstream Froude Number
F_3 = Downstream Froude Number

Also, by the continuity and momentum principals, the following relationship between the Froude Number, wave angle, and wall angle is found to be:

$$\tan\theta = \frac{\tan\beta_1[(1+8F_1^2\sin^2\beta_1)^{1/2}-3]}{2\tan^2\beta_1+(1+8F_1^2\sin^2\beta_1)^{1/2}-1} \qquad \text{(Eq. CH13-118)}$$

Where β_1 = Initial wave angle (degrees)

Equations CH13-116, CH13-117, CH13-118 can be used by trial and error to determine the transition length and wall angle. However, Figure CH13-F120 is provided to allow a quicker trial and error solution than by using the equations. The procedure to determine the transition length and wall angle between two predetermined channel sections using Figure CH13-F120 is as follows:

Step 1: Determine the upstream and downstream channel flow conditions including flow depths, velocities, and Froude numbers.

Step 2: If either or both sections are trapezoidal, convert the trapezoidal flow parameters to equivalent rectangular flow parameters by calculating an equivalent flow width equal to the flow area divided by the flow depth. This computed flow width is used for all calculations.

Step 3: Compute Y_3/Y_1

Step 4: Assume a trial wall angle, θ

Step 5: Using θ and F_1, read the values of F_2 and Y_2/Y_1 for Section 1 from Figure CH6-F120. Then, replacing F_1 with F_2 read a second F_2 (really F_3) and second Y_2/Y_1 (really Y_3/Y_2) from Figure CH6-F120 for Section 2.

Step 6: Compute the first trial value of Y_3/Y_1 by multiplying the Y_2/Y_1 for Section 1 by the Y_2/Y_1 (really Y_3/Y_2) for Section 2.

Step 7: Compare the first trial Y_3/Y_1 to the actual Y_3/Y_1 (Step 3). If the trial value Y_3/Y_1 is larger than the actual Y_3/Y_1, assume a smaller θ and redo Steps 5 through 7. If the trial value Y_3/Y_1 is smaller than the actual Y_3/Y_1, assume a larger θ and redo Steps 5 through 7.

Step 8: Repeat the trial and error procedure until the computed Y_3/Y_1 is within the five percent of the actual Y_3/Y_1 .

Step 9: Compute the transition length using Equation CH13-121 and the last assumed value of θ.

Figure CH13-F120 can also be used to determine the wave angle, β, or may be used with the equations to determine the required downstream depth or width parameter if a certain transition length is desired or required.

To minimize the length of the transition section, Y_3/Y_1 should generally be between 2 and 3. However, F_3 should not be less than 1.7 for all transition designs. For further discussion on oblique jumps and supercritical contractions, refer to Chow, 1959.

b) Expanding Transitions

The goal of a properly designed expansion transition is to expand the flow boundaries at the same rate as the natural flow expansion. Based on experimental and analytical data results, the minimum length of a supercritical expansion should be as follows:

$$L_t \geq 1.5(\Delta T_w) F_{r1} \qquad \text{(Eq. CH13-119)}$$

Where L_t = Minimum transition length (feet)
ΔT_w = Difference in the top width of the normal water surface upstream and downstream of the transition
F_{r1} = Upstream Froude number

1.8.2.2 SUPERELEVATION IN BENDS

Bends in supercritical channels create cross waves and superelevated flow in the bend section as well as further downstream from the bend. In order to minimize these disturbances, the radius of curvature in the bend should not cause superelevation of the water surface exceeding two feet. Equation CH13-114 can be modified to determine the allowable radius of curvature of a channel for a given superelevation value. In no case should the radius of curvature be less than 50 feet.

$$r = \frac{C(V^2 T_w)}{(S_e\ g)} \qquad \text{(Eq. CH13-120)}$$

C should equal 1.0 for all trapezoidal channels and for rectangular channels without transition curves. For rectangular channels with transition curves, C should equal 0.5.

1.8.2.3 CIRCULAR TRANSITION CURVES

When a designer desires to reduce the required amount of freeboard and radius of curvature in a rectangular channel, a circular transition curve may be used. The length of the transition curve measured along the channel centerline should be determined as follows:

$$L_c = \frac{0.32\, T_w V}{y^{0.5}} \qquad \text{(Eq. CH13-121)}$$

Where L_c = Length of transition curve (feet)
T_w = Top width of design water surface (feet)
V = Mean design velocity (feet per second)
y = Depth of design flow (feet)

The radius of the transition curves should be twice the radius of the main bend. Transition curves should be located both upstream and downstream of the main bend.

1.8.2.4 FREEBOARD

In supercritical channels, adequate channel freeboard above the designed water surface should be provided and should not be less than that determined by the following:

$$F_b = 1.0 + 0.025\,V(d)^{1/3} \qquad \text{(Eq. CH13-122)}$$

Where F_b = Freeboard height (feet)
V = Velocity (feet per second)
d = depth of flow (feet)

Freeboard should be in addition to superelevation, standing waves, and/or other water surface disturbances.

The channel lining side slopes should be extended, as a minimum, to the freeboard elevation.

1.8.2.5 SLUG FLOW

Slug flow is a series of shallow-water shock waves that occur in steep super critical channels. The resulting wave heights may easily overtop channel linings using the typical freeboard requirements presented in this MANUAL or damage the channel lining. Therefore, all channels should be designed to avoid the occurrence of slug flow. To avoid slug flow when the Froude Number is greater than 2.0, the channel slope should be as follows:

$$S \le \frac{12}{R_e} \qquad \text{(Eq. CH13-123)}$$

Where S = Channel slope (feet per feet)
R_e =Reynolds Number = (VR/v) (Eq. CH13-124)
V = Mean design velocity (feet per second)
R = Hydraulic radius (feet)
v = Kinematic viscosity of water (feet squared per second)

Theoretically, slug flow will not occur with $F_r < 2.0$.

1.9 DESIGN STANDARDS FOR MINOR ARTIFICIAL DRAINAGEWAYS

A minor drainageway is defined as a channel/drainageway with a contributing tributary area of less than 160 acres. Additional flexibility and less stringent standards may be allowed for minor drainageways. Only the differences in a channel type's design as a minor drainageway versus that of a major drainageway are presented in this section.

A minor drainageway is defined as a channel/ drainageway with a contributing tributary area of less than 160 acres. Only the differences in a channel type's design as a minor drainageway versus that of a major drainageway are presented in this section.

1.9.1 GRASS-LINED CHANNELS

1.9.1.1 FREEBOARD

For swales and drainageways with a 100-year flow of equal to or less than 10 cfs, the minimum freeboard requirements is 6 inches.

1.9.1.2 CURVATURE (HORIZONTAL)

The minimum radius for channels with a 100-year runoff of 20 cfs or less should be 25 feet.

1.9.1.3 TRICKLE CHANNEL

For 100-year runoff peaks of 20 cfs or less, trickle channel requirements will be evaluated for each case. Trickle channels help preserve swales crossing residential property. Factors to be considered when establishing the need for trickle channels are: drainage slope, flow velocity, soil type, and upstream impervious area.

1.9.2 WETLAND BOTTOM CHANNELS

1.9.2.1 CURVATURE (HORIZONTAL))

The minimum radius for channels with a 100-year runoff of 20 cfs or less should be 25 feet.

1.9.3 CONCRETE-LINED CHANNELS

1.9.3.1 FREEBOARD

For swales and drainageways with a 100-year flow of equal to or less than 10 cfs, the minimum freeboard requirements is 6 inches.

1.9.3.2 CURVATURE (HORIZONTAL))

The minimum radius for channels with a 100-year runoff of 20 cfs or less should be 25 feet.

1.9.3.3 TRICKLE CHANNEL

For 100-year runoff peaks of 20 cfs or less, trickle channel requirements will be evaluated for each case. Trickle channels help preserve swales crossing residential property. Factors to be considered when establishing the need for trickle channels are: drainage slope, flow velocity, soil type, and upstream impervious area.

1.9.4 RIPRAP-LINED CHANNELS

1.9.4.1 FREEBOARD

For swales and drainageways with a 100-year flow of equal to or less than 10 cfs, the minimum freeboard requirements is 6 inches.

1.9.4.2 CURVATURE (HORIZONTAL)

The minimum radius for channels with a 100-year runoff of 20 cfs or less should be 25 feet.

1.9.4.3 TRICKLE CHANNEL

For 100-year runoff peaks of 20 cfs or less, trickle channel requirements will be evaluated for each case. Trickle channels help preserve swales crossing residential property. Factors to be considered when establishing the need for trickle channels are: drainage slope, flow velocity, soil type, and upstream impervious area.

1.10 CHANNEL APPURTENANCES

Presented in this section are the design standards for appurtenances to improved channels. All improved channels should be designed to include these appurtenances.

1.10.1 MAINTENANCE ACCESS ROAD

A maintenance access road with a minimum passage width of 12 feet should be provided along the entire length of all improved channels with 100-year design capacity equal to or greater than 50 cfs.

A maintenance access road with a minimum passage width of 12 feet should be provided along the entire length of all improved channels with 100-year design capacity equal to or greater than 50 cfs. For such channels with less than 50 feet in top width, one maintenance access should be provided as part of the channel improvements. For channels with greater than 50 feet in top width, the maintenance road should be located in or within 10 feet horizontal distance from the bottom of the channel or on both sides at the channel top.

For channels with the maintenance access road at or near the channel bottom, ramps to said road should be provided at a maximum 10 percent slope. Said ramps should slope down in the down gradient direction of the channel.

1.10.2 SAFETY REQUIREMENTS

The following safety requirements are required for concrete-lined channels. Similar safety requirements may be required for all other channels:

a. A six-foot high galvanized-coated chain link or comparable fence should be installed to prevent unauthorized access. The fence should be located at the edge of the ROW or on the top of the channel lining. Gates, with top latch, should be placed at major access points or 1,320-foot intervals, whichever is less.

b. Ladder-type steps should be installed not more than 1,200 feet apart and should be staggered on alternating sides of the channel to provide a ladder every 600 feet. The bottom rung should be placed approximately 12 inches vertically above the channel invert.

1.10.3 CULVERT OUTLET PROTECTION

If the flow velocity at a culvert or storm sewer outlet exceeds the maximum permissible velocity for the local soil or channel lining, channel protection is required. This protection usually consists of an erosion resistant reach, such as riprap, to provide a stable reach at the outlet in which the exit velocity is reduced to a velocity allowable in the downstream channel.

The following basin sizing procedure should be used for culvert sizes less than or equal to 36-inches in diameter or equivalent open area and outlet velocities less than 15 fps. For larger culverts or outlet velocities greater than 15 fps, the outlet protection design provided for in USDOT, 1983 should be used.

1.10.3.1 BASIN CONFIGURATION

The length of the outlet protection (L_a) is determined using the following empirical relationships that were developed for the U.S. Environmental Protection Agency (USEPA, 1976):

$$L_a = \frac{1.8\,Q}{D_o^{3/2}} + 7\,D_0 \,, \quad \text{for TW} < \frac{D_o}{2} \qquad \text{(Eq. CH13-125)}$$

and

$$L_a = \frac{3Q}{D_o^{3/2}} + 7\,D_o \,, \ \text{for TW} \geq \frac{D_o}{2} \qquad \text{(Eq. CH13-126)}$$

Where D_o = Maximum inside culvert width (ft)
Q = Pipe discharge (cfs)
TW = Tailwater depth (ft)

Where there is no well defined channel downstream of the apron, the width, W, of the outlet and of the apron (as shown in Figure CH13-F121) should be as follows:

$$W = 3\,D_o + 0.4\,L_a \,, \ \text{for TW} \geq \frac{D_o}{2} \qquad \text{(Eq. CH13-127)}$$

and

$$W = 3\,D_o + L_a \,, \ \text{for TW} < \frac{D_o}{2} \qquad \text{(Eq. CH13-128)}$$

The width of the apron at the culvert outlet should be at least 3 times the culvert width.

Where there is a well-defined channel downstream of the apron, the bottom width of the apron should be at least equal to the bottom width of the channel and the lining should extend at least one foot above the tailwater elevation and at least two-thirds of the vertical conduit dimension above the invert.

The apron side slopes should be 2:1 or flatter, and the bottom grade should be level.

1.10.3.2 ROCK SIZE

The median stone diameter, d_{50} is determined from the following equation:

$$d_{50} = 0.02\frac{(Q)^{4/3}}{TW(D_o)} \qquad \text{(Eq. CH13-129)}$$

Existing scour holes may be used where flat aprons are impractical. Figure CH13-F122 shows a general design of a scour hole. The stone diameter is determined using the following equations:

$$d_{50} = \frac{0.0125(Q)^{4/3}}{TW(D_o)}, \text{ for } Y = \frac{D_o}{2} \qquad \text{(Eq. CH13-130)}$$

Also,

$$d_{50} = \frac{0.0082(Q)^{4/3}}{TW(D_o)}, \text{ for } Y = D_o \qquad \text{(Eq. CH13-131)}$$

Where Y = depth of scour hole below culvert invert

The other riprap requirements are as indicated in the previous sections for channel lining.

1.10.4 LOW FLOW GRADE CONTROL STRUCTURES

1.10.4.1 INTRODUCTION

With the advent of floodplain management programs, developers and local governments frequently decided to preserve the floodplain. Since urbanization causes more frequent and sustained flows, the trickle/low flow channel becomes more susceptible to erosion even though the overall floodplain may remain stable and able to resist major flood events.

Erosion of the low flow channel, if left uncontrolled, can cause degradation and destabilization of the entire floodplain. Low flow check structures are designed to provide control points and establish stable bed slopes within the base flow channel. The check structures can be small versions of the drop structures described in Chapter 13, Section 6 or in many instances simply control sills across the floodplain. Low flow check structures are not appropriate in instances such as completely incised floodplains or very steep channels.

1.10.4.2 DROP STRUCTURE GRADE CONTROL STRUCTURES

The grouted sloping boulder drop structure and the vertical riprap drop structure designs can be adapted for use as check structures. The analysis steps are the same with the additional consideration of 1) stable bed slope for the unlined trickle or low flow channel and 2) potential overflow erosion during submergence of the check structure and where flow converges back from the main channel sides or below the check structure.

The basic design steps for this type of structure include the following:

a. Determine a stable slope and configuration for the low flow zone. For unlined channels, discharges from full floodplain flow to the dominant discharge should first be considered. The dominant discharge is more fully explained in sediment transport texts such as Simons, Li and Associates (1982).

b. The configuration of the low flow zone, and number and placement of the check structures has to be reviewed. Typically, the floodplain slope is steeper, often on the order of critical conditions. If the checks are widely spaced, the trickle channel depth can be quite deep downstream of the check, leading to concentration of higher flows into the trickle channel and the check. A good rule of thumb is to not have the trickle channel more than 2 feet deep at the crest of the check, or more than 4 feet deep below the check structure (relative to the overbank).

c. A hydraulic analysis should be performed using the discharge that completely fills the check structure at its crest (the primary design flow).

d. The secondary design flow is that flow which causes the worst condition for lateral overflow around the abutments and back into the basin or trickle channel below. The goal is to have the check structure survive such an event with minimal or reasonable damage to the floodplain below. The best approach is to estimate unit discharges, velocities and depths along overflow paths. The unit discharges can be estimated at the crest or critical section for the given total flow. Estimating the overflow path around the check abutment is difficult and requires practical judgment. Slopes can be derived for the anticipated overflow routes and protective measures devised such as grouted rock.

e. Seepage control is also important, as piping and erosion through or around these structures is a frequent problem. It is advisable to provide a cutoff which extends laterally at least 5 to 10 feet into undisturbed bank at minimum and has cutoff depth appropriate to the profile dimensions of the check.

1.10.4.3 CONTROL SILL GRADE CONTROL STRUCTURES

Another type of check structure that can be used to stabilize low flow channels within wide, relatively stable floodplains is the control sill shown in Figure CH13-F123. The sill can be constructed by filling an excavated trench with concrete, if soil conditions are acceptable for trenching, or forming a simple wall if a trench will not work.

The sill crosses the low flow channel and should extend a significant distance into the adjacent floodplain on both sides. The top of the sill conforms to the top of the ground at all points along its length. Riprap or other erosion control methods can then be added as erosion occurs.

The basic design steps are:

a. Determine a stable slope as described above.

b. Determine spacing of the sills based on the difference in slope between the natural and projected stable slope and the amount of future drop to be allowed (not to exceed 3 feet).

1.11 EXAMPLE APPLICATION

1.11.1 EXAMPLE

Problem:

An open channel is to be constructed for Doe Creek downstream of John Boulevard and north of Rose Subdivision.
Assume the following conditions for this problem.

Q_{100} = 191 cfs
Invert elevation downstream of John Boulevard = 4,918
Invert elevation downstream of Rose Subdivision = 4,917
Channel improvement length = 900 feet

Due to aesthetics and sufficient right-of-way, a grass-lined channel should be constructed.

Side Slope = z = 3
Bottom Width = b = 10 feet
n = 0.035 for grass-lined channel

Since the 100-year, 24-hour flow is less than 200 cfs, a trickle channel should be constructed in the proposed channel bottom.

Solution:

Step 1: Determine the depth of water during a 100-year flow event.

$$\text{Slope} = \frac{4918 - 4917}{900} = 0.0011 \text{ feet/ feet}$$

The Manning Equation can be re-written so that the depth of flow, y, in a trapezoidal channel is on one side of the equation.

$$\frac{(by + zy^2)^{5/3}}{\left(b + 2y(1 + z^2)^{1/2}\right)^{2/3}} = \left(\frac{Q}{S^{1/2}}\right)\left(\frac{n}{1.49}\right)$$

Solving by trial and error,

Y= 3.7 feet

Step 2: Calculate the water velocity in the proposed channel during a 100-year flow event using the Manning Equation.

$$V = \frac{1.49}{n} S^{1/2} R^{2/3}$$

$$= \left(\frac{1.49}{0.035}\right) * (.0011)^{1/2} * \left(\frac{(10 + 3 * 3.7) * 3.7}{10 + 2 * 3.7 * (1 + 3^2)^{1/2}}\right)^{2/3}$$

$$= 2.5 \text{ fps}$$

Since the water velocity of the proposed channel (2.5 fps) is less than the maximum permissible water velocity in a grass-lined channel, a grass-lined channel can be used at this location.

Step 3: Design the trickle channel.

Assume dimensions for a concrete trickle channel:

Bottom width = 5 feet
Depth = 1 foot
Side Slopes = vertical

The capacity of the trickle channel is:

$$Q = \left(\frac{1.49}{n}\right)\left(S^{1/2}\right)\left(R^{2/3}\right)(A)$$

$$Q = \left(\frac{1.49}{n}\right)\left(S^{1/2}\right)\left(\frac{by}{b + 2y}\right)^{2/3}(by)$$

$$Q = \left(\frac{1.49}{0.015}\right) * (.0011)^{1/2} * \left((5*1)/\left(5 + (2*1)\right)\right)^{2/3} * (5*1)$$

$$Q = 13.16 \text{ cfs}$$

Step 4: Verify that trickle channel has sufficient capacity.

The minimum capacity of the trickle channel is:

$$\text{Min. } Q_{T_c} = 0.05 * Q_{100}$$

$$\text{Min. } Q_c = 9.6 \text{ cfs}$$

CHAPTER 13
HYDRAULIC ANALYSIS AND DESIGN

SECTION 1
OPEN CHANNELS

Since the capacity of the proposed trickle channel (13.2 cfs) is greater than the required capacity (9.6 cfs), the proposed trickle channel is adequate.

Step 5: Determine the freeboard required for the proposed channel.

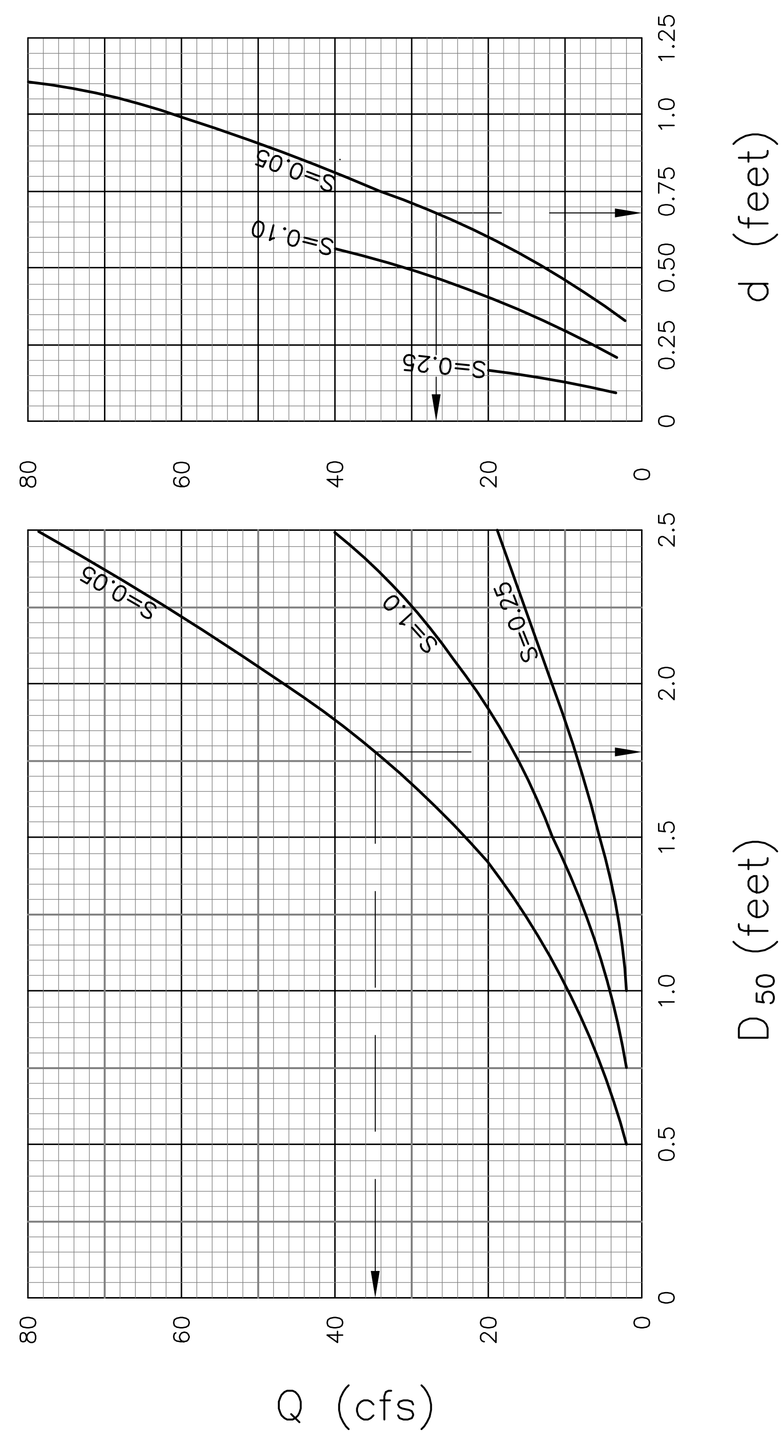
$$F_b = 0.5 + \frac{V^2}{2g}$$

$$F_b = 0.5 + \frac{(2.5)^2}{2 * 32.2} = 0.6 \text{ feet},$$

but minimum = 1.0 feet

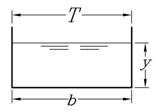
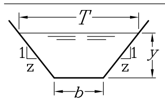
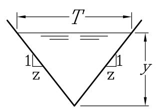
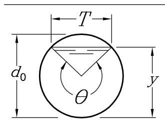
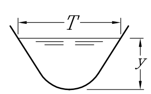
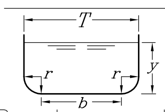
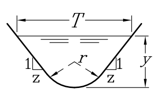
Therefore use F_b = 1.0 feet.

Step 6: The cross-section of the proposed channel is shown in Figure CH13-F124.



VERSION: JANUARY 2006	REFERENCE: Simons, Li and Assoc., 1989	**FIGURE CH13-F112** STEEP SLOPE RIPRAP DESIGN, TRIANGULAR CHANNEL, 2:1 SIDE SLOPES

GEOMETRIC ELEMENTS OF CHANNEL SECTIONS

SECTION	AREA, A	WETTED PERIMETER, P	HYDRAULIC RADIUS, R	TOP WIDTH, T	HYDRAULIC DEPTH, D	SECTION FACTOR, Z
 Rectangle	by	$b+2y$	$\frac{by}{b+2y}$	b	y	$by^{1.5}$
 Trapezoid	$(b+zy)y$	$b+2y\sqrt{1+z^2}$	$\frac{(b+zy)y}{b+2y\sqrt{1+z^2}}$	$b+2zy$	$\frac{(b+zy)y}{b+2zy}$	$\frac{[(b+zy)y]^{1.5}}{\sqrt{b+2zy}}$
 Triangle	zy^2	$2y\sqrt{1+z^2}$	$\frac{zy}{2\sqrt{1+z^2}}$	$2zy$	$\frac{y}{2}$	$\frac{\sqrt{2}}{2}zy^{2.5}$
 Circle	$\frac{1}{8}(\theta-\sin\theta)d_0^2$	$\frac{1}{2}\theta d_0$	$\frac{1}{4}\left(1-\frac{\sin\theta}{\theta}\right)d_0$	$(\sin\frac{1}{2}\theta)d_0$ or $2\sqrt{y(d_0-y)}$	$\frac{1}{8}\left(\frac{\theta-\sin\theta}{\sin\frac{1}{2}\theta}\right)d_0$	$\frac{\sqrt{2}}{32}\frac{(\theta-\sin\theta)^{1.5}}{(\sin\frac{1}{2}\theta)^{0.5}}d_0^{2.5}$
 Parabola	$\frac{2}{3}Ty$	$T+\frac{8}{3}\frac{y^2}{T}$ *	$\frac{2T^2y}{3T^2+8y^2}$ *	$\frac{3}{2}\frac{A}{y}$	$\frac{2y}{3}$	$\frac{2}{9}\sqrt{6}\,Ty^{1.5}$
 Round-cornered rectangle (y>r)	$\left(\frac{\pi}{2}-2\right)r^2+(b+2r)y$	$(\pi-2)r+b+2y$	$\frac{(\pi/2-2)r^2+(b+2r)y}{(\pi-2)r+b+2y}$	$b+2r$	$\frac{(\pi/2-2)r^2}{b+2r}+y$	$\frac{[(\pi/2-2)r^2+(b+2r)y]^{1.5}}{\sqrt{b+2r}}$
 Round-bottom triangle	$\frac{T^2}{4z}-\frac{r^2}{z}(1-z\cot^{-1}z)$	$\frac{T}{z}\sqrt{1+z^2}-\frac{2r}{z}(1-z\cot^{-1}z)$	$\frac{A}{P}$	$2[z(y-r)+r\sqrt{1+z^2}]$	$\frac{A}{T}$	$A\sqrt{\frac{A}{T}}$

*Satisfactory approximation for the interval $0<x\leq 1$, where $x=4y/T$. When $x>1$, use the exact expression $P=(T/2)[\sqrt{1+x^2}+1/x\ \ln(x+\sqrt{1+x^2})]$

REFERENCE:
Chow, V.T. Open-Channel Hydraulics, McGraw Hill Book Company, 1959.

TYPICAL ROUGHNESS COEFFICIENTS FOR OPEN CHANNELS

TYPE OF CHANNEL AND DESCRIPTION	MINIMUM	NORMAL	MAXIMUM
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 plans in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony 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 on 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			
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 as above, but highest state of flow	0.045	0.070	0.110
4. Dense brush, high state	0.080	0.100	0.140

TYPICAL ROUGHNESS COEFFICIENTS FOR OPEN CHANNELS

TYPE OF CHANNEL AND DESCRIPTION	MINIMUM	NORMAL	MAXIMUM
NATURAL STREAMS			
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, but lower stages, and 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			
1. Bottom: gravel, cobbles, and few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070
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	0.105	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
Major streams (top width at flood state 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

TYPICAL ROUGHNESS COEFFICIENTS FOR OPEN CHANNELS

TYPE OF CHANNEL AND DESCRIPTION	MINIMUM	NORMAL	MAXIMUM
LINED OR BUILT-UP CHANNELS			
a. Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Gunite, good section	0.016	0.019	0.023
4. Gunite, wavy section	0.018	0.022	0.023
b. Concrete bottom float finished with side of			
1. Dressed stone in mortar	0.015	0.017	0.020
2. Random stone in mortar	0.017	0.020	0.024
3. Dry rubble or riprap	0.020	0.030	0.035
c. 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
d. Asphalt			
1. Smooth	0.013	0.013	--
2. Rough	0.016	0.016	--
e. Grassed	0.030	0.040	0.050

REFERENCE:
Chow, V.T. Open-Channel Hydraulics, McGraw Hill Book Company, 1959.

TABLE CH13-T102C
TYPICAL ROUGHNESS COEFFICIENTS FOR OPEN CHANNELS

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MAXIMUM PERMISSIBLE MEAN CHANNEL VELOCITY

MATERIAL / LINING	MAXIMUM PERMISSIBLE MEAN VELOCITY (fps)
NATURAL & IMPROVED UNLINED CHANNELS	
Erosive Soils:	
Loams, Sands, Noncolloidal Silts	3.0
Less Erosive Soils:	
Clays, Shales, Cobbles, Gravel	5.0
FULLY LINED CHANNELS	
Unreinforced Vegetation	5.5
Loose Riprap	10.0
Grouted Riprap	15.0
Gibbons	15.0
Soil-Cement	15.0
Concrete	35.0

NOTES:

1. For composite lined channels, use the lowest of the maximum mean velocities for the materials used in the composite lining.
2. Deviations from the above values are only allowed with appropriate engineering analysis and/or suitable agreements for maintenance responsibilities.
3. Maximum permissible velocities based upon non-clear water conditions.

CLASSIFICATION AND GRADATION OF LOOSE RIPRAP

RIPRAP CLASS DESIGNATION	% SMALLER THAN GIVEN SIZE BY WEIGHT	RIPRAP GRADATION (Inches)	d_{50}* (Inches)
Class 150	100 35 - 50 0 - 15	10 6 2	6**
Class 300	100 35 - 50 0 - 15	20 12 4	12
Class 400	100 35 - 50 0 - 15	26 16 6	16
Class 550	100 35 - 50 0 - 15	37 22 8	22
Class 700	100 35 - 50 0 - 15	45 28 10	28
Class 900	100 35 - 50 0 - 15	57 35 14	35

*d_{50} = mean stone size

** Bury Class 150 riprap with native top soil and re-vegetate to protect from vandalism

REFERENCE:
Draft, State of Nevada, Department of Transportation Standard Specifications for Road and Bridge Construction, 1996

TABLE CH13-T104
CLASSIFICATION AND GRADATION OF LOOSE RIPRAP

GRADATION FOR GRANULAR RIPRAP BEDDING

RIPRAP DESIGNATION	GRANULAR BEDDING SIEVE SIZE (MM)	GRANULAR BEDDING PERCENT PASSING BY WEIGHT
Class 150	37.5	100
	19	35 - 100
	12.5	15 - 80
	9.5	5 - 60
	4.75	0 - 35
	1.18	0 - 5
Class 300	100	100
	37.5	30 - 100
	25	15 - 80
	12.5	0 - 50
	4.75	0 - 20
	2.36	0 - 5
Class 400	125	100
	50	30 - 100
	37.5	20 - 80
	19	0 - 45
	6.3	0 - 20
	4.75	0 - 10
Class 550	150	100
	75	35 - 100
	50	15 - 80
	25	0 - 50
	12.5	0 - 30
	6.3	0 - 10
Class 700	200	100
	75	25 - 85
	50	5 - 70
	19	0 - 40
	9.5	0 - 15
	6.3	0 - 5
Class 900	250	100
	100	25 - 90
	75	15 - 75
	25	0 - 35
	12.5	0 - 15
	6.3	0 - 5

REFERENCE:
Draft State of Nevada, Department of Transportation, Standard Specifications for Road and Bridge Construction, 1996

TABLE CH13-T105
GRADATION FOR GRANULAR RIPRAP BEDDING

CLASSIFICATION AND GRADATION OF ROCK FOR GROUTED RIPRAP

RIPRAP DESIGNATION	% SMALLER THAN GIVEN SIZE BY WEIGHT	INTERMEDIATE ROCK DIMENSION (Inches)
Class 400	100	26
	35 - 50	16
	0 - 5	12
Class 550	100	37
	35 - 50	22
	0 - 5	16
Class 700	100	45
	35 - 50	28
	0 - 5	20
Class 900	100	57
	35 - 50	35
	0 - 5	28

DESIGN D_{50} VALUES

D_{50} DETERMINED FROM DESIGN CURVE (FT)	MINIMUM DESIGN D_{50} (FT)
< 0.25	0.25
0.26 - 0050	0.50
0.51 - 0.75	0.75
0.76 - 1.00	1.00
1.01 - 1.25	1.25
1.26 - 1.50	1.50
1.51 - 1.75	1.75
1.76 - 2.00	2.00
2.01 - 2.25	2.25
2.26 - 2.50	2.50
2.51 - 2.75	2.75
2.76 - 3.00	3.00

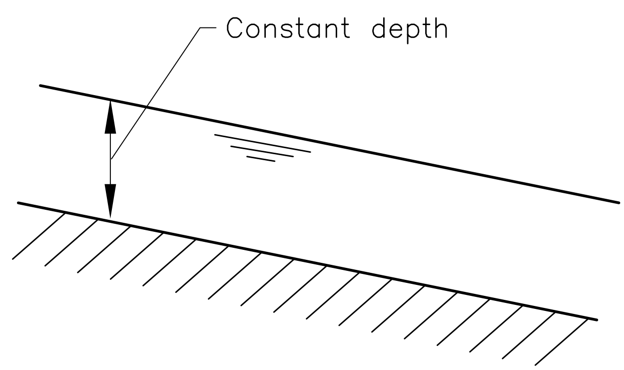
RIPRAP GRADATION FOR STEEP SLOPES

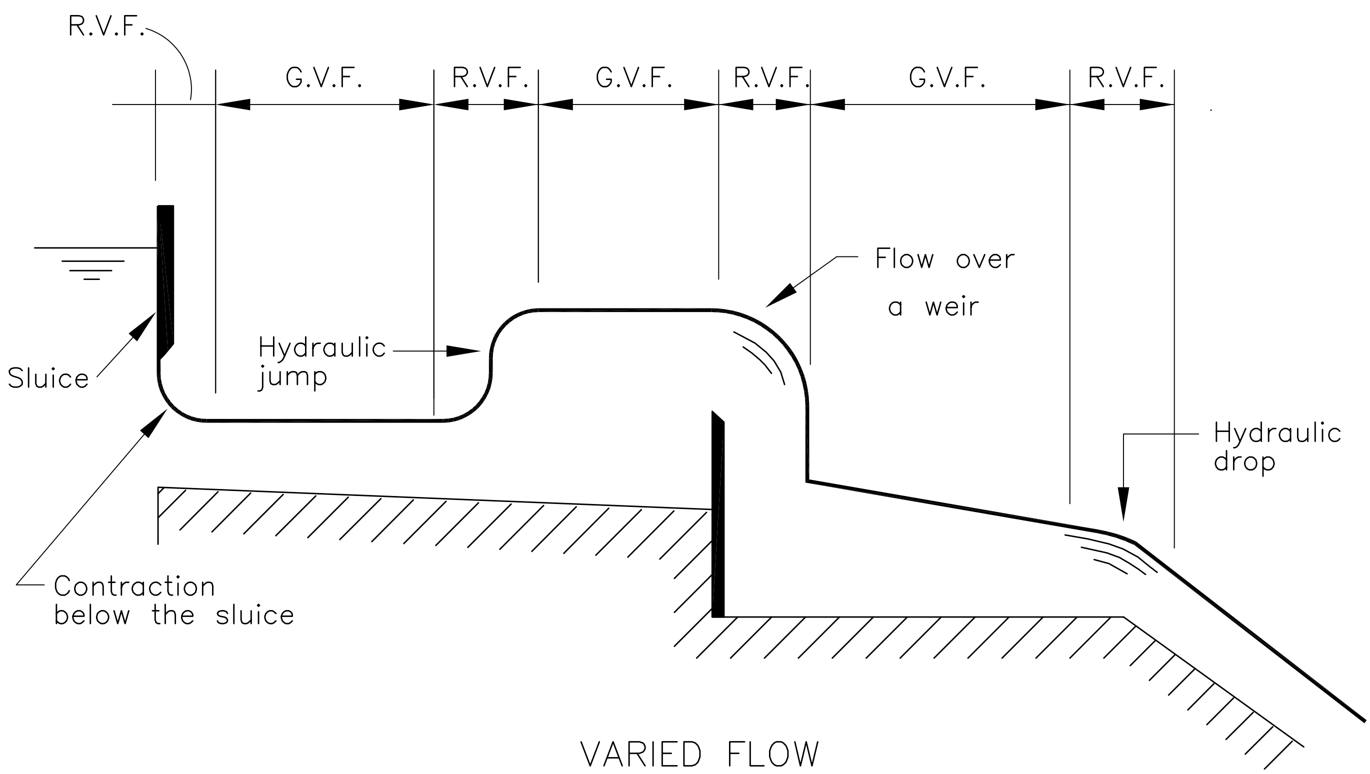
$$\frac{D_{MAX}}{D_{50}} = 1.25$$

$$\frac{D_{50}}{D_{20}} = 2.0$$

$$\frac{D_{50}}{D_{10}} = 3.0$$

REFERENCE:
Simons, 1989



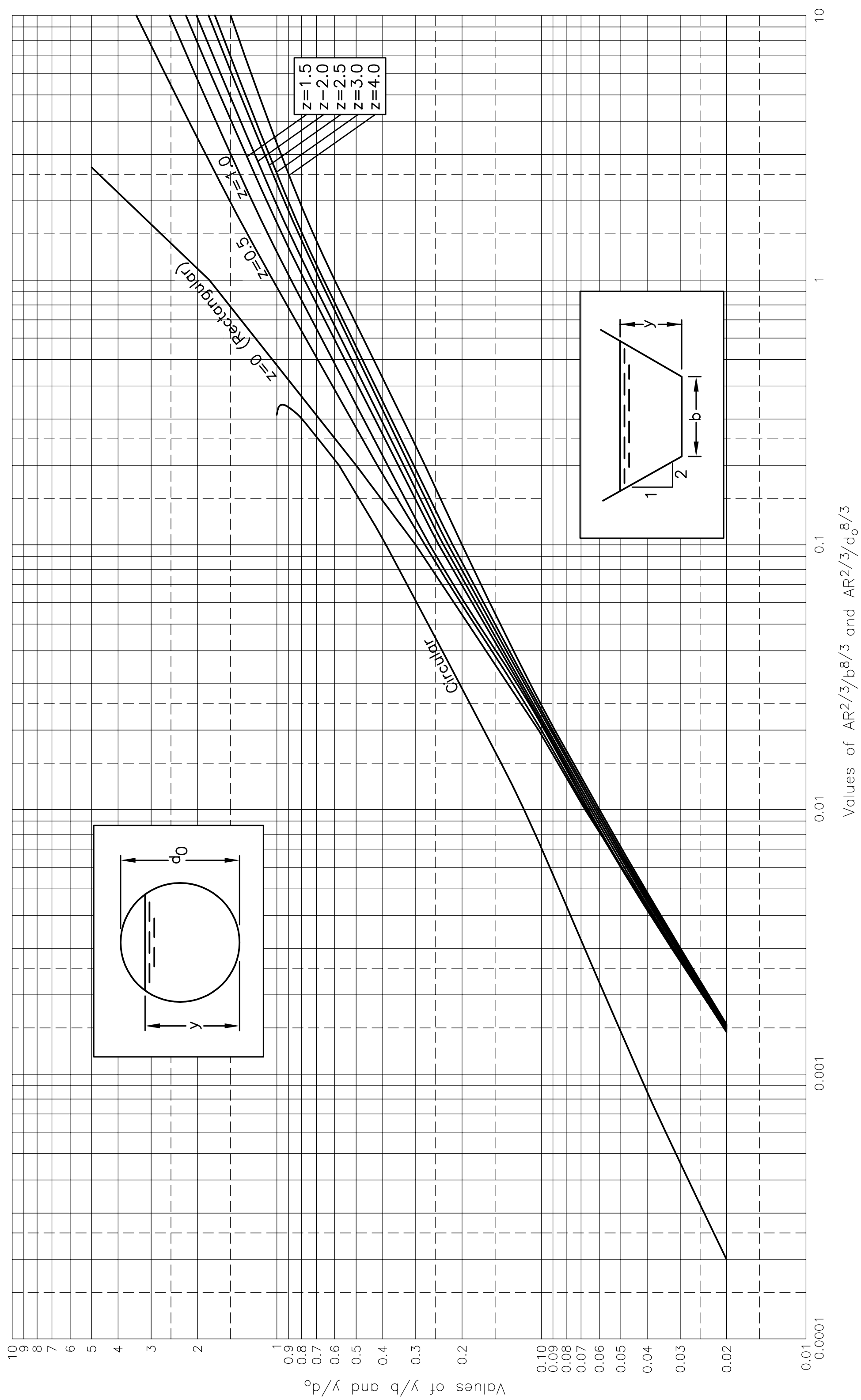


G.V.F. – Gradually Varying Flow
R.V.F. – Rapidly Varying Flow

VERSION: JANUARY 2006

REFERENCE:
Chow, V.T., Open-Channel Hydraulics
McGraw Hill Book Company 1959

FIGURE CH13-F101
OPEN-CHANNEL FLOW CONDITIONS



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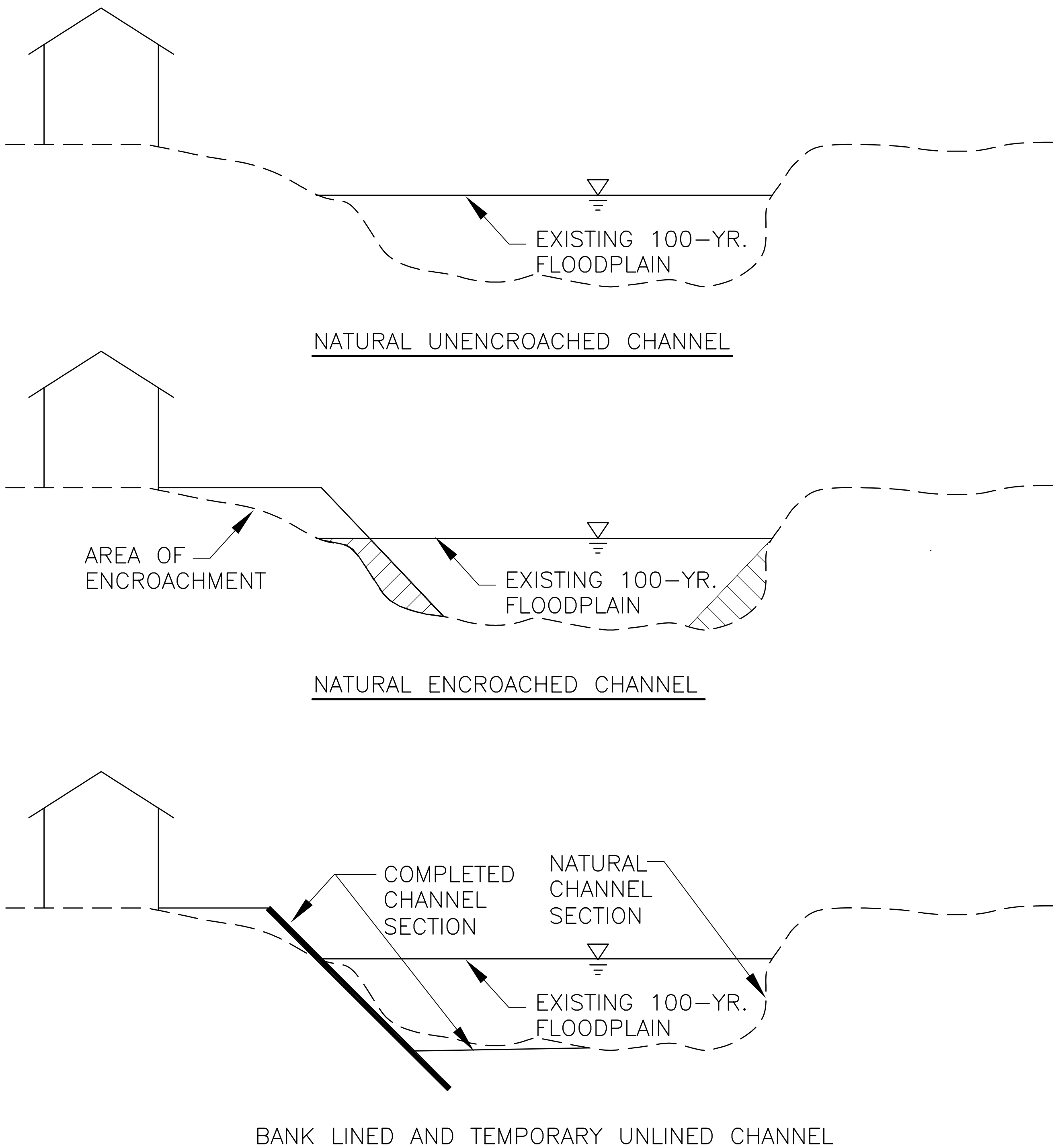
VERSION: JANUARY 2006

REFERENCE:
Chow, V.T., Open-Channel Hydraulics
McGraw Hill Book Company 1959

FIGURE CH13-F102
CRITICAL DEPTH FOR TRAPEZOIDAL AND CIRCULAR SECTIONS



O:/2120/FIGURES/CHAP13-1.DWG, CH13-F103 – 1/6/06 – GPB

VERSION: JANUARY 2006	REFERENCE:	**FIGURE CH13-F103** TYPICAL OPEN-CHANNEL DESIGN SECTIONS (NATURAL CHANNELS)

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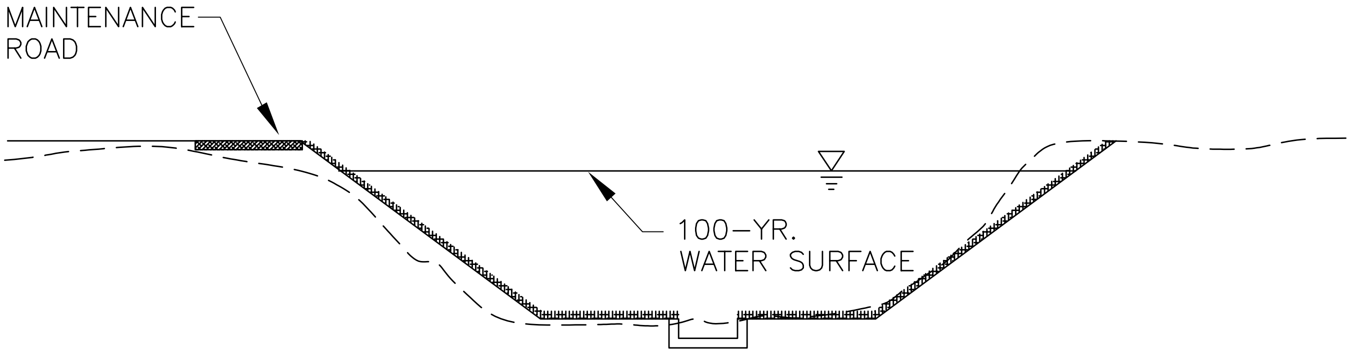
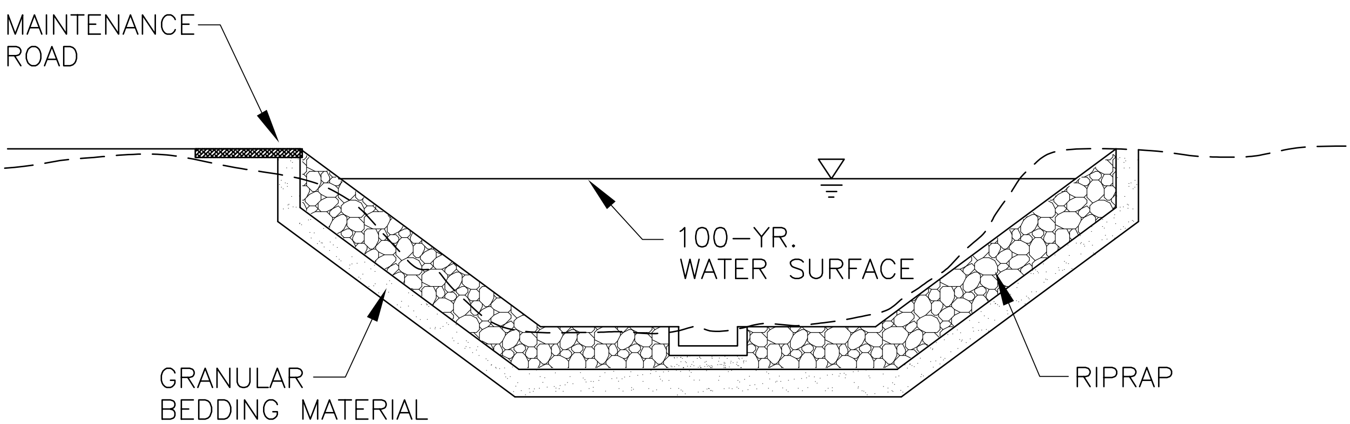


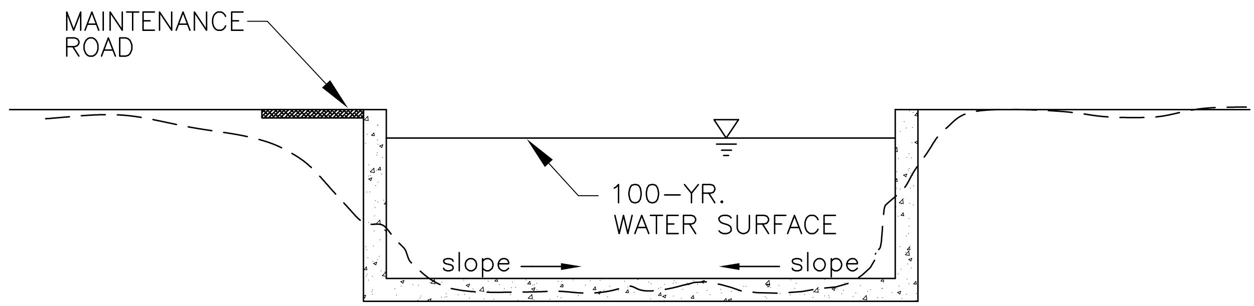
UNLINED AND GRASS-LINED CHANNEL



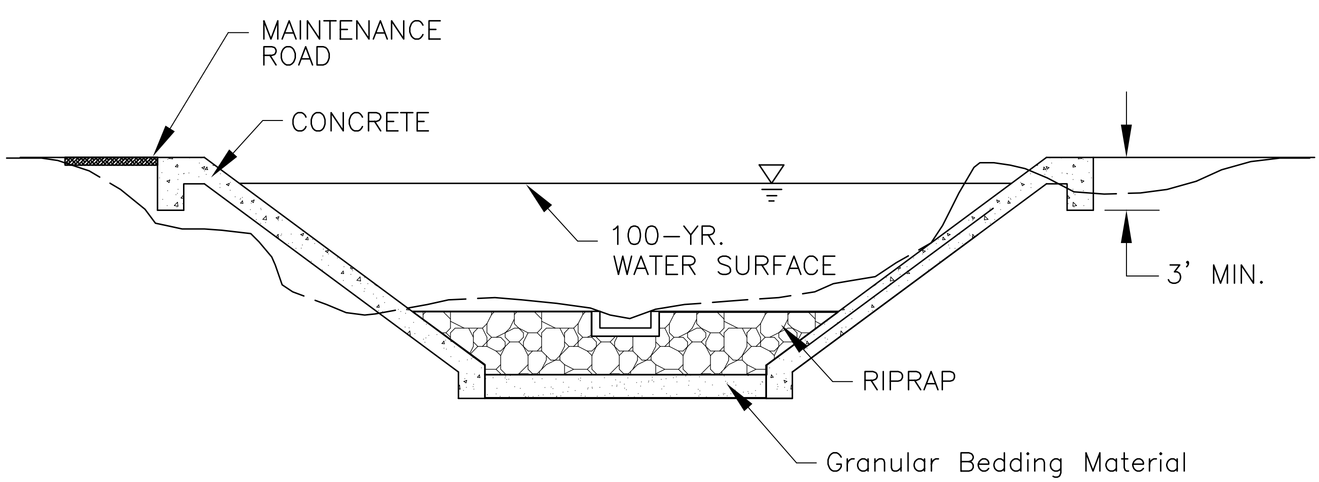
RIPRAP-LINED CHANNEL

O:/2120/FIGURES/CHAP13-1.DWG, CH13-F104A - 1/6/06 - GPB

VERSION: JANUARY 2006	REFERENCE:	**FIGURE CH13-F104A** TYPICAL OPEN-CHANNEL DESIGN SECTIONS (IMPROVED CHANNELS)

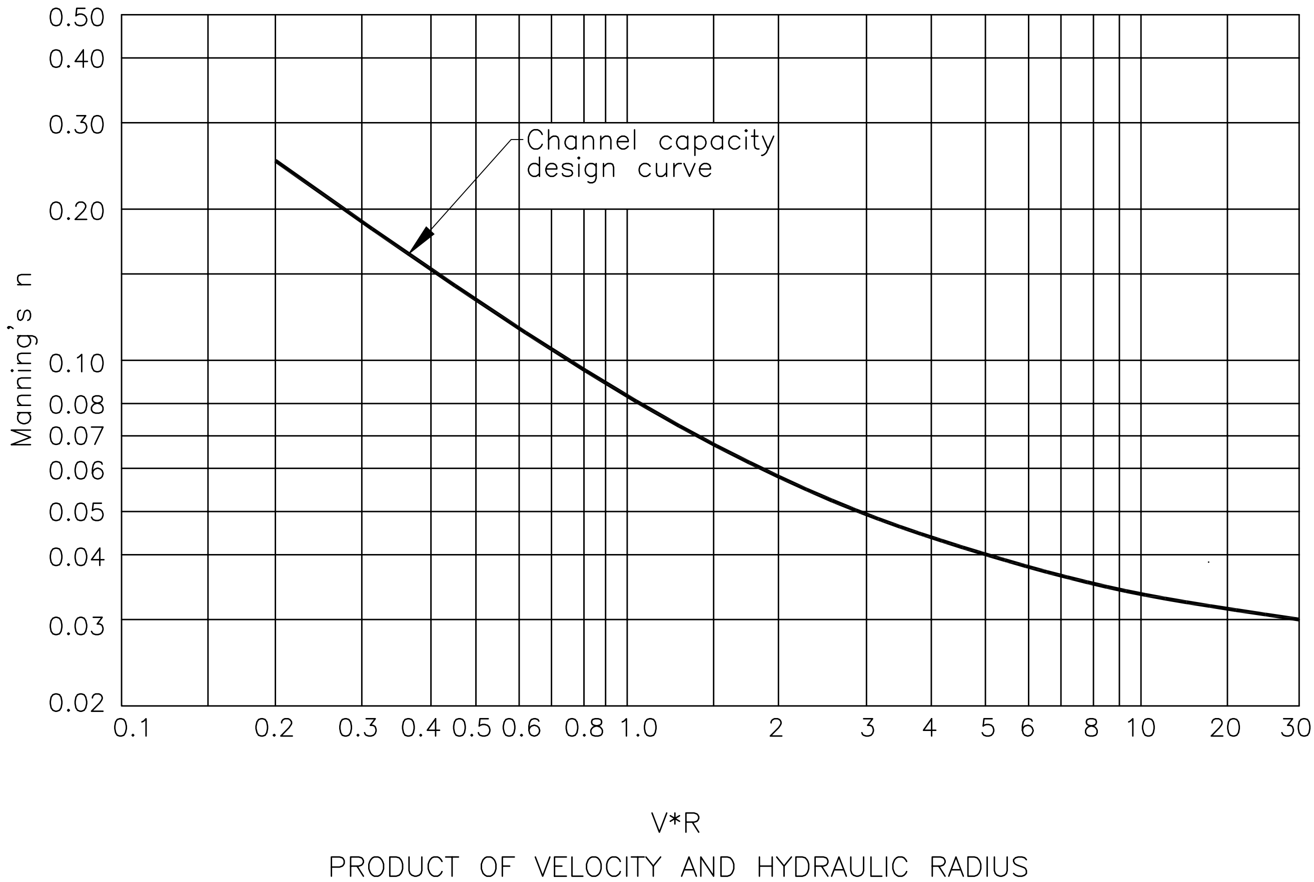


CONCRETE-LINED CHANNEL



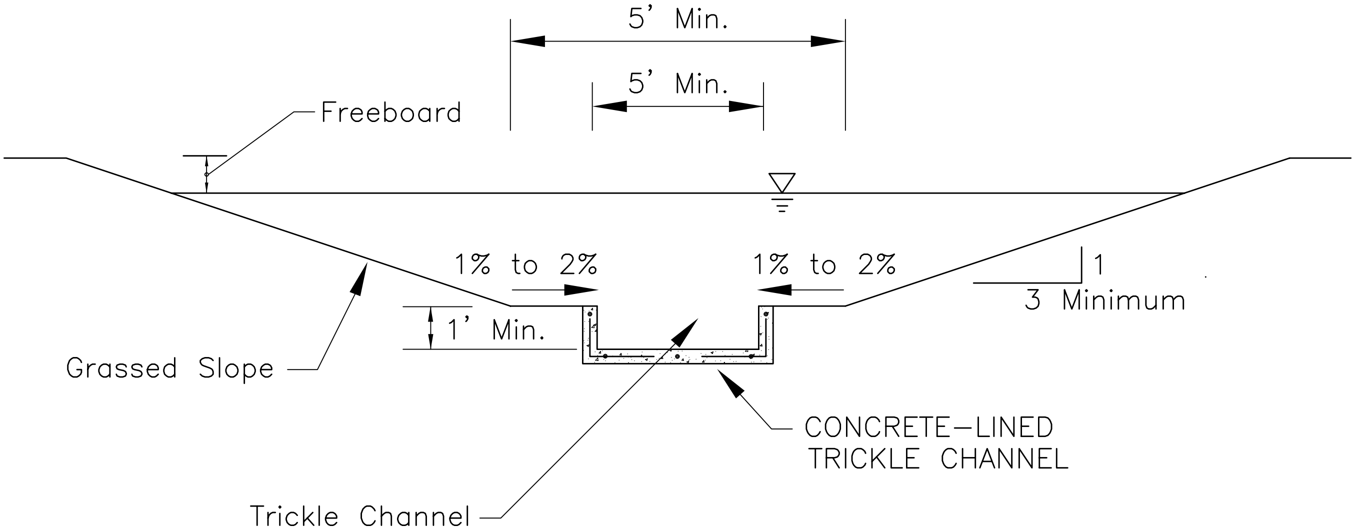
COMPOSITE-LINED CHANNEL

O:/2120/FIGURES/CHAP13-1.DWG, CH13-F104B - 1/6/06 - GPB



From "Handbook of Channel Design For Soil and Water Conservation," U.S. Department of Agriculture, Soil Conservation Service, No. SCS–TP–61 March, 1947, Rev. June, 1954

O:/2120/FIGURES/CHAP13-1.DWG, CH13-F105 – 1/6/06 – GPB

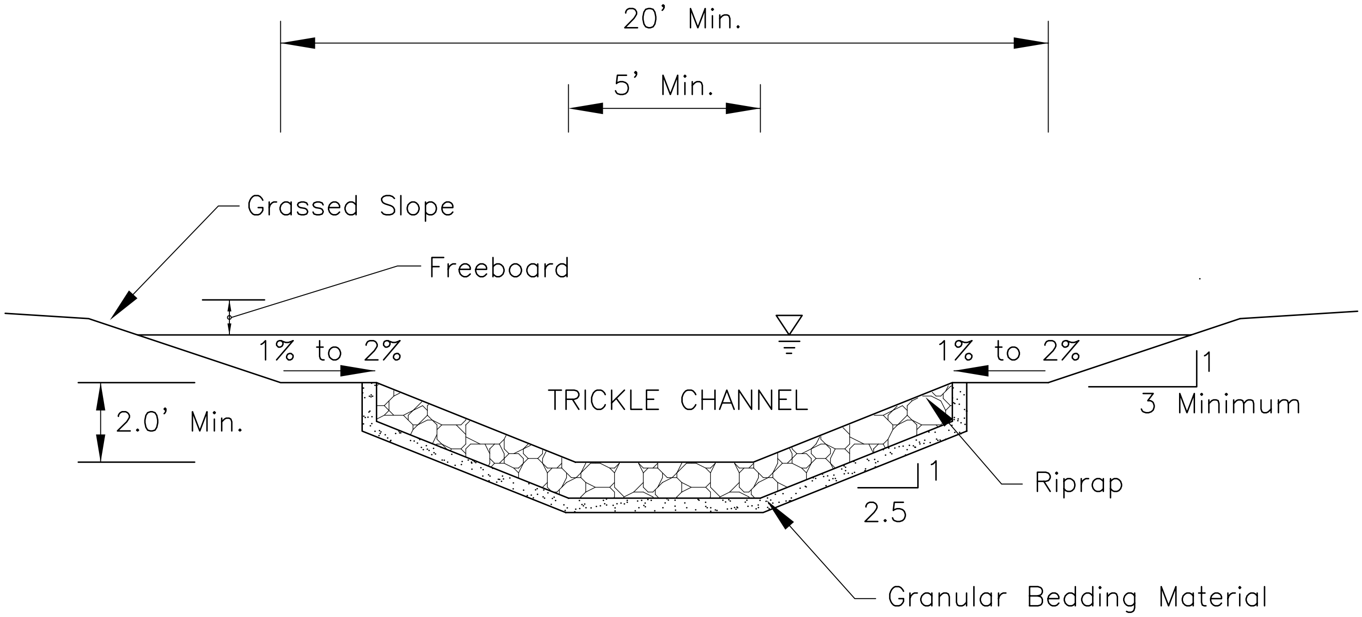


O:/2120/FIGURES/CHAP13-1.DWG, CH13-F106 -1/6/06 - GPB

REFERENCE:

FIGURE CH13-F106

TYPICAL CROSS-SECTION OF CONCRETE-LINED TRICKLE CHANNEL



VERSION: JANUARY 2006	REFERENCE:	**FIGURE CH13-F107** TYPICAL CROSS-SECTION OF RIPRAP-LINED TRICKLE CHANNEL

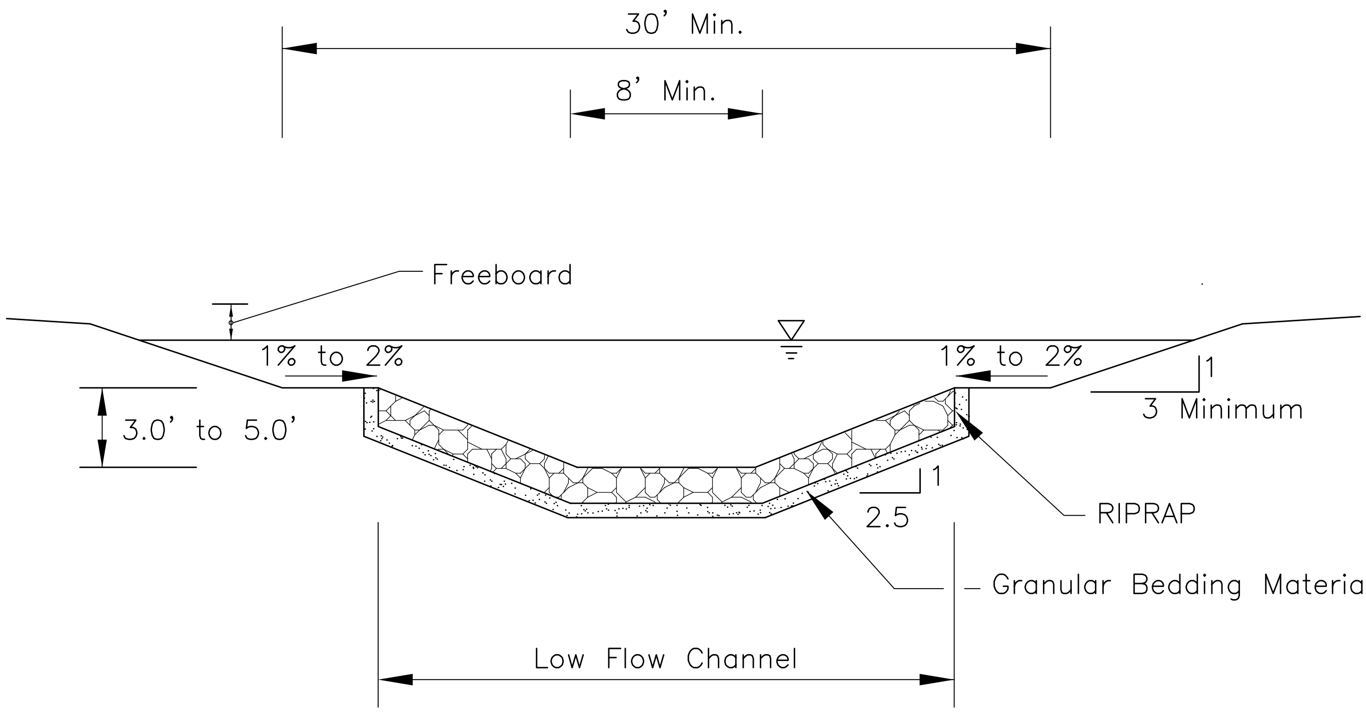
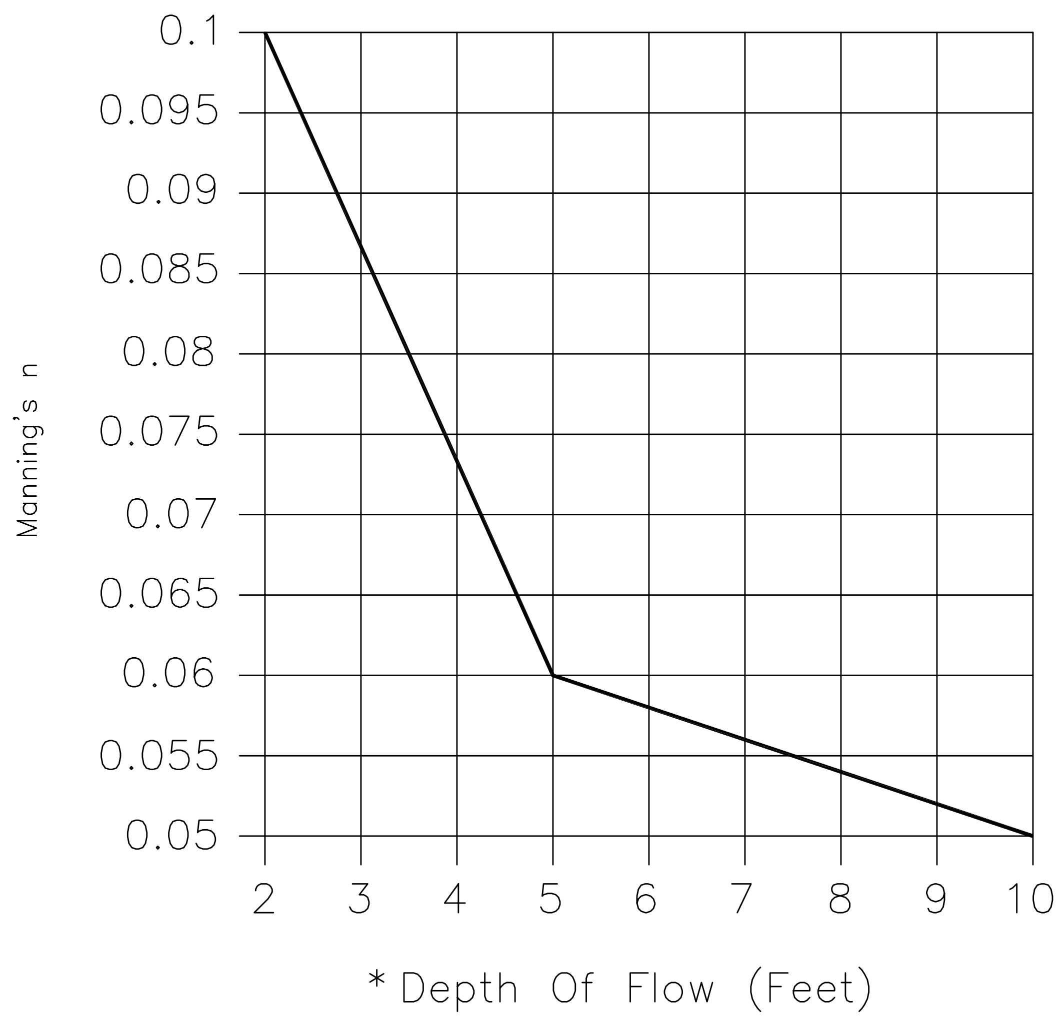


FIGURE CH13-F108

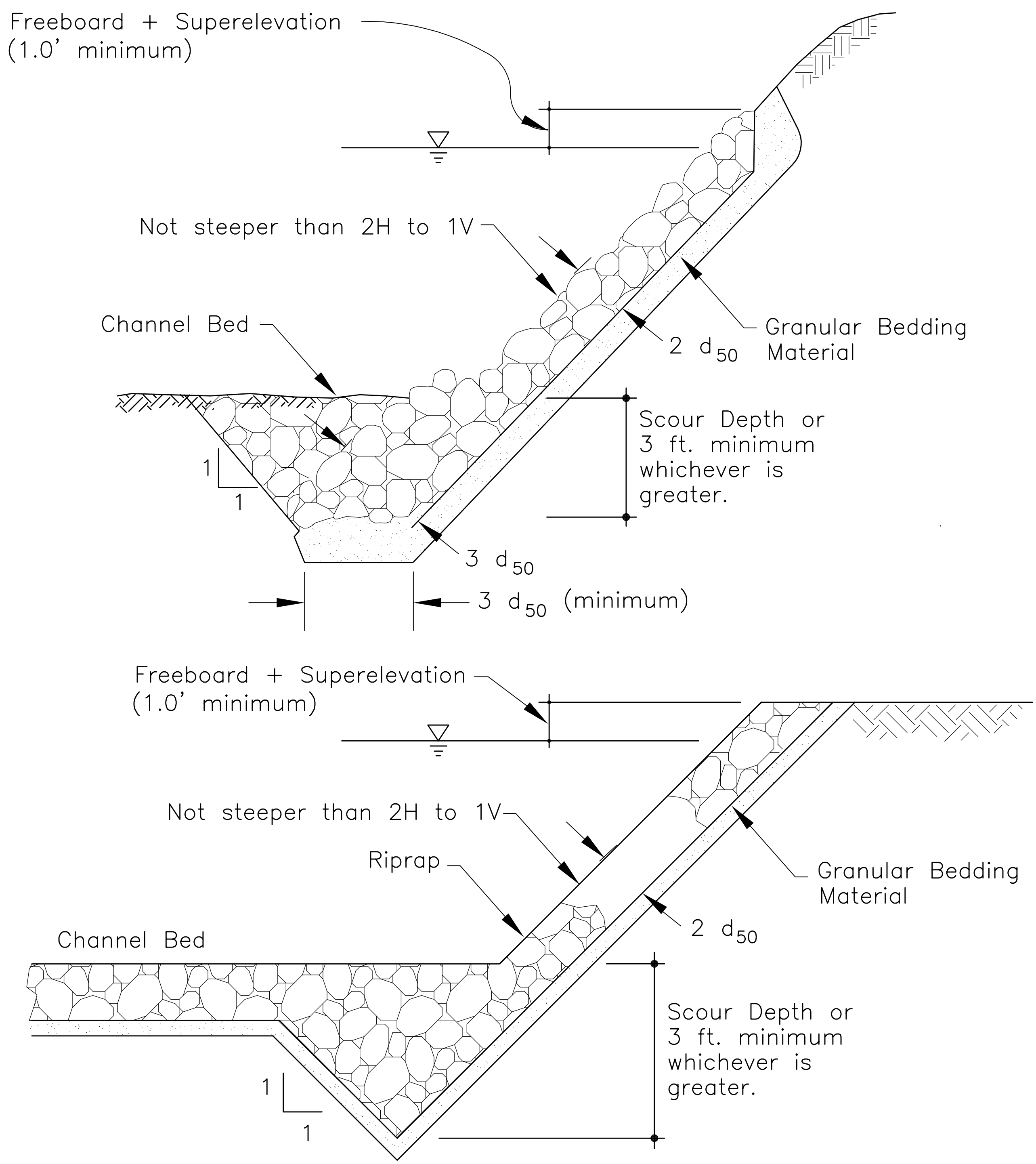
TYPICAL CROSS-SECTION OF LOW-FLOW CHANNEL



* Use normal depth, ignoring all backwater effects

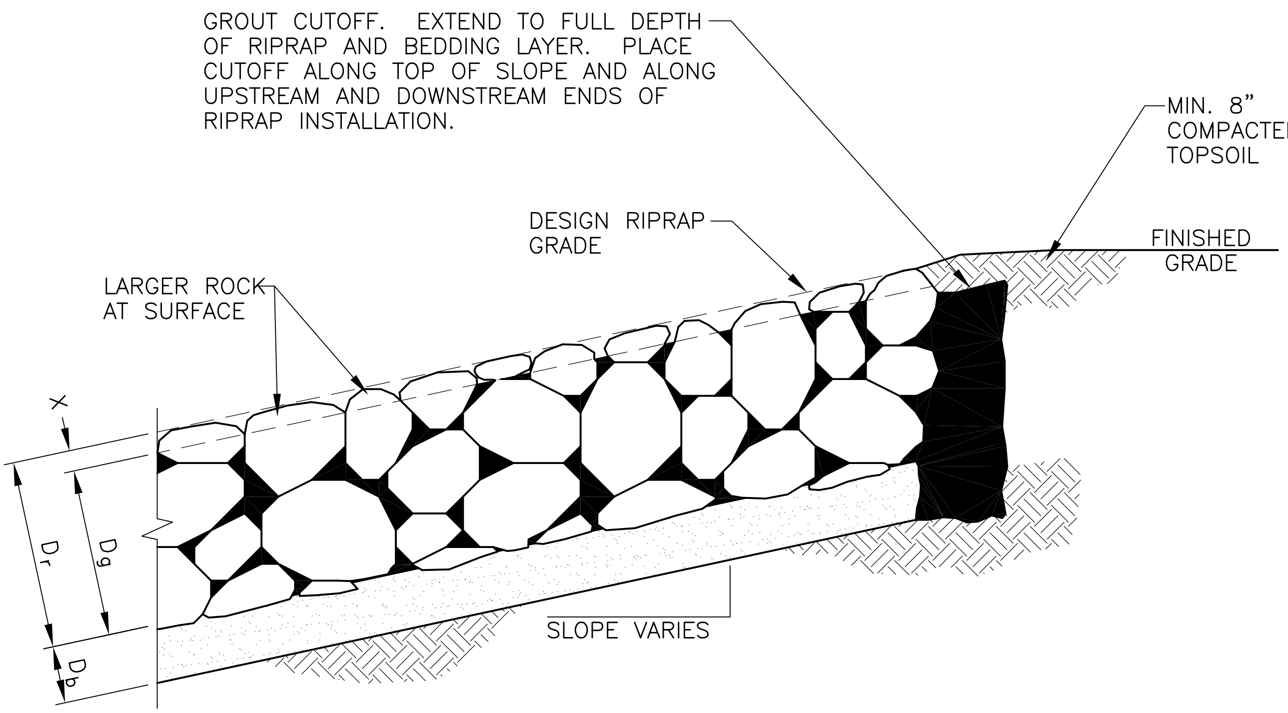
O:/2120/FIGURES/CHAP13-1.DWG, CH13-F109 - 1/6/06 - GPB

FIGURE CH13-F109
MANNINGS ROUGHNESS COEFFICIENT FOR WETLAND BOTTOMS



VERSION: JANUARY 2006	**REFERENCE:** **Modified from H.E.C. No. 11, USDOT, 1989**	**FIGURE CH13-F110** **TYPICAL CROSS-SECTIONS FOR RIPRAP-LINED CHANNELS**

O:/2120/FIGURES/CHAP13-1.DWG, CH13-F110 - 1/6/06 - GPB



LEGEND:

D_b = DEPTH OF BEDDING MATERIAL

D_g = DEPTH OF GROUT LAYER

D_r = DEPTH OF RIPRAP LAYER

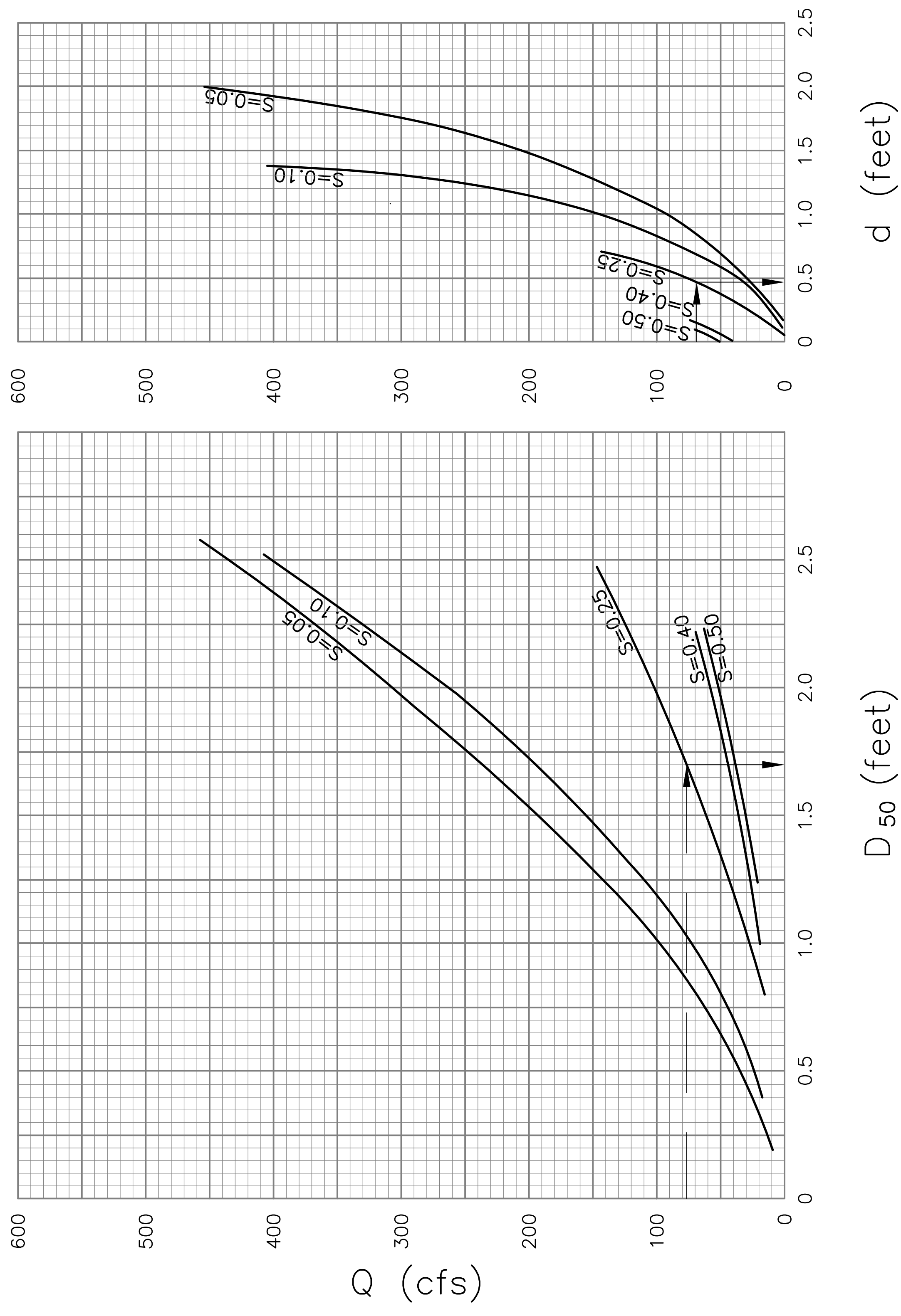
X = DEPTH FROM RIPRAP SURFACE TO GROUT SURFACE

NOTES:

1. FINAL PLACEMENT OF RIPRAP TO BE APPROVED BY ENGINEER PRIOR TO GROUTING.
2. BEFORE GROUTING, CLEAN ALL DIRT AND MATERIALS FROM ROCK THAT COULD PREVENT THE GROUT FROM BONDING TO ROCK.
3. PLACE GROUT BY INJECTION METHODS AND USE A PENCIL VIBRATOR TO FILL VOIDS TO THE SPECIFIED GROUT DEPTH. CLEAN EXCESS GROUT FROM ALL EXPOSED SURFACES. PROVIDE A BROOM FINISH FOR GROUT SURFACE.
4. THE CONTRACTOR SHALL CONTROL GROUT MIX AND PLACEMENT PROCEDURES TO ACHIEVE THE SPECIFIED THICKNESS, PENETRATION AND GRADE OF THE GROUT LAYER.

FIGURE CH13-F111

TYPICAL CROSS-SECTION FOR GROUTED RIPRAP LINING



d (feet)
0
0.5
1.0
1.5
2.0
2.5
S=0.05
S=0.10
S=0.25
S=0.40
S=0.50
D 50 (feet)
Q (cfs)
0
100
200
300
400
500
600

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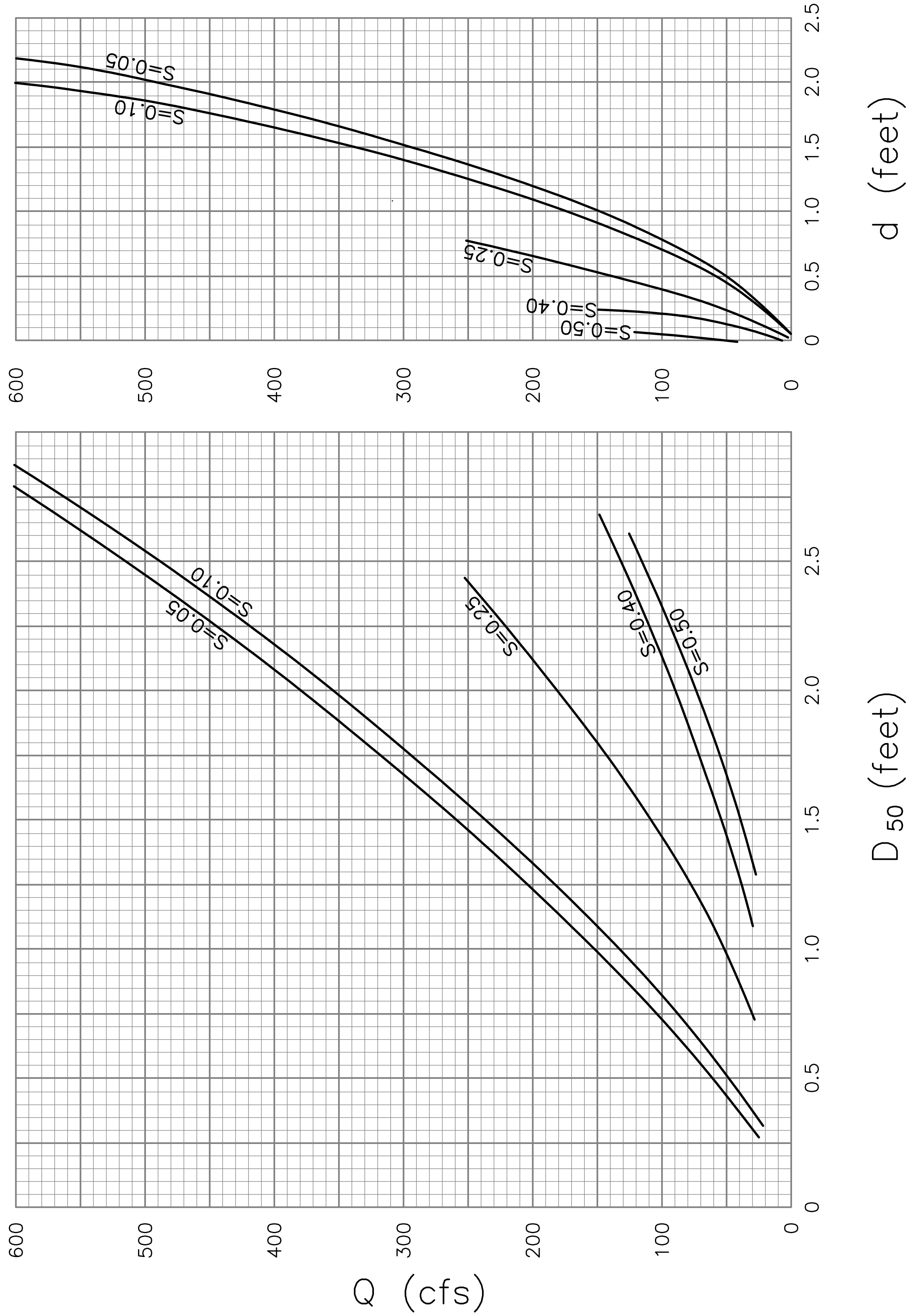
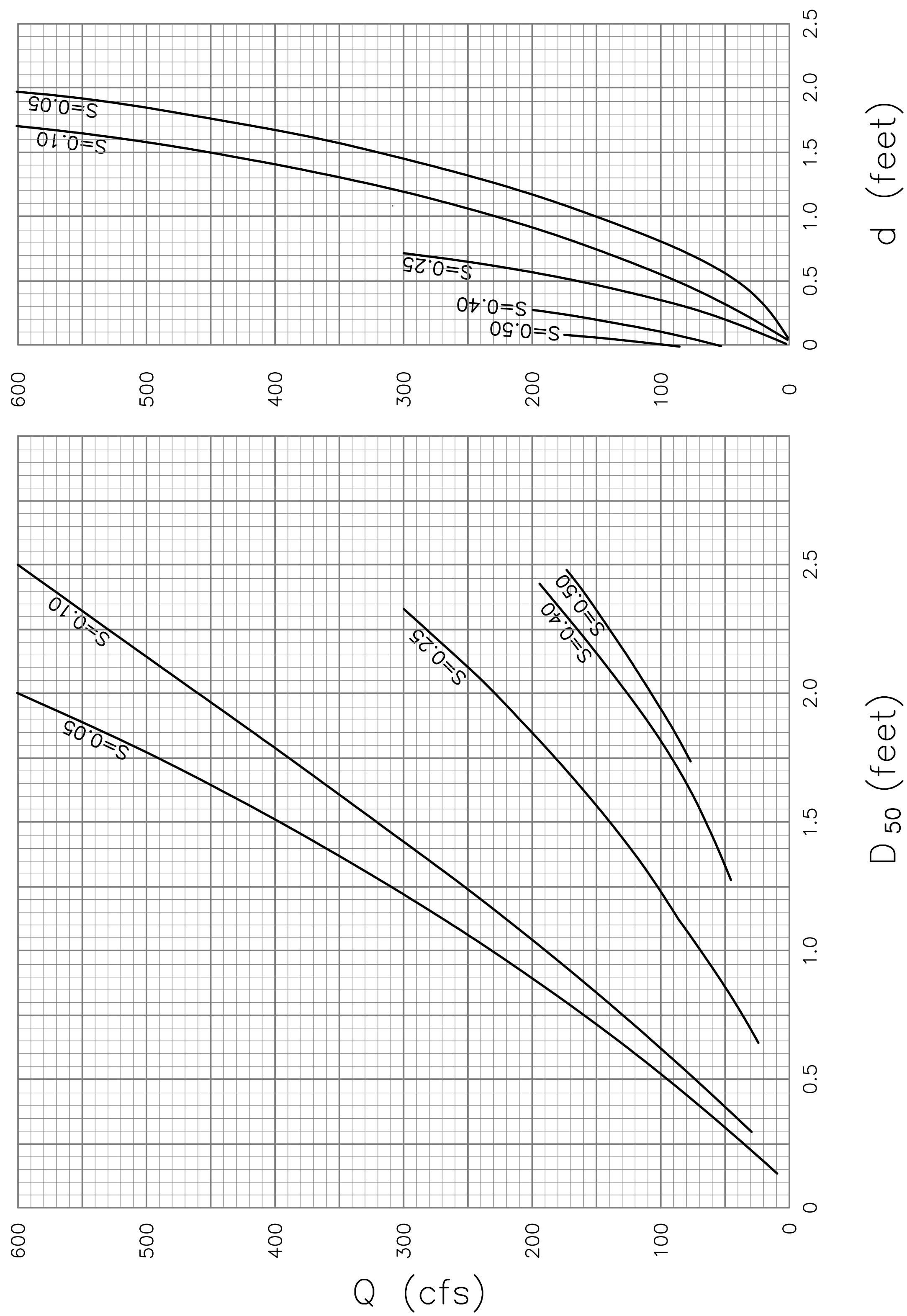


VERSION: JANUARY 2006	REFERENCE: Simons, Li and Assoc., 1989	**FIGURE CH13-F114** STEEP SLOPE RIPRAP DESIGN, TRAPEZOIDAL CHANNEL, 2:1 SIDE SLOPES, 10-FOOT BOTTOM WIDTH



VERSION: JANUARY 2006	REFERENCE: Simons, Li and Assoc., 1989	FIGURE CH13-F115 STEEP SLOPE RIPRAP DESIGN, TRAPEZOIDAL CHANNEL, 2:1 SIDE SLOPES, 14-FOOT BOTTOM WIDTH

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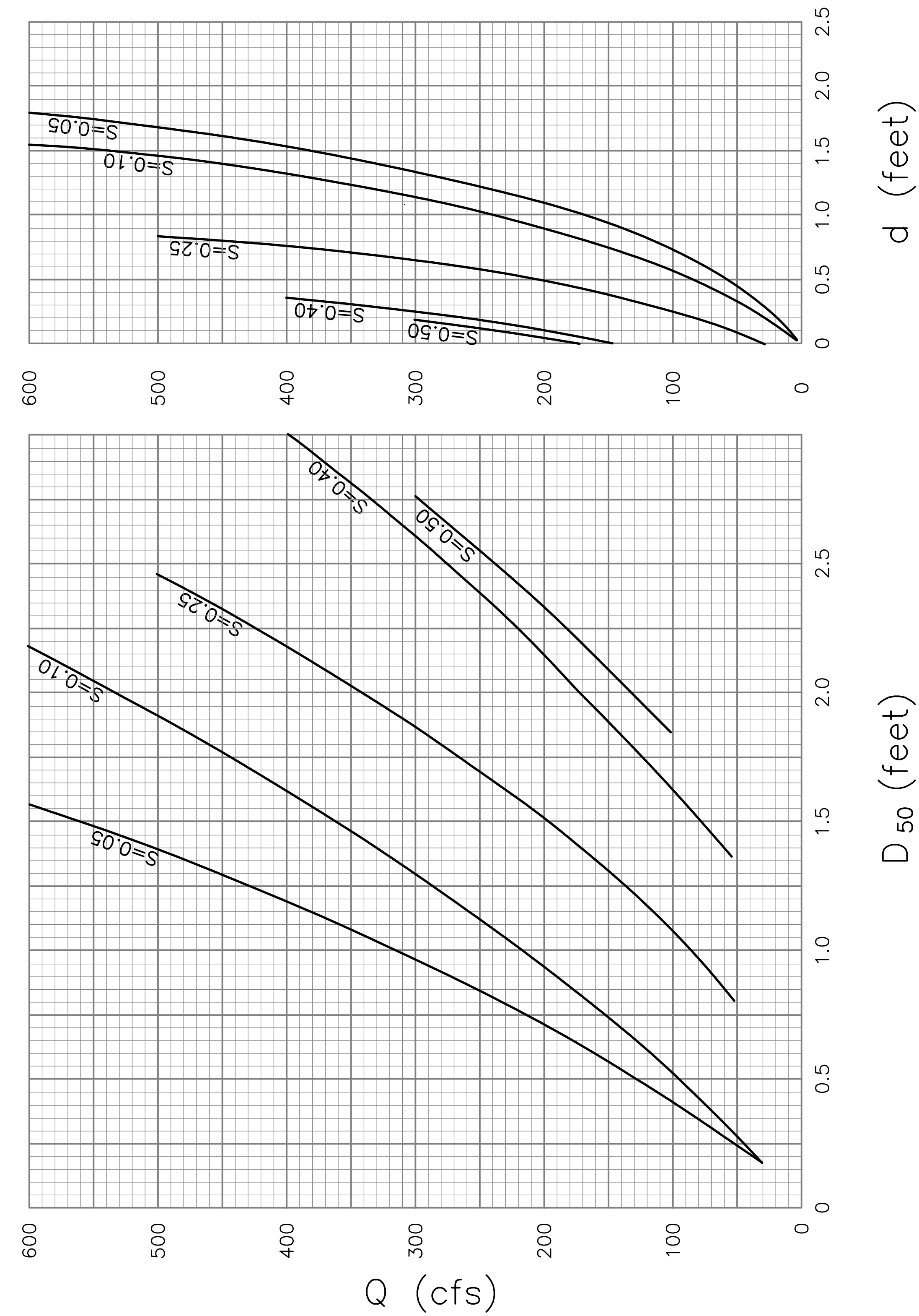
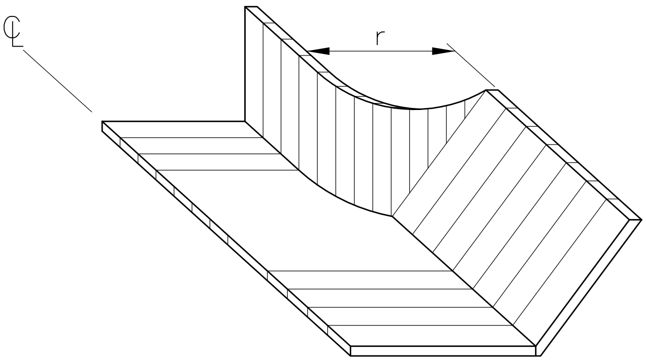
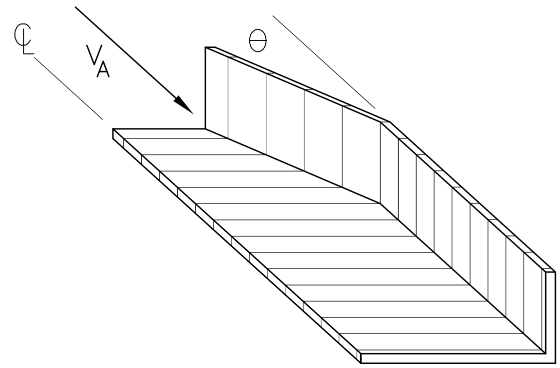


VERSION: JANUARY 2006	REFERENCE: Simons, Li and Assoc., 1989	**FIGURE CH13-F116** STEEP SLOPE RIPRAP DESIGN, TRAPEZOIDAL CHANNEL, 2:1SIDE SLOPES, 20-FOOT BOTTOM WIDTH

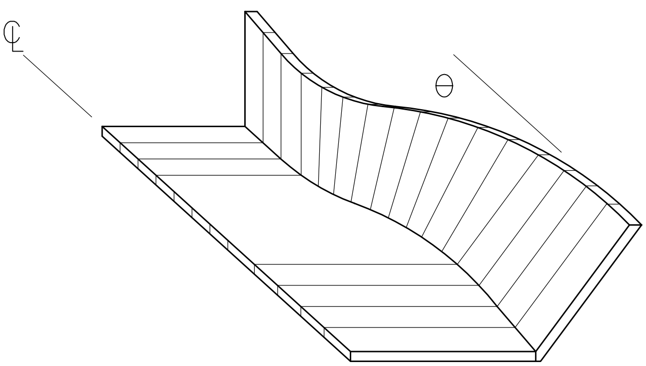
$$\text{Froude Number} = \frac{V_A}{\sqrt{g y_A}}$$



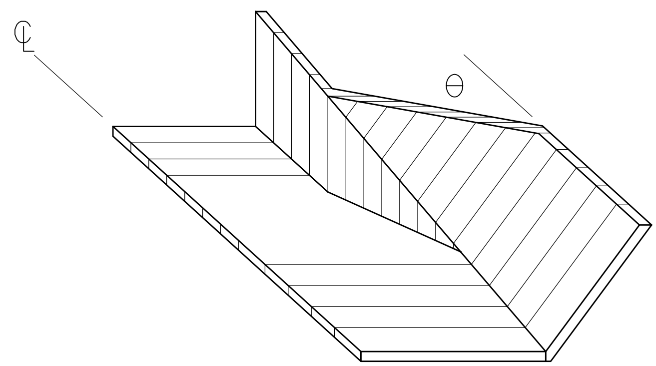
CYLINDRICAL QUADRANT



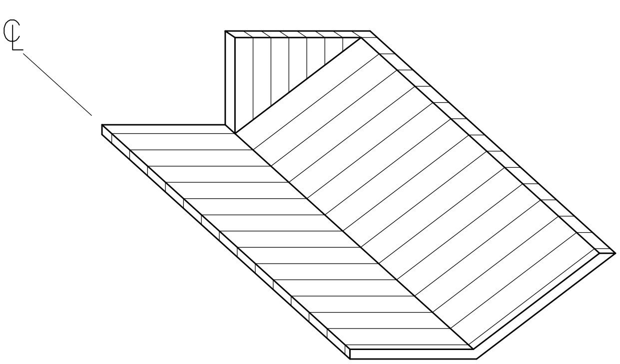
STRAIGHT LINE



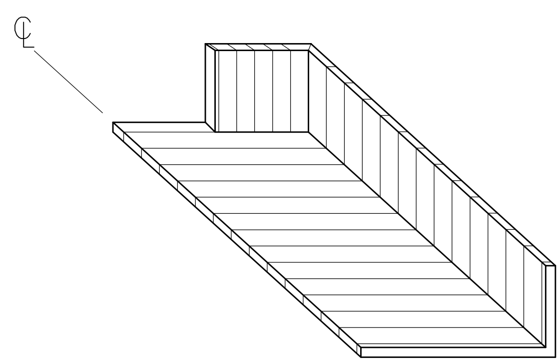
WARPED



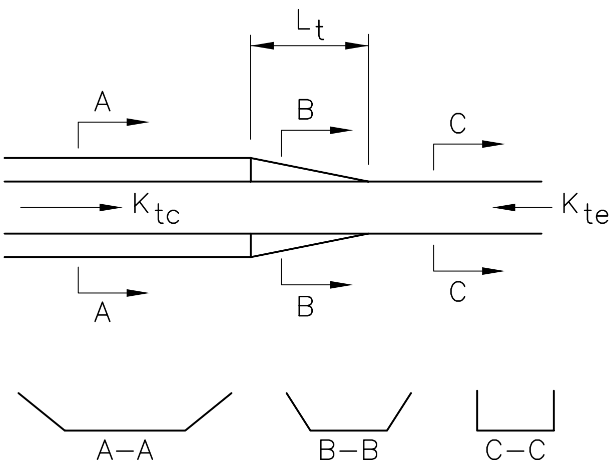
WEDGE



SQUARE–ENDED



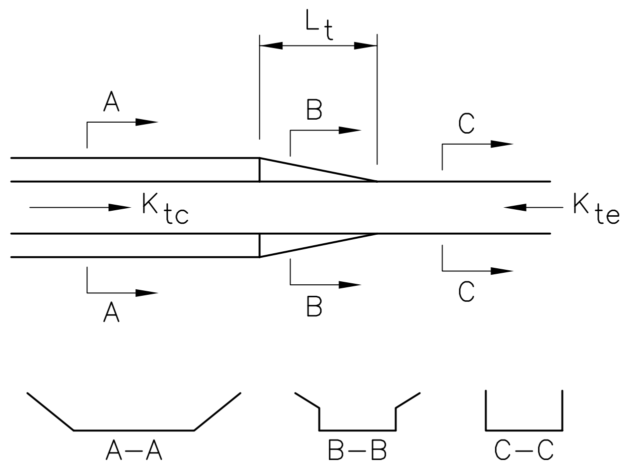
SQUARE–ENDED



WARPED TRANSITION

K_{tc} (CONTRACTION)=0.1

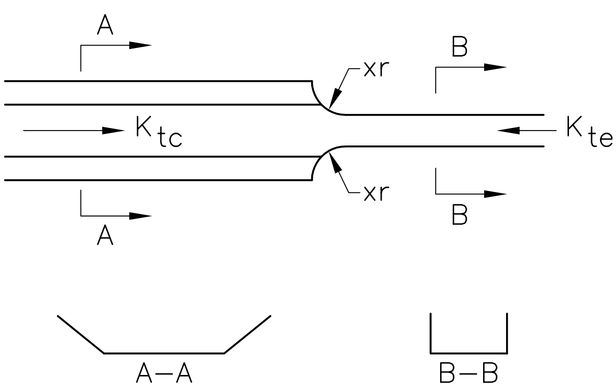
K_{te} (EXPANSION)=0.2



STRAIGHT–LINE TRANSITION

K_{tc} (CONTRACTION)=0.3

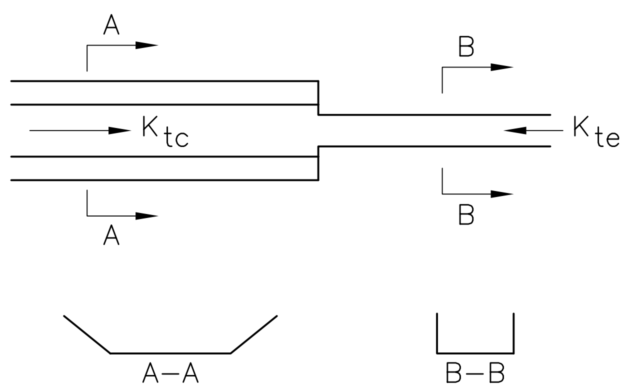
K_{te} (EXPANSION)=0.5



CYLINDER–QUADRANT

K_{tc} =0.15

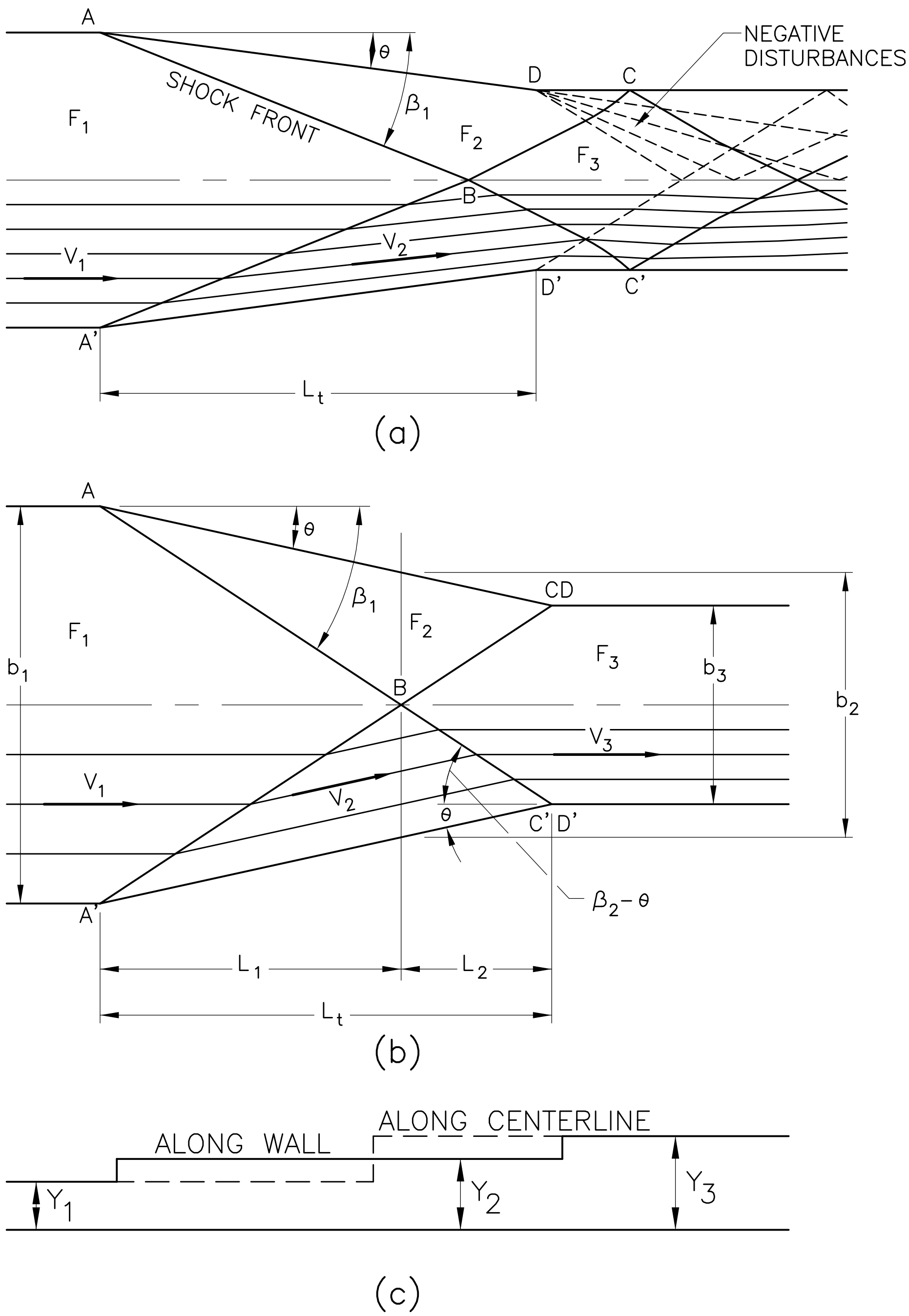
K_{te} =0.25



SQUARE–ENDED TRANSITION

K_{tc} =0.30

K_{te} =0.75



NOTES:

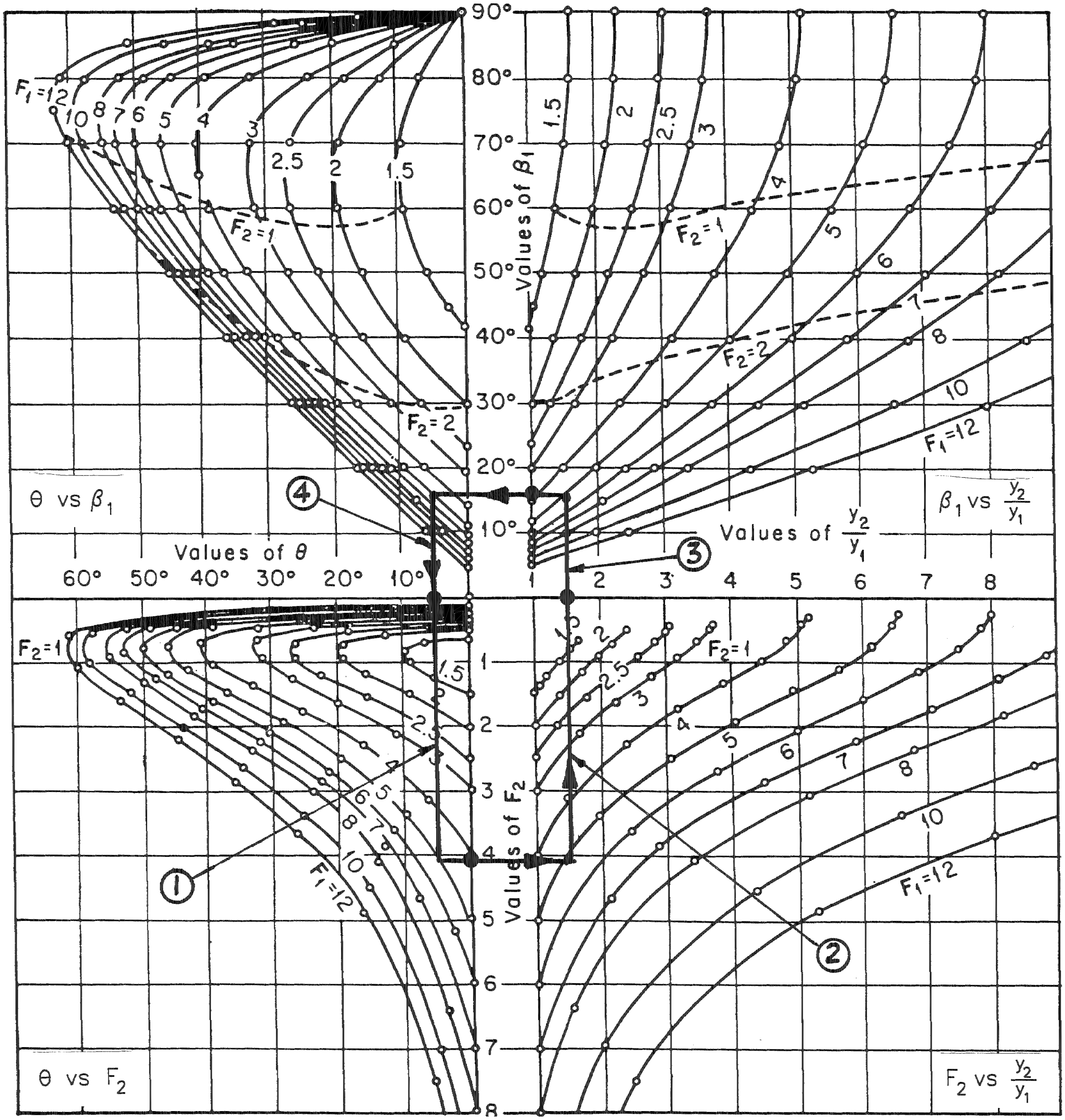
(a) GENERAL DISTURBANCE PATTERNS

(b) MINIMUM DOWNSTREAM DISTURBANCE

(c) SCHEMATIC PROFILE

REFERENCE:
Chow, V.T., Open-Channel Hydraulics, McGraw Hill Book Company, 1959

FIGURE CH13-F119
TYPICAL CONTRACTING TRANSITION FOR SUPERCRITICAL FLOW



EXAMPLE: For θ = 5° and F_1 = 5.0

1. Read F_2 = 4.1
2. Read y_2/y_1 = 1.5
3. Read β_1 = 16°
4. Read θ = 5° (check)

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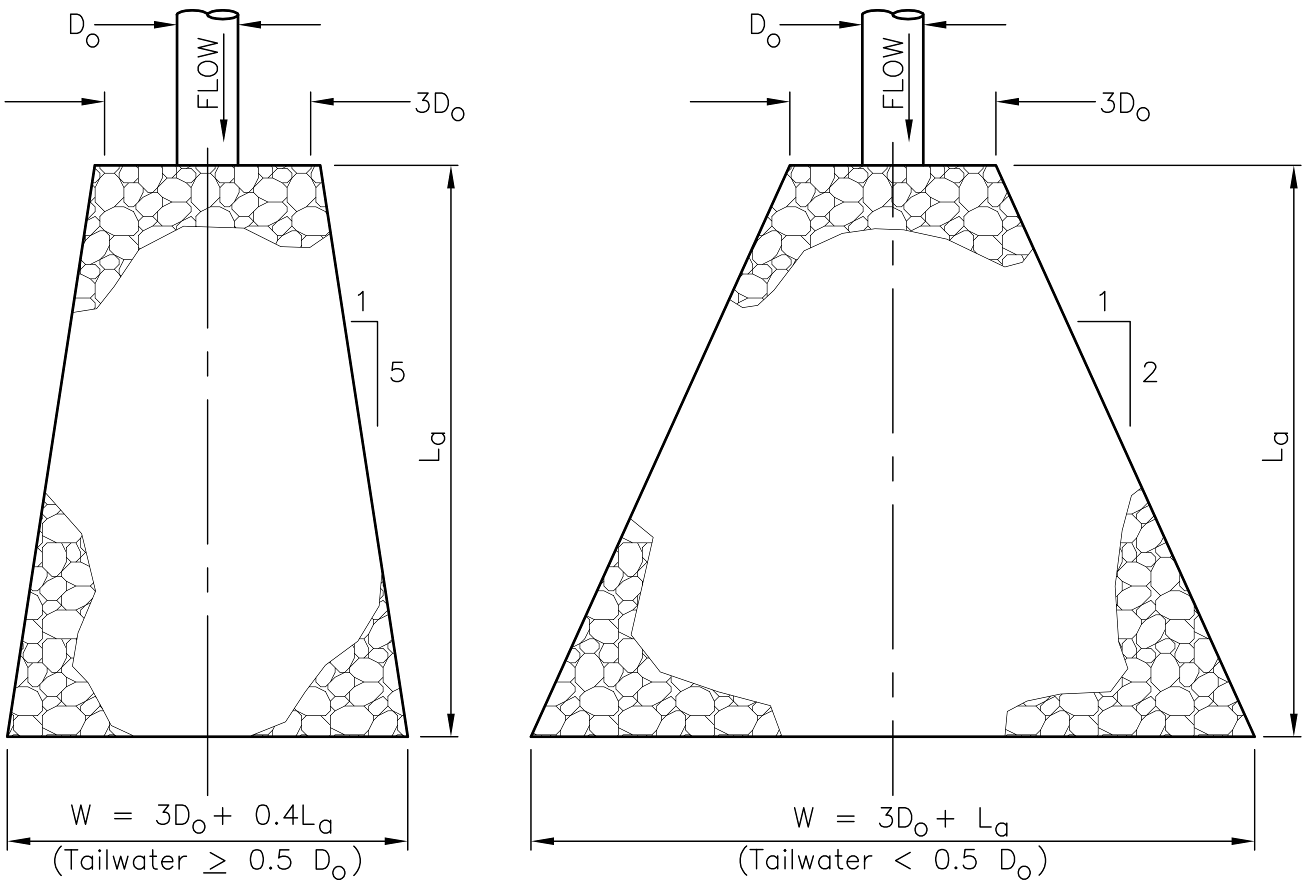
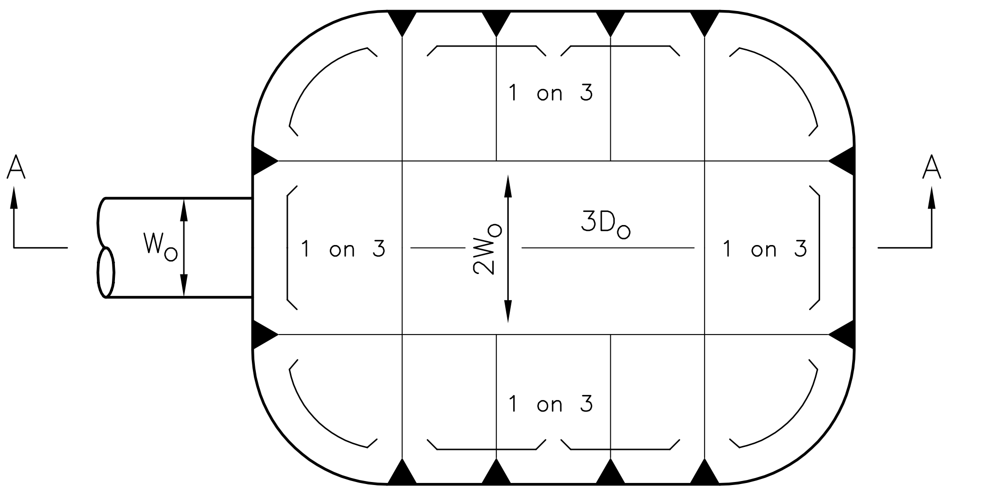
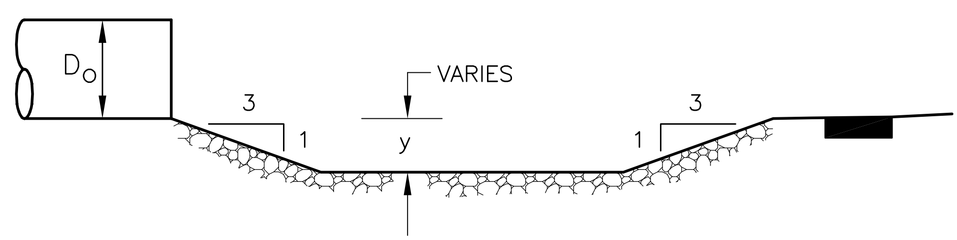


O:/2120/FIGURES/CHAP13-2.DWG, CH6-F121 -1/6/06 - GPB

VERSION: JANUARY 2006	**REFERENCE:** U.S. EPA, 1976	**FIGURE CH13-F121** CONFIGURATION OF CULVERT OUTLET PROTECTION

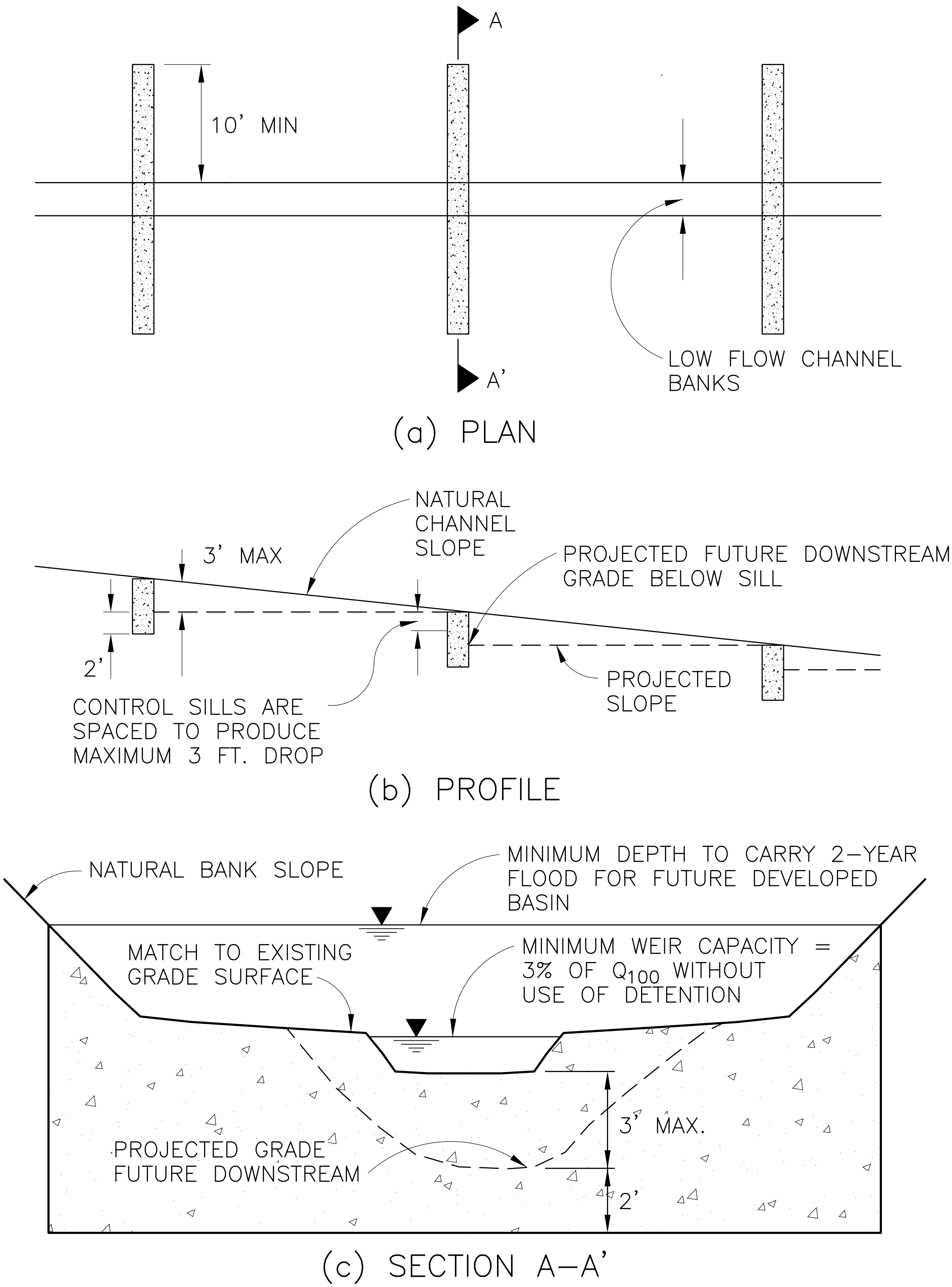


PLAN VIEW



SECTION VIEW

VERSION: JANUARY 2006	REFERENCE: ASCE, 1975	**FIGURE CH13-F122** PREFORMED SCOUR HOLE



VERSION: JANUARY 2006	REFERENCE: UDFCD, 1990	FIGURE CH13-F123 CONTROL SILL GRADE CONTROL STRUCTURE

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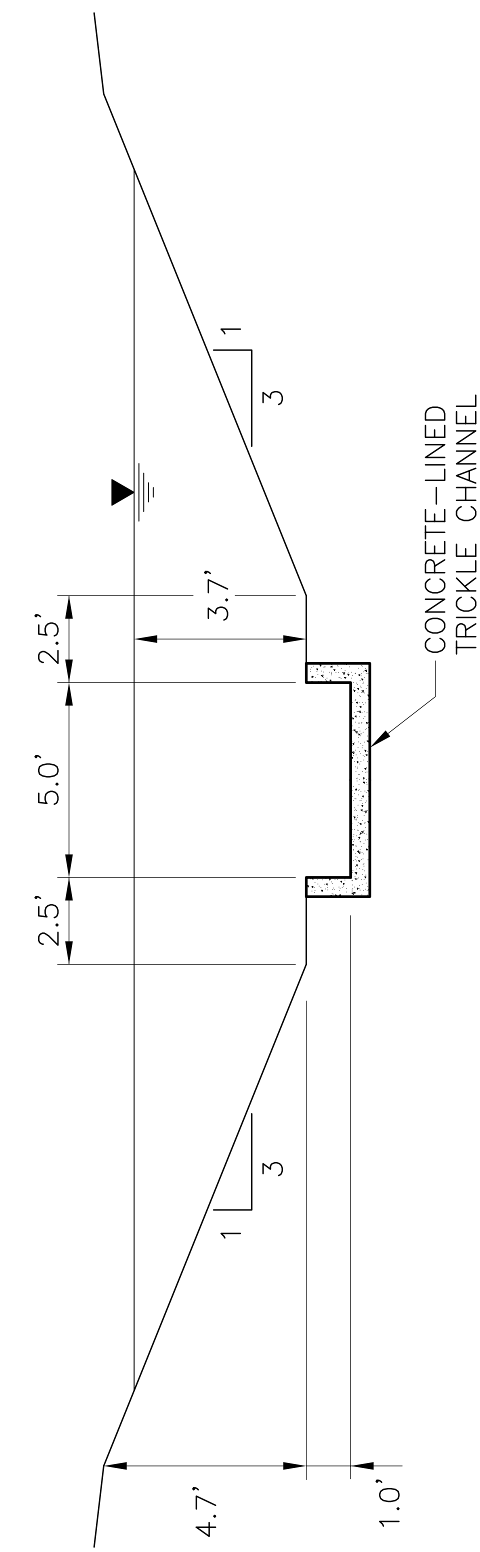

VERSION: JANUARY 2006	REFERENCE:	FIGURE CH13-F124
		EXAMPLE: CROSS-SECTION OF DOE CREEK

O:/2120/FIGURES/CHAP13-2.DWG, CH6-F124 – 1/6/06 – GPB

CHAPTER 13

HYDRAULIC ANALYSIS AND DESIGN

SECTION 2

BRIDGES AND CULVERTS

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SECTION 2
BRIDGES AND CULVERTS

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HYDRAULIC ANALYSIS AND DESIGN

SECTION 2
BRIDGES AND CULVERTS

2.1 INTRODUCTION

All new and replacement culverts and bridges should be designed to not adversely impact surrounding properties by increasing the water surface elevations and/or by diverting flows out of the channel to a different flow path.

Culverts and bridges are widely used to convey surface water through or beneath roadways, railroads, other embankments, and engineered structures. The size, material, alignment, and support structures of bridges or culverts directly affect the flow conveyance capacity of the overall drainage system.

Inadequately designed culverts or bridges can force flows out of the conveyance system, and the flows may take an alternate path and cause damage away from the channel. Undersized structures can also cause increased flow depths upstream of the crossing location. All new and replacement culverts and bridges should be designed to not adversely impact surrounding properties by increasing the water surface elevations and/or by diverting flows out of the channel to a different flow path. Placement of culverts and bridges within the designated floodway may be allowed only if it can be proven through a detailed hydraulic analysis that it will not increase the 100-year water surface elevation.

The primary distinction between a culvert and a bridge is the change in flow conveyance area from the upstream channel cross-section. A culvert is usually designed to allow the upstream water surface elevation to be greater than the top of the culvert, while bridge design generally provides freeboard between the design floodwater surface and the low chord of the bridge.

2.2 DESIGN STANDARDS FOR CULVERTS

All culverts within the State of Colorado should be designed using the following standards. The analysis and design should consider the design flow rate, culvert size and material, culvert length and slope, upstream channel and entrance configuration, downstream channel and outlet configuration, and erosion protection. Maintenance access for culvert maintenance and cleaning should be provided at all culvert locations.

All culverts should be designed, at a minimum, to withstand an HS-20 loading.

Culverts should be structurally designed to withstand the design loads including earth, pavement, and traffic loads. The structural design of culverts should conform to those methods and criteria recommended by the manufacturer for the culvert type and for the conditions found at the installation site. The minimum standards set forth in the current American Association of State Highway and

Transportation Officials (AASHTO) Standard Specifications for Highway Bridges should be adhered to. All culverts should be designed, at a minimum, to withstand an HS-20 loading. For large structures or where groundwater is a problem, the design should include necessary provisions to resist hydrostatic uplift forces that could result in failure of the culvert structure.

2.2.1 CULVERT SIZING CRITERIA

In most instances, culverts have direct impacts on the resulting water surface elevations and the flow conveyance capacity of the overall drainage system. Therefore, it is imperative that culverts are properly sized to convey the design flows at or below the required water surface elevations. Larger culverts do not encroach into the channel cross-section as much as smaller culverts and will cause a smaller rise in water surface elevations. The trade-off is that larger culverts are more expensive to construct than small culverts.

All new and replacement culvert structures, including street overflow sections where permitted, are recommended to be designed to confine and convey the 100-year flows. As a general rule, a 10 % bulking/clogging factor should be added to the estimated 100-year peak flow rate to account for sediment and debris loads

2.2.1.1 DESIGN FREQUENCY

All new and replacement culvert structures, including street overflow sections where permitted, are recommended to be designed to confine and convey the 100-year flows.

Sediment and debris loads associated with a 100-year flood event should be considered in the culvert design. As a general rule, a 10 % bulking/clogging factor should be added to the estimated 100-year peak flow rate. For drainage-ways with known substantial sediment deposition problems, sediment and debris loads should be determined using historic flood/debris information documented by CWCB or local officials. Where appropriate, sediment/debris trap basins should be constructed upstream of the culvert structure.

2.2.1.2 ALLOWABLE CROSS STREET FLOW

The maximum allowable flow overtopping limits during a 100-year event for various street classifications are outlined below.

Street Classification	Max. Depth at the Street Crown (Ft.)	Max. Flow Velocity (fps)
Local	1 ft.	6 fps
Collector	1 ft.	6 fps
Arterial	No Overflow	No Overflow
Freeway & Highway	No Overflow	No Overflow

The minimum guidelines for the design of street overflow section are outlines below.

- Using the allowable overflow limits specified above, the allowable overflow for a 100-year event should be determined based on the street classification and profile. In most instances, the roadway overtopping section may be treated as a broad-crested weir.

- The culvert is then sized for the difference between the 100-year peak flow rate and the allowable flow over the street.

- If the resulting culvert size is smaller than what is required by the Colorado Department of Transportation (CDOT) as summarized in the following table, adjust the culvert size to comply with the CDOT criteria.

Cross Drainage Type	Design Storm Frequency
Multilane Roads-	
including Interstate	
In Urban areas	100 years
In Rural areas	50 years
Two-Lane Roads	
In Urban areas	100 years
In Rural areas	
$Q50 \geq 4000$ cfs	50 years
$Q50 < 4000$ cfs Design ADT > 750	25 years
$Q50 < 4000$ cfs Design ADT < 750	10 years

- If only a small increase in culvert size is required to prevent overtopping during a 100-year event, then the larger culvert is recommended.

- Street overflow will not be allowed if the street in question is the only excess for an area during a 100-year flood event.

In all cases, culverts should be adequately sized and designed to not adversely impact adjacent properties by increasing the water surface elevations and/or by diverting flows out of the channel to a different flow path.

2.2.1.3 MINIMUM CULVERT SIZE

The minimum culvert size should be 18-inch diameter for a round pipe or should have a minimum flow conveyance area of 2.2 square feet for other pipe shapes. The minimum inside dimension for elliptical or arched pipes should be no less than 12 inches.

2.2.2 CONSTRUCTION MATERIALS

Culverts can be constructed with many different types of materials for variety of sizes and shapes. Culverts used in the State of Colorado should be constructed with reinforced concrete, PVC, HDPE, or corrugated metal.

Corrugated metal pipe culverts are available in round or arch cross-sections. Sections of corrugated metal can also be bolted together to form several other cross sectional shapes, such as elliptical and pear shapes. Corrugations also come in various dimensions, which affect the hydraulics of the pipe flow. The wall thickness of corrugated metal pipe (CMP) should be determined based on many factors including, design loads, cover depth, culvert size, and corrugated dimension. Please refer to the Handbook of Steel Drainage and Highway Construction Products published by The American Iron and Steel Institute for the design standards. Site-specific soil tests are required for the placement of CMP's. If soil tests identify the presence of corrosive soil conditions, appropriate pipe coatings will be required.

Reinforced Concrete Box Culverts (RCBC) can be constructed (cast-in-place) for generally any rectangular cross-section with the only limitations being the physical site constraints and the structural requirements. Pre-cast reinforced concrete box and pipe culverts and are also available in several standard dimensions.

The Colorado Water Conservation Board (CWCB) may allow other materials to be used for the construction of culverts. Design and material testing documentations should be submitted for review and approval by CWCB. Supporting documentations should demonstrate that the subject pipe material has a design life similar to the approved materials and that the interior lining, if any, will maintain the design Manning's roughness coefficient ("n") value for the life of the pipe material. Typical Manning's "n" values for different culvert materials and shapes are provided in Table CH13-T202.

Typical Manning's "n" values for different culvert materials and shapes are provided in Table CH13-T202.

2.2.3 VELOCITY LIMITATIONS AND INLET/OUTLET PROTECTION

All culverts should be designed to provide a minimum flow velocity of 3 fps at the culvert outlet for the 5-year storm event condition. In addition, the culvert slope should be a minimum of 0.25 percent.

Design flow velocities through the culvert structure should be determined, at a minimum, for 5- and 100-year storm events. If the flow velocity is too slow, sediment deposition may occur within the culvert decreasing the effective conveyance area of the culvert and increasing the frequency of required maintenance. All culverts should be designed to provide a minimum flow velocity of 3 fps at the culvert outlet for the 5-year storm event condition. In addition, the culvert slope should be a minimum of 0.25 percent, if site conditions allow.

If the flow velocity exiting the culvert is too high, channel erosion and scour at the outlet will take place, possibly jeopardizing the integrity of the culvert and roadway embankment. The design criteria of outlet erosion protections for natural and unlined channels are as follows:

Outlet Velocity (fps)	Required Outlet Protection
Less than 5	Minimum riprap protection (Section 1.10.3, Chapter 13)
Between 5 and 15	Riprap protection (Section 1.10.3, Chapter 13) or Energy dissipater (Section 6, Chapter 13)
Greater than 15	Energy dissipater (Section 6, Chapter 13)

For lined channels, the outlet discharge velocity should not exceed the maximum allowable channel design velocity. Otherwise, additional outlet erosion protection measures should be provided as outlined above.

Headwalls and wingwalls or flared-end sections should be provided for all culverts at both inlets and outlets. Guardrails and/or handrails should also be provided in conformance with the local building codes and roadway design safety requirements. Street overflow sections, when used, should be designed to adequately confine and convey the 100-year flows into the downstream channel. Adequate erosion protection measures should be provided to prevent degradation of the roadway and embankments.

2.2.4 HEADWATER CRITERIA

The extent of impacts on adjacent properties from the 100-year backwater created by culvert installations should be analyzed for all culverts. Culverts should be designed to properly convey the design flows at or below the required water surface elevations. Ponding at the culvert entrance will not be allowed if such ponding will cause property or roadway damage, saturation of fills, significant upstream deposits of debris, or inundation of existing or future facilities.

The maximum headwater for the 100-year design flow should be 1.5 times the culvert height for all culverts taller than 36" with standard inlet and outlet configurations. The maximum headwater for culverts with a height of 36" or less should be 5 feet.

If site conditions are such that the maximum headwater limits cannot be met, additional engineering analysis should be performed. The additional analysis is necessary to determine scour potential, embankment stability and any other factors that may influence the long-term stability of the structure. Additional erosion protection around the culvert inlet or other design considerations should be included as appropriate to ensure the long-term stability of the culvert and approaches.

The maximum headwater for the 100-year design flows should be 1.5 times the culvert height for all culverts taller than 36" with standard inlet and outlet configurations. The maximum headwater for culverts with a height of 36" or less should be 5 feet.

Culverts that do not include a street overtopping section should have a minimum of 1-foot freeboard from the hydraulic grade line at the culvert entrance to the edge of pavement elevation. Levees should not be used to provide increased headwater at culvert inlets.

2.2.5 ALIGNMENT

Alignment of the culvert with respect to the natural channel is very important for proper hydraulic performance. Culverts may pass beneath the roadway normal to the centerline or they may pass at an angle (skewed). Whenever possible, culverts should be aligned with the natural channel. This reduces inlet and outlet flow transition problems.

Whenever possible, culverts should be aligned with the natural channel.

Where the natural channel alignment would result in an exceptionally long culvert, modification of the natural alignment may be necessary. Since such modifications will change the natural stability of the channel, proposed modifications should be thoroughly investigated. Although the economic factors are important, the hydraulic effectiveness of the culvert should be given major considerations. Improper culvert alignment may cause erosion to adjacent properties or siltation within the culvert. Culvert alignment considerations are shown in Figure CH13-F201.

Roadway alignment also affects the culvert design. The vertical alignment of roadways may define the maximum culvert diameter that can be used. Low vertical clearance may require the use of elliptical or arched culverts, or the use of a multiple-barrel culvert system. All culverts should have a minimum of 1.5 foot of cover from top of asphalt (or gravel for gravel road) to outside top of pipe. Culverts with less than 1.5 feet of cover will require additional structural analysis and other provisions (i.e. full depth concrete paving to compensate for the loss of proper cover.

2.2.6 MULTIPLE-BARREL CULVERTS

If the available embankment fill height limits the size of culvert necessary to convey the flood flows, multiple culverts can be used. If each barrel of a multiple-barrel system is of the same type and size, and constructed such that all hydraulic parameters are equal, the total flow should be assumed to be equally divided among each of the barrels.

2.2.7 TRASH RACKS/SAFETY GRATES

Trash racks or safety grates may be necessary at the upstream inlet of some culverts. During the culvert design, engineering judgments should be used to determine if trash racks or safety grates should be included. Factors that may influence whether or not trash racks or safety grates should be used include the following:

- Tributary Land Use (urban, rural, forest)
- Location (urban/rural)
- Design flow rate
- Size of culvert
- Anticipated debris loading
- Performance of nearby existing structures

Trash racks should be used on any size or length of culvert where the horizontal or vertical alignment does not allow for an unobstructed view through the culvert. Trash racks/safety grates should be hinged at the top to permit the grate to be lifted and allow the culvert and grate to be cleaned. The grate/rack should slope at 2:1 to 5:1 (horizontal to vertical) to permit the debris to float up the grate as the water level rises. The bar spacing should prevent a child from passing through the openings. The net open area through the rack/grate below the design water surface should be at least four times the design flow area of the culvert.

2.2.8 AIR VENTS

All culverts greater than 48 inches in diameter for which both the inlet and outlet are sealed by water under less than full flow conditions should include an air vent pipe to prevent air accumulation/partial vacuums. Said vent should have a diameter equal to or greater than one-sixth of the culvert pipe diameter.

2.3 CULVERT HYDRAULICS

This section presents the general procedures for hydraulic design and evaluation of culverts. The user is assumed to possess a basic working knowledge of culvert hydraulics and is encouraged to review the textbooks and other technical literature on the subject. The following is a short list of some of the culvert hydraulics publications.

- U.S. Department of Transportation, Federal Highway Administration, Hydraulic Design of Highway Culverts, Hydraulic Design Series No. 5, September 1985.
- U.S. Department of Transportation, Federal Highway Administration, Hydraulic Charts for the Selection of Highway Culverts, Hydraulic Engineering Circular No. 5, December 1965.
- U.S. Department of Transportation, Federal Highway Administration, Capacity Charts for the Hydraulic Design of Highway Culverts, Hydraulic Engineering Circular No. 10, November 1972.
- U.S. Department of Transportation, Federal Highway Administration, Hydraulic Design of Improved Inlets for Culverts, Hydraulic Engineering Circular No. 13, 1972.

The two categories of flow in culverts are inlet control and outlet control. Under inlet control, the flow through the culvert is controlled by the headwater of the culvert and the inlet geometry. Under outlet control, the flow through the culvert is controlled primarily by culvert slope, roughness, and the tailwater elevation.

> When designing a culvert, the designer should evaluate both inlet and outlet control conditions for the given design constraints. Culvert hydraulic calculations should be performed using rating nomographs and/or culvert hydraulic analysis programs.

When designing a culvert, the designer should evaluate both inlet and outlet control conditions for the given design constraints (e.g. headwater depth, flow capacity, etc.). The control condition that produces the greater energy loss for the design conditions determines the appropriate control to use for culvert design. Culvert hydraulic calculations should be performed using rating nomographs and/or culvert hydraulic analysis programs.

2.3.1 INLET CONTROL CONDITION

Inlet control for culverts may occur in two ways (see Figure CH13-F202):

1. Unsubmerged - The headwater is not sufficient to submerge the top of the culvert and the culvert invert slope is supercritical. The culvert entrance acts like a weir (Condition A, Figure CH13-F202).

2. Submerged - The headwater submerges the top of the culvert but the pipe does not flow full. The culvert inlet acts like an orifice (Condition B and C, Figure CH13-F202).

   The inlet control rating for typical culvert shapes and inlet configurations are presented in Figures CH13-F203 to CH13-F206. Additional nomographs are available in the U.S. Department of Transportation's Hydraulic Design Series Number 5 (USDOT, 1985). These nomographs were developed empirically by pipe manufacturers, Bureau of Public Roads, and the Federal Highway Administration. The nomographs should be used rather than the orifice and weir equations, due to the uncertainty in estimating the orifice and weir coefficients.

2.3.2 OUTLET CONTROL CONDITION

Outlet control will govern if the headwater and/or tailwater is deep enough, the culvert slope is relatively flat, and the culvert is relatively long. There are three types of outlet control culvert flow conditions:

1. The headwater submerges the culvert top, and the culvert outlet is submerged by the tailwater. The culvert will flow full (Condition A, Figure CH13-F202).

2. The headwater submerges the top of the culvert and the culvert is unsubmerged by the tailwater (Condition B or C, Figure CH13-F202).

3. The headwater is insufficient to submerge the top of the culvert. The culvert slope is subcritical and the tailwater depth is lower than the pipe critical depth (Condition D, Figure CH13-F202).

The factors affecting the capacity of a culvert in outlet control include the headwater elevation, the inlet geometry and associated losses, the culvert material friction losses, and the tailwater condition.

The capacity of the culvert is calculated using the conservation of energy principal (Bernoulli's Equation). An energy balance exists between the total energy of the flow at the culvert inlet and at the culvert outlet, which includes the inlet losses, the friction losses, and the velocity head (see Figure CH13-F207). The equation is then expressed as:

$$H = h_e + h_f + h_v \qquad \text{(Eq CH13-200)}$$

Where H = Total energy difference, inlet through outlet (ft)
h_e = Entrance head losses (ft)
h_f = Friction losses (ft)
h_v = Velocity head = $V^2/2g$ (feet) (Eq CH13-201)

For inlet losses, the governing equation is:

$$h_e = k_e\,(V^2/2g) \qquad \text{(Eq CH13-202)}$$

Where k_e is the entrance loss coefficient. Typical entrance loss coefficients recommended for use are given in Table CH13-T201.

Friction loss is the energy required to overcome the roughness of the culvert and is expressed as follows:

$$h_f = (29n^2L/R^{1.33})(V^2/2g) \qquad \text{(Eq CH13-203)}$$

Where n = Manning's coefficient (see Table CH13-T202)
L = Length of culvert (ft)
R = Hydraulic radius (ft)
V = Velocity of flow (fps)
G = Gravitational acceleration constant (32.2 ft/s^2)

Substituting equivalent terms from equations CH13-201, CH13-202, and CH13-203 into equation CH13-200 and simplifying the terms results in the following equation:

$$H = [k_e + (29n^2L/R^{1.33}) + 1]\,V^2/2g \qquad \text{(Eq CH13-204)}$$

Equation CH13-204 can be used to calculate the culvert capacity directly when the culvert is flowing under outlet control conditions A or B as shown on Figure CH13-F202. The actual headwater (Hw) is calculated by adding H to the tailwater elevation (see Figure CH13-F207). For conditions C or D in Figure CH13-F202, the hydraulic grade line at the outlet is approximated by averaging the critical depth and the culvert diameter. This value is used to compute headwater depth (Hw) if it is greater than the tailwater depth (Tw). This is an approximate method and is more fully described in HDS No. 5. Estimates of critical depth for box culverts, circular pipe, and elliptical pipe can be obtained from Figures CH13-F208, CH13-F209, and CH13-F210 respectively.

A series of outlet control nomographs for various culvert shapes have been developed by pipe manufacturers, Bureau of Public Roads, and the Federal Highway Administration. The nomographs are presented in Figures CH13-F211 to CH13-F214. Additional nomographs are available in HDS No. 5. When rating a culvert, either the outlet control nomographs or Equation CH13-204 can be used to calculate the headwater requirements.

When using the outlet control nomographs for corrugated metal pipe, the data should be adjusted to account for the variation in the “n” value between the nomographs and the culvert being evaluated. The adjustment is made by calculating an equivalent length according to the following equation:

$$L^1 = L\,(n^1/n)^2 \qquad \text{(Eq CH13-204)}$$

Where L^1 = Equivalent length
L = Actual length
n = Manning's "n" value shown on Figures CH13-F211 to CH13-F214
n^1= Actual "n" value of the culvert

2.3.3 HYDRAULIC DATA

The hydraulic data provided in Table CH13-T201 and CH13-T202 should be used in the hydraulic design of all culverts. The design capacity of culverts should be calculated using the computation sheet provided as Standard Form CH13-SF201. Manning's roughness coefficients ("n") used for velocity and capacity calculations should be those presented in Table CH13-T202. Alternatively, computer programs may be used for hydraulic analysis. However the designer should thoroughly review the modeling results to determine if the analysis has properly modeled the hydraulic conditions.

2.4 DESIGN STANDARDS FOR BRIDGES

All bridges should be designed in accordance with the "Standard Specifications for Highway Bridges" by AASHTO. Hydraulic design and analysis should be in accordance with the following criteria.

2.4.1 BRIDGE SIZING CRITERIA

All new bridges should be designed to pass the 100-year estimated peak flows. Additionally, the design water surface elevation within the bridge should be at least 2 feet below the bridge low chord or appropriate measures should be taken to avoid floatation of the bridge due to debris blockage. Additional freeboard may be necessary for various special hydraulic conditions.

> *All new bridges should be designed to pass the 100-year estimated peak flows. If possible, replacement bridges should also be designed to pass the 100-year estimated peak flows as discussed above.*

If possible, replacement bridges should also be designed to pass the 100-year estimated peak flows as discussed above. If site-specific conditions do not allow a replacement bridge to be designed to convey the 100-year flows, the design engineer should coordinate with the appropriate agencies to determine the acceptable bridge design capacity. Hydraulic analyses should be performed to demonstrate that the bridge placement will not adversely affect adjacent properties.

2.4.2 VELOCITY LIMITATIONS

The velocity limitation through a bridge opening is controlled by the scour potential and subsequent channel erosion protection measures provided.

The 100-year design flow velocity through the bridge and approaches should not exceed the allowable velocity for the channel lining type as discussed in Section 1.5.2, Chapter 13. If the design velocity through the bridge is greater than the maximum allowable velocity of the natural channel, appropriate channel protection measures should be provided.

2.5 BRIDGE HYDRAULICS

2.5.1 HYDRAULIC ANALYSIS

The procedures for analysis and design as outlined in the following publications should be used for the hydraulic design and scour analysis of all bridges.

- U.S. Department of Transportation, Federal Highway Administration, Hydraulics of Bridge Waterways, Hydraulic Design Series No. 1, 1978.
- U.S. Department of Transportation, Federal Highway Administration, Evaluating Scour at Bridges, Hydraulic Engineering Circular No. 18, 1993.
- U.S. Department of Transportation, Federal Highway Administration, Stream Stability at Highway Structures, Hydraulic Engineering Circular No. 20, 1991.

This analysis should be supplemented by an appropriate backwater analysis using HEC-RAS or HEC-2 to verify the resulting hydraulic performance of the bridge. The extent of the bridge backwater should be shown on a topographic map.

2.5.2 INLET AND OUTLET CONFIGURATION

The design of all bridges should include adequate wingwalls of sufficient length to minimize abutment erosion and to provide slope stabilization from the embankment to the channel. Erosion protection on the inlet and outlet transition slopes should be provided to protect the channel from the erosive forces of eddy currents.

2.6 EXAMPLE APPLICATION

2.6.1 EXAMPLE: CULVERT SIZING

Problem: Determine the culvert size necessary to convey the 100-year, 24-hour peak flow in Doe Creek beneath John Boulevard. The results of this analysis are provided in Table CH13-T203.

Top of road elevation	4928 feet
Culvert inlet elevation	4920 feet
Culvert outlet elevation	4918 feet
Culvert length	200 feet
Inlet	Groove end with headwall and wingwalls at 45 degrees
Outlet	Groove end with headwall and wingwalls at 45 degrees
Flow	191 cfs
Tailwater Depth	4 feet

Solution:

Step 1: Assume a pipe diameter or box culvert dimensions and determine the headwater to depth ratio for inlet control conditions.
Assuming a 5-foot diameter reinforced concrete pipe (RCP), the headwater to depth ratio, is 1.38 (see Figure CH13-F215).

Step 2: Calculate the headwater assuming inlet control conditions. Multiply the pipe diameter times the headwater to depth ratio.

Headwater = $HW_I = D*HW/D = 5*1.38 = 6.9$ feet

Step 3: Estimate the critical depth, d_c, from Figure CH13-F209 (see Figure CH13-F216).

$d_c = 3.9$ feet

Step 4: Since the tailwater depth is less than the culvert diameter, compute the estimated water depth at the culvert outlet assuming the tailwater does not control the outlet conditions.

Outlet Depth = $(d_c + D)/2 = (3.9 + 5.0)/2 = 4.5$ feet

Step 5: Determine the flow depth at the culvert outlet, h_o. The estimated depth is the maximum value of the tailwater depth and the water depth assuming no tailwater.

$h_o = 4.5$ feet

Step 6: Estimate the head, H, for outlet control conditions from Figure CH13-F212.

H = 2.6 feet (see Figure CH13-F217).

Step 7: Calculate the headwater depth for outlet control conditions.

$HW_o = H + h_o + LS_o = 2.6 + 4.5 - 2.0 = 5.1$

Step 8: Determine if the culvert is under inlet control or outlet control and provide the resulting headwater depth and elevation.

Since HW_I is greater than HW_o, the culvert is under inlet control.

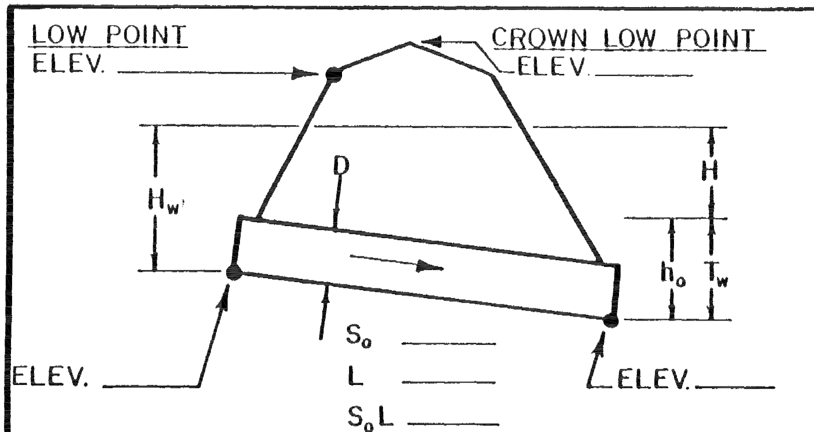
HW= 6.9

Step 9: Calculate the outlet velocity by an appropriate method, and determine the type of outlet protection needed.

V = 10.0 fps

Riprap protection or an energy dissipater is necessary.

PROJECT: ______________ LOCATION: ______________ STATION: ______________



CULVERT DATA

TYPE: ______________ n: ______
INLET: ______________ Q_{FULL}: ______
K_e: ______________ V_{FULL}: ______

OUTLET CONTROL EQUATIONS

(1) $H_w = H + h_o - LS_o$

(2) For $T_w < D$; $h_o = \frac{d_c + D}{2}$ or T_w (whichever is greater)

$T_w \geq D$; $h_o = T_w$

(3) For Box Culvert: $d_c = 0.315(Q/B)^{2/3} \leq D$

Q	STORM EVENT	INLET CONTROL		OUTLET CONTROL						TYPE OF CONTROL	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY
						$T_w < D$		$T_w \geq D$				
		$\frac{H_w}{D}$	H_w	H	T_w	d_c	$\frac{d_c + D}{2} = h$	h_o	H_w			
1	2	3	4	5	6	7	8	9	10	11	12	13

HYDRAULIC DATA FOR CULVERTS
CULVERT ENTRANCE LOSSES

TYPE OF ENTRANCE	ENTRANCE COEFFICIENT, K_E
Pipe	
Headwall	
Grooved edge	0.20
Rounded edge (0.15D radius)	0.15
Rounded edge (0.25D radius)	0.10
Square edge (cut concrete and CMP)	0.40
Headwall & 45° Wingwall	
Grooved edge	0.20
Square edge	0.35
Headwall with Parallel Wingwalls Spaced 1.25D apart	
Grooved edge	0.30
Square edge	0.40
Beveled edge	0.25
Projecting Entrance	
Grooved edge (RCP)	0.25
Squared edge (RCP)	0.50
Sharp edge, thin wall (CMP)	0.90
Sloping Entrance	
Mitered to conform to slope	0.70
Flared-end Section	0.50
Box, Reinforced Concrete	
Headwall Parallel to Embankment (no wingwalls)	
Square edge on 3 edges	0.50
Rounded on 3 edges to radius of 1/12 barrel dimension	0.20
Wingwalls at 30° to 75° to barrel	
Square edge at crown	0.40
Crown edge rounded to radius of 1/12 barrel dimension	0.20
Wingwalls at 10° to 30° to barrel	
Square edge at crown	0.50
Wingwalls parallel (extension of sides)	
Square edge at crown	0.70

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TYPE OF CHANNEL & DESCRIPTION	MINIMUM	NORMAL	MAXIMUM
Brass, smooth	0.009	0.010	0.013
Steel:			
Lockbar and welded	0.010	0.012	0.014
Riveted and spiral	0.013	0.016	0.017
Cast Iron:			
Coated	0.010	0.013	0.014
Uncoated	0.011	0.014	0.016
Wrought Iron:			
Black	0.012	0.014	0.015
Galvanized	0.013	0.016	0.017
Corrugated Metal:			
Sub-drain	0.017	0.019	0.021
Storm Drain	0.021	0.024	0.030
Lucite	0.008	0.009	0.010
Glass	0.009	0.010	0.013
Cement:			
Neat, surface	0.010	0.011	0.013
Mortar	0.011	0.013	0.015
Concrete:			
Culvert, straight and free of debris	0.010	0.011	0.013
Culvert with bends, connections, and some debris	0.011	0.013	0.014
Finished	0.011	0.012	0.014
Sewer with manholes, inlet, etc., straight	0.013	0.015	0.017
Unfinished, steel form	0.012	0.013	0.014
Unfinished, smooth wood form	0.012	0.014	0.016
Unfinished, rough wood form	0.015	0.017	0.020
Wood:			
Stave	0.010	0.012	0.014
Laminated, treated	0.015	0.017	0.020
Clay:			
Common drainage tile	0.011	0.013	0.017
Vitrified sewer	0.011	0.014	0.017
Vitrified sewer with manholes, inlet, etc.	0.013	0.015	0.017
Vitrified subdrain with open joint	0.014	0.016	0.018
Brickwork:			
Glazed	0.011	0.013	0.015
Lined with cement mortar	0.012	0.015	0.017
Sanitary sewers coated with sewage slime with bends and connections	0.012	0.013	0.016
Paved invert, sewer, smooth bottom	0.016	0.019	0.020
Rubble masonry, cemented	0.018	0.025	0.030

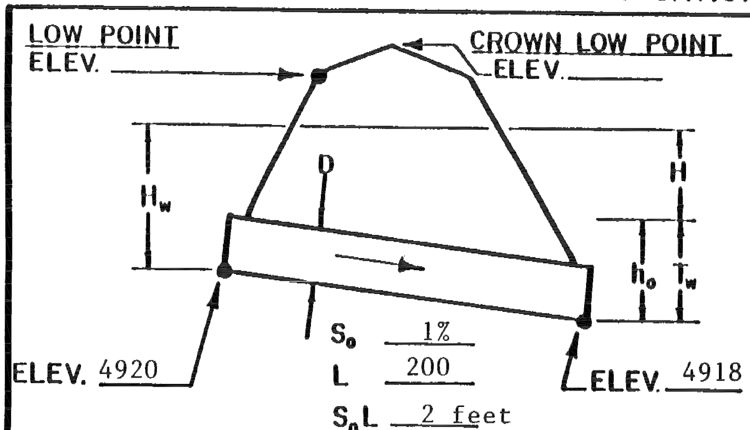
REFERENCE:

HEC-2 User's Manual & Chow, 1959

TABLE CH13-T202
TYPICAL MANNING'S N VALUES FOR CULVERTS

CULVERT RATING

PROJECT: JOHN BOULEVARD LOCATION: DOE CREEK STATION: ______



CULVERT DATA

TYPE: 5 foot Dia. RCP n: ______

INLET: Groove end w/headwall Q_{FULL}: ______

K_e: 0.2 V_{FULL}: ______

OUTLET CONTROL EQUATIONS

(1) $H_w = H + h_o - LS_o$

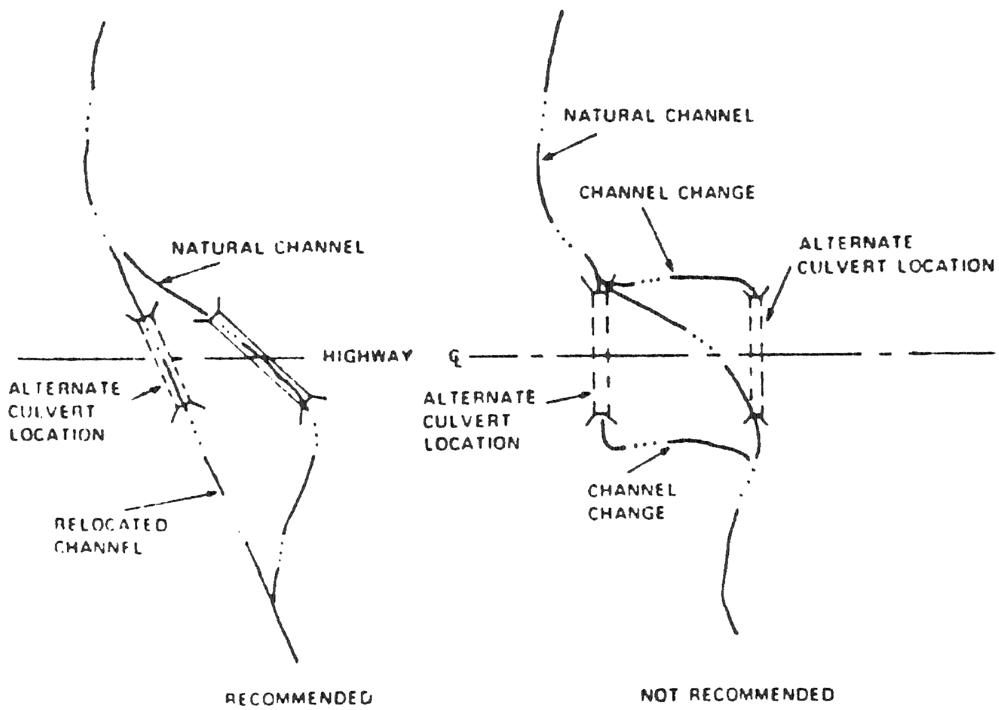
(2) For $T_w < D$; $h_o = \frac{d_c + D}{2}$ or T_w (whichever is greater)

$T_w \geq D$; $h_o = T_w$

(3) For Box Culvert: $d_c = 0.315(Q/B)^{2/3} \leq D$

Q	STORM EVENT	INLET CONTROL $\frac{H_w}{D}$	INLET CONTROL H_w	OUTLET CONTROL H	OUTLET CONTROL T_w	$T_w < D$: d_c	$T_w < D$: $\frac{d_c + D}{2} = h$	$T_w \geq D$: h_o	OUTLET CONTROL H_w	TYPE OF CONTROL	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY
1	2	3	4	5	6	7	8	9	10	11	12	13
191	100-YR	1.38	6.90	2.60	4.0	3.9	4.5	4.5	5.1	Inlet	4926.9	10

COLORADO
FLOODPLAIN AND STORMWATER CRITERIA MANUAL



VERSION: JANUARY 2006	REFERENCE: HYDRAULIC DESIGN OF HIGHWAY CULVERTS FHWA-NHI-01-020	FIGURE CH13-F201 CULVERT ALIGNMENT

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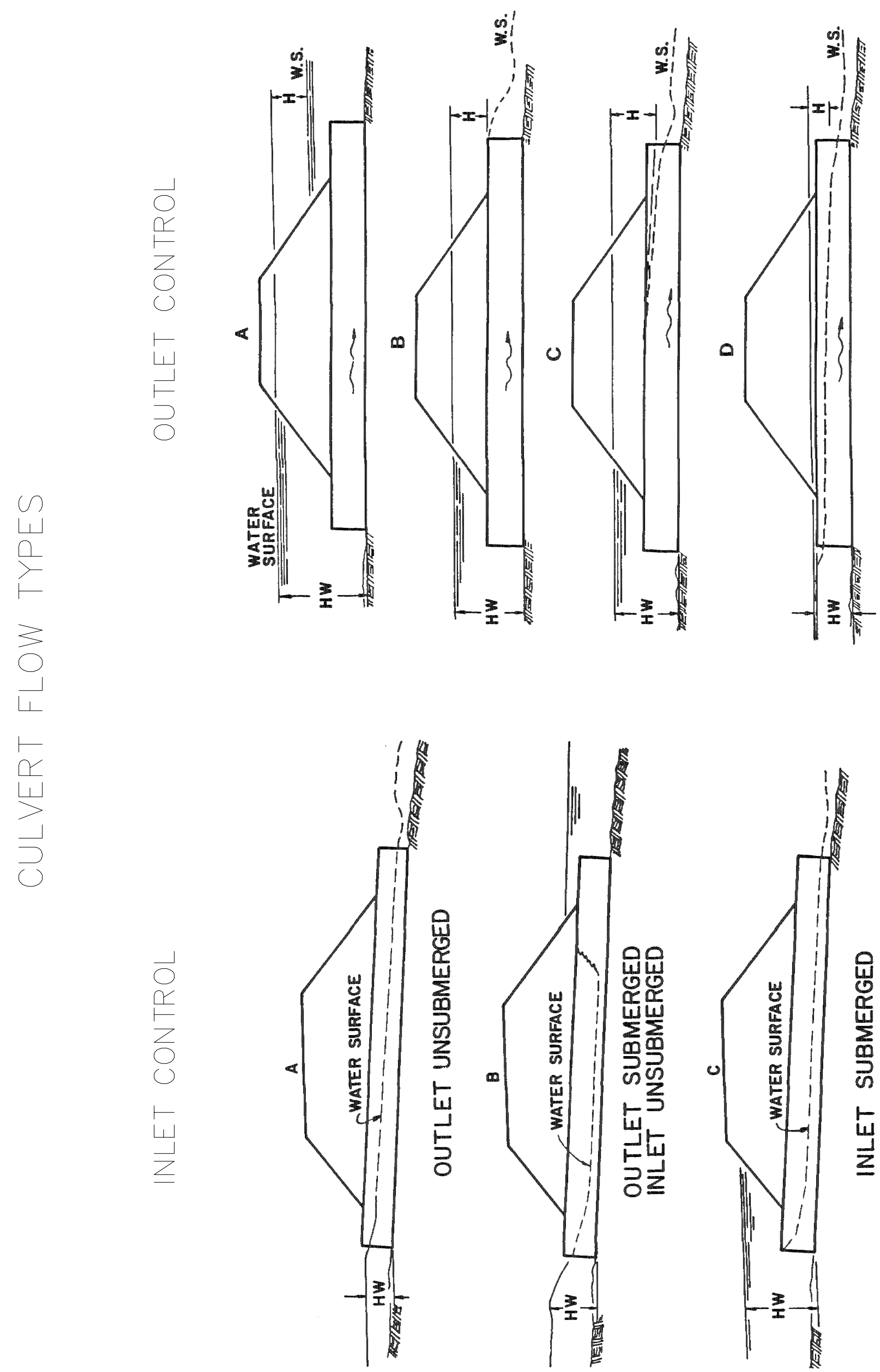

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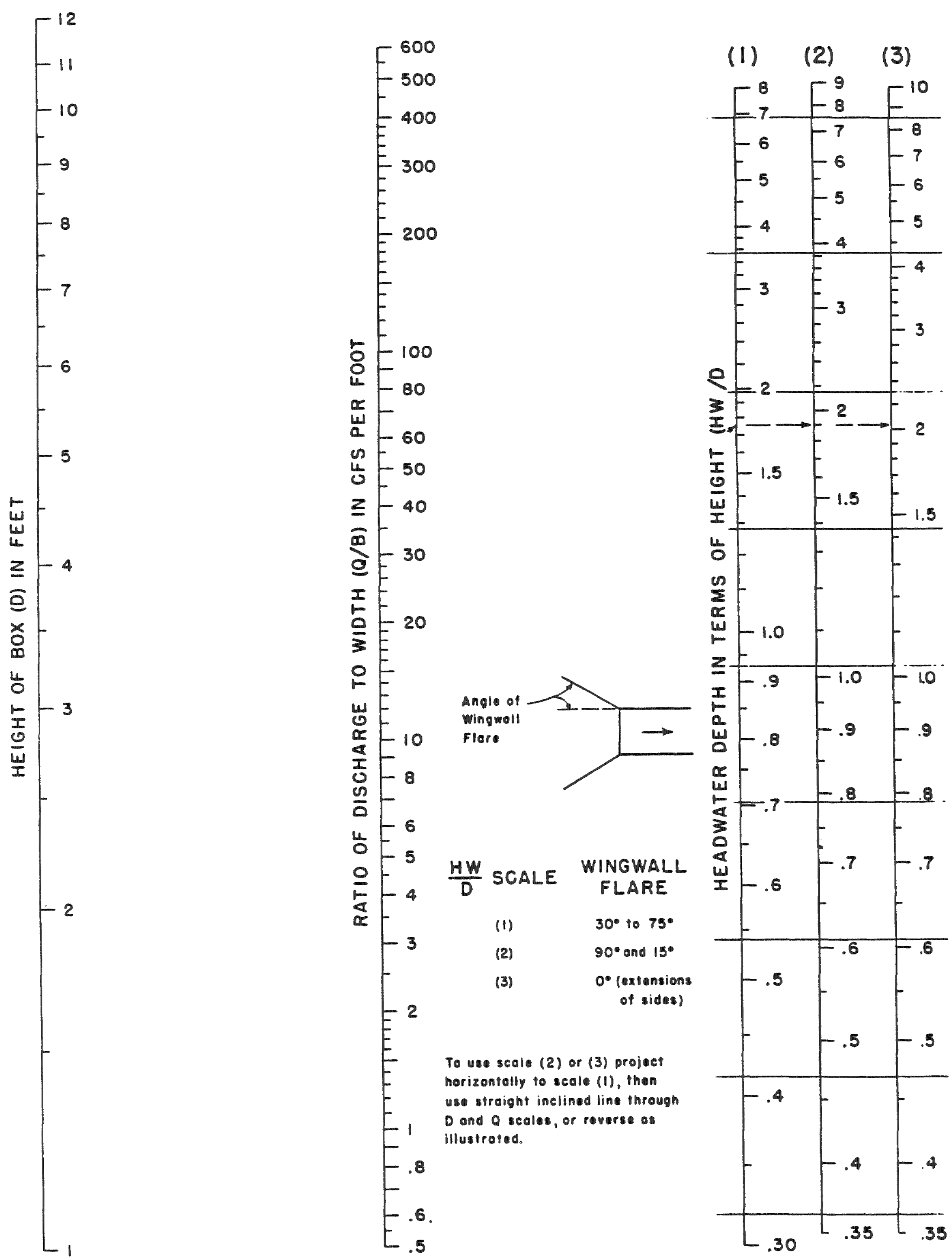


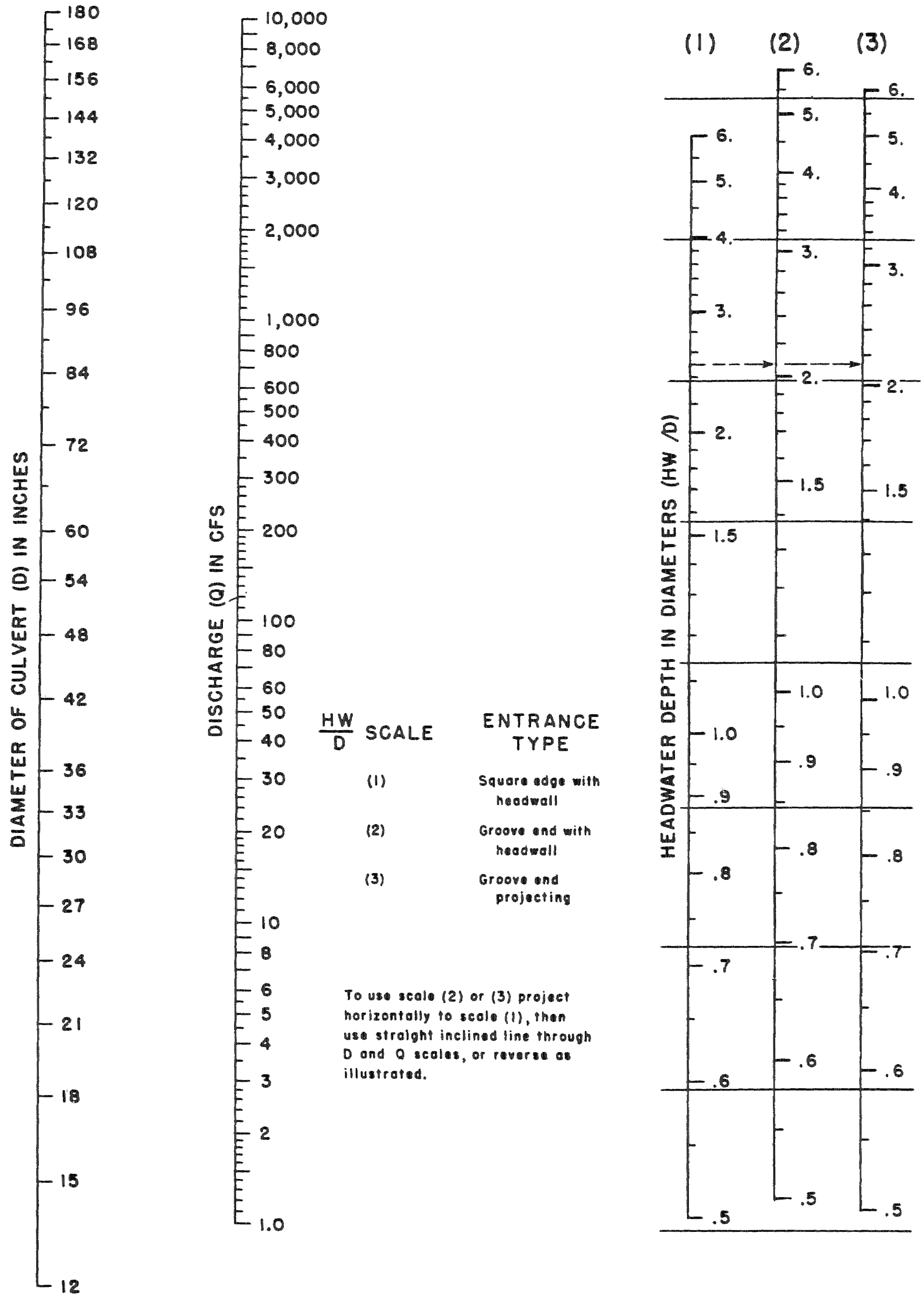
VERSION: JANUARY 2006

REFERENCE:
HYDRAULIC DESIGN OF HIGHWAY CULVERTS
FHWA-NHI-01-020

FIGURE CH13-F203
NOMOGRAPH-INLET CONTROL BOX CULVERT

COLORADO

FLOODPLAIN AND STORMWATER CRITERIA MANUAL



VERSION: JANUARY 2006	REFERENCE: HYDRAULIC DESIGN OF HIGHWAY CULVERTS FHWA-NHI-01-020	FIGURE CH13-F204 NOMOGRAPH-INLET CONTROL RCP

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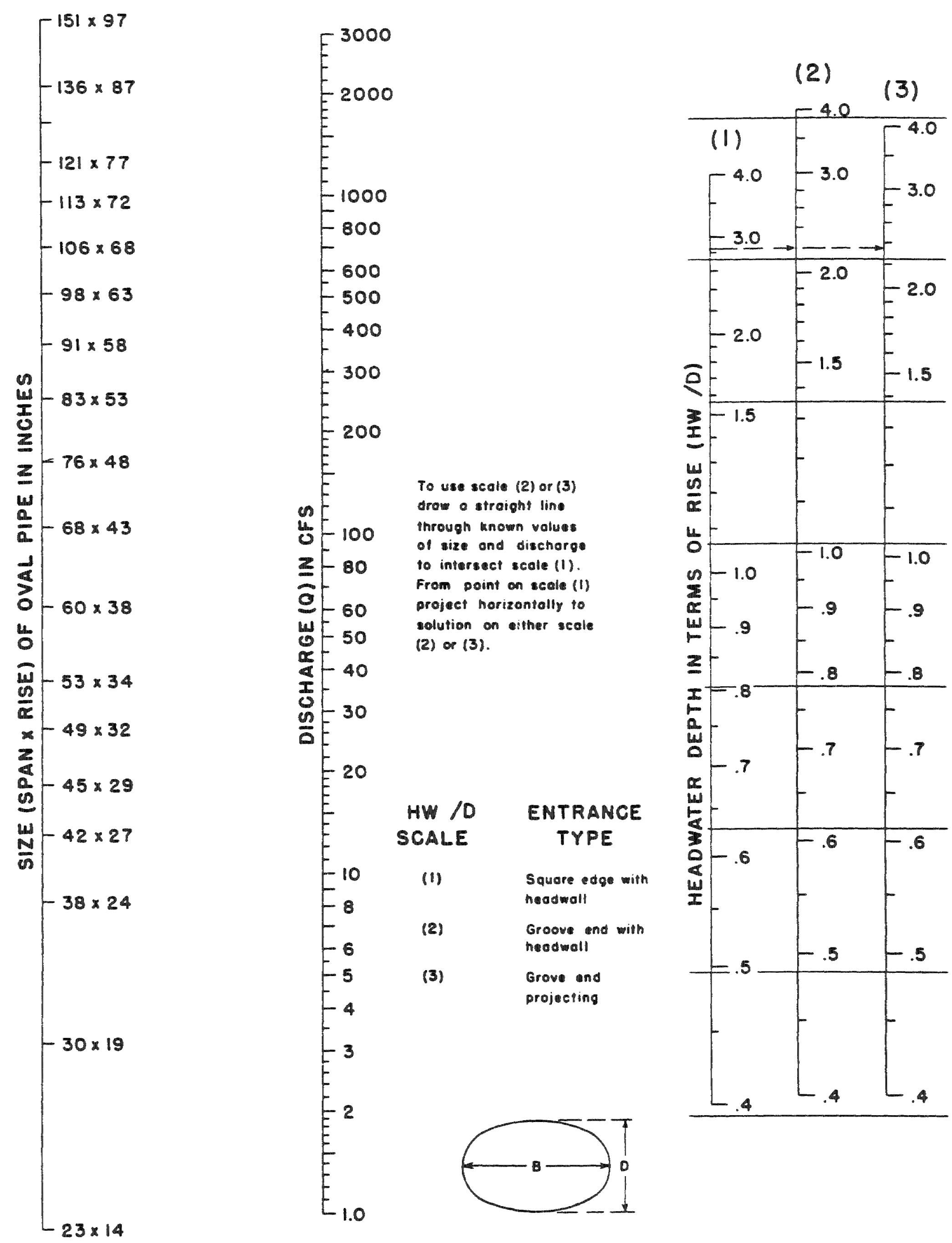

VERSION: JANUARY 2006	REFERENCE: HYDRAULIC DESIGN OF HIGHWAY CULVERTS FHWA-NHI-01-020	FIGURE CH13-F205 NOMOGRAPH-INLET CONTROL ELLIPTICAL PIPE

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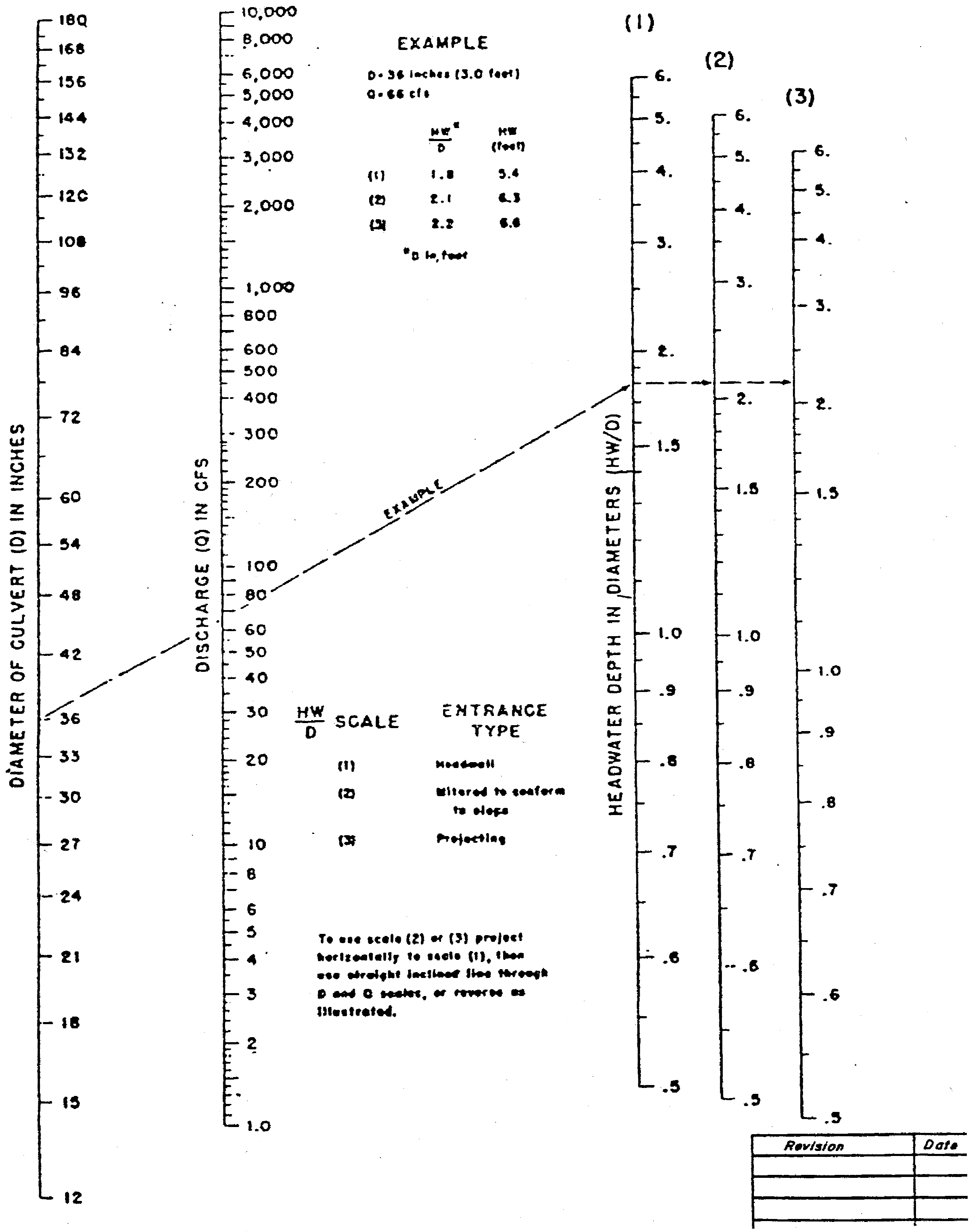
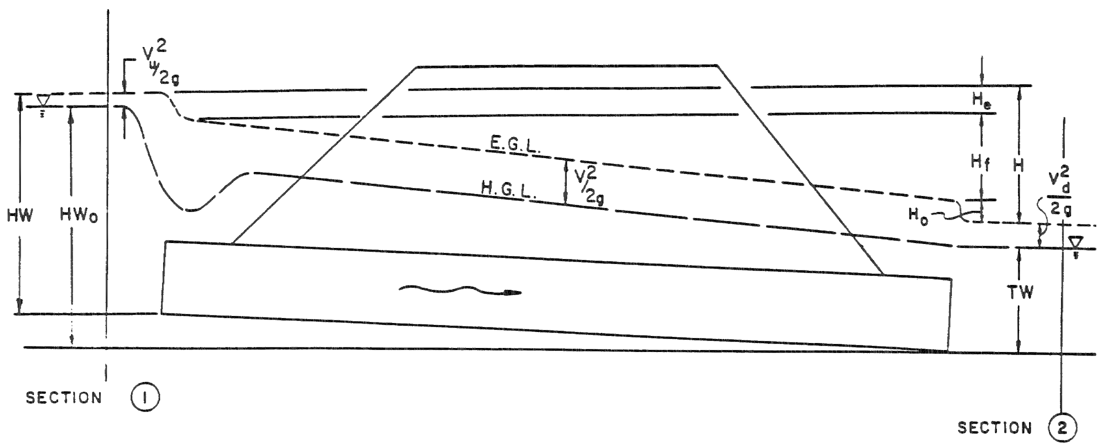


REFERENCE:
HYDRAULIC DESIGN OF HIGHWAY CULVERTS
FHWA-NHI-01-020

FIGURE CH13-F206
NOMOGRAPH-INLET CONTROL CMP



REFERENCE:
HYDRAULIC DESIGN OF HIGHWAY CULVERTS
FHWA-NHI-01-020

FIGURE CH13-F207
OUTLET CONTROL CULVERT HYDRAULICS

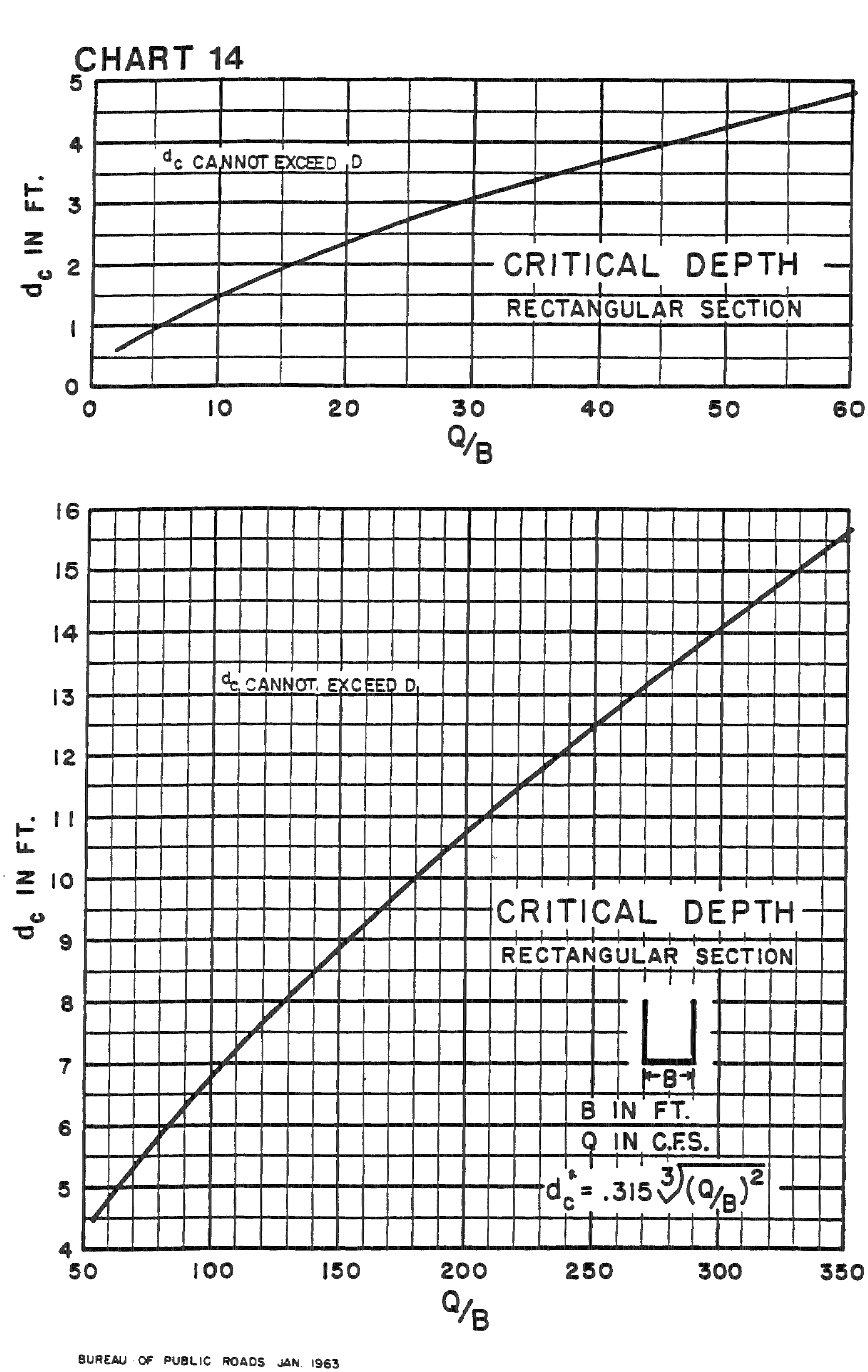
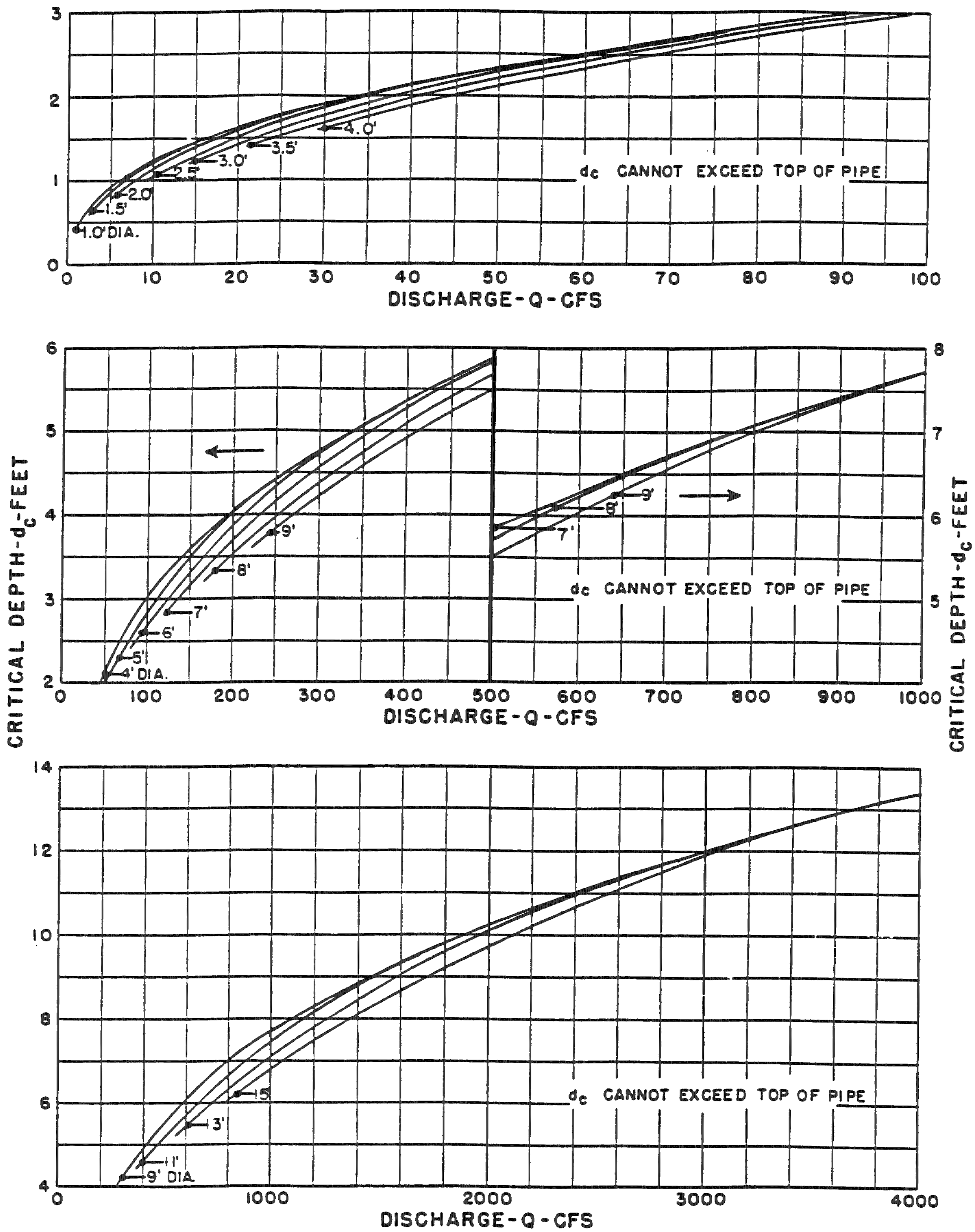


CHART 14
dc CANNOT EXCEED D
CRITICAL DEPTH
RECTANGULAR SECTION
dc IN FT.
Q/B
0
10
20
30
40
50
60
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
50
100
150
200
250
300
350
B
B IN FT.
Q IN C.F.S.
dc = .315 ∛(Q/B)²
BUREAU OF PUBLIC ROADS JAN. 1963

CHART 4



BUREAU OF PUBLIC ROADS
JAN. 1964

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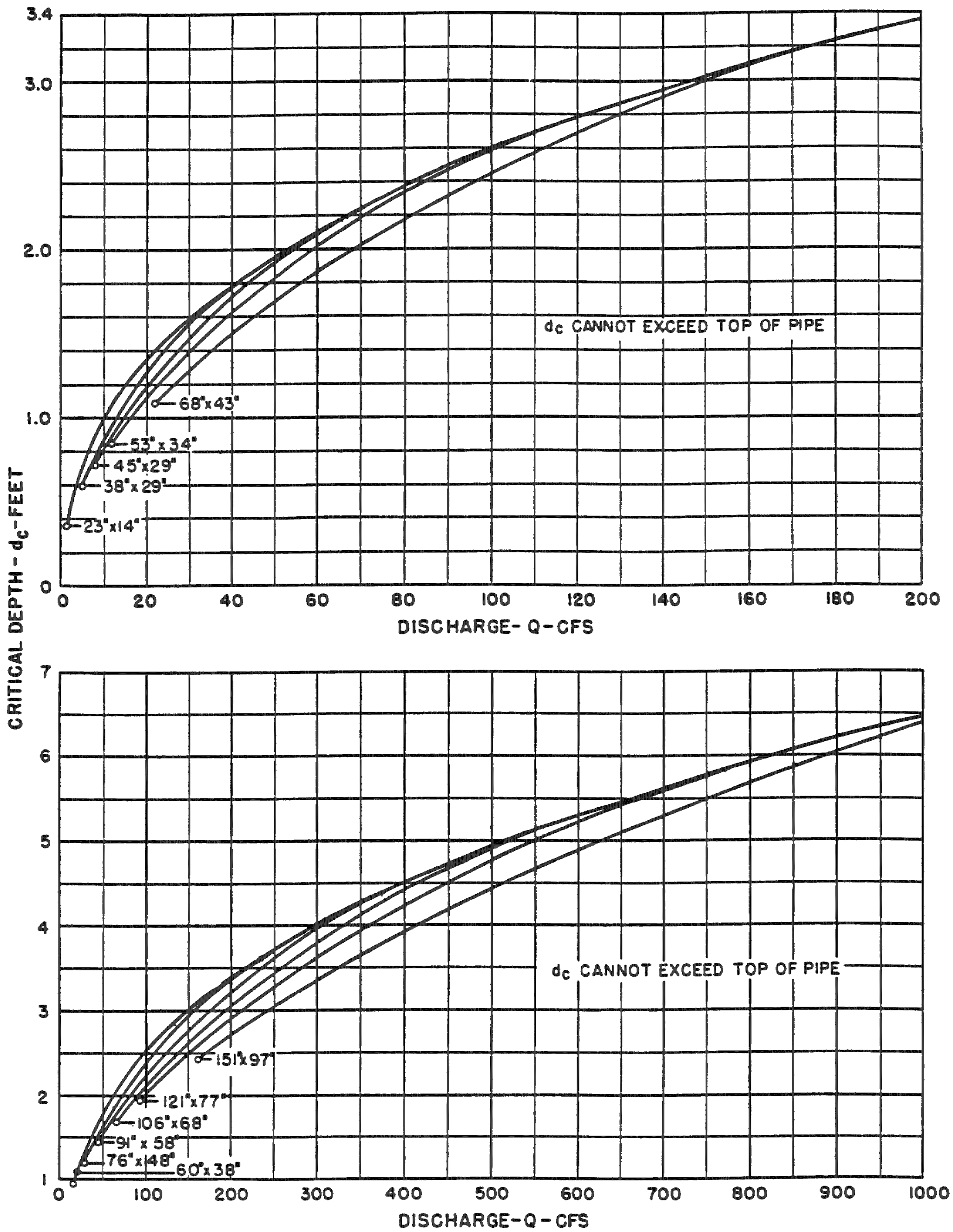
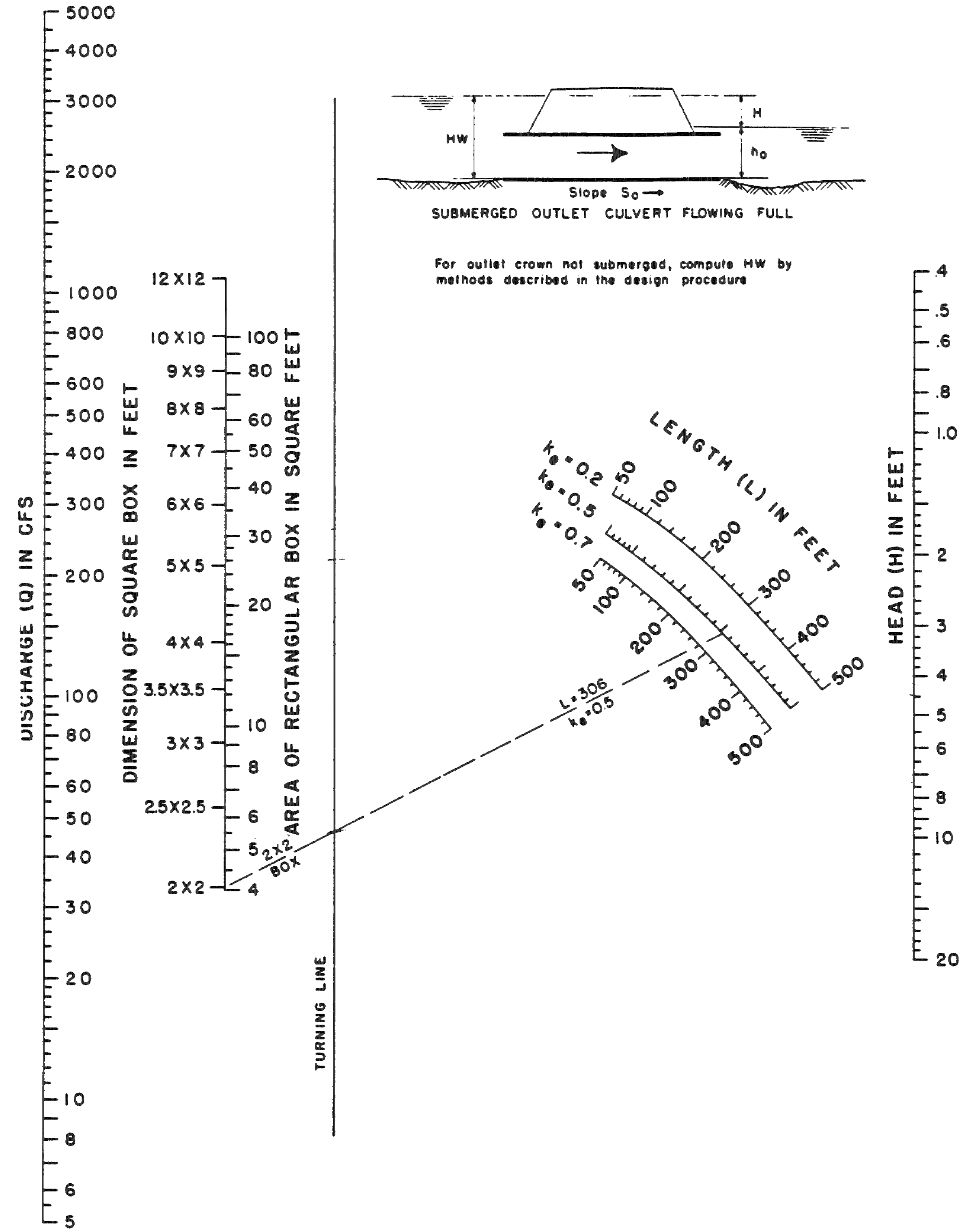
VERSION: JANUARY 2006	REFERENCE: HYDRAULIC DESIGN OF HIGHWAY CULVERTS FHWA-NHI-01-020	FIGURE CH13-F209 NOMOGRAPH-CRITICAL DEPTH RCP

CHART 31



BUREAU OF PUBLIC ROADS

JAN. 1964



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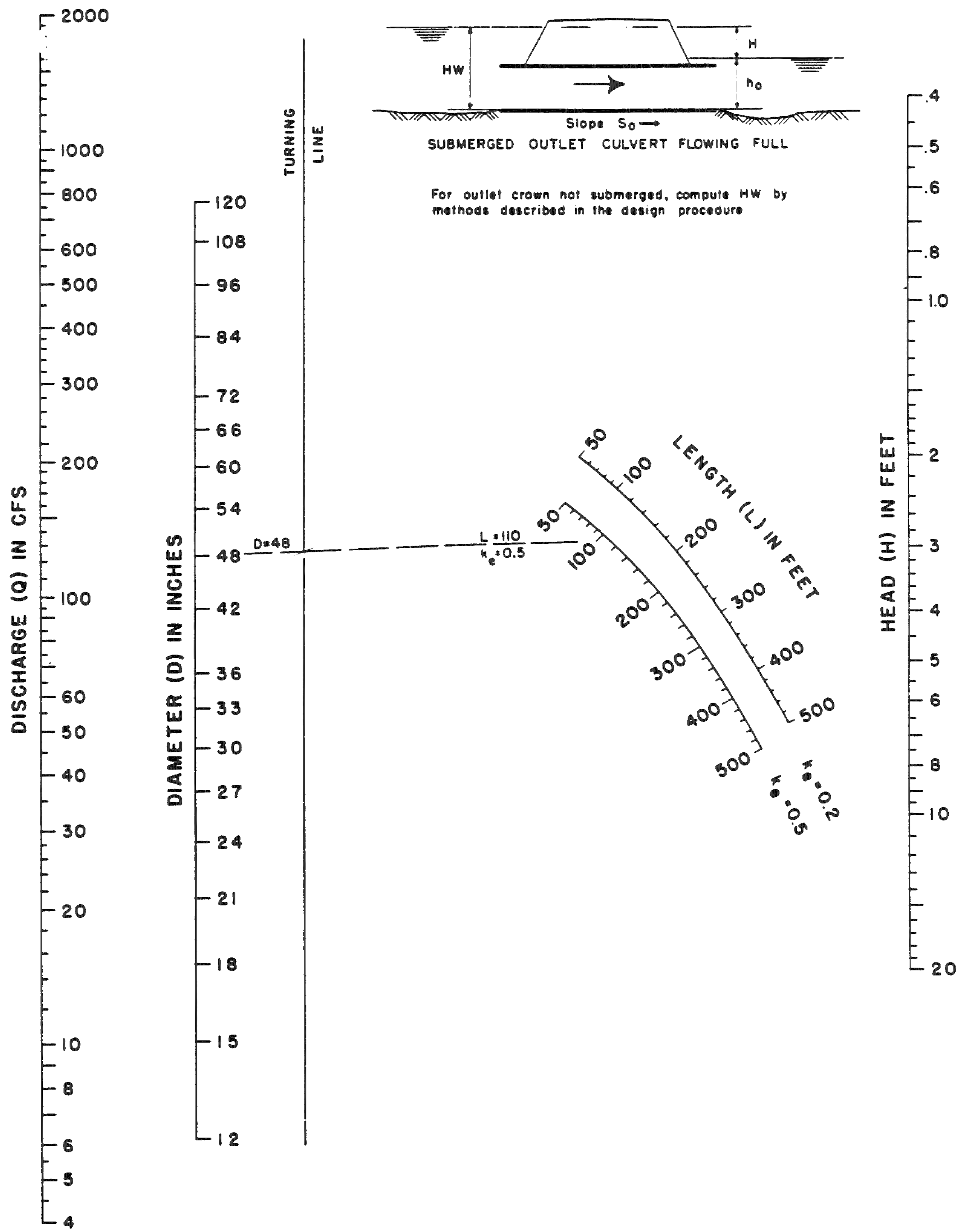
REFERENCE:
HYDRAULIC DESIGN OF HIGHWAY CULVERTS
FHWA-NHI-01-020

FIGURE CH13-F211
NOMOGRAPH-OUTLET CONTROL
BOX CULVERT

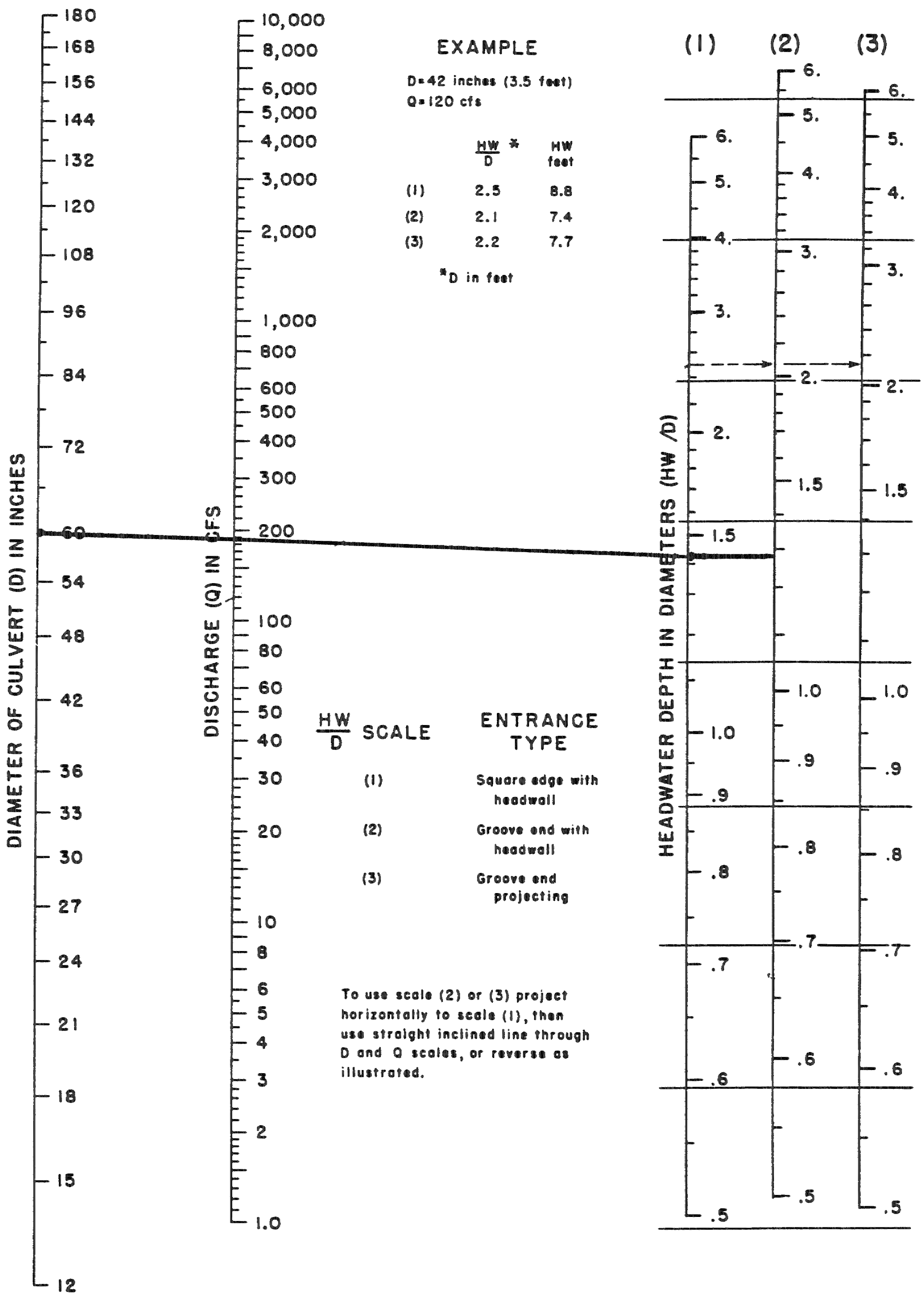


REFERENCE:
HYDRAULIC DESIGN OF HIGHWAY CULVERTS
FHWA-NHI-01-020

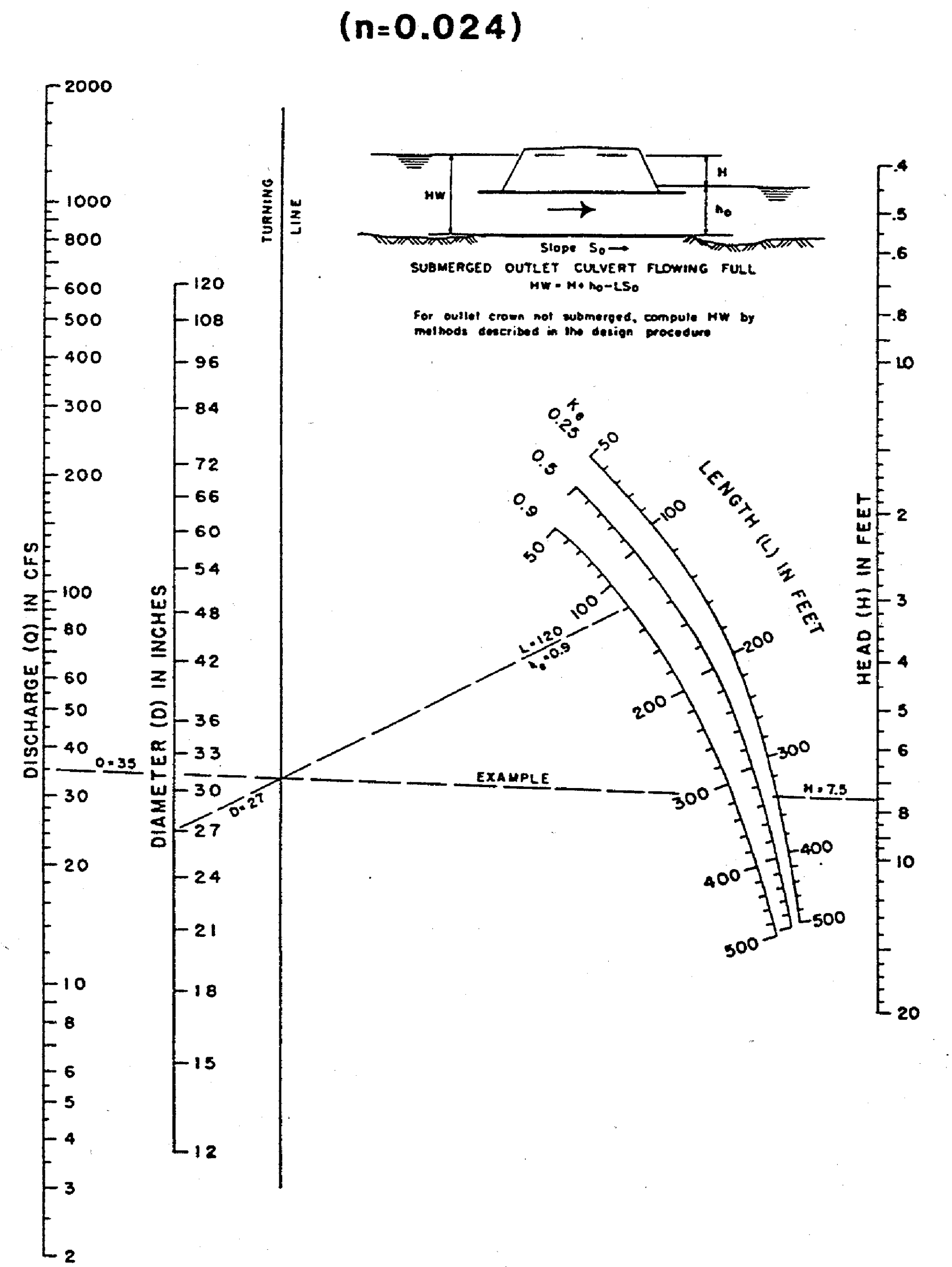
FIGURE CH13-F212

NOMOGRAPH-OUTLET CONTROL RCP

COLORADO
FLOODPLAIN AND STORMWATER CRITERIA MANUAL



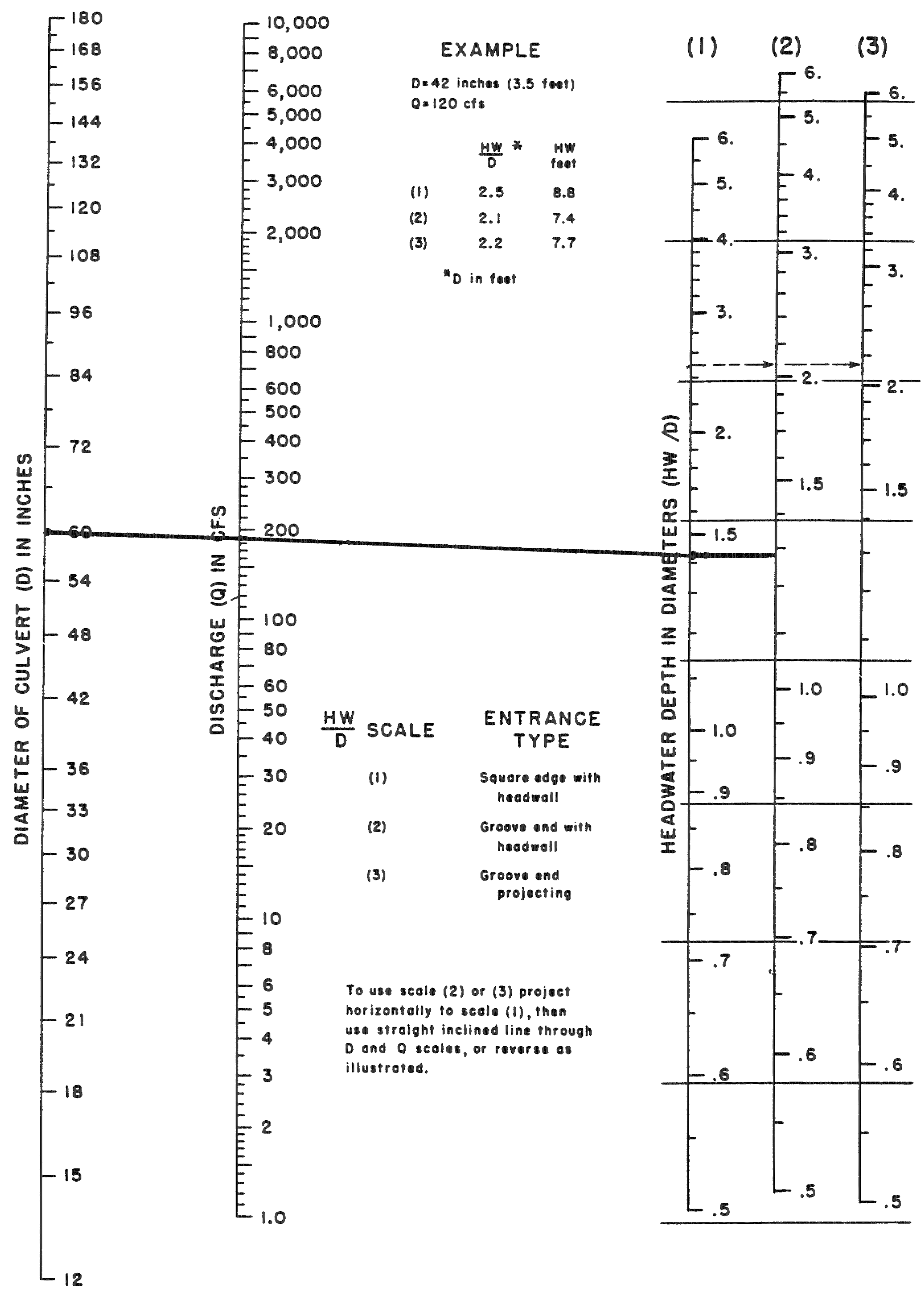
VERSION: JANUARY 2006	REFERENCE: HYDRAULIC DESIGN OF HIGHWAY CULVERTS FHWA-NHI-01-020	**FIGURE CH13-F213** NOMOGRAPH-OUTLET CONTROL ELLIPTICAL PIPE



REFERENCE:
HYDRAULIC DESIGN OF HIGHWAY CULVERTS
FHWA-NHI-01-020

FIGURE CH13-F214

NOMOGRAPH-OUTLET CONTROL CMP



VERSION: JANUARY 2006

REFERENCE:
HYDRAULIC DESIGN OF HIGHWAY CULVERTS
FHWA-NHI-01-020

FIGURE CH13-F215
EXAMPLE: NOMOGRAPH-INLET CONTROL 5-FOOT DIAMETER RCP

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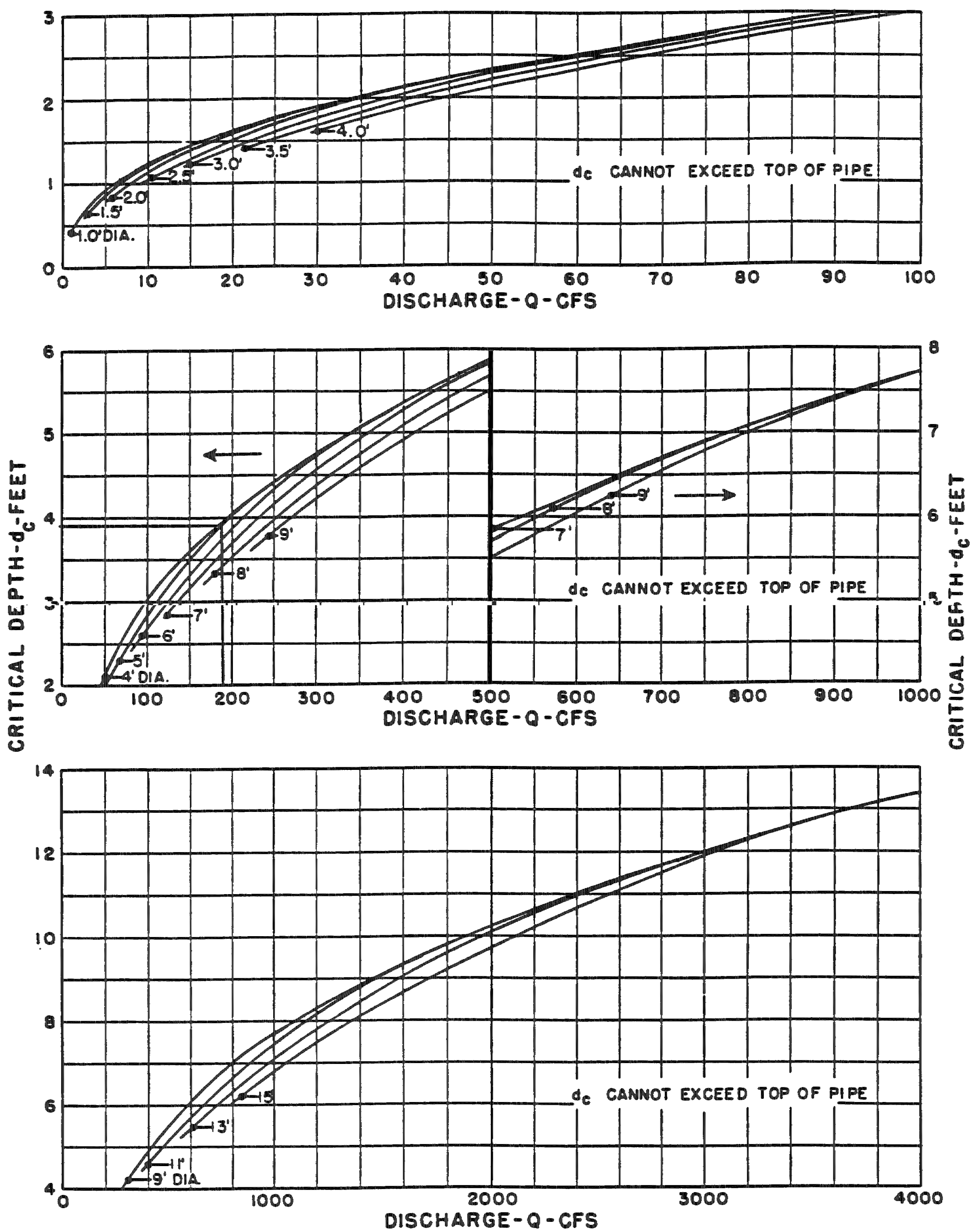
CHART 4



BUREAU OF PUBLIC ROADS
JAN. 1964

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VERSION: JANUARY 2006	REFERENCE: HYDRAULIC DESIGN OF HIGHWAY CULVERTS FHWA-NHI-01-020	FIGURE CH13-F216 EXAMPLE: NOMOGRAPH-CRITICAL DEPTH IN 5-FOOT DIAMETER RCP

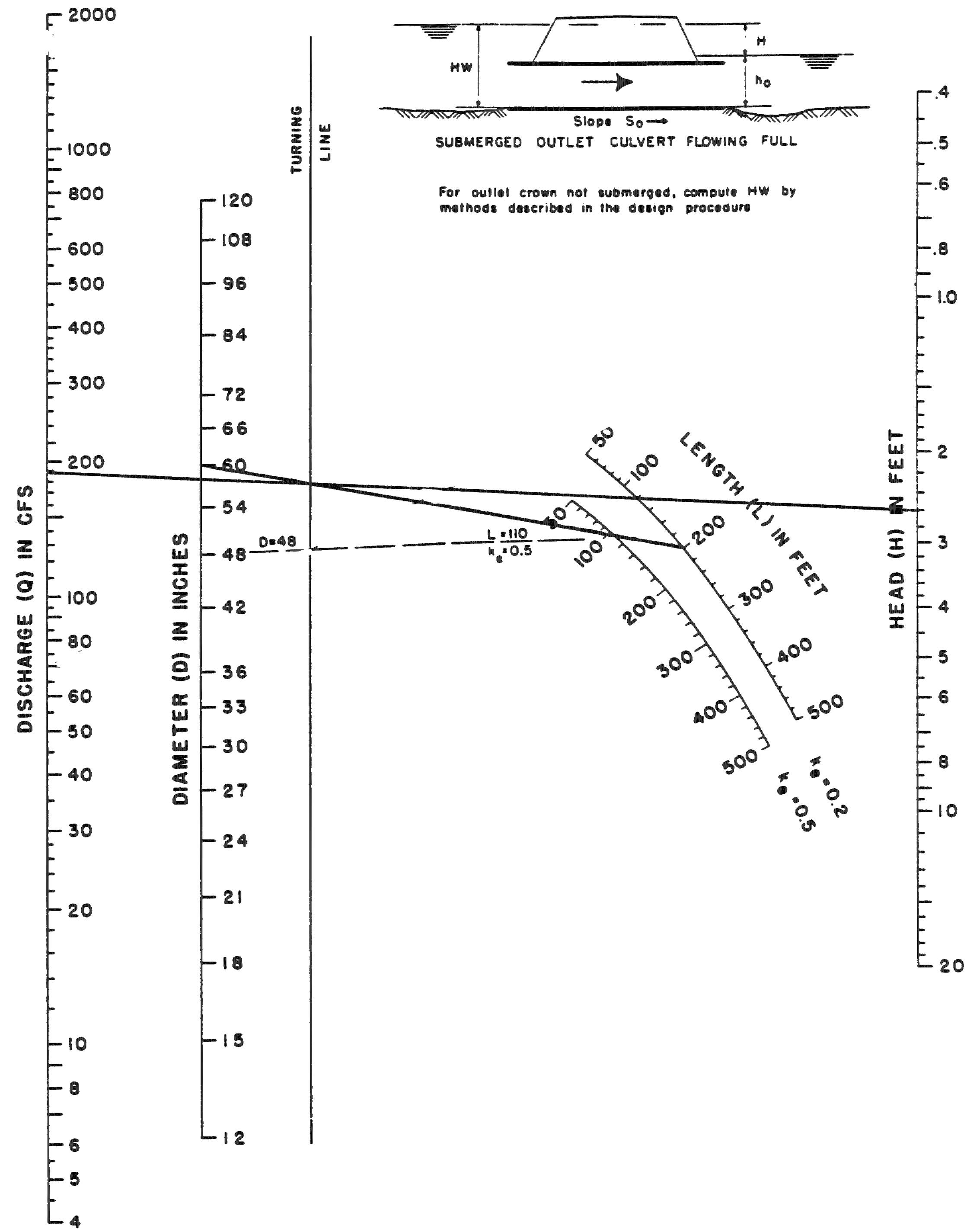

FIGURE CH13-F217
EXAMPLE: NOMOGRAPH-OUTLET CONTROL 5-FOOT DIAMETER RCP

CHAPTER 13

HYDRAULIC ANALYSIS AND DESIGN

SECTION 3

DAMS AND RESERVOIRS

CHAPTER 13
HYDRAULIC ANALYSIS AND DESIGN

SECTION 3
DAMS AND RESERVOIRS

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CHAPTER 13
HYDRAULIC ANAYLSIS AND DESIGN

SECTION 3
DAMS AND RESERVOIRS

3.1 INTRODUCTION

A dam is a man-made embankment that allows temporary or permanent impoundment of water above the natural ground, and a reservoir is a body of water (pond or lake) stored by a dam or a depression of natural ground. Dams and reservoirs can serve a single or multiple purposes including flood control, raw water supply (agricultural, municipal, and industrial), recreation, hydropower, environmental enhancement, water conservation, fish and wildlife, and others. Larger dams and reservoirs are usually designed to serve multiple purposes. Dams that serve a single purpose may include flood control dams, diversion dams, erosion control dams, and others.

This section is intended to provide practical guidelines for determining flood attenuation/storage benefits of dams and reservoirs for the purpose of determining downstream flow rates and associated floodplain boundaries. For detailed discussions on the design and analysis requirements for the jurisdictional and non-jurisdictional dams and reservoirs, please refer to the following publications:

- Office of State Engineer, State of Colorado, Rules and Regulations for Dam Safety and Dam Construction, September 30, 1988
- Office of State Engineer, State of Colorado, Dam Safety Project Review Guide, Third Revision June 1, 2000
- U.S. Department of Interior, Bureau of Reclamation, Design of Small Dams, 3rd Edition, 1987

3.2 STATE DAM SAFETY PROGRAM

Dams within the State of Colorado are classified as either "Jurisdictional Dams" or "Non-jurisdictional Dams" by the State Engineer's office based on the height of the embankment above the natural ground, the surface area of the reservoir, or the total reservoir storage capacity.

Although properly designed dams and reservoirs can provide many great benefits to communities, the problems of dam safety and the related hazard of the emergency spillways have been brought to the attention of the public by many dam failures nationwide. In order to enhance the safety of dams in the State of Colorado, the authority was granted to the State Engineer (Colorado Dept. of Natural Resources, Division of Water Resources) to implement the dam safety program. The state dam safety program is administered through the implementation of "Rules and Regulations for Dam Safety and Dam Construction" (Dam Safety Rules) by the Dam Safety Branch of the Division of Water Resources. The Dam Safety Rules apply to all dams within the State of Colorado that are constructed and/or operated for the purpose of storing water temporarily or permanently.

Dams within the State of Colorado are classified as either "Jurisdictional Dams" or "Non-jurisdictional Dams" by the State Engineer's office based on the height of the embankment above the natural ground, the surface area of the reservoir, or the total reservoir storage capacity. All existing and new dams meeting the criteria outlined below are classified as "Jurisdictional Dams" and those that don't meet the criteria are classified as "Non-jurisdictional Dams".

" *A "Jurisdictional Dam" is a dam which impounds water above the elevation of the natural surface of the ground creating a reservoir with a capacity of more than 100 acre-feet, or creates a reservoir with a surface area in excess of 20 acres at the high-water line, or exceeds 10 feet in height measured vertically from the elevation of the lowest point of the natural surface of the ground where that point occurs along the longitudinal centerline of the dam up to the flowline crest of the emergency spillway of the dam. (Dam Safety Rule 4.A. (6))*"

3.2.1 CLASSIFICATION OF DAMS

Dams are categorized by the State Engineer into four classes based on the potential damages to properties and human lives resulting from failure of a dam assuming the reservoir is full to the crest of the emergency spillway.

- Class I Dam – A dam for which loss of human life is expected in the event of failure of the dam (Dam Safety Rule 4.A. (5)).
- Class II Dam - A dam for which significant damage is expected to occur, but no loss of human life is expected in the event of failure of the dam. Significant damage is defined as damage to structures where people generally live, work, or recreate, or public or private facilities exclusive of unpaved roads and picnic areas. Damage means rendering the structures uninhabitable or inoperable (Dam Safety Rule 4.A. (5)).
- Class III Dam – A dam for which loss of human life is not expected, and damage to structures and public facilities will not be significant in the event of failure of the dam (Dam Safety Rule 4.A. (5)).
- Class IV Dam – A dam for which no loss of human life is expected, and which damage will occur only to the dam owner's property in the event of failure of the dam (Dam Safety Rule 4.A. (5)).

Dams are also categorized as minor, small, intermediate, or large structures depending on the height of embankments or the storage capacity. Dams may be re-categorized, if developments occur within the dam failure hazard areas or as modifications to existing dam structures occur.

An applicant proposing to construct or modify a jurisdictional dam is required to obtain approval from the State Engineer's Office based on their submittal guidelines, prior to beginning construction.

3.3 JURISDICTIONAL DAMS

An applicant proposing to construct or modify a jurisdictional dam is required to obtain approval from the State Engineer's Office based on their submittal guidelines, prior to beginning construction. The design, analysis, construction, maintenance, and submittal guidelines for jurisdictional dams are specified in the State Engineer's Dam Safety Rules. The spillway capacity requirement and the recommended hydrologic analysis method for jurisdictional dams are also specified.

The list and description of structures exempted from the State rules and regulations are provided in Rule 18, Dam Safety Rules. The applicant should verify the current Dam Safety Rules as the rules may change from time to time.

3.4 NON-JURISDICTIONAL DAMS

Although smaller than jurisdictional dams, depending on the location of the dam structure, a non-jurisdictional dam failure can result in substantial damages to properties and even loss of human lives. The applicant proposing to construct a non-jurisdictional dam should notify the State Engineer's Office at least 10-days prior to construction using the forms provided by the State. It should be noted that the State Engineer might require that a non-jurisdictional or exempted dam to be designed based on the same design guidelines for a jurisdictional dam, if the site-specific conditions warrant such requirements. It is recommended that the project engineer coordinate with the State Engineer's Office during the early phase of the design to determine the appropriate design criteria for the dam.

In general, non-jurisdictional dams are not required to comply with the jurisdictional dam design criteria provided in the State Engineer's Dam Safety Rules. However, all non-jurisdictional dams should be designed and constructed to safely collect and store the design flows without structural failures. Emergency spillways should be provided to control and confine the overflows. The design elements including, but not limited to, protection of embankment slopes, primary and emergency spillways, stability of embankment and foundation, seepage, compaction of fill, potential settlement, and maintenance access should be addressed. It is the design engineer's responsibility to design the dam to withstand the hydraulic, seismic and other loadings and to ensure the stability of the dam. The readers of this manual are encouraged to review the following publications in addition to the State Dam Safety Rules for detailed dam design guidelines:

- U.S. Department of Interior, Bureau of Reclamation, Design of Small Dams, 3rd Edition, 1987
- U.S. Army Corps of Engineers, Engineering and Design, EM 1110-2-1603, Hydraulic Design of Spillways, January 1990
- U.S. Army Corps of Engineers, Engineering and Design, EM 1110-2-2300, Earth and Rock-Fill Dams – General Design and Construction Considerations, September 1986

3.4.1 HYDROLOGIC ANALYSIS

For detailed discussions on the rainfall-runoff analysis methods and procedures, please refer to Chapter 9, Hydrologic Analysis.

3.4.2 SPILLWAYS

Emergency spillways should be provided to control and confine the overflows. Spillways should be sized, as a minimum, to handle the 100-year peak flows with a minimum freeboard of one foot.

3.4.3 DAM EMBANKMENT

The minimum top width of a fully compacted earthen dam embankment should be 12 feet and the side slopes should not be steeper than 3H:1V. Steeper embankment side slopes may be allowed only if the design engineer can demonstrate the stability of embankments and foundations based on acceptable engineering analyses. However, under no circumstances, should an embankment side slope steeper than 2H:1V be used. All dam structures should be designed to minimize required maintenance and to allow access by equipment and workers to perform maintenance.

Spillways should be sized, as a minimum, to handle the 100-year peak flows with a minimum freeboard of one foot.

The minimum top width of a fully compacted earthen dam embankment should be 12 feet and the side slopes should not be steeper than 3H:1V.

3.5 FLOOD CONTROL DAMS

Dams and reservoirs can be designed to help reduce the downstream flooding by capturing and storing a portion of or the entire design storm runoff from the upstream watersheds. Dams and reservoirs designed to provide flood protection for the downstream area should have the required floodwater storage capacity reserved, and the dam operation plan should clearly identify the flood control regulation purpose of the dam.

3.5.1 DETENTION DAMS

Unless an agreement can be reached with the downstream water rights holders, flood detention dam outlets should be sized to drain the stored floodwater within 24 hours of a storm event.

The majority of flood control dams are designed to detain flood flows and limit the peak outflows to the downstream receiving drainage facilities. The main purpose of a detention dam facility is to temporarily impound runoff behind the dam and reduce the downstream flow rate by allowing flows to be discharged through the primary spillway (usually a culvert) at a controlled outflow rate. The controlled outflow rate is usually determined based on either the downstream receiving facility conveyance capacity or a limit on the increase in flows over pre-development conditions. However, unless an agreement can be reached with the downstream water rights holders, flood detention dam outlets should be sized to drain the stored floodwater within 24 hours of a storm event.

The controlled detention dam outlet capacity has direct influence on the required size of the detention dam. For a given design storm event, the smaller the outlet capacity, the larger the required storage capacity of a dam. For detailed discussions on the design requirements of detention basins, please refer to Chapter 13, Section 5.

3.5.2 RETENTION DAMS

Depending on the flow conveyance capacity of the downstream drainage facility and site-specific conditions, it may be necessary to design a retention dam to capture and store the entire design storm runoff. Retention dams can be designed to either permanently or temporarily store the runoff from the upstream drainage basins.

A retention dam can be designed with a controlled outlet gate to capture the entire runoff, and later release the stored water at a controlled rate when the downstream facility can safely convey the outflows from the dam. Retention dams designed to permanently store the storm runoff are not desirable for the flood control purposes since the storage capacity available for back-to-back storm events will be minimized.

The design and construction of retention dams should not adversely impact the water rights of downstream users.

The design and construction of retention dams should not adversely impact the water rights of downstream users, unless an agreement can be reached with all impacted downstream users. Further, retention dams should have valid storage rights that can be exercised to store water when such rights are in priority.

3.6 ROADWAY AND RAILROAD EMBANKMENTS

Intentionally or unintentionally, some roadway and railroad embankments are used to store flood flows behind the embankments during storm events. Depending on the topography of the site, size of culverts, and the total runoff from the upstream drainage basins, the depth and/or the amount of water temporarily impounded behind the embankment may exceed the non-jurisdictional dam size limit.

The use of roadway and railroad embankments for flood detention purposes is exempted from the State Engineer’s Dam Safety Rules. However, if the embankment height or the storage capacity meets the State’s definition of “Jurisdictional Dams”, the project engineers should coordinate with the State Engineer’s Office during the early phase of the design to determine the appropriate design criteria for the roadway/railroad embankment.

3.7 AREAS PROTECTED BY DAMS AND RESERVOIRS

Properly designed and maintained dams and reservoirs can significantly reduce the downstream flooding problems by capturing and storing a portion of or the entire design storm runoff from the upstream watersheds. The flood attenuation and storage benefits of a dam or a reservoir should be included in the hydrologic and hydraulic analysis of the downstream drainageway, if the dam/reservoir is:

- Owned, operated, and maintained by a public agency or privately owned but publicly controlled
- Designed and operated, either in whole or in part, for flood control purposes

Dams and reservoirs constructed for other purposes (i.e. gravel pits, water supply reservoirs, etc.) may provide flood protections for the downstream areas inadvertently. However, the available flood storage capacity of these dams cannot be relied upon, since the flood storage availability cannot be guaranteed. Dams and reservoirs not specifically designed and operated, either in whole or in part, for flood control purposes should not be included in the hydrologic and hydraulic analysis of the downstream drainageway unless such a dam/reservoir aggravates downstream flooding conditions. The downstream peak flow rates and floodplain boundaries should be determined assuming such a dam/reservoir does not exist.

However, if adequate assurances have been obtained to preserve the flood routing capabilities of such a dam, then the delineation of the floodplain below the dam may, but need not, be based on the assumption that the reservoir formed by the dam will be filled to the elevation of the dam's emergency spillway. The project engineer should coordinate with appropriate government agencies and CWCB in determining whether a non-flood control dam should be included in the analysis or not.

3.7.1 STORAGE ROUTING METHOD

The flow attenuation effect of a dam/reservoir can be determined using the Modified Puls Routing Method. The Modified Puls Routing Method can be used in HEC-1, HEC-HMS, and UDSWM computer programs to route hydrographs through dams and reservoirs. Only the storage specifically reserved for the flood attenuation purposes should be included in the analysis.

Detailed discussions on the Modified Puls routing method and the use of HEC-1 and CUHP computer programs are provided in Chapter 9, Section 5.

CHAPTER 13

HYDRAULIC ANALYSIS AND DESIGN

SECTION 4

LEVEES

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LEVEES

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HYDRAULIC ANAYLSIS AND DESIGN

SECTION 4
LEVEES

4.1 INTRODUCTION

A levee is a man-made embankment that can provide flood protection from occasional flood events up to and including the duration and magnitude of the design flood event. Typically, levees are designed to provide flood protection from an estimated 1% annual chance (100-year) flood event and only for a short period of time. Levees are normally not designed to provide flood protection for a prolonged period.

The use of levees for flood control and flood mitigation projects is not encouraged by the CWCB. The CWCB does not endorse the use of levees as a form of floodplain reduction for areas along streams where new development is planned.

The use of levees for flood control and flood mitigation projects is not encouraged by the CWCB, unless other mitigation alternatives are not feasible or cost effective. Setback levees should be designed whenever possible to maintain the natural channel and some natural floodplain areas. The CWCB does not endorse the use of levees as a form of floodplain reduction for areas along streams where new development is planned.

The CWCB is not responsible for the design, examination, operation, certification, or maintenance of levee systems. The levee owner is responsible for these activities and this Section is intended to provide guidance to aid levee owners in their efforts.

Presented in this section are the general criteria, standards for the hydraulic analysis, and design of earthen levees. There are many factors which should be considered in the design of earth levee systems and these factors differ substantially from one project site to another. The site-specific geological, hydraulic, environmental, and other design factors should be identified and incorporated into the levee design. The following is a short list of some of the levee design factors that should be considered:

- Design peak flow rate, duration of flood, and water surface elevations
- Flow velocity
- Embankment height and freeboard
- Opening closures (culverts, etc.)
- Interior drainage
- Embankment erosion protection
- Embankment and foundation stability
- Under and through seepage
- Settlement
- Other site-specific factors
- Operations and maintenance

The federal minimum standard design for levees is codified in 44CFR65.10. Any levee that is to be accredited or recognized as providing flood protection on a FEMA Flood Insurance Rate Map must meet and continue to meet the criteria outlined in 44CFR65.10. Furthermore, the CWCB has adopted the criteria of 44CFR65.10 as the basis for flood damage reduction planning within its jurisdiction. By these criteria the minimum design is that a levee withstand the forces and degenerative processes associated with the 1% annual chance flood. It is important to note that no levee provides full and complete protection, indefinitely, from flooding.

4.1.1 LEVEE FAILURE

Throughout the United States, levees are used to protect properties within and adjacent to the natural floodplains. Properly designed levee systems can be an effective tool in reducing the risks to people and property associated with flooding.. Despite the design level to which a levee is constructed, changing landscapes and climatic patterns can produce flood events that exceed the design or ability of a levee to hold and failures can occur. It is understandable that levee design can be exceeded prompting levee failure but there are other mechanisms that also can also erode the integrity of a levee and its ability to provide protection against flood events.

Due to the lack of adequate levee maintenance, improperly designed levee embankments and foundations, and the occurrence of storm events greater than the design event, many levee failures have occurred throughout the United States. The most common reasons of levee failures are:

- Embankment erosion and scour
- Levee overtopping
- Seepage and piping
- Deferred maintenance

Levee failures can result in great flood losses for many communities. The importance of proper levee design practices, adequate ongoing maintenance and operations, and early flood warning programs cannot be over emphasized.

4.2 EMBANKMENT AND FOUNDATION DESIGN

The primary purpose of levees is to provide flood protection from flood events. Levees perform best against floods that occur infrequently with short durations of time. Therefore, the levee embankment and foundation are typically designed to withstand the continuous hydraulic forces for periods up to just a few days. If the site and flood conditions require the earthen levee to withstand the hydraulic loading for an extended period, the levee embankment and foundation should be designed in accordance with the design criteria outlined for earthen dams (Chapter 13, Section 3).

The design of all levee embankments and foundations should be in accordance with the guidelines established by Federal Emergency Management Agency (FEMA) and the US Army Corps of Engineers. Levee design elements including, but not limited to, closure structures, protection of embankment slopes, stability of embankments and foundations, compaction of fill, and potential settlement should be addressed. The readers of this manual are referred to the following levee design publications for detailed design guidelines:

- US Army Corps of Engineers, Engineering and Design, Design and Construction of Levees, Engineer Manual, EM 1110-2-1913, April 2001.
- US Army Corps of Engineers, Engineering and Design, Settlement Analysis, Engineer Manual, EM 1110-1-1904, September 1990.
- Federal Emergency Management Agency, Guidelines and Specifications for Flood Hazard Mapping Partners , Appendix H, Guidance for Mapping of Areas Protected by Levee Systems, April 2003.

The minimum top width of 12 feet for a fully compacted earthen levee embankment is recommended and should not be less than 10 feet under any conditions. The earthen levee embankment side slopes should not be steeper than 3H:1V. Embankment side slopes flatter than 3H:1V may be necessary depending on the site-specific design conditions. Steeper embankment side slopes may be allowed only if the design engineer can demonstrate the stability of embankments and foundations based on appropriate engineering analyses. However, under no circumstances, should an embankment side slope steeper than 2H:1V be used.

The minimum top width of 12 feet for a fully compacted earthen levee embankment is recommended and should not be less than 10 feet under any conditions. The earthen levee embankment side slopes should not be steeper than 3H:1V.

4.2.1 EMBANKMENT PROTECTION

Levee embankments should be protected against erosion and scour problems associated with a 1% annual-chance-flood event. The following is a list of some of the general factors that should be addressed in the design of embankment protections:

- Flow velocities
- Channel migration

- Sediment and debris loading
- Embankment and foundation materials
- Duration and depth of flooding
- Embankment alignments
- Transitions and bends
- Embankment widths and side slopes

If possible, environmentally friendly erosion protection measures (e.g., grass cover or grass cover with a geo-mat under layer, etc.) should be used. Please refer to Chapter 13, Section 1 for detailed discussions on the allowable maximum flow velocities of various materials and the design procedures for erosion protection measures.

It is important to evaluate the flow velocities associated with smaller storm events since these events may produce higher flow velocities, especially where flows are constricted by structures including culverts and bridges.

4.2.2 SETTLEMENT

Potential levee settlement should be evaluated and addressed during the levee design, especially when the embankment and foundation materials contain highly compressible soils. The detailed settlement analysis procedures can be found in the Army Corps of Engineers, Settlement Analysis, Engineer Manual, EM 1110-1-1904, dated September 1990.

The estimated settlement amount should be incorporated into the top of the levee grade to ensure the required freeboard will be maintained after settlement has occurred.

4.3 DESIGN TOP OF LEVEE ELEVATION

Levees in the State of Colorado should be designed to safely confine and convey, at a minimum, peak flows associated with a 1%annual-chance-flood event. The final design top of levee elevations should be set to include the required minimum freeboard.

Levees in the State of Colorado should be designed to safely confine and convey, at a minimum, peak flows associated with a 1% annual-chance-flood event also referred to as base flood. The detailed procedures for determination of the 100-year peak flow rate for a design point are provided in Chapter 9 – Hydrologic Analysis.

Once the design base flood hydrograph has been determined, appropriate hydraulic analyses should be performed to establish the base flood elevation profile based on the proposed levee alignments and channel configurations. It may be necessary to perform several iterations of hydraulic modeling in order to refine the levee alignment and design. The Army COE hydraulic computer HEC-RAS program is recommended for determination of the design water surface profiles.

The top of levee embankment grades should be set sufficiently above the calculated design water surface elevations to account for the uncertainties in design peak flow rates, water surface elevations, settlements, and other unforeseen site conditions.

The final design top of levee elevations should be set to include the required minimum freeboard. A method employing deterministic risk and uncertainty analysis can be performed to directly account for hydraulic and design uncertainties and to set the top of levee grades instead of utilizing the required freeboard.

4.3.1 FREEBOARD

The following levee freeboard requirements should be used for levees within the State of Colorado. The freeboard criteria are consistent with the FEMA requirements at the time of this criteria manual publication.

“Levees should provide a minimum freeboard of three feet above the water-surface level of the base flood (1%annual-chance-flood). An additional one foot above the minimum is required within 100-feet in either side of structures (such as bridges) riverward of the levee or wherever flow is constricted. An additional one-half foot above the minimum at the upstream end of the levee, tapering to not less than the minimum at the downstream end of the levee, is also required. (44CFR65.10)”

If the site conditions prevent conformance to the above minimum levee freeboard requirements, lesser freeboard may be allowed in accordance with the FEMA criteria set forth in 44CFR65.10. A CLOMR (see Chapter 5) may be required before the use of lesser freeboard is accepted. However, freeboard of less than 2 feet will not be allowed under any circumstances.

For levees of small drainage-ways with the 1%annual-chance-flood design peak flow rate of 100 cfs or less, the minimum required levee freeboard should be 2 feet.

4.3.2 RISK AND UNCERTAINTY ANALYSIS

In place of utilizing the freeboard requirements outlined above, a deterministic risk and uncertainty analysis may be performed to directly account for uncertainties associated with hydrology, hydraulic analysis, and embankment and foundation design. The analysis can be used to directly establish the design top-of-levee profile.

Currently, the Army Corps of Engineers does not use the freeboard concepts for design of their levee projects. The readers of this manual are referred to the following publications for detailed discussions on the risk and uncertainty analysis.

- US Army Corps of Engineers, Water Resources Support Center, Institute for Water Resources, Guidelines for Risk and Uncertainty Analysis in Water Resources Planning, Volumes I and II, March 1992.

4.4 INTERIOR DRAINAGE

The areas protected by levees may still experience flooding from other sources including runoffs from local drainage basins and backwater through levee openings. Since the levee embankments are usually higher than the adjacent protected areas, the runoff from the local interior drainage basins cannot surface drain into the channel/river on the other side of the levee. Also, during a flood event, underground storm drain outlets will be closed to prevent backflows from the channel/river, again preventing discharge of local runoffs into the channel.

Interior drainage systems should be designed to minimize human intervention, and backup systems should be provided to the extent feasible.

Interior drainage systems should be provided to drain flows from the local drainage basins into the channel/river during flood events. An interior drainage system associated with a levee system may include, but is not limited to, temporary flow retention areas with controlled outlets, various pump stations, gravity outlets to a downstream channel location, or a combination thereof. Interior drainage systems should be designed to minimize human intervention, and backup systems should be provided to the extent feasible. If human intervention is necessary, the necessary procedures and responsibilities should be clearly defined in the officially adopted maintenance and operations plan for the levee system. If the areas protected by the levee and interior drainage systems are to be removed from the flood hazard designation, the guidelines provided in 44CFR65.10 should be followed.

Interior drainage systems should be adequately sized to handle the flows from the local contributing drainage basins for the following two scenarios:

- Sized to handle expected flows from the contributing drainage basins during a base flood event of the river/channel. The expected flows from the interior contributing basins should be determined based on the joint probability of the interior and exterior flooding.
- Sized to handle base flood flows from the interior contributing basins (in combination with the other drainage facilities including storm drains & etc.) with expected water levels on the other side of the levee (channel/river). The flows and associated water surface elevations of the receiving channel/river should be determined based on the joint probability of the interior and exterior flooding.

4.4.1 CLOSURES

All levee openings including culverts should be adequately designed to not adversely impact the embankment stability and should be provided with closure devices that can prevent flood flows from flowing though the openings to inundate the areas protected by the levee system.

Culvert closure devices can be classified as automatic or manual. Automatic closure devices include flap gates, Tideflex check valves, and other devices not requiring human intervention. Manual closure devices include slide type gates, sluice gates, and other devices that require human intervention.

Automatic closure devices should be used for openings where the water level can rise in a short time and for situations where the gates cannot be easily accessed. The flap-gates should not be used to provide opening closures where debris can easily prevent the flap gates to close completely. Manual closure devices may be used where flood flows rise slowly allowing ample time for safe operations. If the site conditions warrant, a secondary emergency gate may be necessary to minimize the risk of backflows through the opening.

4.5 OPERATIONS AND MAINTENANCE

In order for levees to be recognized as providing flood protection, levees should be designed in accordance with the guidelines set forth in this section. In addition, the following levee ownership and operations and maintenance requirements should be followed.

4.5.1 OWNERSHIP

Levees operated and/or maintained by a private party will not be recognized as providing flood protection. Levees for which the local, state, or federal government has responsibility for operations and maintenance may be considered as providing flood protection provided that the other criteria outlined in this section are satisfied.

4.5.2 CERTIFICATION

In accordance with FEMA NFIP regulations 44 CFR 65.10(a) (www.fema.gov), it is the responsibility of the community or other party seeking recognition of a levee system at the time of a flood risk study or restudy to provide the data outlined in 44 CFR Section 65.10. Neither CWCB nor FEMA will be conducting detailed examinations of levees to determine how a structure or system will perform in a flood event.

Data submitted to support that a given levee system complies with the structural requirements set forth in paragraphs (b)(1) through (7) of 44 CFR Section 65.10 must be certified by a registered professional engineer. Also, certified as-built plans of the levee must be submitted. In lieu of these structural requirements, a Federal agency with responsibility for levee design may certify that the levee has been adequately designed and constructed to provide protection against the 1% annual chance flood.

Levees that have been certified through the aforementioned procedures may require recertification at during subsequent flood mapping updates or as a result of changing physical characteristics of the flooding source or levee itself. When recertifying levees the levee owner must obtain the existing certification documentation, and demonstrate that the levee has been adequately maintained through inspection and maintenance records. There also needs to be a current survey of the levee by a registered PE or PLS indicating that the conditions of the levee have not changed.

If the existing certification documentation can not be obtained, there are no inspection and maintenance records, or the current survey shows that conditions have changed, the levee [65.10 paragraph (b)(1) through (7)] will need to be recertified by a registered Professional Engineer or Federal Agency with the responsibility for levee design.

4.5.3 HUMAN INTERVENTION

Levees that require human intervention during or shortly before a flood event (i.e., sandbagging, earthfill, flashboards, etc.) in order to increase the levee heights to the required 1% annual-chance-flood design top of the levee grades (including freeboard) should not be considered as providing the base flood protection. Human intervention necessary for the operation of opening closures and mechanical functions of internal drainage systems (i.e., manual backup of pumping stations & etc.) may be considered only if the operation procedures are clearly defined in an officially adopted operations and maintenance manual.

4.5.4 OPERATIONS AND MAINTENANCE PLAN

For levee systems to be recognized as providing the designed 1% annual-chance-flood protection, comprehensive operations and maintenance plans should be prepared, followed and officially adopted by local, state, or federal agencies. The operations and maintenance plan criteria outlined in the FEMA NFIP regulations 44 CFR 65.10 (www.fema.gov) should be followed.

Levees should be inspected periodically, and after storm events, and any considerable damage should be repaired promptly.

4.6 FLOODPLAIN DELINEATION OF AREAS PROTECTED BY LEVEES

If the levee satisfies the appropriate requirements, as outlined in Chapter 13, Section 4.5.2, the protected area (landward side of the levee) is to be designated as shaded Zone X or the appropriate zone determined by the interior drainage analysis (e.g., Zone AH). If an interior drainage analysis does not exist or has been determined to be insufficient in the levee investigation, further analysis and investigation of the residual flood risk associated with interior drainage shall be coordinated with the CWCB. The CWCB may opt to authorize additional analyses or make a recommendation for the flood hazard mapping on the protected side of the levee.

If the subject levee does not meet the requirements stated in Section 65.10 of the NFIP regulations, the levee owner shall recompute the 1-percent-annual-chance flood elevations as if the levee did not exist. None of the subject levee will be recognized as providing 1-percent-annual-chance flood protection unless there are portions of the levee system that can meet requirements of Section 65.10 of the NFIP regulations independent of the remaining levee system. The levee owner shall consider the 1-percent-annual-chance flood levels on the unprotected side (river side) of the levee to be equal to the 1-percent-annual-chance water-surface elevations computed with the levee in place.

If the 1-percent-annual-chance flood level, with the levee in place, is higher than the top of the levee, the levee owner shall use either the computed 1-percent-annual-chance flood levels on the riverside of the levee or the top-of-levee elevation, if appropriate. The 1-percent-annual-chance flood levels shall then be recomputed for the landward side of the unrecognized levee as if the levee did not exist.

If water-surface elevations of the 10-, 2-, and 0.2-percent-annual-chance floods on the river side of the levee are higher than the top-of-levee elevations, the levee owner also shall consider those elevations to be equal to the top-of-levee elevations. If those elevations are lower than the top-of-levee elevations, the levee owner shall use the elevations as computed on the Flood Profile. The levee owner shall not make further analyses for the conditions without the levees shall not be made for floods with frequencies less than the 1-percent-annual-chance flood.

For the levees that do not satisfy the minimum requirements, the levee owner might draw a maximum of five Flood Profiles on the profile sheet, representing the 10-, 2-, and 1-percent-annual-chance floods with levee elevations, and the 1- and 0.2-percent-annual-chance floods without levee elevations.

If the "with levee" base (1-percent-annual-chance) flood elevations (BFEs) are higher than the "without levee" BFEs, the levee owner shall show a line running along the levee centerline, separating the areas of different BFEs, on the floodplain mapping. Otherwise, the levee owner shall show only "without levee" BFEs on the floodplain mapping.

If the levees do not meet the requirements of Section 65.10 of the NFIP regulations, the levee owner shall compute the regulatory floodway widths for the without levee condition using the equal conveyance reduction method. In the "Regulatory" column in the Floodway Data table, the levee owner shall include two BFEs, representing river side and land side conditions, if the former elevation is higher than the latter elevation. Otherwise, the levee owner shall show without levee BFEs in the Floodway Data table. At a tributary confluence with the main stream, the levee owner shall show the BFEs from the main stream as the regulatory elevations if they are higher than the river side or land side BFEs of the tributary.

The above procedures for the determination of BFEs and regulatory floodways also apply to the conditions where levees exist on both sides of the stream. In these cases, the evaluation shall include the possibility of simultaneous levee failure, failure of only the left side, and failure of only the right side, and shall consider simultaneous levee failure for both the BFE and regulatory floodway computations. The levee owner shall contact the CWCB for guidance on the evaluation of levee systems under these circumstances.

Regulatory floodway boundaries are to be delineated at the landside toe of mainline and tributary levees that are credited with providing 1-percent-annual-chance flood protection. Thus, the community's floodplain management ordinance must prohibit encroachment on the levee, which could jeopardize levee integrity or effectiveness. It may also be appropriate to place regulatory floodways at levees providing a lower level of protection if encroachment on the river side of the levee is of concern to the community. The levee owner that is performing the analysis shall consult with community officials and the CWCB to resolve this situation.

For levee systems where an area of land may be totally or partially surrounded by levees or where two or more flooding sources join that have levees on both sides of the stream, the levee owner that is performing the analysis shall contact the CWCB before proceeding with any analyses for levee failures. For these complex situations, the flood hazard in the area that would have been protected by the non-failed levee(s) must be based on selection of failure scenarios that yield the highest BFE or flood hazard.

4.7 SETBACK LEVEES

Properly designed levee systems can effectively provide the designed flood protection for many communities and allow existing developments to be removed from the floodplains. Levees have been used because they usually cost less and require relatively small amounts of land when compared to other flood control options. Also, there may be site-specific constraints that prevent the use of other flood control options. However, when and if the levees fail, the resulting flooding can be devastating for many communities.

Levees should be used only if other reasonable and safer flood control methods (i.e., relocation, channel modification/improvement, fill, elevation, acquisition, etc.) cannot be utilized due to the site-specific constraints or if other methods were determined economically impractical. If levees are to be used, setback levees should be used where possible. Setback levees are less susceptible to failures because levees are placed substantially away from the channel, allowing flood flows to spread out thereby reducing the flow velocity acting on the levee embankments. Setback levees can also allow some natural channel migration to occur without impacting the levee embankment and foundation and usually results in less environmental impacts.

4.8 FLOODWALLS

When the right-of-way necessary for the construction of new levees or enlargement of existing levees is not available or too expensive, floodwalls may be used in place of earthen levees. Floodwalls are considerably more expensive to design and construct compared to earthen levees and therefore floodwalls are rarely used outside of urban areas.

Most commonly used floodwall types are cantilever T-type and cantilever I-type walls and they are shown schematically on Figure CH13-F401. Floodwalls should be structurally designed to withstand the hydraulic forces and other loadings. The top of floodwall grades should be determined following the same guidelines as the earthen levee as outlined in Chapter 13, Section 4.3.

If floodwalls are used to confine flood flows and remove areas out of natural floodplains, a CLOMR should be obtained prior to the construction to allow local and state agencies and FEMA to review and comment on the design prior to the wall construction.

For detailed discussions on the design of floodwalls, readers are referred to the following publications:

- US Army Corps of Engineers, Engineering and Design, Retaining and Flood Walls, Engineer Manual, EM 1110-2-2502, Sept. 1989

CHAPTER 13

HYDRAULIC ANALYSIS AND DESIGN

SECTION 5

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DETENTION BASINS

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5.1 INTRODUCTION

Detention of flood flows for all development and redevelopment projects is recommended in accordance with the policies presented in Chapter 3, Section 2 of these CRITERIA. The main purpose of a detention facility is to store the excess stormwater runoff associated with increased basin imperviousness and discharge this excess at a rate similar to the rate experienced from the basin without development. "Grandfathering" existing imperviousness is not acceptable, and comparison of runoff should be made against pre-developed conditions.

Rapid urban runoff can equate to higher downstream peak flows. Detention reduces the peak of the hydrograph and is considered a viable method to reduce urban drainage infrastructure costs. Detaining the peak of the storm runoff can significantly reduce downstream peak flow and flood hazards, as well as reduce pipe and channel sizes downstream. Storage also provides for sediment and debris collection, which helps to maintain water quality in downstream channels and streams.

This chapter provides the criteria for design and evaluation of all detention facilities. The CRITERIA strongly encourages integration of detention and water quality treatment requirements in accordance with the strategies presented in Chapter 15. All detention facilities must have adequate maintenance access and be maintained on a regular basis.

5.2 DESIGN CRITERIA

5.2.1 DESIGN STORM EVENT FREQUENCY

It is recommended that detention facilities be designed to control significant (minor event) runoff, with provisions to safely route flooding up to a 100-year storm (major event). Recommended design events are the 10- and 100-year recurrence interval floods, and may be combined with the water quality capture volume (WQCV), which controls up the first ½-inch of runoff. Designs which account for all three events include multi-stage outlets and are very effective at protecting downstream properties from flooding and protecting receiving drainageways from erosion and instability, commonly a result from development.

Designing detention facilities for less than a 100-year event in effect creates a "residual floodplain" in a major event. For example, if detention ponds and conveyance facilities are sized for a 25-year event, then there will be quantifiable surface flooding in a 100-year event. The Urban Drainage & Flood Control District completed a detailed evaluation of alternatives in the "Big Dry Creek Northern Tributaries Outfall Systems Plan Update" that considered *potential* cost savings by sizing conveyance structures for less than a 100-year event. However, by including the cost of land encumbrance from the residual floodplain, there may not be any savings.

5.2.2 SIZING METHODOLOGY FOR VOLUMES AND RELEASE RATES

Routing calculations are needed to design storage facilities. Some municipalities utilize empirical equations to size detention volumes and release rates for on-site storage facilities on small developments. All storage facilities for basins larger than 90 tributary acres must be analyzed with reservoir routing techniques. The Federal Aviation Administration (FAA) detention method is acceptable as long as a discharge rate, which varies with flood stage, is used. Table 5.1 summarizes acceptable methodologies for sizing detention facilities. Input and output listings used with software programs shall be provided in electronic and hard copy formats.

Table 5.1. Detention Sizing Methodologies

Method	Site Conditions	Comments
Simplified Method Based on Empirical Equations	Small basins less than 90 acres. Do not use when off-site flows are present. Use with care when multi-stage controls are used.	This method has limited application subject to the site conditions.
Hydrograph Routing Procedures (HEC-1, HEC-HMS, Colorado Urban Hydrograph Procedure (CUHP)/Stormwater Management Model (SWMM), EPA-SWMM. UD-Pond Wizard or UD-Detention Spreadsheet are available for free download from www.udfcd.org)	Larger basins greater than 90 acres. Required when upstream detention facilities are present in watershed.	A historic imperviousness of 2% or less must be used in this procedure. The Natural Resources Conservation Service (NRCS) soil classification for the land area must also be used. Off-site tributary areas to the facility must be included in sizing volumes.

The maximum allowable unit release rates for the 10- and 100-year volumes shall be based on the predominant soil type at a site in accordance with Table 5.2. If NRCS soil surveys are not available for a site, then site-specific soils evaluation shall be completed.

**Table 5.2.
Maximum Allowable Unit Flow Release Rates (cfs)
per Tributary Area (/acre)**

Design Return Period	NRCS Soil Group and Release Rate (cfs/acre)		
	A	B	C&D
10-year	0.13	0.23	0.30
100-year	0.50	0.85	1.00

Above all, the release rate cannot exceed a non-hazardous discharge capacity of the downstream drainage system. Some regulatory jurisdictions require an analysis of pre-developed hydrology rather than using the simplified release rates based on soil type shown above.

5.2.3 ROADWAY EMBANKMENTS

> *Only regional, publicly-owned and maintained detention facilities should be considered in regional hydrologic computations.*

Inadvertent detention often occurs upstream of roadway embankments if culverts are undersized. Large storm events will impound stormwater upstream of the culverts and the "spillway" is overtopping of the roadway. Unless these roadway impoundments are dedicated drainage facilities, their impact on reducing the downstream flow rate cannot be considered. The difficulty in quantifying the effects of inadvertent detention facilities is the virtual impossibility of assurance of their continued long-term performance or existence. There is generally no guarantee that the culverts will not be replaced in the future with larger structures. Only regional, publicly-owned and maintained detention facilities should be considered in hydrologic computations.

5.2.4 SITE CONSIDERATIONS

Impacts to upstream and downstream properties relative to proposed detention facilities shall be considered and minimized through appropriate facility design. If an adequate outfall does not exist or if some portions of the proposed development drain directly off-site, then it may be necessary for the new development to over-detain, thereby incorporating more restrictive release rates and larger detention volumes.

Designs shall take into account the location of structures near detention facilities and plan accordingly to prevent seepage into basements and structural damage.

5.2.5 MAINTENANCE

Maintenance is extremely important to long-term function of stormwater facilities. All detention facilities shall be designed with adequate maintenance access provisions and in a manner that facilitates ease of maintenance. Appropriate measures (typically an all-weather access road to the basin bottom) shall be included to allow for access by maintenance equipment. As a general rule of thumb, inspect all detention ponds and outlets one a year, preferably during wet weather, mow as required (at least twice a year), and remove accumulated sediment (after site construction in the tributary basin, and at least every 5 to 10 years once the basin is developed and stabilized).

Utilizing a forebay at all outfalls into the pond concentrates the largest pollutants and heavy sediments in one location for ease of maintenance. A forebay will reduce the frequency of dredging the detention pond Otherwise; regular maintenance is required throughout the pond site.

5.3 DETENTION METHODS

Inadvertent detention often occurs upstream of roadway embankments if culverts are undersized. Large storm events will impound stormwater upstream of the culverts and the "spillway" is overtopping of the roadway. Unless these roadway impoundments are dedicated drainage facilities, their impact on reducing the downstream flow rate cannot be considered. The difficulty in quantifying the effects of inadvertent detention facilities is the virtual impossibility of assurance of their continued long-term performance or existence. There is generally no guarantee that the culverts will not be replaced in the future with larger structures. Only regional, publicly-owned and maintained detention facilities can be considered in hydrologic computations.

5.3.1 ON-LINE VERSES OFF-LINE

In-line storage facilities are located within the flow path of the drainageway or conveyance system. Low and high flows pass through an on-line detention facility. Off-line systems are adjacent to the drainageway and only fill when a specific flow level is exceeded, and empties when sufficient conveyance becomes available in the downstream system.

5.3.2 DRY VERSES WET

This wet pond in Basalt combined stormwater management into a community amenity.

A majority of detention ponds are designed to empty completely between storms. However, sometimes it is desirable to maintain a permanent pool in the stormwater facility during dry weather for habitat, recreation and/or aesthetic reasons. Wet ponds are usually more expensive than dry detention basins and usually serve a large watershed. Stormwater surcharges the permanent pool of the wet pond for controlled release after the storm. The key to a wet pond design is to maintain the permanent pool. Aeration can be an additional consideration to avoid the negative impacts of stagnation. A water budget, which compares inflows and outflows, is critical to a wet pond design and evaluates rainfall, runoff, infiltration, exfiltration, evaporation and outflow.

5.3.3 ON-SITE VERSES REGIONAL

There are two basic approaches to designing storage facilities: "on-site" and "regional". When runoff storage facilities are planned on an individual site basis, they are referred to as "on-site." Larger facilities that have been identified and sized as a part of some overall regional plan are categorized as "regional" facilities. In addition, the regional definition can also be applied to storage facilities that address moderately sized watersheds to encompass multiple land development projects. This chapter focuses primarily on on-site detention facilities. In order to consider regional facilities, the following criteria must be met:

1. The regional detention facility is designed to accommodate the fully developed flows from the upstream watershed.

2. The regional detention facility is constructed, or will be constructed in phases with the development; otherwise, temporary detention must be provided.
3. Legally-binding ownership and maintenance responsibilities by a public entity are clearly defined to ensure the proper function of the facility in perpetuity.
4. There is adequate conveyance of the fully developed flows from the site to the regional detention basin.
5. Design is completed in accordance these criteria:
    a. Multi-use (e.g., recreation) shall be considered in the design of detention basins.
    b. The creation of jurisdictional dams shall be strongly discouraged.
    c. Regional Detention Basins shall be located on publicly-owned lands whenever possible for long-term operations and maintenance.

5.3.4 APPROACHES TO DETENTION

Criteria for the following four approaches to on-site detention are presented in this chapter:

1. Surface ponds (preferred approach),
2. Inundation of Parking lots,
3. Underground storage, or
4. Retention as a temporary measure.

Underground detention is only allowed in ultra-urban settings where redevelopment is taking place and when no other on-surface methods are practicable. In these cases, underground detention must meet strict criteria.

Underground detention is only allowed in ultra-urban settings where redevelopment is taking place and when no other on-surface methods are practicable.

5.4 DESIGN STANDARDS FOR ABOVE-GROUND DETENTION BASINS

5.4.1 STATE ENGINEER'S OFFICE

Any dam constructed for the purpose of storing water, with a surface area, volume, or dam height as specified in Colorado Revised Statutes 37-87-105 as amended, shall require the approval of the plans by the State Engineer's Office. Those facilities subject to state statutes shall be designed and constructed in accordance with the criteria of the state, in addition to these CRITERIA.

5.4.2 GRADING REQUIREMENTS

As a general rule, slopes should be as flat and the depths as shallow as site conditions and safety considerations allow. However, obtaining the required storage volume within a tight site often forces the pond design deep with steep side slopes. Safety must dictate, and if a person were to fall into the facility, slopes should be flat enough that they can easily climb out. Slope terracing with benches may be beneficial and improve the aesthetics.

Wherever possible, slope stabilization should be with vegetation and be traversable when wet.

Grading requirements for embankments shall be in accordance with Table 5.3. All earthen embankments shall be covered with topsoil and revegetated with grass. Mowing is difficult on 3:1 slopes, and 4:1 to 6:1 is the maximum slope that can be effectively mowed, depending on the equipment.

Table 5.3. Grading Criteria for Embankments

Embankment Height	Criteria
5 feet in height or less	No steeper than 4 (horizontal) to 1 (vertical).
Higher than 5 feet	Slopes shall not be steeper than 3 (horizontal) to 1 (vertical), but 4 (horizontal) to 1 (vertical) is preferred.
Riprapped embankments	No steeper than 2 (horizontal) to 1 (vertical).
Grassed detention facilities	Minimum bottom slope shall be 1.0 percent measured perpendicular to the trickle channel.

5.4.3 USE OF RETAINING WALLS

The use of retaining walls within detention basins is generally discouraged; however, if walls are unavoidable, low-height walls less than 30 inches that are constructed of natural rock or landscape block are preferred. Long-term maintenance access, safety and aesthetics are important design considerations. Maintenance equipment must be able to safely reach the bottom of the facility and have adequate space to operate and turn. If several retaining walls are used, a separation of at least 4 feet shall be provided. Any future outfalls to the basin shall be designed and constructed concurrently with the detention basin. This eliminates future disturbance of the retaining walls, which may jeopardize the wall's structural integrity, in order to construct the future outfall. Foundation walls of buildings shall not be used as detention basin retaining walls.

Any retaining walls exceeding a height of 30 inches (as measured from the ground line to the top of the wall) may require handrails. All handrails/guardrails shall be designed to meet International Building Code (IBC) requirements.

Walled-in or steep-sided basins should be located away from major pedestrian routes and emergency egress routes should be provided. Site lighting may also be required to discourage illicit activity in walled-in basins.

A licensed professional engineer shall perform a structural analysis of the retaining wall for the various loading conditions the wall may encounter. The wall design and calculations shall be stamped by the professional engineer. The structural design details and requirements for the retaining wall(s) shall be included in the construction drawings.

5.4.4 FREEBOARD REQUIREMENTS

For sites greater than or equal to 5 acres, the elevation of the top of the embankment shall be a minimum of 1.0 foot above the water surface elevation when the emergency spillway is conveying the maximum design or emergency flow. For sites less than 5 acres, the minimum required freeboard is 1.0 foot above the computed 100-year water surface elevation in the detention facility.

5.4.5 INLET CONFIGURATION

Forebays shall be provided at all pipe inlets into the detention pond to concentrate trash and sediment deposition and reduce sediment loading to the facility. Forebays require frequent and regular maintenance.

5.4.6 TRICKLE CHANNEL (LOW FLOW)

A low flow or trickle channel constructed across the facility bottom from the inlet to the outlet is recommended to convey low flows, and prevent standing water conditions. Concrete valley gutters should be sloped a minimum of 0.5% toward the outlet works. Grassed bottom detention basins require a concrete trickle channel; otherwise, either the low flow area becomes marshy or scours a notch through the turf.

5.4.7 OUTLET CONFIGURATION

To control the release rate, a multi-stage outlet is commonly used. For example, the WQCV may have a slow release rate, the 10-year event has a separate release, and the 100-year have a much larger outlet. A control orifice plate at the entrance of the pipe may be required to control the discharge of the design flow. The trash rack must be designed to prevent pinning a person during an extreme event. Clogging of the outlet with trash and debris is a particularly important concern, and a dedicated and stabilized emergency overflow is necessary to safely route flows in a big event. A simple plastic trash bag can completely block an outlet, and cause flooding.

5.4.8 TRASH RACKS

Trash racks (a grate in front of the perforated plate or outlet to capture large debris) may be needed for safety or to prevent small orifices from clogging. The trash rack opening should be at a minimum 4 times larger than the outlet pipe to reduce the velocity of water through the trash rack. Otherwise, fast moving water could pin a person against the outlet. For large openings with less potential for clogging, a trash rack may be more of a hazard due than a benefit. As a general rule of thumb, if the outlet can be seen by looking through the inlet, a person is safer being flushed through the pipe and a trash rack is discouraged. If the pipe is long with bends, by comparison the trash rack may be less of a hazard and is recommended.

5.4.9 EMBANKMENT PROTECTION/SPILLWAY REQUIREMENTS


This concrete spillway safely routes emergency flows down the embankment and was landscaped to have the general appearance of a staircase.

Whenever a detention basin uses an embankment to contain water, the embankment shall be protected from catastrophic failure due to overtopping. Overtopping can occur when the basin outlets become obstructed or when a larger than 100-year storm occurs. The emergency spillway of a storage facility should be designed to pass flows in excess of the design flow of the outlet works. When the storage facility falls under the jurisdiction of the Colorado State Engineer's Office (SEO), the spillway's design storm is prescribed by the SEO (SEO 1988). If the storage facility is not a jurisdictional structure, the size of the spillway design storm should be based upon the risk and consequences of a facility failure. Generally, embankments should have spillways that, at a minimum, are capable of conveying the total peak 100-year storm discharge from a fully developed total tributary catchment, including all off-site areas, if any. Frequently, however, analysis of potential downstream hazards indicates that the spillway design storm should be larger than the 100-year event, especially if loss of life could occur as a result of floodwaters going around the spillway.

Failure protection for the embankment may be provided in the form of a buried heavy soil riprap layer on the entire downstream face of the embankment or a separate emergency spillway. Structures shall not be located in the path of the emergency spillway or overflow. The invert of the emergency spillway should be set equal to or above the 100-year water surface elevation. If a roadway becomes overtopped by emergency overflow, the cross street flow shall be limited to 6-inch depth maximum at the crown to minimize hazards to traffic.

5.4.10 LANDSCAPING REQUIREMENTS

Rather than a concrete box in the middle of the pond, this outlet was effectively landscaped into the embankment.

Water diversion/detention areas and embankments should be designed and constructed to integrate with their surroundings, creating site amenities rather than eyesores. In open space or natural areas, techniques to be considered include creation of topographic changes that mimic natural conditions (including a variety of slope changes), using natural materials such as stone, blending with the textures and patterns of the surrounding landscape, and using materials that match the local environment. Existing drainage patterns should be preserved whenever possible. Grading from the toe of the slope to the first foot should be gradual to provide a broad area identified as the littoral zone. This area is critical to support wetland functions and emergent vegetation for improved water quality. A diversity of vegetation is encouraged to support wildlife diversity that requires food and cover. For urban areas, a formal treatment in shape and vegetation can be appropriate. All above-ground detention basins shall be revegetated. Native grass species, either drill seeded or broadcast is more desirable than sod. Turf grass cannot survive in saturated conditions and requires the additional maintenance of mowing and fertilizing. Fertilizer has a negative impact on water quality and algae growth because of the additional loading of nitrogen and phosphorous.

Landscaping improvements may be provided in the basin to enhance the aesthetics of the basin. When determining landscaping, long-term maintainability of the facility should be a high priority. The following is a list of guidelines for basin landscaping:

- Detention areas should have attractive natural-looking features, fit into the surrounding landscape and add to the overall character of an area. The shape of the detention basin should be as natural looking as practical, with terracing of the slopes and bottom. The tops and the toes of slopes should vary, and there should be an undulation in the shape and grading of the sides of the detention area.
- Slopes should vary and be well vegetated to prevent erosion. The use of appropriate groundcovers and grasses at the top of the slope help to soften the appearance of the detention area and can incorporate the detention area into the landscape design. Appropriate plant material, such as wetland species or drought tolerant species, should be planted in the detention area and on the slopes. Shrubs and trees should be planted back from the top of the slope. Native and perennial species should be used to the extent practical.
- Soil amendments should be considered since most facilities will be built in either urban soils or disturbed soils. This improvement aids the success of establishing vegetation since nutrients will be available and improved soil structure will ensure root stability.

- Vegetation zones in response to elevation and proximity to water is desirable. For instance the establishment of riparian plant communities adjacent to the water or inundated with each storm occurrence. Upland prairie species which are drought tolerant occur at higher elevations. Vegetation species selection should include consideration of evergreen attributes that provide cover all year long and fruit that provide food for multiple wildlife species. Since Colorado has a diversity of landscape habitat types, it is critical to select appropriate indigenous native vegetation species that thrive within proximity to the project area.
- Use of rock or wood mulch in and adjacent to detention facilities is discouraged because of its potential to be displaced and clog outlet structures. Mulch placed over filter fabric is particularly susceptible to displacement and should not be used on slopes greater than 6 (horizontal) to 1 (vertical) or below the 100-year water surface elevation.
- Rundowns, which convey runoff from streets and parking lots into channels or storage facilities, should be incorporated into the overall design and be attractively designed.
- Temporary irrigation should be considered for successful vegetation establishment.

5.4.11 MULTIPLE USE CONSIDERATIONS

Picnic benches and other public open space amenities are commonly located in stormwater detention facilities.

Multiple uses of detention facilities are encouraged; however, it is critical that the uses of these areas be taken into account to ensure that usage conflicts are minimized. For example, areas used as soccer fields or golf courses need to drain within a reasonable timeframe to prevent soggy fields that are incompatible with recreational use. Other park and detention facility conflicts may relate to safety in areas used for child play, West Nile virus concerns, and/or protection and enhancement of wildlife. Specific factors that shall be considered for multiple use facilities include:

- Compatibility with design, historic designation or other protective constraints including wildlife habitat and protection.
- Compatibility with recreational uses. The level of organized and informal activity in a park must be considered.
- Technical constraints and opportunities including soil characteristics, turf management, or terrain.
- Potential for new natural areas and wildlife corridors.
- Size and configuration of the park.
- Maintenance and operations, funding resources, successful techniques for dealing with silt, debris, etc.
- The configuration and easements for underground utilities and their impact on the existing park land.
- Potential for total rehabilitation of existing sites to accommodate multi-purpose uses.

- Impacts on all aspects of the open space system: natural areas including potential areas such as along gulches, traditional parks, and other publicly owned lands.

This stormwater facility using a sand bed infiltration system was integrated into the neighborhood pocket parks at Stapleton in Denver. Special turf was required in the pond bottom that could tolerate short-term saturated conditions.

5.5 DESIGN STANDARDS FOR PARKING LOT DETENTION

5.5.1 DEPTH LIMITATION

The maximum allowable design depth of ponding in parking lots for the 100-year flood is 12 inches.

5.5.2 OUTLET CONFIGURATION

A drop inlet may be used to discharge to a storm sewer or drainageway. A weir and a small diameter outlet through a curb may also be used. The size and shape of the outlet are dependent on the discharge/storage requirements.

5.5.3 PERFORMANCE

To assure that the detention facility performs as designed, maintenance access shall be provided. The outlet shall be designed to minimize unauthorized modifications, which affect function. Any repaving of the parking lot shall be evaluated for impact on volume and release rates and is subject to approval.

5.5.4 FLOOD HAZARD WARNING

All parking lot detention areas shall have multiple signs posted identifying the detention basin area. The signs shall have a minimum area of 1.5 square feet and containing the following message:

WARNING
This area is a detention basin and is subject to periodic flooding to a depth of (provide design depth).

5.6 DESIGN STANDARDS FOR UNDERGROUND DETENTION

Underground detention is strongly discouraged for the following reasons:

- Underground detention is not visible; therefore, it tends to be "out-of-sight, out-of-mind." As a result, these devices do not typically receive regular maintenance, nor is their performance periodically monitored.
- Maintenance access is often poor, which can be a deterrent to maintenance.
- Anaerobic (absence of dissolved oxygen) conditions in bottom sediments are more likely to develop in underground devices. This condition can release pollutants that were bound to the sediment and cause bad odors.

Nevertheless, there are some cases where the use of such facilities is necessary due to extreme space constraints in smaller, ultra-urban redevelopment sites. The use of underground detention will be considered under these circumstances; however, the applicant must comply with the following restrictions prior to receiving authorization for its use:

- Clear evidence must be provided documenting why detention cannot be provided on the ground surface and why the use of an underground facility is the best choice for the site, considering factors such as initial installation, maintenance, and ability to assure long-term function.
- Any water quality treatment must still be provided above-ground, even if detention is provided below ground.

When no other alternative is practicable, the requirements for underground detention are provided below.

Dry wells, which are underground vaults with a porous open base to promote infiltration, typically do not have adequate storage volume for significant storm events. Although they may function for frequent smaller storm events, their reliability for infiltration and ability to manage large storm events are questionable.

5.6.1 MATERIALS

Underground detention shall be constructed using corrugated aluminum pipe (CAP), reinforced concrete pipe (RCP), concrete vaults or approved equivalents. Galvanized or aluminumized pipes are not acceptable. The pipe thickness, cover, bedding, and backfill shall be designed to withstand HS-20 loading.

5.6.2 CONFIGURATION

Pipe or vault segments shall be sufficient in number, height, and length to provide the required minimum storage volume. The minimum headroom height of the pipe or vault segments shall be 48 inches to permit maintenance. If parallel pipes are used, the pipe segments shall be placed side by side and connected at both ends by elbow and tee fittings. The pipe segments shall be continuously sloped at a minimum of 0.25% to the outlet. Manholes for maintenance access shall be placed in the tee fittings, bends and in the straight segments of the pipe, when required.

Permanent buildings or structures shall not be placed directly above the underground detention.

5.6.3 INLET AND OUTLET DESIGN

Inlets to detention facilities can be surface inlets, pipes and/or a local private storm sewer system.

Outlets from underground detention shall be designed with ease of maintenance to prevent clogging. A two-pipe outlet may be required to control both minor and major design return periods. The invert of the lowest outlet pipe shall be set at the lowest point in the detention vault. The outlet pipe(s) shall discharge into a standard manhole or standard inlet or into an open drainageway with erosion protection. If an orifice plate is required to control the release rates, the plate(s) shall have a hinge on one side to open into the detention pipes to facilitate back flushing of the outlet pipe(s) and be firmly bolted or secured to the wall to prevent leakage around the edges.

5.6.4 MAINTENANCE ACCESS

Access easements to the detention facility shall be provided. Maintenance access designs shall take into consideration Occupational Safety and Health Administration (OSHA) requirements for confined space entry.

5.7 DESIGN STANDARDS FOR RETENTION PONDS

5.7.1 ALLOWABLE USE

> *Retention ponds are strongly discouraged as a permanent solution for drainage problems since they may be full when needed.*

A retention facility (a pond with a zero release rate or a very slow release rate when a trickle outflow can be tolerated) is used when there is no formal drainageway available within a reasonable distance of the site or one that is grossly inadequate. When designing a retention facility, the hydrologic basis of design is difficult to describe because of the random nature of rainfall events. Thus, sizing for a given set of assumptions does not ensure that another scenario produced by nature (e.g., a series of small storms that add up to large volumes over a week or two) will not overwhelm the intended design. For this reason, retention ponds are strongly discouraged as a permanent solution for drainage problems. Retention ponds should be designed as temporary facilities, with an ultimate conversion to a detention system.

When a retention pond is proposed as a temporary solution to an evolving drainage problem, the pond shall be sized to capture, as a minimum, the runoff equal to 100-year, 24-hour storm plus 1-foot freeboard. The facility also shall be situated and designed so that when it overtops, no human-occupied or critical structures (e.g., electrical vaults) will be flooded, and no catastrophic failure at the facility (e.g., loss of dam embankment) will occur. Retention facilities shall be as shallow as feasible to encourage infiltration and other losses of the captured urban runoff. Retention ponds should be designed to drain between storms and release the water back to the stream system. The pond should preferably drain within 72 hours. If the storage volume cannot be infiltrated within this time frame, a secondary outlet should be designed to provide additional releases from the pond.

5.7.2 CALCULATION OF RETENTION VOLUME

Retention ponds shall be sized to completely contain the 100-year, 24-hour rainfall, which can be obtained from the NOAA Atlas (see maps Chapter 9, Section 4). No reduction in volume will be allowed for infiltration during the storm event. In other words, assume the tributary basin is 100% impervious. Minimum required pond volume is simply [Tributary Area] multiplied by [Rainfall Depth].

5.7.3 DESIGN STANDARDS FOR RETENTION PONDS

Side slopes of the pond shall be no steeper than 3 (horizontal):1 (vertical). A stabilized emergency overflow section capable of passing the full 100-year event at a minimum shall be provided that will safely route stormwater away from downstream development, which may be a significant design challenge if no formal downstream drainageway exists.

Design standards for retention ponds must comply with specific site development, flood-proofing, site investigation and physical design considerations, as described below.

1. Site Development: The total development site area must be accounted for when planning for the retention of stormwater runoff. Provide grading for the entire site development to drain to the retention pond. Any off-site basins that historically flow through the site must be provided flow routes around the site and returned to the natural drainageway. Colorado state law maintains that "a property within a natural drainageway is subservient to the historic drainage from upper lands." Off-site drainage cannot be excluded if there is no other discharge location to be used; therefore, in volume calculations, include all off-site drainage basin areas that cannot otherwise be rerouted around the development and returned to the natural drainage path.

A property within a natural drainageway is subservient to the historic drainage from upper lands.

2. Floodproofing: The construction of a retention pond is essentially creating an isolated floodplain on the property. Delineate the limits of the 100-year flood area on the design drawing. Provide 1 foot of freeboard from the 100-year maximum water surface elevation of retention pond volume. Provide a 100-year emergency release overflow route from the site, which returns the flow back to its natural drainage path. Ensure finished floor elevations are at least 1.0 feet above the water surface elevation when the emergency spillway is conveying the maximum design flow or emergency flow.

The construction of a retention pond is essentially creating an isolated floodplain on the property.

3. Site Investigation: Site selection for infiltration retention ponds is critical. Factors for evaluating site suitability include:
   - Location of groundwater table
   - Location of bedrock
   - Seasonal fluctuation of water table
   - Soil permeability and porosity
   - Soil profile
   - Environmental conditions (e.g., contaminated soils)
   - Proximity to structures (e.g., basements)

   The following factors would preclude the site's use as a retention infiltration pond:
   - Groundwater of less than 4 feet below pond bottom
   - Bedrock within 4 feet of the pond bottom
   - Pond location over fill
   - Surface and underlying soils classified as NRCS Hydrologic Group D (having little or no infiltration capacity)
   - Saturated infiltration rate less than 0.3 inch per hour

   A thorough geotechnical and geohydrological investigation shall be performed to determine site suitability. The following shall be included in the investigation:
   - Soil borings to a depth of 10 feet or to bedrock
   - Percolation tests
   - Soil classification

5.8 CHECKLIST AND DESIGN AIDS

Several key considerations that the designer must take care to address include:
1) Grade earth slopes 4:1 or flatter.
2) Provide minimum freeboard of 1 foot.
3) Provide trickle channels in above-ground detention areas.
4) Protect embankment from overtopping conditions.
5) Provide proper trash racks at all outlet structures.
6) Provide signs as required.
7) Provide maintenance access.
8) Provide emergency spillway and check emergency overflow path.
9) Check finished floor elevation of any structure near the detention basin.
10) Ensure failure of underground detention is clearly evident from above ground.