CITY OF FLAGSTAFF
STORMWATER MANAGEMENT
DESIGN MANUAL

City of Flagstaff Engineering Division
Stormwater Management Section
211 West Aspen Avenue
Flagstaff, Arizona 86001

March, 2009

FOREWARD

The information presented in this manual is to be used for the preparation of drainage reports, flood studies, engineered plans, and other stormwater related analyses required by the City of Flagstaff. Other appropriate procedures not presented in this manual may also be used, provided that approval for their use is first obtained from the Stormwater Manager.

The information contained in this manual is based on what is believed to be the best procedures and techniques available at the time this document was prepared. Every attempt was made to define the applicability and the limits of the policies, criteria, and procedures presented herein. It is however, the responsibility of the user of this manual to exercise proper engineering judgement in the application of the procedures presented herein for the analysis and design of stormwater and flood control improvements.

As further progress is made in applied hydrology and hydraulics, appropriate modifications will be made to this manual. The user should therefore contact the Stormwater Management Section, prior to the use of this manual, to obtain any modifications and errata sheets that may apply.

ACKNOWLEDGEMENTS

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DEFINITIONS

For the purposes of use in this manual, the following terms are defined as indicated:

AGGREGATION - a progressive buildup or raising of a channel or stream bed due to sediment deposition.

ARTERIAL STREET - a Type I or Type II street as defined in the City of Flagstaff Engineering Design and Construction Standards & Specifications.

ATTENUATION - a reduction in downstream peak flood discharges induced by storage provided within stormwater detention facilities, channels, and overbank areas.

BASE FLOOD - means that flood event having a one (1) percent chance of being equaled or exceeded in any given year. Also referred to as the 100-year flood event.

CARRYOVER FLOW - is that portion of gutter flow which bypasses an inlet on a continuous grade. Also referred to as "bypass flow".

CITY - shall mean the City of Flagstaff.

CHANNEL - a natural or man-made conveyance for water in which the water surface is exposed to the atmosphere and the gravity force component in the direction of motion is the driving force.

COMBINATION INLET - a pavement drainage structure typically comprised of a curb-opening and a grate inlet.

COLLECTOR STREET - a Type III street as defined in the City of Flagstaff Engineering Design and Construction Standards & Specifications.

CRITICAL DEPTH - is that depth of flow at which the specific energy of a given flow rate is at a minimum.

CROWN - synonymous with "soffit". Not to be confused with a street crown.

CURB - a concrete barrier found at the edge of the street pavement, usually six to eight inches in height.

CURB-OPENING INLET - a pavement drainage inlet consisting of an opening in the vertical curb of a roadway section.

CULVERT - a relatively short, closed conduit typically designed hydraulically to take advantage of submergence to increase hydraulic capacity used for the purpose of conveying surface runoff under an embankment or roadway fill section.

DEGRADATION - a progressive lowering of a channel bed due to scour.

DESIGN CRITERIA - are the standards by which a policy is carried out or placed into effect.

DETENTION FACILITY - a permanent flood control system or stormwater management facility whose primary purpose is to temporarily store stormwater and release the stored stormwater runoff at controlled rates. Also used as a means of attenuating the effects of increased runoff caused by development by temporarily storing runoff and metering the discharge at pre-development rates, thereby lengthening the duration of flow.

DEVELOPMENT - means any land change, including but not limited to subdivisions, buildings, other structures, mining, dredging, clearing, grubbing, stripping, grading, paving, excavation, drilling, transporting and filling of land.

DROP INLET - a drainage inlet with a horizontal or nearly horizontal opening.

DROP STRUCTURE - a hydraulic structure used in an open channel or conduit for grade control or energy dissipation purposes.

EMBANKMENT - a man-made earth fill structure constructed for the purpose of roadway elevation, detention and/or impoundment of water.

ENERGY DISSIPATOR - any device designed to protect downstream areas from erosion by reducing the velocity of flow to acceptable limits.

ENGINEER - a registered professional engineer in the State of Arizona who demonstrates proficiency in the specific area of design.

EROSION - the process of removal and transport of soil particles or land surface by the action of wind, water, ice, gravity or any combination thereof.

EROSION AND SEDIMENT CONTROL - the control of solid material, both mineral and organic, during a land disturbing activity or development to prevent its transport out of the disturbed area or development.

FEMA - an abbreviation for the Federal Emergency Management Agency.

FILTER BLANKET (LAYER) - a layer of graded, intermediate size gravel placed between fine-grained (natural) material and riprap to prevent the erosion of the finer material. Also referred to as a "granular filter".

FILTER FABRIC - a layer of synthetic material that serves the same purpose as a filter blanket.

FLARED WINGWALLS - the part of a culvert headwall which serves as a retaining wall for the roadway embankment or channel banks. The walls form an angle to the centerline of the culvert.

FLOODPLAIN - is those areas adjoining the channel of a watercourse including areas where drainage is or may be restricted by man-made structures which have been or may be covered partially or wholly by floodwater from the one hundred year flood.

FLOODWAY - is the channel of a river or other watercourse and the adjacent land areas that must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation more than a designated height.

FLOODWAY FRINGE - is that portion of the Special Flood Hazard Area (100-year floodplain) located outside of the Regulatory Floodway.

FLUME - an open or closed channel used to convey water, typically down an embankment.

FREEBOARD - is the additional vertical distance between the calculated maximum level water surface in a culvert, reservoir, tank, detention basin, channel, canal, or wash and the top of the confining structure.

FROUDE NUMBER - a dimensionless number that represents the ratio of inertial forces to gravitational forces. High Froude numbers are indicative of high velocity and high potential for scour.

GRADE CONTROL STRUCTURE - a hydraulic structure constructed across an open channel or stream from bank to bank to control bed slope and prevent channel degradation and headcutting.

GRADING AND DRAINAGE PLAN - the set of drawings and other documents that comprise all the information and specifications for the drainage system, structures, concepts and techniques that will be used to control stormwater runoff, erosion, and sediment transport.

GRATE INLET - a drainage structure consisting of an opening in the gutter, covered by one or more metal grates, set flush with the pavement or gutter or located at the roadside in a low point, swale or channel.

GUTTER - that portion of the roadway section adjacent to the curb which is utilized to convey stormwater runoff. It may include a portion or all of a traveled lane, shoulder or parking lane, and a limited width adjacent to the curb may be of different materials and have a different cross slope.

HEADCUTTING - is an abrupt vertical drop in an earthen channel bottom or embankment caused by erosive flows. This type of erosion typically moves upstream due to changes in discharge and sediment load characteristics.

HEADWALL - the structural appurtenance usually applied to the end of a culvert to control an adjacent roadway embankment and protect the culvert end.

HEADWATER - is that depth of water impounded upstream of a culvert due to the influence of the culvert constriction, friction, and configuration.

HYDRAULIC GRADE LINE (HGL) - a line which represents the static head plus the pressure head of flowing water and is equal to the total energy grade line minus the velocity head at any point along a storm drain. Typically referred to as the locus of elevations to which the water would rise in successive piezometer tubes if the tubes were installed along a pipe run.

HYDRAULIC JUMP - an abrupt rise in the water surface caused by abrupt transitions from supercritical flow regime to a subcritical flow regime in the flow direction.

IMPERVIOUS - the condition of being impenetrable to water.

IMPROVED INLET - a flared, depressed or tapered inlet that has entrance geometry that decreases the flow constriction at the inlet and thus increases the capacity of a culvert entrance.

INVERT - the floor, inside bottom or the lowest portion of a conduit or channel.

LAND DISTURBING ACTIVITY - any use of the land that results in a change in the natural cover or topography that may cause erosion and contribute to sediment and alter the quality and/or quantity of storm water runoff emanating from that land.

LOCAL SCOUR - erosion which occurs as the result of high velocity flow at a culvert, storm drain outlet, pier, or abutment which extends only a limited distance downstream.

LOW CHORD - the elevation of the lowest portion of a bridge deck structure used to determine the open area below the bridge available for freeboard and flow conveyance.

MANNING'S "n" - a coefficient of roughness, typically used in a formula for estimating the capacity of an open channel or conduit to convey water.

MILD SLOPE - a slope where the critical depth is less than the normal depth.

MUTLI-PURPOSE FACILITY - a detention facility which provides other benefits in addition to flood control, such as recreation, parking, visual buffers, and stormwater water treatment.

NORMAL FLOW - is open channel flow which occurs when the discharge, velocity, depth, slope, and channel cross-section are uniform throughout the reach. The water surface profile and channel bottom are parallel.

NPDES - acronym for the National Pollutant Discharge Elimination System program and permitting process administered by the Environmental Protection Agency for the purpose of improving the quality of the nation's waters by mitigating non-point source pollution in urban stormwater runoff.

OBSTRUCTION - is any physical alteration in, along, across or projecting into any channel, watercourse, or floodway which may impede or divert floodwaters, or may itself be carried downstream to cause damage to life or property. Examples include but are not limited to: any dam, wall, embankment, bridge, conduit, culvert, building, structure, levee, dike, abutment, projection, excavation, fill, stockpile, fence, wire, rock, gravel, refuse, vehicle, equipment, or vegetation.

POLICY - is a set of goals that establish a definite course or method of action and that are established to guide and determine present and future decisions. Policy is implemented through design criteria established as standards for making decisions.

PRESSURE FLOW - the flow of water within a closed conduit without a free surface open to atmospheric pressure.

RESERVOIR ROUTING - flood routing of inflow and outflow hydrographs through a reservoir or detention basin.

RETENTION BASIN - is a stormwater storage facility which is drained by subsurface infiltration instead of a positive outlet.

RIPRAP - rock or stones placed in a loose assemblage along the banks/bed of a channel or outlet of a conduit to inhibit erosion and scour.

RIVERINE - means relating to, formed by, or resembling a river (including tributaries), stream, wash, or brook.

SAG - synonymous with "sump".

SEDIMENT TRAP - an area within a stormwater detention facility or construction site which is designed to trap incoming sediments for the purpose of facilitating maintenance.

SEDIMENTATION - the process involving the deposition of soil particles which have been carried by flood waters or stormwater runoff.

SEEPAGE - the movement of water through pores and voids of pervious material such as soil, gravel, filter fabric, etc.

SETBACK - the minimum horizontal distance between a structure and a channel, stream, wash, riverine, natural watercourse, or detention basin as measured from the top edge of the highest bank.

SHALL - means a required element.

SHEET FLOW - shallow, diffuse runoff typically characterized by an approximately equal depth of runoff across a broad width of flow without concentrating in gullies and streams (often referred to as overland flow).

SHOULD - means a recommended element.

SLOTTED DRAIN INLET - a pavement drainage inlet composed of a continuous slot built into the soffit of a pipe which serves to intercept, collect and transport the flow.

SOFFIT – is the inside top of a culvert or storm drain conduit. Also refereed to as the "crown".

SOIL STABILIZATION - the installation of vegetative, synthetic, or structural measures to protect soil from erosive forces of raindrop impact and flowing water.

SPREAD - the width of flow measured laterally from the roadway face of curb.

STEADY FLOW - flow characteristic in which the discharge passing a given cross section remains constant in time.

STEEP SLOPE - a slope where the critical depth is greater than the normal depth.

STORM DRAIN - a combination of underground conduits (laterals, trunks, pipes) and surface inlet structures utilized for the purpose of removing stormwater runoff from the ground surface or street pavement and conveying it to a downstream discharge point.

STORM DURATION - the period or length of a storm event.

STORMWATER MANAGEMENT - the collection, conveyance, storage, treatment and disposal of stormwater runoff in a manner to minimize channel erosion, flood damage, and/or degradation of water quality and in a manner to enhance and ensure public health, safety, and general welfare, which shall include a system of vegetative or structural measures, or both, that control the increased volume and rate of runoff caused by manmade changes to the land.

STORMWATER MANAGER - is the duly designated head of the City of Flagstaff Stormwater Management Section, or his/her duly authorized representative or agent.

STORMWATER RUNOFF - is the direct response of a watershed or drainage area to precipitation from a storm event and/or snow melt and includes surface and subsurface runoff or drainage that enters a watercourse, street, storm drain or other concentrated flow during and following precipitation.

SUBCRITICAL FLOW - flow regime which occurs when the Froude number is less than one (1), flow depths are greater than critical depth, small water surface disturbances travel both up and downstream, and the control of the flow depth is always located downstream.

SUMP - a low point typically found within a street profile where stormwater runoff collects. Also referred to as a “sag”.

SUPERCRITICAL FLOW - flow regime which occurs when the Froude number is greater than one (1), depths are less than critical depth, small water surface disturbances are always swept downstream, and the location of the flow control is always upstream.

TAILWATER - the water surface elevation or depth of flow in a channel, watercourse, or other receiving water body directly downstream of a culvert or storm drain outlet.

TIME OF CONCENTRATION (T_C) - is the time required for stormwater runoff to flow from the hydraulically most remote point of a watershed or drainage area to the point of interest or watershed outlet. The most remote point is the point from which the time of runoff is the greatest. Thus, the T_C is the maximum time for water to travel through the watershed, which is not always the maximum distance from the outlet to any point in the watershed.

UNIFORM FLOW - flow characteristic in which the flow rate and depth remain constant along the length of the channel reach.

WASH - is a natural watercourse which is essentially undisturbed by development.

WATERCOURSE - is a naturally occurring river, riverine, stream, creek, wash, lake or other body of water or channel consisting of banks and bed through which continuous or periodic flows occur. This may include any depression serving as a conveyor of stormwater.

WATERSHED - the catchment area for rainfall which is delineated as the drainage area producing stormwater runoff to a given point. It is assumed that the base flow in a stream also originates from the same area.

WEIR - is a depression or notch in the side or top of an outlet structure or a depression of specific shape in the embankment of a stormwater detention facility. Weirs are classified in accordance with the specific shape of the notch such as rectangular, V-notch, trapezoidal, parabolic, or proportional.

CHAPTER 1: INTRODUCTION

1.1. PURPOSE

This manual has been developed to assist in the design and evaluation of stormwater management facilities within the Corporate Limits of the City of Flagstaff, Arizona. Stormwater management policies, design criteria, and recommended design procedures are presented herein for conducting hydrologic and hydraulic studies, design, and evaluations. Although the intent of this manual is to establish uniform design practices, it neither replaces the need for engineering judgement nor precludes the use of information not presented. Other accepted engineering procedures may be used to conduct hydrologic and hydraulic studies if prior approval from the Stormwater Manager is obtained.

The overall goals for the development of this manual are to: (1) ensure compliance with applicable floodplain and stormwater management regulations, policies, and design criteria; (2) minimize public expenditures for drainage projects; (3) minimize the review time of drainage report and plan submittals; and (4) provide consistent policies and criteria that will result in uniform practices and drainage infrastructure within the City of Flagstaff.

1.2. APPLICABILITY

The policies, design criteria, and procedures presented in this manual are applicable to the design and analysis of drainage facilities of both public improvement and private development projects within the City of Flagstaff. However, the applicability of many of the items contained herein may have limited ranges.

The hydrologic design criteria presented in this manual is based on accepted engineering procedures and criteria. Due to the lack of gauged runoff data for the City of Flagstaff, these methods do not have any point of reference to any historic events and in no way ensure that design flows are reasonable, correct, or calibrated.

1.3. USE OF THIS MANUAL

The policies, design criteria, and procedures presented in this manual are to be utilized in the preparation of drainage reports, flood studies, grading and drainage plans, and public improvement plans required by the City of Flagstaff.

The use of this manual is intended to act as guidance to the policies and design criteria specific to the City of Flagstaff. There is no intent to inhibit sound innovative engineering design or use of new techniques, procedures, or data.

It is the responsibility of the design engineer to utilize proper engineering judgement when applying any procedure or criteria contained in this manual to a specific site or condition.

This manual is dynamic in nature and shall be periodically reviewed and updated to keep it up-to-date with new legal and technical developments in the field of stormwater management. It is the intent of the City of Flagstaff to periodically issue revisions to this manual which incorporate new data, methods, criteria and such other information as may be deemed appropriate.

Use of this manual does not supersede the need for acquiring various permits or authorizations required by the Federal Emergency Management Agency, Environmental Protection Agency, U.S. Army Corps of Engineers, State of Arizona, Coconino County, or the City of Flagstaff.

CHAPTER 2: DRAINAGE REPORTS AND PLANS

The purpose of this chapter is to present criteria for submittals of drainage reports, floodplain studies, grading and drainage plans, and public drainage improvement plans to the City of Flagstaff for review and approval.

Drainage reports and engineered grading and drainage plans are required to: (1) analyze the impact that the proposed development will have on stormwater discharges; (2) provide adequate data to ensure that the development is designed to be protected from flooding and conforms to applicable floodplain and stormwater management regulations; and (3) provide data for the design of public and private drainage facilities.

Drainage reports shall be of sufficient detail to demonstrate that the development or project will not create drainage or flooding problems and that any on-site drainage facilities are properly sized to detain and/or convey the design storm flows.

2.1. DRAINAGE REPORT REQUIREMENTS

Drainage reports will be required for the following land development activities:

1. Residential, commercial, and industrial subdivisions.
2. Application for rezoning.
3. Any multi-family residential, commercial, or industrial development; parking lot; or park.
4. Public improvements involving new streets, culverts, storm drains, open channels, and private/public detention facilities or other drainage infrastructure.
5. Application for Conditional Letter of Map Revision (CLOMR) or Letter of Map Revision (LOMR) to the Federal Emergency Management Agency (FEMA).

A drainage report may also be required for application for a building permit, floodplain use permit, or grading permit if site conditions warrant or if drainage dictates the development of the site.

Drainage reports submitted to the City for review and approval shall be prepared and sealed by an Arizona Registered Professional Civil Engineer.

2.1.1. Rezoning Applications

Drainage reports and plans for rezoning applications need to address the manner in which stormwater is to be managed in conjunction with development of the project. Off-site drainage and flood prone areas should be identified and required drainage improvements shown in general on the development plan. The information needs to be adequate enough to demonstrate that the site can be developed in compliance with City of Flagstaff (COF) Floodplain Management Regulations and stormwater management requirements. Detailed hydrologic and hydraulic analyses will not be required for most rezoning applications, unless the Stormwater Manager determines that the site drainage is a limiting factor in the successful development of the subject site.

2.1.2. Subdivision Applications

A preliminary drainage report is required for all Preliminary Plat submittals. This report should contain the following information at a minimum: (1) a description of how the proposed development will comply with COF stormwater management, floodplain management, and detention requirements; (2) a description of any existing drainage conveyances or problems such as natural watercourses, floodplains, and drainage from adjacent lands; (3) preliminary drainage conveyances, detention facility, sizing, and location(s); and (4) the effects of proposed detention location(s) on any resource protected areas, as defined in the COF Land Development Code. Detailed hydrologic and hydraulic analyses are typically not required at this stage, however, the information provided must be adequate to demonstrate compliance with applicable regulations.

2.1.3. Drainage Report Content and Format

Drainage reports shall contain the following information, at a minimum, and shall be presented in the format outlined below:

1. <u>Cover Sheet</u>
   - Title of report.
   - Date of report completion/submittal and any revisions.
   - Project name, address, and COF File number.
   - Name, address, and phone number of client.
   - Name, address, and phone number of engineering firm responsible for report.
   - Seal/signature of the Arizona Registered Professional Civil Engineer responsible for preparing the report.

2. <u>Table of Contents</u>
   - All report pages shall be numbered sequentially including any appendices.
   - List of all tables and illustrations.
   - Table of contents sealed by responsible engineer.

3. <u>Introduction</u>
   - Location map showing the project in relation to adjacent properties, streets, and nearby watercourses.
   - Legal description of subject property (can be placed in Appendix if too lengthy).
   - Description of the existing and proposed land use/project, drainage patterns, natural watercourses, drainage problems, and floodplain status within the development.
   - Summary of any previous hydrologic/hydraulic studies or other information which pertain to the development.
   - Description of potential impacts both upstream and downstream.
   - Effect of proposed construction on major drainage conveyances.

4. Objectives and Procedures Section

- Brief summary of the purpose of the report in relation to the project (e.g., rezoning, Preliminary plat, subdivision, commercial development, etc.)
- Description of the methodologies, assumptions, and procedures used in preparing the report.
- Description of all applicable development standards, policies, detention requirements, and floodplain regulations to which the proposed development must adhere.

5. Hydrology Section

- Drainage maps (drawn to scale) for pre and post-development conditions which clearly depict contributing watersheds, sub-basins, concentration points, flow patterns, measured flow lengths, elevations, and contours.
- Hydrologic data sheets, for both pre and post-development conditions, for each concentration point including time of concentration calculations, rainfall intensities, runoff coefficients or curve numbers, and peak discharges.
- Pre and post-development hydrology.
- Summary table listing all concentration points, corresponding drainage areas, calculated peak discharges for pre and post-development conditions, and differences in discharges.

6. Hydraulics Section

- Open channel design and capacity computations in accordance with the policies and criteria outlined in Chapter 4.
- Design computations for all culverts in accordance with the policies and criteria outlined in Chapter 5.
- Design computations for all storm drains, inlets, and street sections in accordance with policies and criteria outlined in Chapter 7. Storm drain design shall include a labeled schematic of storm drain network, design discharges, pipe capacities, profiles, outlet velocity, and hydraulic grade line.
- All supporting data, printouts, tables, nomograph, etc., which are referenced in report.

7. Detention Design Section

- Site plan (to scale) which clearly shows dimensions and locations of all proposed development and detention system(s) including but not limited to the following:
  a. Location, size, and type(s) of inflow and outflow structures.
  b. Location and size of access and maintenance access ramps and roadways, if applicable.
  c. Boundaries of Common Areas or private drainage easements, if applicable.
  d. Maximum water surface elevations, limits of ponding, and typical facility cross-section(s).
  e. Flow arrows, drainage divides, contours, and finished grades.
  f. Roof drainage direction(s) and finish floor elevations of all buildings.
- Description of how the overall detention design will comply with grading criteria, resource preservation, and landscaping requirements.

- Description of maintenance schedule and responsibility.
- Detention volume estimate computation(s).
- Detailed reservoir-routing calculation sheets for all required design storms.
- Plotted inflow and outflow hydrographs (preferably superimposed).
- If retaining walls are utilized, include free-body diagrams showing all forces, moments and computations required for determining factors of safety against sliding and overturning.

8. Summary and Conclusions
   - A brief summary of the analyses and conclusions presented in the report.
   - A brief description of how the proposed development and/or public improvements will adhere to applicable stormwater detention and/or floodplain regulations and mitigate any impacts created by the development.

9. References
   - Provide a listing of pertinent sources of analysis and design procedures used.

10. Appendices
   - Appendices may be used for hydrologic, hydraulic, reservoir-routing calculations, etc., and other material not suited for inclusion in the main body of the report.

2.2. FLOODPLAIN STUDIES AND MAP REVISIONS

Detailed floodplain studies (i.e., Technical Data Notebooks) are required for the following applications to the FEMA:

- Conditional Letter of Map Revision (CLOMR),
- Letter of Map Revision (LOMR), and
- Physical Map Revision (PMR).

Floodplain studies may also be required by the Stormwater Manager for a Conditional Letter of Map Amendment (CLOMA) or Conditional Letter of Map Revision Based on Fill (CLOMR-F). Applications for Letters of Map Amendment (LOMA) and Letters of Map Revision based on Fill (LOMR-F) will typically not require a floodplain study, however, the FEMA application must be reviewed by the Stormwater Manager prior to submittal to FEMA. Flood studies may also be required by the Stormwater Manager for other development(s) that may adversely affect floodplain depths.

Floodplain studies submitted to the City for review and approval shall be prepared and sealed by an Arizona Registered Professional Engineer.

The City of Flagstaff will review all floodplain studies for technical compliance and completeness. Arizona Department of Water Resources (ADWR) review and approval may be required for all floodway revisions and new hydrologic studies. All local and federal review fees associated with map revision requests are the responsibility of the applicant.

Specific guidelines for Flood Insurance Studies can be found in FEMA 37, *Guidelines and Specification for Study Contractors,* January, 1995 or latest version and *Addendum to Guidelines and Specifications for Study Contractors,* April, 1993 or latest version.

Floodplain study formats shall be in accordance with ADWR State Standard 1-97, "Requirement for Flood Study Technical Documentation" and applicable FEMA applications forms.

Hydrologic modeling performed in flood studies shall utilize U.S. Army Corps of Engineers, *HEC-1 Flood Hydrograph Package* program or other hydrologic models approved by FEMA.

Hydraulic modeling performed in flood studies shall utilize U.S. Army Corps of Engineers, *HEC-2 Water Surface Profiles* program. The *HEC-RAS River Analysis System* program is acceptable if the entire wash is modeled in *HEC-RAS* or the reach being modeled is hydraulically independent. The engineer is encouraged to contact FEMA for a list of accepted models.

2.2.1. General Guidelines

Hydraulic models required for map revision requests to FEMA are typically:

1. Duplicate Effective Model (natural and floodway models) -
   - using the same computer program (e.g., HEC-2), run the model on your computer and check the result with the output to make sure the model was duplicated
   - assures the baseline is correct and the revised model will tie back into the effective model upstream of the revised reach.

2. Corrected Effective Model -
   - using a newer version of the same program
   - more detailed cross sections that reflect conditions that existed when the original model was developed
   - fix technical errors
   - an improved computer model or improved bridge routine
   - add bridges, culverts, or other structures that existed but were not modeled
   - becomes the new base model to measure impacts of development/construction that occurred since the original model was developed

3. Existing Conditions Model -
   - update the corrected effective model to include existing conditions
   - natural changes in the floodplain
   - reflects fill in the floodway fringe since original model was developed
   - other channel improvements
   - other bridges and culverts
   - used as baseline model to measure the effects solely attributed to the "project" as reflected by the post-project model

4. Post-Project Model -
   - reflects the project (built or proposed) and determines the impacts of the project

Floodway analysis must include a Method 4 and/or 5 optimization and Method 1 floodway delineation runs.

River stationing shall correspond to river miles above the confluence with the applicable downstream watercourse as identified in the effective Flood Insurance Study.

When no discernable breaks in the geometry (e.g., change in bank slope) exists for determination of channel and overbank areas, channel and overbank limits should be determined based on similar "n" values.

Cross-sections should be located at places that fully describe the geometry of the reach and shall be oriented to be perpendicular to the flow lines and/or contours.

If the overall cross-section is skewed more than 18 degrees from the perpendicular of the flow line, either the cross-section needs to be resurveyed or reduced by an appropriate multiplier.

Cross-sections are not permitted to cross each other.

Obstructions or buildings within cross-sections should be physically modeled when possible or ineffective flow options utilized. Adjustment of the overbank "n" values may also be necessary.

Reach lengths should be determined based on the horizontal distance between the centers of mass of the overbank cross-sectional areas.

2.3. GRADING AND DRAINAGE PLANS

Grading and Drainage plans, prepared and sealed by an Arizona Registered Civil Engineer, are required for the following land development activities:

1. Residential, commercial, and industrial subdivisions.
2. Any multi-family residential, commercial, or industrial development; parking lot; or park.
3. Public improvements involving new streets and/or drainage facilities.
4. Application for CLOMR or LOMR.
5. Application for grading permit.

Grading and drainage plans may also be required by the Stormwater Manager or Building and Safety Director for building permit or floodplain use permit applications if site conditions warrant.

The engineer shall submit one reproducible (mylar) copy of the approved grading and drainage plan(s) as permanent public record.

2.3.1. Grading and Drainage Plan Requirements

Grading and Drainage Plans shall include the following information at a minimum:

- Vicinity map including north arrow, scale, boundary lines of site, and other information necessary to locate the development site.
- Name of subdivision or project and COF Project Number.
- Date(s) of preparation and revisions.
- Seal/signature of responsible engineer.
- Property lines, lot lines, right-of-way lines of streets, easements, and other rights-of-way, with accurate bearings and distances.
- Existing and proposed contours at 2' intervals. Spot elevations or 1' contour intervals where 2' contours do not show on the property or where needed to depict the grading.
- Floodplain and floodway locations, if applicable.
- Existing and proposed buildings or structures on the property and within 15 feet of the property limits, roof drainage directions, paved and landscaped areas, and dimensions of same.
- Finished floor and grade at foundation elevations of all structures.
- Location, dimensions, elevations, contours, characteristics, cross sections, profiles, and details for all existing and proposed drainage facilities, retaining walls, cribings, and other protective devices.
- Construction notes, specifications, and design details.
- Cross-sections of all open channels and detention basins, including design water surface elevation(s).
- Detention basins which show capacity, discharge(s), spillways, and the 100-year water surface elevation (WSE). Shading of the area inundated by the 100-year WSE is recommended.
- Recommendations included in the soils engineering or engineering geology report incorporated in the plans and/or specifications, if applicable.
- Dates and reference number of the soils report(s) together with the names, addresses and phone numbers of the firm(s) or individual(s) who prepared the report(s).
- Cut slopes no steeper than 2 horizontal (H):1 vertical (V) unless soils report states that the cut slope will be stable and will not create a hazard to public or private property.
- Fill slopes not constructed on natural slopes steeper than 2H:1V unless soils report recommends otherwise. Fill slopes shall not exceed 2H:1V.
- Top of cut slopes no nearer to the site boundary line than one fifth the vertical height of cut with a minimum of 2 feet and a maximum of 10 feet.
- Toe of fill slope(s) no nearer to the site boundary line than one half the height of the slope with a minimum of 2 feet and a maximum of 20 feet.
- Erosion control measures for all cut and fill slopes. (Note: referencing the landscape plan does not meet this requirement).
- Detention facility design details and cross-sections.
- Cut and fill quantities.
- Limits of grading or disturbance.

- Established benchmark of known elevation to which every other elevation is referenced.
- Horizontal control.
- Landscape Designer review block with signature.
- The engineer must review and sign the landscape plan for potential conflicts with grading and drainage plan.
- The following statement is required on all grading & drainage plans:

  "Adequate drainage, erosion and sediment control measures, best management practices, and/or other stormwater management facilities shall be provided and maintained at all times during construction. Damages to adjacent property and/or the construction site caused by the contractor's or property owner's failure to provide and maintain adequate drainage and erosion/sediment control for the construction area shall be the responsibility of the contractor and/or property owner."

2.4. PUBLIC IMPROVEMENT PLANS - DRAINAGE FACILITIES

In addition to plan presentation requirements set forth in Title 6 of the City of Flagstaff Engineering Design and Construction Standards & Specifications for public construction plans, the following information, at a minimum, shall also be included on public improvement plans for drainage facilities.

2.4.1. Storm Drain Systems:

- storm drain profile(s).
- the design frequency, discharge, and pipe capacity.
- pipe size, length, type, and slope(s), inlet/outlet invert elevations.
- outlet treatment.
- invert elevations in and out and rim elevations for all manholes and junction structures.
- existing and proposed grades and pipe cover.
- proposed utility crossings and vertical separations.
- typical trench detail(s)

2.4.2. Open Channels:

- the design frequency, design discharge, and channel capacity.
- velocities at the design discharge for all grades.
- channels grade(s).
- typical cross-section(s).
- transition details.
- HGL and available freeboard.
- channel lining(s) and details.
- drainage easement or right-of-way widths and setbacks.

2.4.3. Culverts:

- design frequency and design discharge.
- culvert slope and design velocities.
- inlet and outlet invert elevations.
- design and allowable headwater and tailwater elevations.
- headwalls.
- inlet and outlet protection measures.
- plotted headwater elevation(s) with contours in plan view.
- culvert profile w/controlling headwater elevation, pipe size and type, and slope.
- typical trench detail.
- temporary erosion and sediment control measures for channel reconstruction or culvert/bridge crossings shall also be included on public improvement plans

2.5. QUALITY OF SUBMITTALS

All drainage report and plan submittals presented to the City of Flagstaff for review shall be prepared and sealed by an Arizona Registered Professional Civil Engineer.

The engineer shall be held solely responsible for the correctness and adequacy of all data, drawings, calculations, and reports submitted to the City for review and approval. In addition, the engineer shall comply with all local, state, and federal floodplain regulations in the design of a development.

The Engineering Division will review drainage report and plan submittals for completeness and general compliance with all applicable local, state, and federal requirements. Approval by the City does not necessarily imply that the design is appropriate, nor that the development is in strict compliance with all applicable regulations and standards. Review and approval of drainage submittals shall not create liability on the part of the City or its employees for any flood damages that may result from reliance upon any administrative decision made by the City or its employees. When design procedures, equations, and data not included in this manual are used, the engineer must provide the City enough information on the methods and data to enable City staff to evaluate their applicability.

CHAPTER 3: HYDROLOGY

This chapter provides an overview of urban hydrologic methods and procedures used in the City of Flagstaff. The information presented herein is intended to provide the design engineer with guidance to the methods and procedures, their data requirements, and their applicability and limitations. Most of these methods and procedures can be applied using commonly available computer programs.

3.1. RATIONAL METHOD

The Rational formula is one of the most commonly used (and misused) simplified methods of estimating peak discharges for small uniform drainage areas. This method is typically used to size drainage structures for the peak discharge of a given return period. An extension of this method is often used to estimate the shape of the runoff hydrograph to design detention facilities and/or design a drainage structure that require routing of the hydrograph though the structure.

3.1.1. Rational Equation

The Rational Equation is expressed as follows:

$$Q = C_f \, C \, I \, A \tag{3.1}$$

where:
- Q = maximum rate of runoff, cfs
- C_f = antecedent precipitation factor[1]
- C = runoff coefficient
- I = rainfall intensity, in/hr
- A = drainage area tributary to the design location, acres.

[1]Less frequent, higher intensity storms will require modification of the runoff coefficient due to infiltration and other losses which have a proportionally smaller effect on runoff (Wright-McLaughlin, 1969). Thus, an adjustment to the Rational Equation (see Table 3-1) is required to account for antecedent precipitation conditions for storms greater than the 10-year event. **Note:** The product of C_f times C should not exceed 1.0.

TABLE 3-1: ANTECEDENT PRECIPITATION FACTORS

Storm Frequency	Factor
25 Year	1.1
50 Year	1.2
100 Year	1.25

3.1.2. Rational Method Assumptions

The following assumptions are inherent when using the Rational Equation:

1. The peak flow occurs when the entire watershed is contributing to the flow.
2. The rainfall intensity is the same over the entire watershed.
3. The rainfall intensity is uniform over a duration equal to the time of concentration.
4. The frequency of the computed peak flow is the same as that of the rainfall intensity (e.g., the 25-year rainfall intensity is assumed to produce the 25-year peak flow).
5. All the land uses within a drainage area are uniformly distributed throughout the area.

3.1.3. Rational Method Limitations

The following limitations shall apply to the Rational Method in the City of Flagstaff:

1. The total drainage area must be less than or equal to 20 acres.
2. The time of concentration cannot be less than 5 minutes or greater than 60 minutes.
3. The land use of the contributing watershed must be fairly consistent over the entire drainage area and uniformly distributed throughout the area. That is, the contributing area should not consist of a large percentage of two or more land uses (e.g. 50% commercial and 50% undeveloped forest).
4. The contributing watershed cannot have drainage structures or facilities which would require flood routing to estimate the discharge at the point of interest.
5. If it is important to locate a specific land use within the drainage area then another hydrologic method should be used where hydrographs can be generated and routed through the drainage system.

In cases where the Rational Method is not applicable, more appropriate hydrologic methods shall be used as outlined in Sections 3.2 and 3.3 of this chapter.

3.1.4. Estimation of Area

An adequate topographic map of the subject site and surrounding drainage area is required to define the drainage boundary in acres. A field inspection of the area should also be made to determine if natural drainage divides have been altered.

It is generally recommended to consider the largest reasonable drainage area, especially in urbanized areas. The contributing drainage area can increase with more intense storm events as larger storms can overtop existing street crowns, curbs, or other drainage facilities.

3.1.5. Estimation of Rainfall Intensity

The rainfall intensity (I) in Equation 3.1 is estimated in inches/hour for the specified return period having a duration equal to the time of concentration for the drainage basin.

The rainfall intensities for the City of Flagstaff are shown in Table 3-2. These values were developed using the rainfall depth-duration frequency statistics for Arizona from NOAA Atlas 2, Volume VIII, Arizona (Miller et al, 1973).

3.1.6. Time of Concentration Estimation

The time of concentration (T_c) is the time necessary for the runoff to travel from the hydraulically most remote point of the drainage area to the point of interest. The T_c is not necessarily determined by the longest travel path. Several different T_c computations may be necessary. The duration of rainfall is then set equal to the estimated T_c and is used to obtain the average rainfall intensity (I).

The T_c used in the Rational Method typically consists of an inlet time (e.g., sheet flow or shallow concentrated flow) to a point where the flow enters a storm drain or open channel plus the time of flow in the storm drain or channel to the point of interest. Formulas for computing the time or composite time of concentration, as given in Equations 3.2 through 3.8, shall be used.

3.1.6.1. Overland or Sheet Flow

Sheet flow is the shallow mass of runoff on a planar surface with uniform depth across a sloping surface. The sheet flow length shall not exceed 100 feet in urban areas and 300 feet in natural, rural or wooded areas.

For sheet flow, Mannings kinematic solution, Equation 3.2, shall be used:

$$\mathbf{T_t = [0.007\ (nL)^{0.8} / (2.0)^{0.5}\ S^{0.4}]} \qquad \textbf{(3.2)}$$

where: T_t = sheet flow travel time, hr
n = Mannings roughness coefficient (see Table 3-3)
L = flow length, ft
s = land slope, ft/ft

3.1.6.2. Shallow Concentrated Flow

After short distances, sheet flow tends to concentrate in rills and then gullies. The velocity of such shallow concentrated flow can be estimated by the following equations for slopes greater than 0.005 ft/ft:

Unpaved: $$\mathbf{V = 16.1345\ (S)^{0.5}} \qquad \textbf{(3.3)}$$

Paved: $$\mathbf{V = 20.3282\ (S)^{0.5}} \qquad \textbf{(3.4)}$$

where: V = average velocity, ft/s
S = slope of hydraulic grade line, ft/ft

3.1.6.3. Open Channel Flow

Velocities for open channel can be estimated by Manning's equation:

$$\mathbf{V = (1.49\, r^{2/3}\, s^{1/2})/n} \qquad \textbf{(3.5)}$$

where:

	V	= average velocity, ft/s
	r	= hydraulic radius (a/wp), ft
	a	= cross-sectional area, ft^2
	wp	= wetted perimeter, ft
	s	= slope of hydraulic grade line, ft/ft
	n	= Manning's roughness coefficient for open channel flow

Average velocity shall be determined based on the bank-full water surface elevation.

3.1.6.4. *Street Gutter Flow*

Flow velocities for concrete gutters can be estimated by:

$$\mathbf{V = 86\, S^{0.5}\, d^{0.67}} \textbf{ or} \qquad \textbf{(3.6)}$$

If d = 6"(0.5'): $\mathbf{V = 54\, S^{0.5}}$ **(3.7)**
If d = 4"(0.33'): $\mathbf{V = 41\, S^{0.5}}$ **(3.8)**

where:

	S	= longitudinal slope of gutter, ft/ft
	d	= depth of water surface, ft
	v	=velocity, ft/s

Note: These equations do not apply when depth of water is above the top of curb.

TABLE 3-2: CITY OF FLAGSTAFF RAINFALL INTENSITIES, INCHES/HOUR

Duration	Frequency, In Years					
	2	5	10	25	50	100
5-min.	3.96	5.04	5.76	6.84	7.68	8.52
10-min.	3.06	3.90	4.50	5.34	6.00	6.66
15-min.	2.48	3.20	3.48	4.40	4.92	5.48
30-min.	1.58	2.06	2.40	2.86	3.22	3.58
1-hour	0.95	1.25	1.46	1.76	1.98	2.21
2-hour	0.56	0.73	0.85	1.02	1.15	1.28
3-hour	0.41	0.53	0.62	0.74	0.83	0.92
6-hour	0.24	0.31	0.36	0.43	0.48	0.53
12-hour	0.14	0.19	0.21	0.26	0.29	0.32
24-hour	0.08	0.11	0.12	0.15	0.17	0.19

TABLE 3-3: MANNING'S "n" FOR SHEET FLOW[1]

Surface Description			‘n’ value
Concrete			0.012
Asphalt			0.011
Fallow (no residue)			0.05
Cultivated soils:	Residue cover ≤ 20%	0.06	
	Residue cover > 20%	0.17	
Grass:	Short grass, prairie		0.15
	Dense grasses		0.24
	Bermuda grass[1]		0.41
	Bluegrass sod		0.45
Range (natural)			0.13
Woods[2]:	Light underbrush		0.40
	Dense underbrush		0.80

Note: These values were determined specifically for sheet flow conditions and are not appropriate for open channel flow calculations. These “n” values are a composite of information compiled by Engman (1986), with additions from the Florida Department of Transportation Drainage Manual (1986).

[1]Includes species such as weeping lovegrass, buffalo grass, blue grama grass, and native grass mixtures.

[2]When selecting “n”, consider cover to a height of about 0.1 ft. This is the only part of plant cover that will obstruct sheet flow.

Rational Runoff Coefficient

Perhaps the most important variable in the Rational Equation is selection of the dimensionless runoff coefficient (C) which represents that fraction of rainfall that appears as surface runoff from a tributary area. This fraction of rainfall runoff is independent of rainfall intensity or volume for impervious areas, such as streets, rooftops, and parking lots. However, for pervious areas, the fraction of runoff varies with rainfall intensity and the accumulated volume of runoff. Therefore, the selection of a coefficient that is appropriate for the storm, soil type, slope, and land use conditions is critical. The design engineer should always document how and why a particular runoff coefficient was chosen. Thought should also be given to future changes in land use that might occur during the service life of the proposed drainage facility which can result in an inadequate drainage system.

Runoff coefficients based on surface type can be found in Table 3-4. Runoff coefficients based on land use, slope, and soil type are to be chosen from Table 3-5. The Hydrologic Soil Groups (HSG) used in Table 3-5 were developed by the Soil Conservation Service (now the Natural Resources Conservation Service) based on infiltration rates and are described as follows:

> <u>Group A</u> - Soils having low runoff potential due to high infiltration rates even when thoroughly wetted. These soils consist primarily of deep, well to excessively drained sands or gravels and have a high rate of water transmission (greater than 0.3 in/hr).
>
> <u>Group B</u> - Soils having moderate infiltration rates when thoroughly wetted and consist primarily of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of transmission (0.15 - 0.30 in/hr).
>
> <u>Group C</u> - Soils having low infiltration rates when thoroughly wetted and consist primarily of soils with a layer that impedes downward movement of water and soils moderately fine to fine textures. These soils have a low rate of water transmission (0.05 - 0.15 in/hr).
>
> <u>Group D</u> - Soils having high runoff potential. They have very low infiltration rates when thoroughly wetted and consist primarily of clay soils with high swelling potential, soils with permanently high water tables, soils with a claypan or clayey layer at or near the surface, and shallow soils over nearly impervious parent material. These soils have a very low rate of water transmission (0 - 0.05 in/hr).

HSG soils textures can be classified as:

> A - Sand, loamy sand, or sandy loam
> B - Silt loam or loam
> C - Sandy clay loam
> D - Clay loam, silty clay loam, sandy clay, silty clay, or clay

TABLE 3-4: RUNOFF COEFFICIENTS (C) BY SURFACE TYPE

Surface Description	Runoff Coefficients
Streets	0.95
Asphaltic Concrete	0.95
Concrete	0.95
Brick Pavers	0.90
Compacted ABC roadways/shoulders	0.50 - 0.70
Drives and Sidewalks	0.95
Gravel (open)	0.50
Roofs	0.95

Surface Description	SLOPE Flat < 2%	 Avg. 2% - 7%	 Steep > 7%
Lawns			
Sandy Soils	0.10	0.20	0.30
Gravelly Soils	0.15	0.25	0.35
Clay Soils	0.20	0.30	0.40
Dense Vegetation			
Sandy Soils	0.07	0.14	0.20
Gravelly Soils	0.11	0.20	0.27
Clay Soils	0.15	0.25	0.35
Woods			
Sandy Soils	0.05	0.10	0.15
Gravelly Soils	0.07	0.12	0.17
Clay Soils	0.10	0.15	0.20

The coefficients in Table 3-4 are based on the assumption that the design storm does not occur when the surface is frozen. These coefficients are for recurrence intervals less than 25-years, therefore for 25, 50, and 100 year storms, the adjustment factors given in Table 3-1 must be applied to these values. When more than one surface type is present, a composite or weighted runoff coefficient (C_w) value must be used and can be calculated by an area (A) weighted average given in Equation 3.9 below:

$$C_w = (C_1A_1 + C_2A_2 + ... + C_nA_n) / A_{total} \quad (3.9)$$

TABLE 3-5: RUNOFF COEFFICIENTS BY SCS HSG, SLOPE, LAND USE

LAND USE	**A**			**B**			**C**			**D**		
	0-2%	**2-6%**	**6%+**	**0-2%**	**2-6%**	**6%+**	**0-2%**	**2-6%**	**6%+**	**0-2%**	**2-6%**	**6%+**
Bare Soil	0.09	0.14	0.18	0.12	0.17	0.24	0.16	0.21	0.31	0.20	0.25	0.38
Pasture	0.12	0.20	0.30	0.18	0.28	0.37	0.24	0.34	0.44	0.30	0.40	0.50
Meadow	0.10	0.16	0.25	0.14	0.22	0.30	0.20	0.28	.036	0.24	0.30	0.40
Forest	0.05	0.08	0.11	0.08	0.11	0.14	0.10	0.13	0.16	0.12	0.16	0.20
Residential												
Lot size 1/8 ac.	0.25	0.28	0.31	0.27	0.30	0.35	0.30	0.33	0.38	0.33	0.36	0.42
Lot size 1/4 ac.	0.22	0.26	0.29	0.24	0.29	0.33	0.27	0.31	0.36	0.30	0.34	0.40
Lot size 1/3 ac.	0.19	0.23	0.26	0.22	0.26	0.30	0.25	0.29	0.34	0.28	0.32	0.39
Lot size 1/2 ac.	0.16	0.20	0.24	0.19	0.23	0.28	0.22	0.27	0.32	0.26	0.30	0.37
Lot size 1 ac.	0.14	0.19	0.22	0.17	0.21	0.26	0.20	0.25	0.31	0.24	0.29	0.35
Industrial	0.67	0.68	0.68	0.68	0.68	0.69	0.68	0.69	0.69	0.69	0.69	0.70
Commercial	0.70	0.71	0.72	0.71	0.72	0.74	0.74	0.75	0.75	0.75	0.80	0.85
Open Space	0.05	0.10	0.14	0.08	0.13	0.19	0.12	0.17	0.24	0.16	0.21	0.28

The coefficients in Table 3-5 are based on the assumption that the design storm does not occur when the surface is frozen. These coefficients are for recurrence intervals less than 25-years, therefore for 25, 50, and 100 year storms, the adjustment factors given in Table 3-1 must be applied to these values.

NOTE: The residential, industrial, and commercial coefficients given in Table 3-5 are intended to be used for estimating discharges for larger areas where using weighted coefficients is impracticable. These land use coefficients shall not be used for smaller drainage basins where weighted runoff coefficients can be more accurately computed using the values given in Table 3-4 and the values for undeveloped areas given in Table 3-5.

3.1.7. Rational Method Hydrograph Shape

The Rational Method is designed solely to determine an estimated peak discharge for a given drainage area. However, it is often necessary to route the storm inflow through a drainage structure such as a detention basin. A procedure for estimating dimensionless hydrograph shapes for urbanized and undeveloped watersheds can be found in Chapter 2 of the Arizona Department of Transportation, Highway Design Manual, Hydrology, Report No. FHWA-AZ-93-281, March 1993.

3.2. SCS TR-55 METHOD

Analysis for drainage areas ranging from 10 to 40 acres in size may be designed using methods described in USDA, SCS, Technical Release 55, "*Urban Hydrology for Small Watersheds",* June 1986. Curve numbers for undeveloped forest shall be per ADWR, *Oak Creek Flood Warning System Hydrology Report,* TR 90-4, September 1990.

3.2.1. Graphical Peak Discharge Method

This method calculates peak flows as a function of drainage basin area, potential watershed storage, and the time of concentration when a hydrograph and/or reservoir routing are not required. Limitations for this method which can impact the accuracy of estimated peak flows are:

- The watershed should be hydrologically homogeneous (i.e., describable by one curve number greater than 40) for this method.
- T_c should be between 0.1 and 10 hours.
- I/P should be between 0.1 and 0.5.
- Watershed should have one main channel or branches with nearly equal times of concentration.
- Neither channel or reservoir routing can be incorporated.
- Adjustment factor (F_p) is applied for ponds/swamps that are not in the T_c flow path.
- Snow, frozen ground, and subsurface flows are not present.

3.2.2. Tabular Hydrograph Method

This method was developed to estimate partial composite flood hydrographs at any point in a watershed and is generally applicable to small, non-homogeneous areas which are beyond the limitations of the Rational Method. It is applicable for estimating the effects of land use change in a portion of the watershed as well as estimating the effects of proposed structures. Chapter 5 of TR-55 provides a detailed description of this method.

Assumptions and limitations inherent in the Tabular Method are as follows:

1. Total area should be less than 2000 acres. Typically, subareas are far smaller than this because the subareas should have fairly homogeneous land use.
2. Subarea T_c is from 0.1 to 2.0 hours and reach travel time (T_t) is less than or equal to three (3) hours.

3. Drainage areas of individual subareas differ by less than a factor of five.
4. Type II rainfall distribution is used for Arizona.

If either the T_c or the T_t limitations are exceeded, the watershed is very complex, or a higher degree of accuracy is required, HEC-1 methodology shall be utilized.

3.3. HEC-1 METHOD

Analysis of large drainage areas shall be Modeled using HEC-1 or HEC-HMS techniques. General guidance on HEC-1 can be found in the HEC-1, *Flood Hydrograph Package,* Users Manual or the Arizona Department of Transportation, *Highway Design Manual, Hydrology,* Report No. FHWA-AZ-93-281, March 1993.

The storm duration (PH Record) for the rainfall input is dependent on the total watershed area and shall be as follows:

1. If the watershed area is less than or equal to 1.0 square mile, the design storm duration shall be 6 hours.
2. If the watershed area is greater than or equal to 1.0 square mile, the design storm duration shall be 24 hours.

Storm durations used for FEMA flood studies shall be in accordance with current FEMA guidelines.

Most watersheds in the Flagstaff area are ungaged, therefore, unit hydrograph and soil loss parameters should be developed using the SCS dimensionless unit hydrograph (UD Record) and SCS curve number (CN) methodology. The SCS Type II rainfall distribution shall be used.

Calibration of the HEC-1 model to observed data should be performed if adequate stream gage and rainfall data is available for the watershed.

CHAPTER 4: OPEN CHANNELS

The purpose of this chapter is to present policies and design criteria for open channels intended to be public and outline channel and erosion protection design procedures. It is recommended that private open channels also be designed in accordance with this chapter.

An open channel is defined as a conveyance in which water flows with a free surface. Open channels in the City of Flagstaff will either be natural washes or artificial. Natural washes will typically consist of a compound cross-section consisting of a low flow channel and adjacent overbank floodplains. Artificial channels typically include drainage ditches, roadside channels, irrigation channels, and swales which have a regular geometric cross-section and can be either lined or unlined.

The principles of open channel flow are the same regardless of the channel type. Flow classifications are generally classified as steady or unsteady, uniform or varied, and subcritical or supercritical. Specific information on open channel flow concepts and total energy relationships can be found in Chow, V.T, *Open Channel Hydraulics*, 1959.

4.1. POLICIES

a. All open channel designs and/or related activities shall meet the minimum requirements and/or design criteria for the City of Flagstaff, Federal Emergency Management Agency, U.S. Army Corps of Engineers 404 permitting, and the Arizona Department of Environmental Quality, as applicable.

b. Safety of the general public shall be considered in the selection of location and cross-sectional geometry of artificial channels.

c. The design of artificial channels shall consider the frequency and type of maintenance expected and make allowance for access of maintenance equipment.

d. Channels shall be designed to follow natural drainage alignments whenever possible. Environmental impacts of modifications to natural channels, including disturbance of wildlife habitat, wetlands, and stream bank stability shall be assessed and disturbance minimized.

e. All channels which are to be maintained by the City of Flagstaff must be dedicated to the City either in fee title or granted as drainage easement.

f. Unless proper authorization from the City of Flagstaff and the adjacent property owner(s) is obtained, open channels must enter and exit a site where the channel historically flowed.

4.2. NATURAL CHANNELS

Natural washes, not designated by the City or FEMA as regulatory floodplains, which cross private property are encouraged be left in their natural state, if possible, upon development. Planning and other design measures must be used to protect development adjacent to natural washes from the flooding and erosion typically associated with increased development. As additional runoff from development is added to a natural channel, these channels can experience erosion and may require on-site detention, grade control, and/or localized bank protection.

Washes designated as Rural Floodplains per the City of Flagstaff Land Development Code must be left undisturbed and in their natural state. Alteration through channelization and FEMA map revisions and/or other development within a designated Rural Floodplains is prohibited.

If relocation of a natural stream channel is unavoidable, the cross section shape, meander pattern, roughness, sediment transport capacity, and slope should conform to the existing conditions insofar as practicable. Some means of energy dissipation may be necessary when existing conditions cannot be duplicated.

U.S. Army Corps of Engineers Section 404 and Section 401 of the Clean Water Act permitting processes may apply for impacts on Waters of the U.S. such as stream channelization, relocation, bank stabilization, or roadways crossings. Approval by the City does not supersede or waive compliance with other applicable Federal and State laws.

Natural channels should be analyzed using field observation, surveyed cross-sections, and Normal Depth/Uniform Flow (Manning's Equation) or step-backwater methods (HEC-2 or HEC-RAS are preferred), as applicable.

4.3. ARTIFICIAL CHANNELS

Artificial open channels have a wide variety of applications ranging from landscaping swales to large flood control facilities. The selection of the open channel type is influenced by numerous factors such as hydraulics, structural features, environmental concerns, sociologic impact, risks and liability, maintenance, and economics. Examples of various types of open channels are depicted in Figure 4-1.

All artificial open channel drainage systems shall be designed for the 25-year design storm at a minimum and checked with the 100-year design storm to determine available freeboard, minimum finished floor for structures elevations adjacent to the channel, and the potential for flood damages.

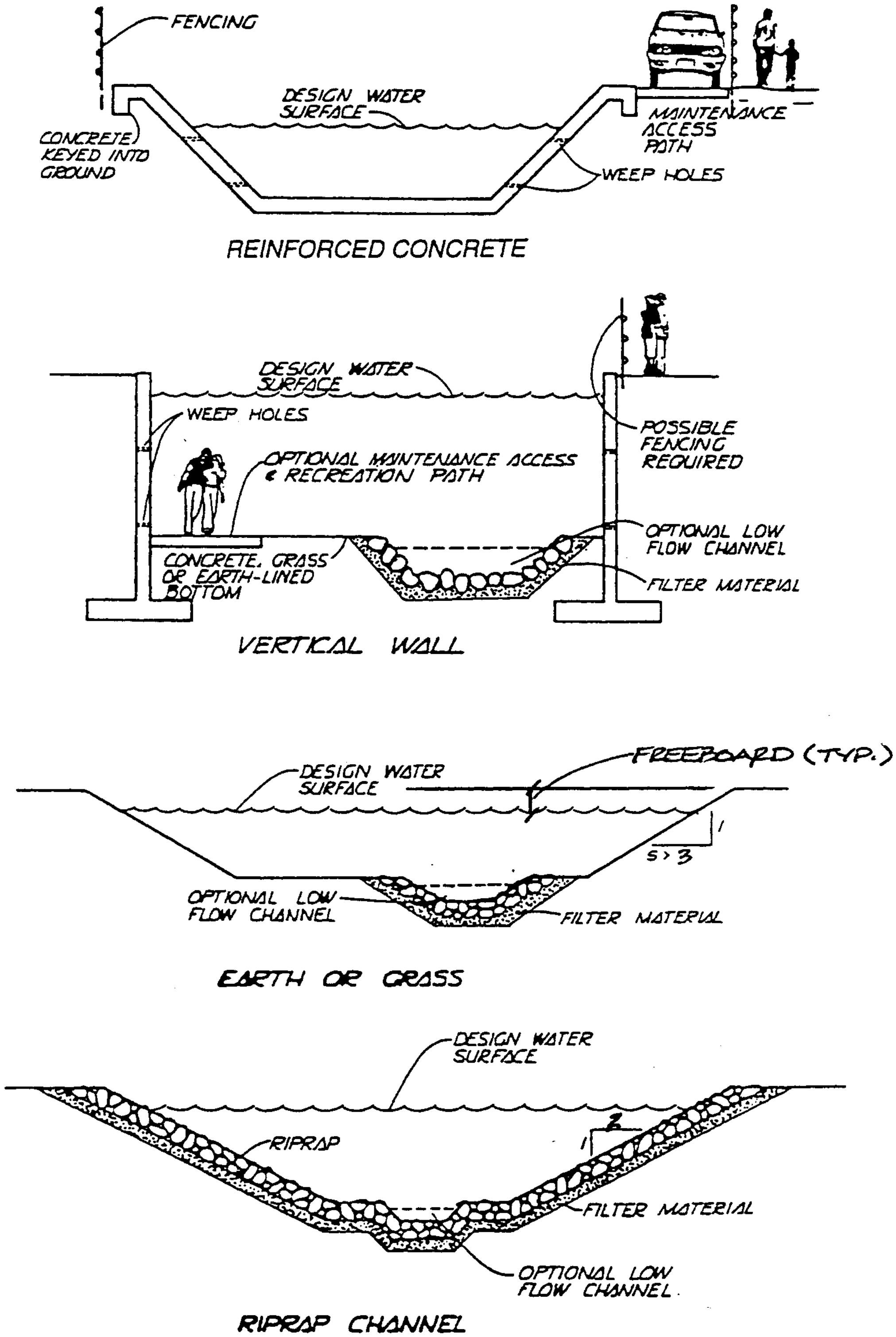


Figure 4-1: Typical Artificial Channels

4.3.1. Maintenance Access and Easements

Minimum width of fee title dedications or drainage easements for public open channels shall be dependent upon the top width of the channel, required setbacks per Uniform Building Code, and the need for maintenance access roads adjacent to the channel.

Maintenance access shall be provided along at least one side of all new open channels intended to become public. The minimum maintenance access width shall be twelve (12) feet. Maintenance access may either be necessary adjacent to the top of the channel bank, down into the channel bottom, or a combination of both. In all cases, the right-of-way or drainage easement must be of sufficient width to allow maintenance vehicles or equipment to operate freely.

4.3.2. Cross-Sectional Geometry

Trapezoidal or compound cross-sections are required for public open channels, unless prior approval of an alternate design is obtained from the Stormwater Manager. Channel side slopes shall be stable throughout the entire length and will be dependent upon the channel bank or lining material. Slope stability analysis may be required in some instances. Side slopes shall be no steeper than 3H:1V for natural vegetal or earth linings and 2H:1V for ungrouted riprap lining. Side slopes for rigid lined channels may be steeper depending on the structural stability of the lining. Channels with bottom widths greater than ten (10) feet should be designed with a minimum cross slope of 12H to 1V.

The depth of public open channels shall be determined by computation of the normal uniform flow depth for the design discharge plus a minimum of one foot of freeboard (see Section 4.3.4). The maximum depth for artificial public channels in residential areas should not exceed three (3) feet, including freeboard, for safety considerations.

Sediment transport requirements must be considered for conditions of flow below the design frequency, especially for multi-use corridors, if applicable. Low and high flow sections shall be considered in the design of channels with large cross sections and/or a design discharge greater than fifty (50) cubic feet per second.

4.3.3. Channel Slope and Velocities

The slope of a proposed channel is typically dependent upon the natural topography. However, variations can be accomplished by altering the channel alignment through a development, by adjusting the elevation of inflow and outflow points, or utilizing grade control. The selected channel configuration, alignment, and slope should result in a stable channel. The minimum allowable channel slope for public channels shall be 0.5 percent. The maximum allowable velocity for any open channel shall not exceed eighteen (18) feet per second.

Abrupt changes in channel slope shall be avoided except when necessary to create a desired hydraulic jump or grade control. When abrupt changes in slope are unavoidable, the slope changes should not cause the channel top width to vary by more than fifteen (15) percent.

Froude Numbers for earth, vegetal, grass, or riprap lined channels should not exceed 0.86 due to instability experienced with critical flow. The Froude Numbers for concrete lined channels should not exceed 2.0 unless supercritical flow is being designed for. Supercritical flow designs must consider "run-up" and/or wave action. Channel designs should also avoid flow regimes that have Froude numbers in the range of 0.86 - 1.13.

All channel designs must identify supercritical or subcritical flow regimes for all portions of the channel reach. Natural, earth lined, vegetal, or riprap lined channels should not be designed for supercritical flow.

4.3.3.1. *Grade Control Structures*

Most channels constructed with earthen bottoms are constructed on slopes that are greater than their equilibrium slopes. In such cases, grade control structures (e.g., cut-off walls or sills, drop structures, chutes, or flumes) may be required to reduce bed erosion, reduce increases in channel slope and flow velocities, and to prevent general streambed degradation.

All grade control structures should consist of a control section, an adjacent protection section, and an energy dissipation section. Use of natural rock grade control structures is encouraged.

Concrete cut-off walls or sills, if required, shall be a minimum of eight (8) inches thick Class A concrete, two (2) feet deep below the channel bottom, and extend across the entire channel.

General Design Procedure: The general procedure for grade control structure design is:

1. Determine the total fall through the design reach.
2. For the selected channel, determine the equilibrium slope.
3. Determine the amount of fall to be controlled by the grade control or drop structures from the tailwater of the upstream structure to the head on the next structure downstream.
4. Select the size, location and type of structures to be used.

Design guidance on grade control structures can be found in the National Engineering Handbook, Section 11, "Drop Spillways" and Section 14, "Chute Spillways", or the Urban Drainage and Flood Control District, Denver Colorado, *Drainage Criteria Manual, Volume 2, "Grouted Sloping Boulder Drop Structures"*.

4.3.4. Freeboard

Freeboard is the additional channel depth required between the calculated water surface elevation and the top of the lowest channel bank. The purpose of freeboard is to protect against hydraulic disturbances such as waves, obstructions of flow, debris, and sediment accumulation in addition to providing a factor of safety for flows greater than the design discharge.

The freeboard, under normal depth conditions, shall be calculated with Equation 4.1 for public channels, with a minimum of one (1) foot.

$$\mathbf{FB = 0.25\ [Y + (V^2 / 2g)]} \qquad \mathbf{(4.1)}$$

where:
- FB = required freeboard, in feet;
- Y = the maximum depth of flow, in feet;
- V = the average velocity of flow, in feet/second;
- g = the acceleration due to gravity (32.2 ft/sec²)

Additional freeboard may be required at junctions and culvert inlets, where backwater effects may occur, and at locations where supercritical flow or hydraulic jumps occur. In areas of channel constriction, decrease of channel slope, or hydraulic jump, the channel depth may need to be increased to accommodate increases in flow depth and minimum freeboard requirements.

Special consideration should be given to the increased velocities and shear stresses that are generated as a result of non-uniform flow in bends. Superelevation of flow at channel bends is another important consideration and although the degree of superelevation is relatively small (usually less than one foot) when compared with the overall flow depth in the bend, it should be considered when establishing freeboard limits for bank protection on sharp bends. For specific design criteria for channel deflections or curves, refer to Brater and King, *Handbook of Hydraulics,* 1976 and Chow, *Open Channel Hydraulics*, 1959.

Channel linings of protected open channels shall extend to the elevation necessary to include the freeboard requirement.

4.3.5. Hydraulic Jump

Hydraulic jumps should only occur in planned locations at hydraulic structures and should be checked in the following cases:

- the slope of the channel abruptly changes from steep to mild;
- at abrupt expansions or contraction in the channel section;
- at locations where obstructions occur, such as a culvert or bridge, in a channel of steep slope;
- at sharp bends;
- dip crossing or culverts; and
- where steep or supercritical channels discharge into another channel.

Recommended procedures for computing hydraulic jump can be found in FHWA, HEC-14, 1983 and/or Chow, *Open Channel Hydraulics,* 1959.

4.3.6. Channel Transitions

Transition sections shall be designed to provide a gradual transition to avoid turbulence and eddies. Energy losses in transitions should be accounted for as part of the water surface profile calculations. Recommended guidance can be found in *U.S. Army Corps of Engineers, EM 1110-2-1601.*

A straight line connecting flow lines at the two ends of the transition should not make an angle greater than 12.5 degrees with the axis of the main channel.

Scour downstream of rigid to natural or steep to mild transition sections should be accounted for through velocity slowing and energy dissipation.

4.4. CHANNEL LININGS

One of the most influencing factors in the design of artificial channels is the channel lining. The most prominent channel lining types include earth (natural), grass, rock, concrete, and other biotechnical or synthetic measures. These linings can be used alone or in combination to form a composite channel. Soil cement linings are not permitted for open channels intended to become public.

It is recommended to incorporate the use of grade control to limit channel slopes/velocities and preclude the need for channel linings. The use of alternate channel treatments other than riprap is strongly encouraged. Methods developed by the International Erosion Control Association for channel protection and streambank stabilization are recommended.

The order of preference for open channels and swales shall be:

1. The use of natural bio-f liters to promote infiltration and water quality treatment.

2. The use of underground facilities.

3. The use of natural land forms. Side slopes not to exceed 2H:1V. The horizontal alignment shall meander to the extent possible. Riprap lining shall be limited to areas where erosion is anticipated.

4.4.1. Earth Lined/Naturally Vegetated Channels

Earth lined and naturally vegetated channels are classified the same for the purposes of this section. This includes the common practice of hydroseeding with native grasses. Allowable velocities for erodible channels are given in Table 4-1. Shear stress and tractive force analysis may be required.

4.4.2. Grass Lined Channels

Grass lined channels require proper maintenance and irrigation to function properly. Non-maintained/non-irrigated grass lined channels will revert back to the earth lined channel classification in Section 4.4.1. Conditions under which vegetal linings may not be acceptable include, but are not limited to:

- Standing or continuous flowing water;
- Lack of regular maintenance;
- Lack of nutrients and/or inadequate topsoil;

- excessive shade; and
- high flow velocities.

Temporary erosion control measures (e.g., jute or straw matting or spray tacking substances) may be required to provide sufficient time for seeding to be established. Seeding and mulch should only be used when the channel's design velocity does not exceed the allowable velocity for bare soil. Allowable velocities for grass lined channels are given in Table 4-2.

4.4.3. Flexible Linings - Riprap Design Criteria

Due to long term maintenance considerations, fully lined riprap channels intended to become public must receive prior approval from the Stormwater Manager and Public Works Department. The use of grade control or other channel linings must be examined first. Riprap lined channels should only be designed for subcritical flow regimes. If the channel is to be designed as a complete riprap channel (side slopes and bottom), the riprap itself will control the velocity and an iterative process between riprap size and channel velocity, due to riprap roughness, is necessary.

The channel cross section design must account for riprap thickness in channel excavation. The riprap must be "inlaid" by over excavating to the depth of the riprap layer thickness. Riprap should not be placed on top of swale or channel bottom.

Wire tied riprap or gabion baskets may be required where higher velocities or steeper channel side slopes are proposed. Rock sizes and basket characteristics should meet the manufacturers specifications.

4.4.3.1. Edge and End Treatment

The edges of riprap revetments (flanks, toe, and head) require special treatment to prevent undermining. The flanks of the revetment shall be designed as illustrated in Figure 4-2. Riprap can be substituted for the compacted fill shown in Section A-A in Figure 4-2.

Undermining of the toe is one of the primary causes of riprap failure. The toe of the riprap shall be designed as illustrated in Figure 4-3a. The toe material should be placed in a toe trench along the entire length of the riprap blanket. Where a toe trench cannot be excavated, the riprap blanket should terminate in a thick, stone toe at the level of the streambed (see alternate design in Figure 4-3a). Under this alternate design, care must be taken to not form a dike at/or along the toe or that the channel's design is not impaired by this toe mound.

The size of the toe trench (or alternate stone toe) is controlled by the anticipated depth of scour along the stream bed and toe. As scour occurs, the stone in the toe will launch into the eroded area as illustrated in Figure 4-3b.

The volume of rock required for the toe must be equal to or exceed 1.5 times the volume of rock required to extend the riprap blanket (at its design thickness on a 2:1 slope) to the anticipated depth of scour. Dimensions should be based on the required volume using the thickness and depth determined by scour depth.

4.4.3.2. Riprap Material Criteria

The shape of the rock riprap shall be blocky or angular rather than elongated or smooth river run rock. Cinder material (e.g. clinkers, tufa, and scoria) shall not be used for rock riprap lining.
Rock material should be hard, dense, and have sufficient durability to withstand abrasive water action and freeze/thaw cycles. Durability index and absorption laboratory tests should be conducted to determine the quality of the rock (see ASTM C127). The durability absorption ration (DAR) is then computed as follows:

$$\mathbf{DAR} = \frac{\textbf{Durability Index}}{\textbf{\% Absorption + 1}} \tag{4.2}$$

DAR specifications are:

1. DAR > 23, material is accepted;
2. DAR < 10, material is rejected;
3. DAR 10 through 23:
   a. DI of 52 or greater, material is accepted; and
   b. DI of 51 or less, material is rejected.

Rock riprap size is typically specified as D_{50}, which is defined as the average diameter of a rock for which 50 percent of the gradation is finer, by weight. The riprap blanket gradation should form an interlocked mass of angular rock with little or no apparent voids or pockets. The recommended maximum rock size is two (2) times the D_{50} and the minimum size is one-third (1/3) of the D_{50}. No rock shall have a length exceeding 3.0 times the calculated D_{50}. Section 4.4.4 provides a recommended design procedure for determining D_{50}.

The riprap blanket thickness shall be a minimum of 1.5 to 2.0 times the calculated D_{50}. A minimum thickness of twelve (12) inches shall be used in all cases. A thickness of $2.0D_{50}$ is recommended to offset the occurrence of segregation of the rock gradation when the riprap is mechanically dumped, rather than hand placed or keyed into place. The thickness shall be measured perpendicular to the slope on which the riprap will be placed.

4.4.3.3. Filter Layers

A filter layer is a transitional layer of gravel, small stone, or fabric placed between the underlying soil and the riprap blanket (or other structure). A filter shall be used whenever the riprap is being placed on noncohesive material subject to subsurface drainage.

Granular Filters: For rock riprap, a filter ratio of five (5) or less between layers will usually result in a stable condition. The filter layer (blanket) ratio is defined as the ratio of the 15 percent particle size (D_{15}) of the filter to the 85 percent particle size (D_{85}) of the base layer (channel bank). An additional requirement for stability is that the ratio of the 15 percent particle size of the filter material to the 15 percent particle size of the bank material should be greater than 5 but less than 40. These requirements are expressed as:

$$\frac{(D_{15})_{filter}}{(D_{85})_{bank}} < 5 < \frac{(D_{15})_{filter}}{(D_{15})_{bank}} < 40 \qquad (4.3)$$

The above relationships must hold between the filter blanket and the base material and between the riprap and filter blankets (USDOT, FHWA, HEC-15, 1988).

If the inequalities are satisfied by the riprap layer itself, then no filter layer is needed. If a single layer of filter material will not satisfy the filter requirements, one or more additional layers of filter material must be used. The filter requirement applies between the bank material and the filter blanket, between successive layers of filter material if more then one layer is used, and between the filter blanket and the riprap. In addition to the filter requirement, the grain size curves for the various layers should be approximately parallel to minimize the infiltration of fine material from the finer layer to the coarse layer. Not more than 5 percent of the filter material should pass the No. 200 sieve.

The thickness of the filter blanket should range from six (6) to fifteen (15) inches for a single layer, and 4 to 8 inches for individual layers of a multiple layer blanket. Where the gradation curves of adjacent layers are approximately parallel, the blanket thickness should approach the minimum. The blanket thickness should be increased above the minimum proportionately as the gradation curves depart from a parallel pattern. Figure 4-4 can be used as an aid in designing granular filters.

Filter Fabric: Synthetic, geotextile filter fabrics can be used as an alternative to granular filters. However, it is not a complete substitute for granular filters. The site conditions, application, and installation procedures must be carefully considered when evaluating filter fabric as a replacement for granular filters. The design criteria for such filter fabrics is dependent upon the permeability and effective opening size of the fabric. Essentially, the permeability of the fabric must exceed the permeability of the underlying soil and the fabric opening size (AOS) must be less than the soil particle size.

The criteria for the AOS is expressed as follows:

1. For soil with less than 50 percent of the particles, by weight, passing a No. 200 sieve, the AOS should be less than 0.6 mm (No. 30 sieve), and

2. For soil with more than 50 percent of the particle, by weight, passing a No. 200 sieve, the AOS should be less than 0.3 mm (No. 50 sieve).

Problems associated with filter fabrics are less resistance to stone movement and tearing. Direct mechanical dumping is not permitted. A minimum of four (4) inches of bedding material is recommended over non-woven filter fabric to prevent damage from mechanically dumped riprap. Experimental evidence indicates that when channel banks are subjected to wave action, non-cohesive bank material has a tendency to migrate downstream beneath the fabric; this tendency was not found with granular filters. Filter fabrics can also induce translational or modified slump failures when used under rock riprap installed on steep slopes.

Filter Fabric Placement: Failures in filter fabrics often occur due to improper installation. Therefore, to provide optimum performance, a properly selected fabric should be installed per the following guidelines or the manufacturer's guidelines, whichever is more restrictive:

a. Heavy riprap may stretch the fabric as it settles. A four (4) to six (6) inch gravel bedding is required to be placed beneath the riprap for gradations having a D_{50} greater than 2 feet.
b. A filter cloth shall not extend into the channel beyond the riprap layer; it should be wrapped around the bottom of the toe material and back into the riprap layer.
c. Adequate overlaps must be provided between individual fabric sheets.
d. A sufficient number of folds should be included during placement to minimize tension and stretching under settlement.
e. Securing pins with washers are required at two (2) to five (5) foot intervals along the midpoint of overlaps.
f. Proper stone placement on the filter requires beginning at the toe and proceeding upslope. Dropping stone from heights greater than two (2) feet is not permitted.

TABLE 4-1: ALLOWABLE VELOCITIES FOR ERODIBLE CHANNELS

MATERIAL	VELOCITY (ft/sec)
Fine Sand (noncolloidal)	2.5
Sandy Loam (noncolloidal)	2.5
Silt Loam (noncolloidal)	3.0
Alluvial Silts (noncolloidal)	3.5
Alluvial Silts (colloidal)	5.0
Ordinary Firm Loam	3.5
Volcanic Ash	2.5
Fine Gravel	5.0
Stiff Clay (very colloidal)	5.0
Graded, Loam to Cobbles (noncolloidal)	5.0
Graded, Silts to Cobbles (colloidal)	5.0
Coarse gravel (noncolloidal)	6.0
Cobbles and Shingles	5.5
Shales and Hard Pans	6.0

For sinuous channels, multiply permissible velocity by:
0.95 for slightly sinuous;
0.90 for moderately sinuous; and
0.80 for highly sinuous

Source: USDOT, FHWA, 1961 and 1983

TABLE 4-2: ALLOWABLE VELOCITIES - GRASS LINED CHANNELS

VEGETATION TYPE	SLOPE[1] (%)	MAXIMUM VELOCITY[2] (ft/s) Erosion Resistant Soils	MAXIMUM VELOCITY[2] (ft/s) Easily Eroded Soils
Bermuda grass	0-5	7.0	5.0
	5-10	7.0	5.0
	> 10	6.0	4.0
Kent. bluegrass	0-5	7.0	5.0
Buffalo grass	5-10	6.0	4.0
	> 10	5.0	3.0
Native grass mixture	0-5[1]	5.0	4.0
	5-10	4.0	3.0
Lespedeza Kudzu, Alfalfa	0-5[3]	3.5	2.5
Annuals[4]	0-5	3.5	2.5
Sod		4.0	4.0
Lapped sod		5.5	5.5

[1] Do not use on slopes steeper than ten percent (10%) except for side slope in combination channel.
[2] Use velocities exceeding 5 ft/s only where good stand can be established and maintained.
[3] Do not use on slopes steeper than five percent (5%) except for side-slope in combination channel.
[4] Annuals - used on mild slopes or as temporary protection until permanent covers are established.

Source: USDA, TP-61, 1954

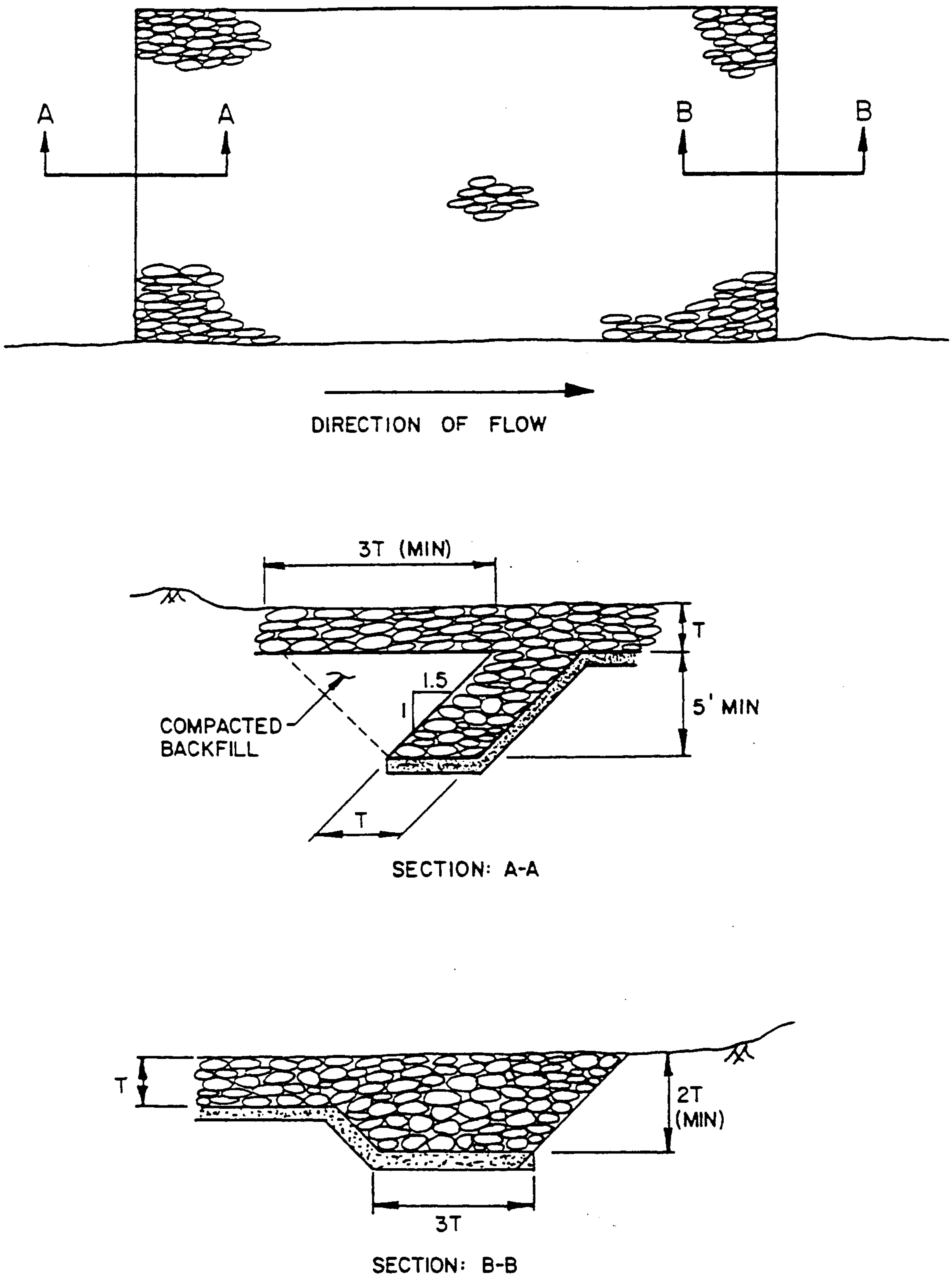


Figure 4-2: Typical Riprap Installation - Plan and Flank Details

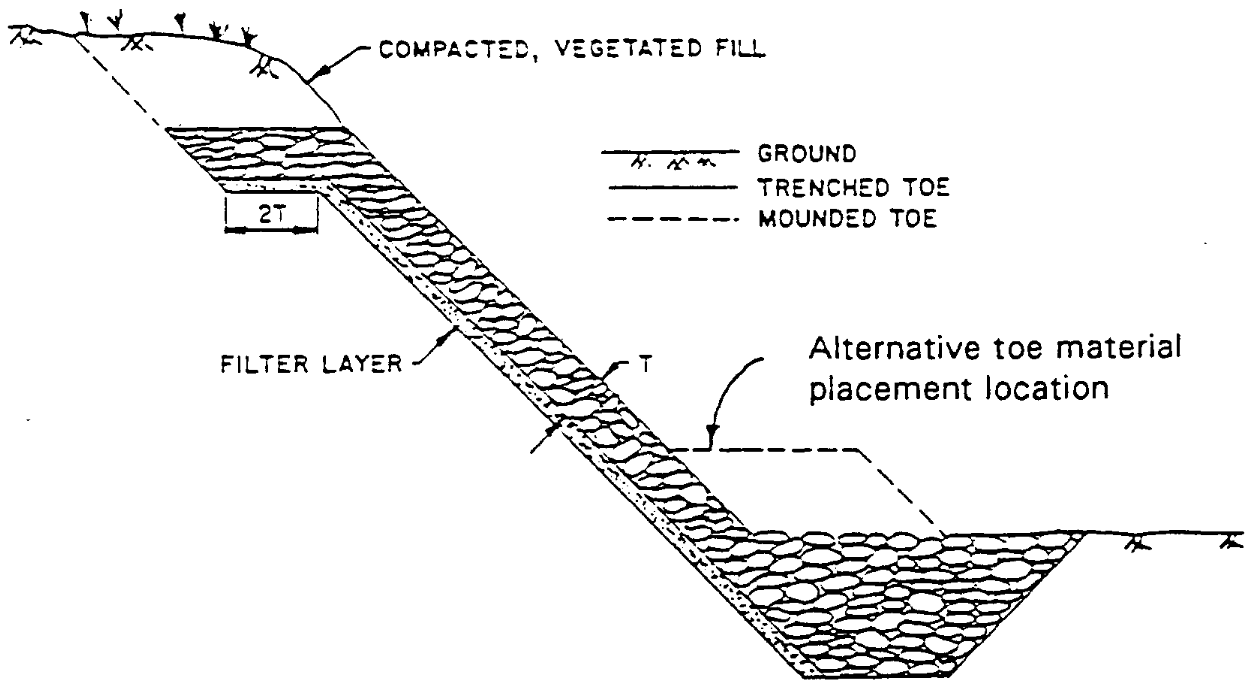


Figure 4-3a: Typical Riprap Installation - Bank Protection Only

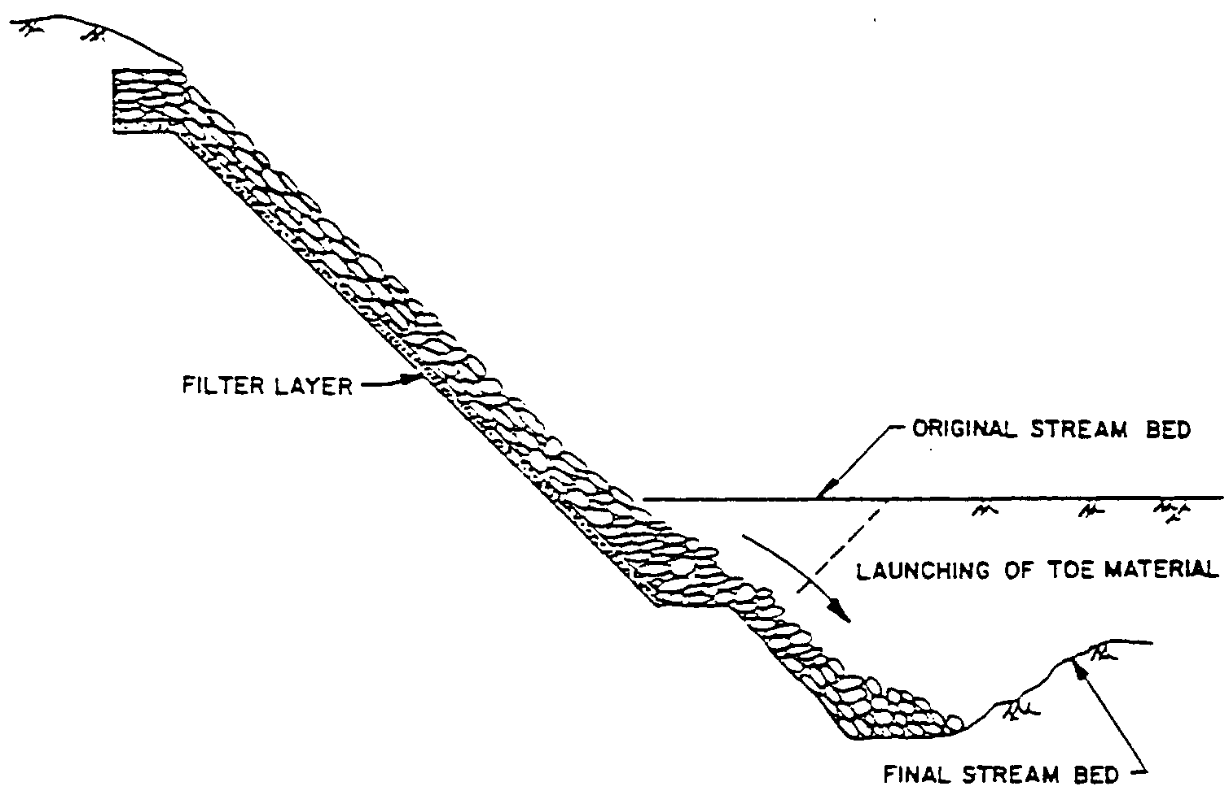


Figure 4-3b: Launching of Riprap Toe Material

PROJECT ____

DESCRIPTION ____

____

Prepared by/Date: : ____/____ ________

Checked by/Date: ____/____ ________

Sheet _____ of _____

GRANULAR FILTER:

LAYER	DESCRIPTION	D_{15} (ft)	D_{85} (ft)	RATIO	$\frac{D_{15}\text{COARSE}}{D_{85}\text{FINE}}$	< 5 <	$\frac{D_{15}\text{COARSE}}{D_{15}\text{FINE}}$	< 40

SUMMARY:

LAYER DESCRIPTION	D_{15}	D_{85}	THICKNESS

FABRIC FILTER:

PHYSICAL PROPERTIES CLASS: ____________________

HYDRAULIC PROPERTIES

PIPING RESISTANCE < 50% PASSING #200 AOS < 0.6 mm

< 50% PASSING #200 AOS < 0.3 mm

PERMEABILITY SOIL PERMEABILITY < FABRIC PERMEABILITY

SELECTED FABRIC FILTER SPECIFICATIONS: ______________________________

Figure 4-4: Granular Filter Design Form

4.4.4. Riprap Design Procedure

The following procedure applies to riprap placement in both natural and prismatic channels and is based on tractive force theory. Additional design information, for discharges less than 50 ft^3/s, can be found in Federal Highway Administration (FHWA), HEC No. 15, *Design of Stable Channels with Flexible Linings,* 1988. Velocity is the primary design parameter and the following assumptions and limitations apply to this procedure:

- minimum riprap thickness equal to d_{100},
- the value of d_{85}/d_{15} less than 4.6,
- Froude Number of less than 1.2,
- side slopes no steeper than 2H:1V,
- a safety factor of 1.2,
- a maximum velocity less than 18 ft/s.
- typical riprap angle of repose of 39 degrees

If significant turbulence is caused by boundary irregularities (e.g., near obstructions or structures) this procedure is not applicable.

4.4.4.1. Riprap Sizing

The median size (D_{50}) of the riprap can be determined using Equation 4.4 below:

$$\mathbf{D_{50} = (0.001\ V^3) / (d^{0.5}\ K^{1.5})} \qquad \textbf{(4.4)}$$

where:
- D_{50} = size of stone, ft
- V = average velocity in the channel, ft/s
- d = the average depth flow in the channel, ft
- K = side slope correction factor found in Equation 4.5

$$\mathbf{K = [1 - (\sin^2 \phi / 0.396)]^{0.5}} \qquad \textbf{(4.5)}$$

where: ϕ = the bank angle with horizontal (e.g., 3H:1V has ϕ = 18.43 degrees)

Equation 4.4 is based on a stone specific weight of 165 lbs/ft^3. If the rock density is significantly different from 165 lbs/ft^3 the D_{50} size found in Equation 4.4 should be multiplied by a specific weight correction factor found in Equation 4.6.

$$\mathbf{C_w = 2.12 / (S_{sg} - 1)^{1.5}} \qquad \textbf{(4.6)}$$

where: S_{sg} = the specific gravity of the rock riprap

For design situations other than a uniform straight channel, the D_{50} size from Equation 4.4 should be multiplied by a Stability Correction Factor found in Equation 4.7.

$$\mathbf{C_{SF} = (SF / 1.2)^{1.5}} \quad \textbf{(4.7)}$$

where: SF = the stability factor to be applied as defined in Table 4-1 and is defined as the ratio of the average tractive force exerted by the flow field and the riprap material's critical shear stress.

NOTE: The correction factors computed in Equations 4.6 and 4.7 are multiplied together to form a single correction factor, C, to be applied to riprap size computed in Equation 4.4.

TABLE 4-3: GUIDELINES FOR SELECTION OF STABILITY FACTORS

Channel Condition	Stability Factor (SF)
Uniform flow; straight or mildly curving reach [curve radius/channel top width, (R_c/T) > 30]; little impact from wave action or debris; little uncertainty in design.	1.0 - 1.2
Gradually varied flow; moderate bend curvature (30 > R_c/T > 10); moderate impact from waves or debris; moderate uncertainty in design parameters.	1.3 - 1.6
Approaching rapidly varied flow; sharp bend curvature (10 > R_c/T); significant impact from waves or debris; high flow turbulence; significant uncertainty in design parameters.	1.61 - 2.0

4.4.5. Grouted Riprap

Grouted riprap provides advantages similar to concrete linings and can be used in applications where high flow velocities and tractive forces could pull away the rock in a typical dumped riprap section. Grouted riprap can also be used on pipe outlets, stilling basins below drop structures, or spillways.

Common problems associated with grouted riprap are settlement, failure to achieve complete penetration between the rocks, seepage, and undermining from runoff around and under the grouted riprap system.

Grouted riprap is considered a rigid structural lining comprised of a blanket of rock riprap that is held together by concrete grout injected into the voids between the rocks. For the grouted riprap to properly function as a monolithic structure, the grout must extend the full thickness of the riprap blanket, instead of being placed or poured on top of the riprap blanket.

Weep holes or subsurface drains may be required to allow for reduction in lift forces and hydrostatic pressure buildup behind the riprap layer.

Subgrade conditions must be considered as with concrete lined channels. If the grouted riprap is intended for channel bank protection only, adequate key-ins and toe protection, below the estimated scour depth, shall be provided as depicted in Figure 4-5.

As the riprap layer is placed, a key-in or cut off trench shall be excavated around the rock section at the top of the slope and the upstream and downstream edges as illustrated in Figure 4-5.

4.4.5.1. Rock Material and Size

Rock used for grouted riprap should meet the same properties and specification as Section 4.4.3, however graded riprap shall not be used for grouting since the smaller rock in a well graded riprap mix will take up the void spaces intended for the grout. To accomplish this, rock smaller than the specified D_{50} is removed. Riprap smaller than twelve (12) inches should not be grouted. Riprap classification and gradation criteria are given in Table 4-4.

The median rock size shall not exceed 0.67 times the blanket thickness and the largest size rock shall not exceed the blanket thickness.

The minimum riprap blanket thickness shall be one (1) foot for channel velocities of 0.0 - 7.0 feet/second and two (2) feet for velocities from 7.0 - 15.0 feet/second.

TABLE 4-4: GROUTED RIPRAP CLASSIFICATION AND GRADATION

Riprap Designation	% Smaller Than Given Size By Weight	Intermediate Rock Dimension (inches)
Type MG	70 - 100	21
	50 - 70	18
	0 - 5	12
Type HG	100	30
	50 - 70	24
	0 - 5	18
Type VHG	100	42
	50 - 70	33
	0 - 5	24

4.4.5.2. Grout Requirements

Grout for riprap installation on public structures shall be in accordance with MAG Section 220.5 and the following criteria:

1. Air entrainment of 5 - 7%,
2. Compressive strength of not less than 3000 psi in 28 days,
3. Slump of 5 to 7 inches, and
4. 1.5 lbs./cy of fibermesh, or equivalent, shall be used to control shrinkage/cracking.
5. Type II cement. Maximum of 25% fly ash may be substituted for Portland cement.
6. Aggregate shall be comprised of 70% natural sand (fines) and 30% 3/8-inch rock (coarse).

4.4.5.3. Grouted Riprap Placement Requirements

Placement of grouted riprap for public structures shall be in accordance with the following criteria:

1. Final placement of riprap shall be approved by Stormwater Manager prior to placement of grout.

2. Fines and smaller gradation rock shall be removed from the rock blanket to ensure better penetration by the grout.

3. The riprap shall be clean of all dirt and deleterious materials and sprayed with clean water immediately prior to grouting.

4. A low pressure (< 10 psi) grout pump with a two (2) inch diameter nozzle shall be used.

5. Grout must be placed by injection methods using a pencil vibrator and grout penetration must extend the full thickness of the rock blanket. Pneumatically placed grout is not permitted. The grout mix and placement procedures must be controlled to achieve the specified thickness, penetration, and grade of the grout layer.

6. The finished level of the grout shall be 1/4 - 1/3 the rock size below the top of the rock. After the grout has been places and vibrated, excess grout should be removed from the exposed rock. A broom finish should be provided for the grout.

7. The finished surface shall be sealed with a clear liquid membrane curing compound per American Society for Testing and Materials (ASTM) C-309.

4.4.6. Concrete Lined Channels

Concrete lined channels are discouraged in residential and/or recreational areas, however, they may become necessary in cases with limited right-of-way, high velociies, and supercritical flow.

Common problems experienced with concrete lined channels are bedding/subgrade failure, liner failures due to seepage or high groundwater, and sedimentation. Lack of proper maintenance can result in vegetation growth through the concrete liner.

Supercritical flow is typically associated with concrete lined channels. Therefore, supercritical channels should not be designed with any curvature or bends and there shall be no reduction in cross sectional area at bridges/culverts or any other obstructions in the flow path. Lateral storm drains entering a concrete lined channel cannot protrude beyond the channel bank, into the channel flow area. The maximum allowable velocity in any concrete channel shall not exceed eighteen (18) feet per second.

Low flow or trickle channels are recommended on the bottom of all concrete lined channels. Low flow channels should be designed to convey the 2-year discharge at a minimum.

4.4.6.1. Concrete Lining Criteria

Side slopes shall be no steeper than 1H:1V unless designed to act as a structurally reinforced retaining wall designed to withstand soil and groundwater forces, or surcharging.

All concrete lining shall be designed to withstand anticipated hydrodynamic and hydrostatic forces. Minimum thickness shall be no less than eight (8) inches for supercritical channels and no less than six (6) inches for subcritical channels. The top of the concrete lining shall be adequately keyed into the channel banks as depicted in Figure 4-6.

Concrete channels shall be constructed of continuously reinforced concrete without transverse joints. The reinforcing shall be continuous both longitudinally and laterally. Expansion or contraction joints shall only be installed where new concrete lining is connected to a rigid structure or to existing concrete lining which is not continuously reinforced.

Longitudinal joints, where required, shall be constructed on the side-walls at least one (1) foot vertically above the channel invert. Construction joints are required for all cold joints and where the lining thickness changes. Reinforcement shall be continuous through the joint. All joints shall be designed to prevent differential movement by utilizing a staggered or step joint.

Reinforcement steel shall be a minimum grade 40 epoxy coated deformed bars. The ratio of steel area to concrete cross-sectional area shall be greater than 0.006. The ratio of transverse steel area to concrete cross-sectional area shall be greater than 0.003. Reinforcing steel shall be placed at he center of the section with a minimum clear cover of three (3) inches adjacent to the ground.

Additional steel shall be provided as needed to meet retaining wall structural requirements.

Earthwork for concrete lined channels shall be compacted to a minimum of ninety-five percent (95%) of maximum density, as determined by ASTM D-698 (Standard Proctor), for the following areas:

a. The twelve (12) inches of subgrade immediately beneath concrete lining (bottom and sides).
b. Top twelve (12) inches of earth surface within ten (10) feet of concrete channel lip.
c. Top twelve (12) inches of maintenance road subgrade.
d. All fill material.

A minimum of six (6) inches of aggregate base course granular bedding is required under channel bottom and side slopes.

Longitudinal underdrains may be required when temporary or permanent high water tables are experienced. Underdrains shall be free draining, consist of six (6) inch minimum perforated pipe in gravel or coarse sand, flap valves at the outlets, and shall daylight at check drops when applicable. Weepholes should be limited when possible, but are necessary on vertical wall sections to relieve hydrostatic pressure.

Side ditches along the tops of the channel should be utilized to intercept sheet flow and convey it to chutes or dip inlets.

Safety requirements for concrete lined channels shall consist of the following measures at a minimum:

a. A six (6) foot high chain link or comparable fence is required to prevent access wherever the 100-year channel concrete depth exceeds three (3) feet or on concrete channels with vertical walls exceeding thirty (30) inches in height. Appropriate numbers of gates and maintenance roads (12 ft. width), shall be placed and staggered where fencing is required on both sides of the channel.
b. Ladder-type steps shall be installed not more than 400 feet apart on alternating sides of the channel. The bottom rung or step shall be placed one (1) foot above the channel bottom.

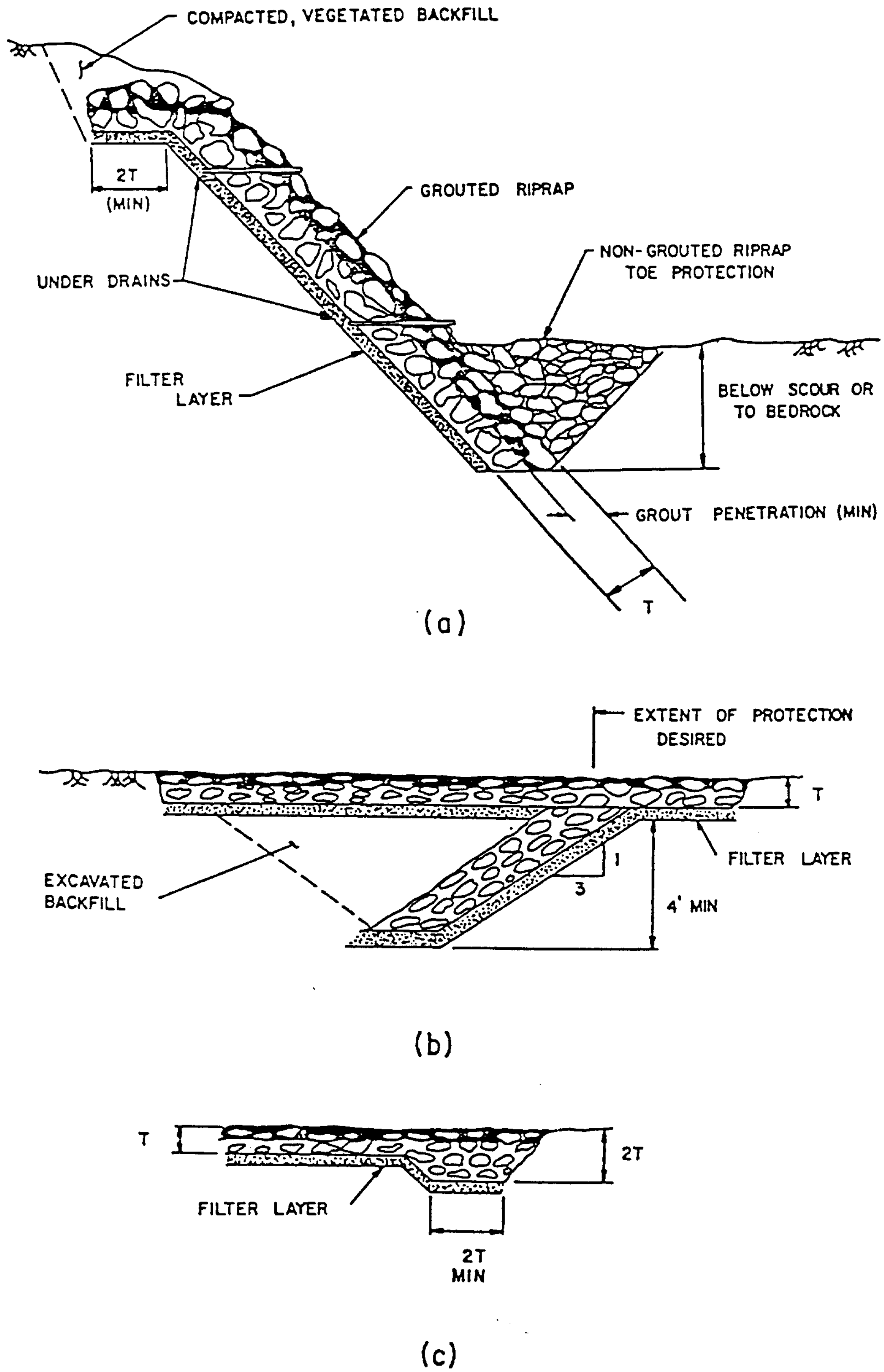


Figure 4-5: Grouted Riprap Sections

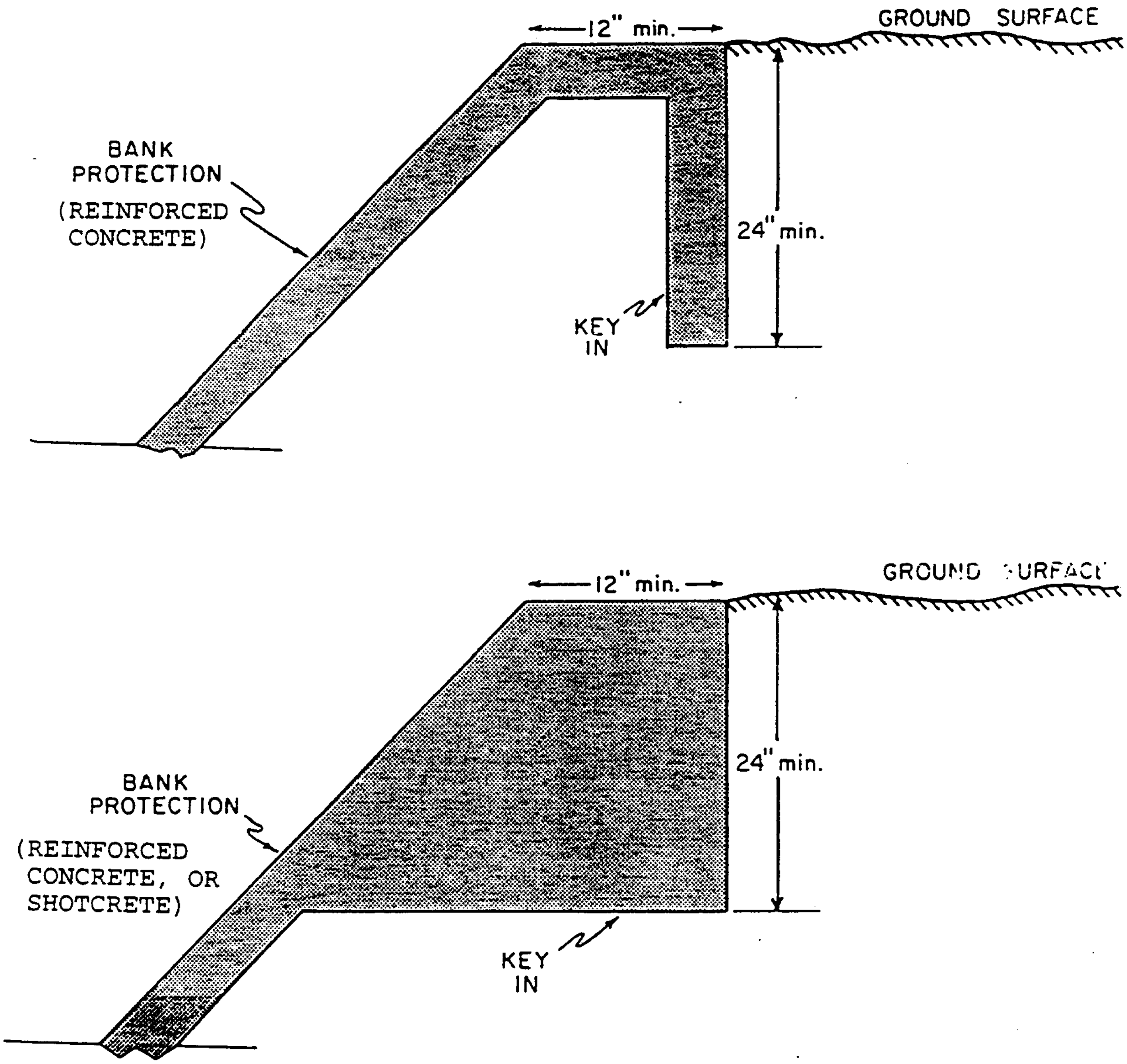


Figure 4-6: Concrete Lined Channels: Typical Key-Ins
Source: Simmons, Li and Associates, 1988

4.5. MANNING'S ROUGHNESS COEFFICIENTS

The Manning's 'n' value is an important variable in open channel design. Variations to the roughness coefficient significantly affect discharge, depth of flow, and velocity estimates. Care and sound engineering judgement must be exercised in the selection process. Factors which should be considered in the selection process are:

1. The physical roughness of the bottom and sides of the channel
2. The value of 'n' depends on the height, density, type of vegetation, and how the vegetation affects the flow through the channel. The 'n' value may increase in the Spring/Summer and diminish in Fall/Winter.
3. Channel shape variations, such as abrupt changes in channel cross sections or alternating small and large cross-sections, will require somewhat larger 'n' values than normal.
4. A significant increase in the 'n' value is possible if severe meandering occurs in the channel alignment. Meandering becomes more important when frequent changes in the direction of the curve occur with relatively small radii of curvature.
5. Active channel erosion or sedimentation will tend to increase the 'n' value, since these processes may cause variations in the shape of the channel. The potential for future erosion or sedimentation in the channel should also be considered.
6. Obstructions, such as log jams or deposits of debris, will increase the 'n' value. The level of this increase will depend on the number, type, and size of the obstruction(s).
7. To be conservative, it is better to use a higher resistance for capacity calculations and a lower resistance for stability calculations. Sensitivity studies may be important.
8. Due to floodplain vegetation, the 'n' value may vary with the depth of flow.

All of the above factors should be considered with respect to the type of channel, degree of maintenance, seasonal requirements, etc. as a basis for making a determination of an appropriate design 'n' value. The probable condition of the channel when the design event will occur should be considered. Values representative of a newly constructed channel are rarely appropriate as a basis for design capacity calculations.

For roughness coefficients for composite channels, refer to Chow, *Open Channel Hydraulics,* 1959.

Recommended Manning's 'n' values for use in open channel design and analysis are shown in Tables 4-5 and 4-6. For more information refer to the *Guide For Selecting Mannings Roughness Coefficients For Natural Channels and Floodplains,* FHWA-TS-84-204, 1984 or Chow, 1959.

TABLE 4-5: MANNING'S ROUGHNESS COEFFICIENTS (n)

ARTIFICIAL CHANNELS

Category	Lining Type	FLOW DEPTH 0-0.5 ft	0.5-2.0 ft	> 2.0 ft
Rigid	Concrete	0.015	0.013	0.013
	Grouted Rip-rap	0.040	0.030	0.028
	Stone Masonry	0.042	0.032	0.030
	Soil Cement	0.025	0.022	0.020
	Asphaltic Concrete	0.018	0.016	0.016
Unlined	Bare Soil	0.023	0.020	0.020
	Rock Cut	0.045	0.035	0.025
Temporary[1]	Woven Paper Net	0.016	0.015	0.015
	Jute Net	0.028	0.022	0.019
	Fiberglass Roving	0.028	0.022	0.019
	Straw with Net	0.065	0.033	0.025
	Curled Wood May	0.066	0.035	0.028
	Synthetic Mat	0.036	0.025	0.021
Gravel Riprap	1-inch D_{50}	0.044	0.033	0.030
	2-inch D_{50}	0.066	0.041	0.034
Rock Riprap	6-inch D_{50}	0.104	0.069	0.035
	12-inch D_{50}	-------	0.078	0.040

Values listed are representative values for their respective depth ranges. Manning's roughness coefficients vary with flow depth.

[1]Some "temporary" linings become permanent when buried.

Source: USDOT, HEC-15, 1988.

TABLE 4-6: UNIFORM FLOW VALUES OF ROUGHNESS COEFFICIENT (n)

TYPE OF CHANNEL AND DESCRIPTION	MINIMUM	NORMAL	MAXIMUM
EXCAVATED OR DREDGED			
Earth, straight, uniform	0.016	0.018	0.020
Clean, recently completed	0.018	0.022	0.025
Clean, after weathering	0.022	0.025	0.030
Gravel, uniform section, clean	0.022	0.027	0.033
Earth, winding and sluggish			
No vegetation	0.023	0.025	0.030
Grass, some weeds	0.025	0.030	0.033
Dense weeds/plants in deep channels	0.030	0.035	0.040
Earth bottom, rubble sides	0.025	0.030	0.035
Stony bottom, weedy sides	0.025	0.035	0.045
Cobble bottom, clean sides	0.030	0.040	0.050
Dragline excavated or dredged			
No vegetation	0.025	0.028	0.033
Light brush on banks	0.035	0.050	0.060
Rock cuts			
Smooth and uniform	0.025	0.035	0.040
Jagged and irregular	0.035	0.040	0.050
Unmaintained channels, weeds/brush uncut			
Dense weeds, high as flow depth	0.050	0.080	0.120
Clean bottom, brush on sides	0.040	0.050	0.080
Same, highest stage of flow	0.045	0.070	0.110
Dense brush, high stage	0.080	0.100	0.140
NATURAL STREAMS			
Minor streams (top width @ flood stage < 100')			
Streams on Plain			
Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
Same, more stones and weeds	0.030	0.035	0.040
Clean, winding, some pools and shoals	0.033	0.040	0.045
Same, some weeds and stones	0.035	0.045	0.050
Same, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
Very weedy reaches, deep pools, or floodways w/heavy stands of timber and underbrush	0.075	0.100	0.150

Table 4-6 continued.

Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
Bottom: gravel, cobbles, few boulders	0.030	0.040	0.050
Bottom: cobbles W/large boulders	0.040	0.050	0.070
Floodplains			
Pasture, no brush			
Short grass	0.025	0.030	0.035
High grass	0.030	0.035	0.050
Brush			
Scattered brush, heavy weeds	0.035	0.050	0.070
Light brush, trees, in winter	0.035	0.050	0.060
Light brush, trees, in summer	0.040	0.060	0.080
Medium to dense brush, in winter	0.045	0.070	0.110
Medium to dense brush, in summer	0.070	0.100	0.160
Trees			
Dense willows, summer, straight	0.110	0.150	0.200
Cleared land, tree stumps, no sprouts	0.030	0.040	0.050
Same, heavy growth of sprouts	0.050	0.060	0.080
Heavy stand of timber, few down trees, flood stage below branches	0.080	0.100	0.120
Same, flood stage above branches	0.100	0.120	0.160
Major streams (top width > 100').			
Regular section w/no boulders or brush	0.025	-------	0.060
Irregular and rough section	0.035	-------	0.100

Source: Chow, V.T., 1959

CHAPTER 5: CULVERTS AND BRIDGES

The purpose of this chapter is to present policies and criteria for the design and construction of roadway culverts. The culvert design procedures presented in this chapter are based on FHWA Hydraulic Design Series No. 5 (HDS-5), *Hydraulic Design of Highway Culverts,* 1985. HDS-5 design procedures should be used to expedite the review and approval of culvert designs.

Culverts are typically used to convey stormwater through an embankment or may serve as the primary outlet for detention facilities. Culverts are typically aligned with natural washes, watercourses, or open channels which serve as the primary outfall for local and regional drainageways. The design of culverts is influenced by purpose, hydraulic efficiency, site topography, effects on adjacent property, and cost. Bridges are typically not practical or warranted for most roadway crossings in the Flagstaff area, however bridge design criteria is presented in this chapter.

5.1. POLICIES

a. All culverts shall be hydraulically designed to determine whether inlet and outlet control conditions govern for the design storm discharge(s).

b. Culverts shall be used where:

   1. bridges are not hydraulically required,
   2. debris and ice are tolerable, and
   3. they are more economical than a bridge structure.

   Bridges shall be used:

   1. where culverts cannot be used,
   2. where more economical than a culvert,
   3. to satisfy land use requirements,
   4. to mitigate environmental harm caused by a culvert and fill,
   5. to avoid floodway encroachments, and
   6. to accommodate ice and large debris.

   Bridge structures will require special design and review considerations as approved by the Stormwater Manager.

c. Culverts shall be located and designed to present a minimum hazard to traffic, persons, and property. Projecting ends shall not be permitted for culverts intended to become public.

d. Survey and resource information should include topographic features, channel characteristics, aquatic life, riparian habitat, highwater information, existing structures, and other related site specific information, as applicable.

e. Roadway culverts shall be designed to accommodate debris or proper provisions shall be made for debris maintenance. Where practicable, some means shall be provided for personnel and equipment access to facilitate maintenance.

f. Material selection shall include consideration of service life, hydraulic efficiency, and maintenance and shall not be made using first cost as the sole criteria.

g. Low water or at-grade, dip crossings of FEMA designated/mapped washes or other riverines are not permitted for public or private roadways which serve as the primary access to a development or single family residence.

h. Culvert or bridge crossings of FEMA designated/mapped washes shall be analyzed with HEC-2 Water Surface Profiles or HEC-RAS. It must be demonstrated and certified by the engineer that there will be no significant increases on the base flood elevations(s) and/or limits upstream or downstream of the crossing.

i. Erosion control measures may be required in all construction plans, as determined by the Stormwater Manager. These measures shall include, but not be limited to silt boxes, straw silt barriers, filter cloth, temporary silt fences, and check dams to minimize pollution of streams and damages to wetland areas.

5.2. CULVERT DESIGN CRITERIA

Performance curves shall be developed for all public culverts for evaluating hydraulic capacity versus various headwater depths, outlet velocities, and scour depths.

The culvert length and slope shall be chosen to approximate existing topography, and to the degree practicable, the culvert shall be aligned with the channel bottom and the skew angle of the watercourse.

Multiple barrel culvert crossings should fit onto the natural channel cross-section with minimal widening of the channel so as to avoid conveyance loss and sediment deposition. Multiple barrel culverts shall be avoided where the approach velocity is high, particularly supercritical, to avoid adverse hydraulic jump effects.

The minimum velocity through a culvert should be three (3) feet per second when the culvert is flowing partially full. The maximum allowable velocity for corrugated metal pipe should not exceed twenty (20) feet per second.

5.2.1. Design Storm Criteria

Roadway culverts shall be designed to convey the following frequency flows without roadway overtopping:

Local Streets……………………….. 25-year

Collector/Arterial Streets…………… 50-year

FEMA Mapped Watercourses ………100-year

The weir flow depth for the 100-year design storm shall be limited to 0.5 feet or less for roadways serving as secondary access to a development or subdivision.

Public roadway culvert or bridge crossings of riverine areas with a contributing watershed greater than 1/4 square miles should be designed to convey the 100-year peak discharge with no roadway overtopping. Bridge crossings for all other types of roadway classifications and crossings shall be designed for the 50-year storm event at a minimum.

Private streets or accesses crossing public drainage ways or FEMA designated floodplains shall be designed to convey the channel design discharge or the 100-year discharge whichever is greater.

Public facilities such as underpasses, depressed roadways, etc. where no overflow relief is available shall be designed for the 50-year event.

At-grade, dip crossings which serve as the primary access to a development are not permitted. Secondary access crossings of broad shallow washes, where installation of a culvert or bridge is impractical, may be dipped to allow the entire flow to cross the roadway. The pavement section must have a one-way cross slope in the direction of flow without raised curbs or medians. Cut-off walls and aprons will be required on both the upstream and downstream edges on the pavement to prevent headcutting and erosion.

5.2.2. Skewed Culverts

Culverts should be designed to closely conform to the natural stream in alignment, slope, and width, whenever possible. As a result, culverts are often skewed with respect to the roadway centerline. The culvert skew angle shall not exceed forty-five (45) degrees as measured from a line perpendicular to the roadway centerline.

Headwalls and wingwalls are required on all skewed culverts. Alterations of the normal inlet configuration are generally required due to culvert skew. Inlets are often skewed with respect to the centerline of the culvert so that the headwalls are parallel to the roadway centerline to avoid warping of the roadway embankment fill. Figure 5-1 illustrates typical skewed headwall and wingwall configurations.

5.2.3. Headwater and Tailwater Conditions

5.2.3.1. Allowable Headwater

The allowable headwater (HW) is the depth of water that can be ponded at the upstream end of a culvert and shall be limited to one or more of the following parameters:

1. No damage or inundation to upstream property;
2. No greater than the low point in the road grade;
3. Equal to the elevation where flow diverts around the culvert; or
4. For culverts with cross-sectional area equal to or less than 30 sq. ft. - $HW/D \leq 1.5$; and for culverts with cross-sectional area greater than 30 sq. ft. - $HW/D \leq 1.2$.

A submerged inlet occurs when the headwater is greater than 1.2 times the pipe diameter (D). The ponded headwater elevation should be evaluated and plotted on the public improvement plans to ensure adequate drainage easement is reserved and flooding of adjacent property or buildings will not occur for the design storm.

5.2.3.2. Tailwater Relationship

A submerged outlet occurs where the tailwater elevation is higher than the crown of the culvert. For design purposes, downstream conditions which result in high tailwater should be avoided if possible.

A free outlet has a tailwater equal to or lower than critical depth. For culverts having free outlets, lowering the tailwater has no effect on the discharge or the backwater profile upstream of the tailwater.

The tailwater depth may be computed as the highest value of the following criteria:

1. The normal depth in the downstream channel for subcritical flow regimes;
2. The critical depth and equivalent hydraulic grade line if the outlet is operating with a free outfall;
3. The high water elevation that has the same design frequency if outlet is a detention basin, channel, or other body of water; or
4. The quantity $(d_c + D)/2$; where d_c = critical depth (ft.), and D = pipe diameter (ft.)

If culverts are placed in close proximity to each other, the headwater of the downstream culvert may influence the tailwater depth of the upstream culvert. It may be necessary to use the headwater elevation of a nearby downstream culvert if it is greater than the tailwater depth of the upstream culvert or normal depth in the channel.

5.2.4. Inlet and Outlet Treatment

5.2.4.1. Inlets

Culvert inlets shall match the geometry of the roadway embankment whenever possible. In order to reduce headwater elevations, improve inlet capacity, and prevent damage to roadway embankments and culvert ends, the use of concrete headwalls and wingwalls, side or slope tapered inlets, and beveled edges may be required.

Commercial end sections are permitted on culverts 36" in diameter or less, if the design headwater is acceptable and other embankment protection measures are used. Concrete headwalls are required on all public culverts greater than 36" in diameter. Typical headwall and wingwall configurations are illustrated in Figure 5-1. Details of beveled edge, side tapered, and slope tapered inlets are illustrated in Figure 5-2a, 5-2b, and 5-2c respectively. Entrance loss coefficients (K_e) used for culvert design and analysis are given in Table 5-1.

Aprons may be required if high headwater depths are encountered or the approach velocity in the channel will cause scour. Aprons shall extend at least one pipe diameter beyond the pipe invert and shall not protrude above the normal channel or streambed elevation.

Concrete headwalls and aprons shall be constructed in accordance with MAG Standard Details and/or Arizona Department of Transportation, Highways Division, Structures Section Standard Drawings.

Metal pipe culverts with a span or diameter greater than 48 inches shall have a cut-off wall where the outlet velocity and downstream bed material may result in local scour.

Improved inlets shall only be considered for culverts which operate under inlet control. Information and design procedures for improved inlets can be found in FHWA, *Structural Design Manual For Improved Inlets And Culverts,* 1983 or HDS-5.

Inlet riprap protection for commercial end sections shall extend around and over the top of the inlet a minimum of two (2) feet. Roadways designed for overtopping will require additional slope protection for the upstream and downstream spillway sections.

The use of drop inlets on culverts is discouraged due to problems associated with sediment and plugging of grate inlets. Drop inlets shall only be used when the upstream channel sides and bottom are bank protected and significant sediment loads are not anticipated. It should be noted that HDS-5 inlet control nomographs do not apply to drop inlets since the additional losses caused by the plunging flow are not accounted for. In this case, the analysis for a storm drain inlet should be used.

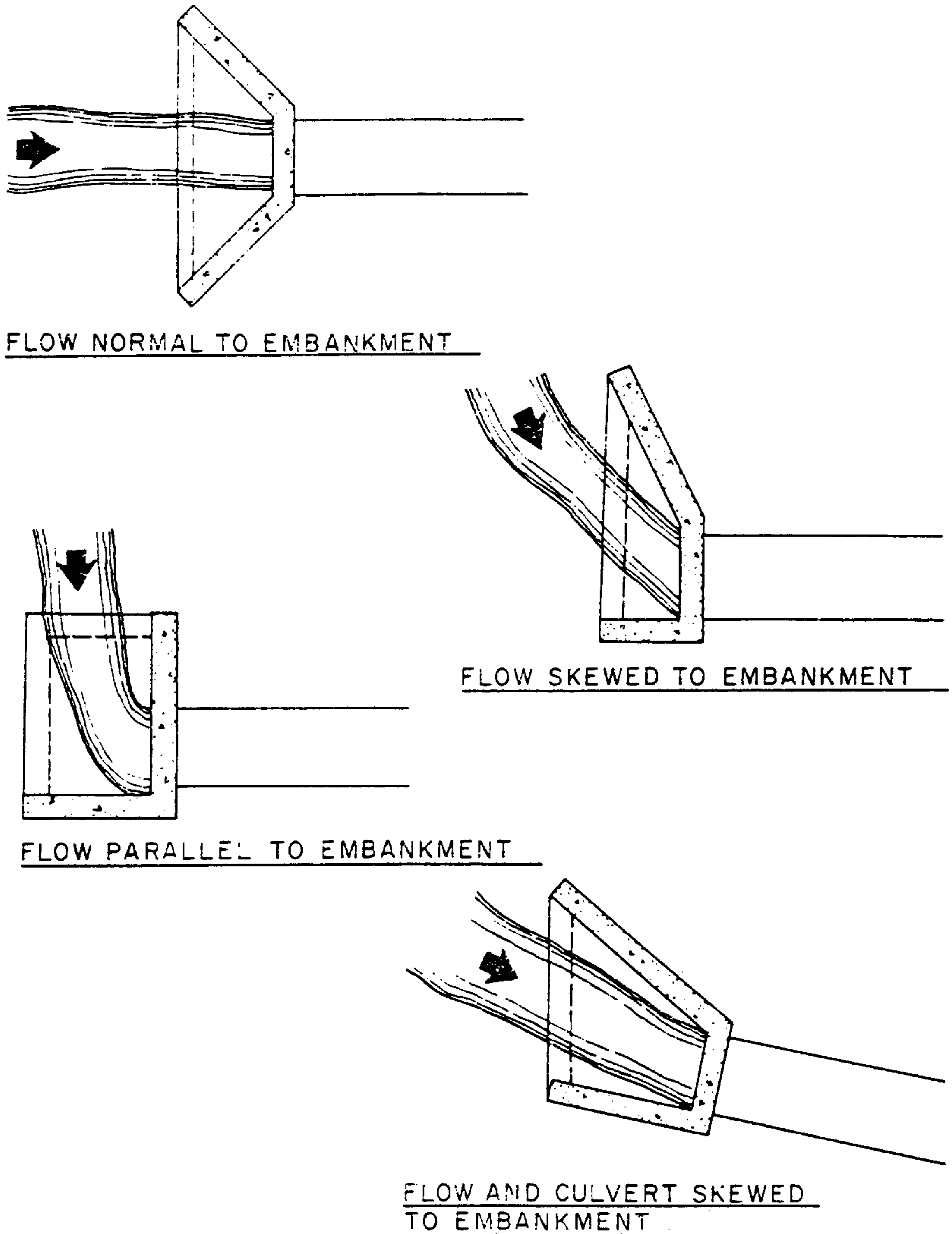


Figure 5-1: Headwall and Wingwall Configurations

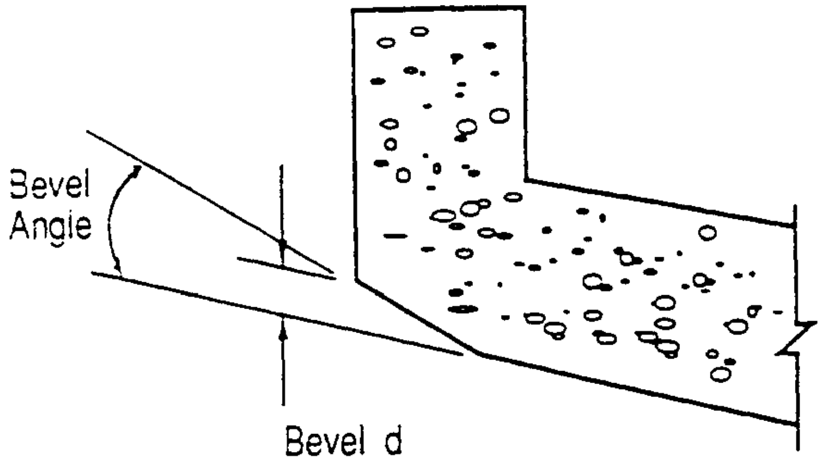


Figure 5-2a: Beveled Edge Detail

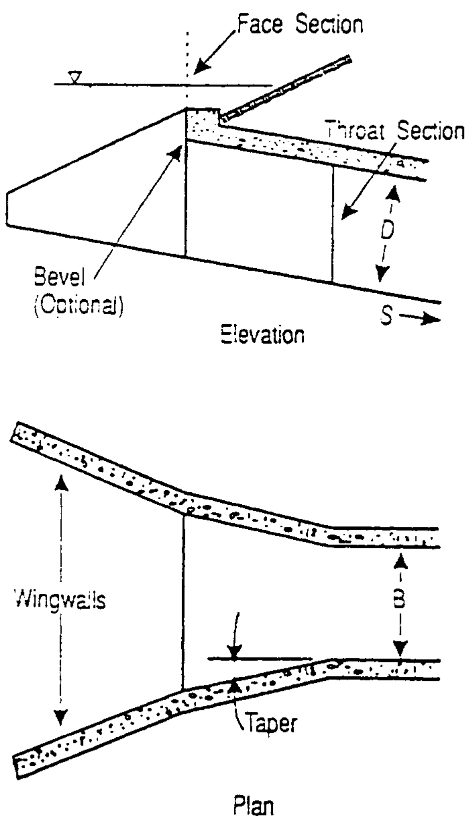


Figure 5-2b: Side Tapered Inlet

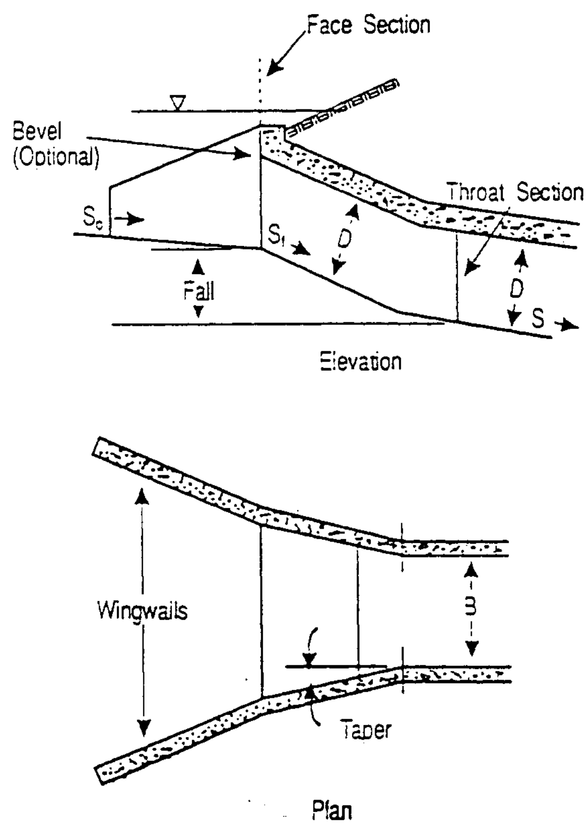


Figure 5-2c: Slope Tapered Inlet

Source: FHWA, HDS-5, 1985

TABLE 5-1: ENTRANCE LOSS COEFFICIENTS

TYPE OF STRUCTURE AND ENTRANCE DESIGN	COEFFICIENT, K_e
Pipe, Concrete:	
Mitered to conform to fill slope	0.7
Projecting from fill, sq. cut end	0.5
Projecting from fill, socket end (grooved end)	0.2
Headwall or headwall w/wingwalls	
Socket end	0.2
Square edged	0.5
Rounded (radius = 1/12D)	0.2
Manufactured end section conforming to slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side or slope tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal:	
Projecting from fill (no headwall)	0.9
Mitered to conform to fill slope, paved or unpaved slope	0.7
Headwall or headwall w/wingwalls, square edged	0.5
Manufactured end section conforming to slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side or slope tapered inlet	0.2
Box, Reinforced Concrete:	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius 1/12 barrel dimension, or beveled top edge	0.2
Wingwalls at 30° to 75° to barrel;	
Square edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side or slope tapered inlet	0.2

Source: USDOT, FHWA, HDS-5, 1985

5.2.4.2. Buoyancy Protection

Buoyancy or flotation is the phenomenon by which a culvert fails due to uplift forces. The buoyancy force is produced by high head on the outside of the culvert and a region of low pressure on the inside of the inlet caused by flow separation. This results in a bending moment exerted on the end of the culvert. This typically occurs on culverts under high head, on steep slopes, with projecting or mitered inlets, with debris blockage, with damaged inlets, or on large culvert skews. Large projecting or mitered corrugated metal pipe culverts are most susceptible to buoyancy. Concrete headwalls, slope paving or other means of anchoring shall be considered for all flexible culverts, particularly when embankment fill heights are less than 1.5 times the pipe diameter or fill slopes are flatter than 1H:1V. Rigid pipes susceptible to separation at the joints should be protected with tie bars. It is recommended to limit the headwater depth to 1.5 times the culvert height to help prevent buoyancy.

5.2.4.3. Outlets

The design of culvert outlets shall be based on structural considerations to protect the culvert outlet and embankment from local scour, bank sloughing, and general channel degradation, rather than hydraulic efficiency. Headwalls are required on all public culvert outlets (see *Inlets* section).

The maximum velocity at the culvert outlet shall be consistent with the velocity in the natural channel. Appropriate protection shall be considered when outlet velocities are between 4.0 and 15 ft/sec. Recommended outlet treatments are shown in Table 5-2.

Outlet velocities greater than 15 ft/sec shall be mitigated with energy dissipation as outlined in Chapter 11 of this manual.

TABLE 5-2 OUTLET PROTECTION MEASURES

OUTLET VELOCITY	LEVEL OF PROTECTION
Less than 4 fps	No protection required
4 to 10 fps	Dumped rock riprap apron
10 to 15 fps	Wire tied rock riprap[1]
Greater than 15 fps	Energy Dissipator

[1] It is recommended to use a concrete energy dissipator or increase the culvert size if velocities are greater than ten (10) feet/second.

5.2.4.4. Riprap Apron Design Procedure

Typical riprap aprons, as illustrated in Figure 5-3, are suitable for use with outlet velocities not exceeding ten (10) feet per second.

The following procedure is based on criteria developed for the U.S. Environmental Protection Agency, 1976. The riprap outlet protection configuration used in this procedure is illustrated in Figure 5-4.

Referring to Figure 5-4, the apron length (L_a) is computed by:

$$\mathbf{L_a = (1.8Q / D^{1.5}) + 7D \qquad \text{for } TW < 0.5D} \tag{5.1}$$

and,

$$\mathbf{L_a = (3Q / D^{1.5}) + 7D \qquad \text{for } TW \geq 0.5D} \tag{5.2}$$

where:
- Q = design discharge, ft/s
- D = maximum inside culvert width, ft
- TW = tailwater depth, ft

Where there is no well defined channel downstream of the culvert apron, the width (W_a) of the downstream end of the apron is computed as follows:

$$\mathbf{W_a = 3D + L_a \qquad \text{for } TW < 0.5D} \tag{5.3}$$

and,

$$\mathbf{W_a = 3D + 0.4L_a \qquad \text{for } TW \geq 0.5D} \tag{5.4}$$

The width of the upstream end of the riprap apron at the culvert outlet should be at least three (3) times the culvert width (D).

The median riprap apron rock size (D_{50}) can be computed by the following equation:

$$\mathbf{D_{50} = [0.02\,(Q)^{4/3}] / [TW\,(D)]} \tag{5.5}$$

5.2.4.5. Safety Considerations

During design and construction, culvert entrances may require safety precautions to protect life, health, traffic, and adjacent property. This may include the use of safety measures such as fencing, handrails, guard rails, warning signs, and safety/trash racks to limit or deter access by the public.

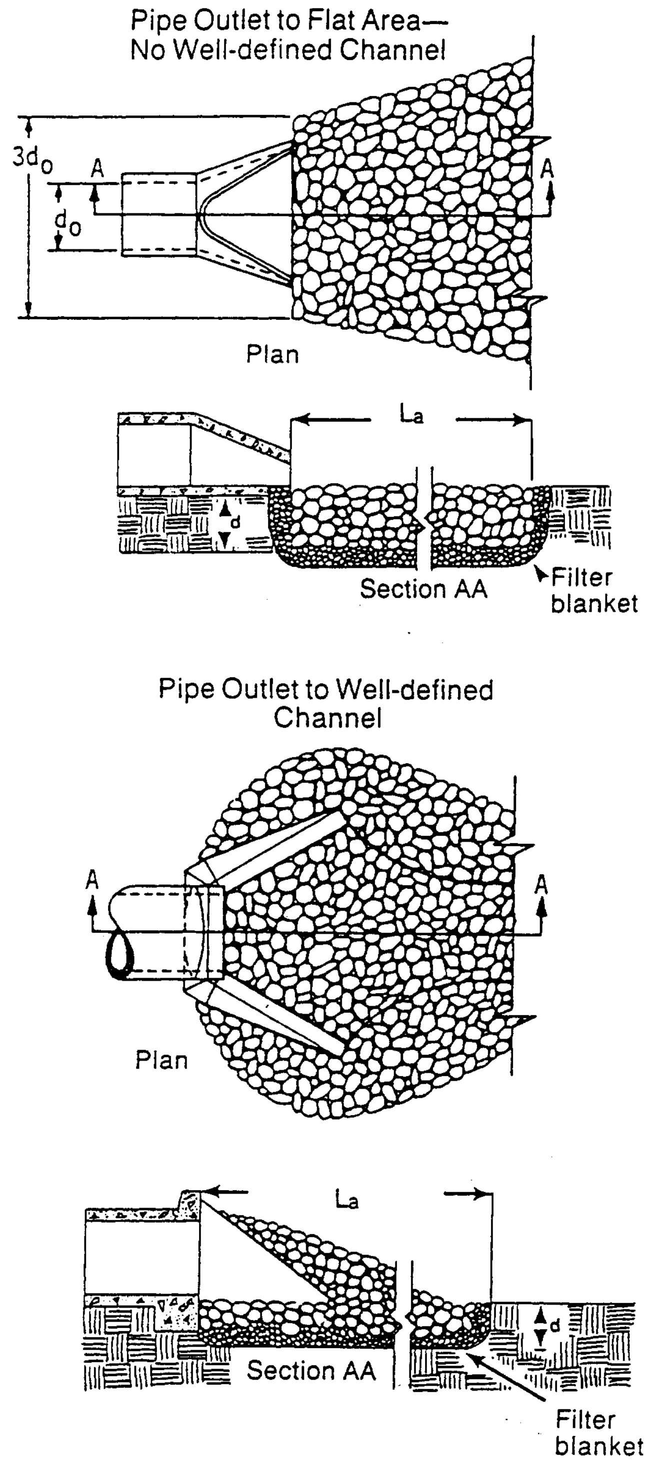


Figure 5-3: Riprap Outlet Protection
Source: North Carolina Erosion and Sediment Control Manual

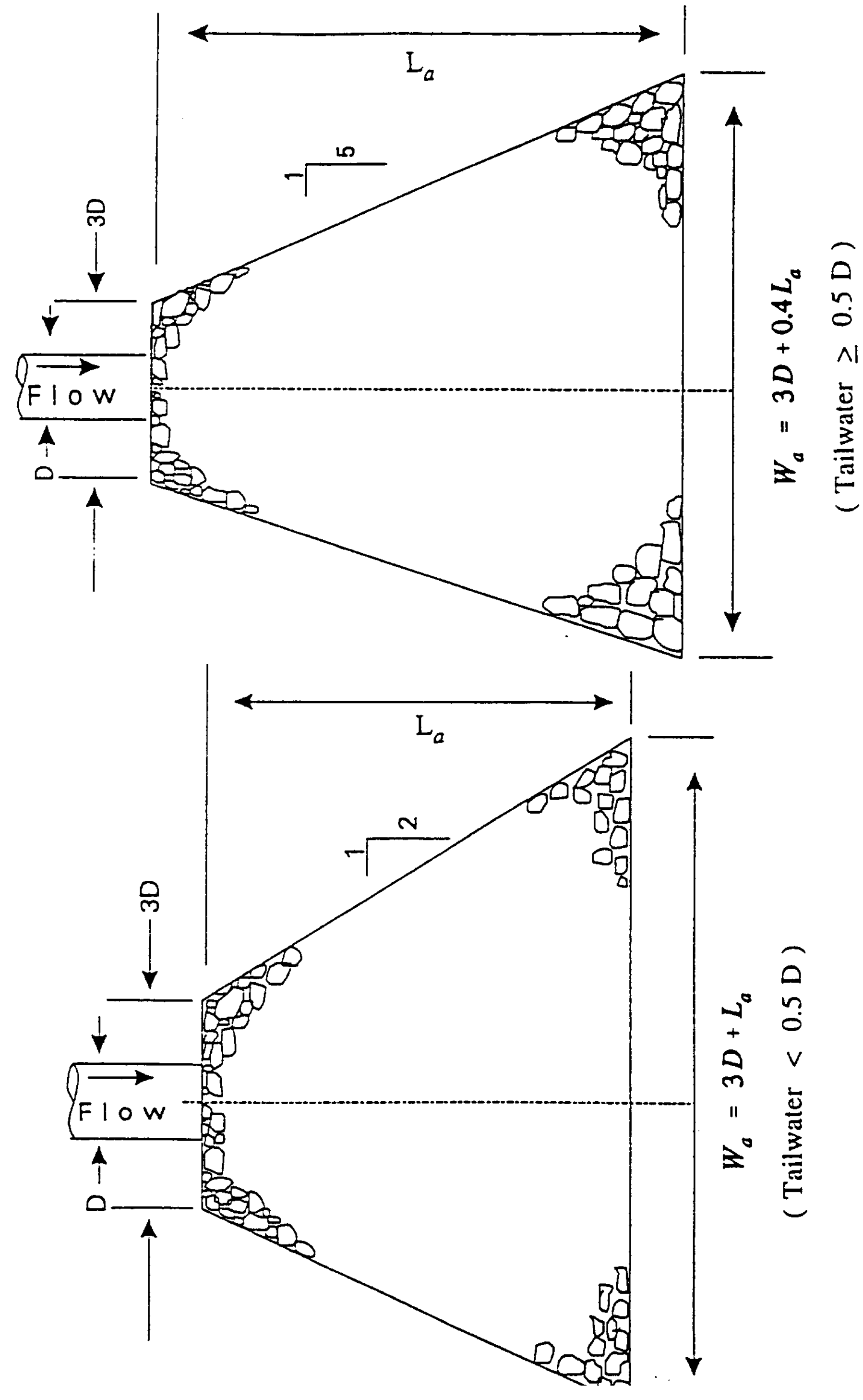


Figure 5-4: Conduit Outlet Protection Detail
Source: U.S. Environmental Protection Agency, 1976

Traffic should be protected from culvert ends as follows:

1. Small culverts (24" in diameter or less) can use a commercial end section or a sloped concrete wingwalls.
2. Culverts greater than 24" in diameter shall receive one of the following treatments:
    a. be extended to the appropriate "clear zone" distance per AASHTO Roadside Design Guide.
    b. Safety treated with a grate if the consequences of clogging and causing a potential flood hazard is less than the hazard of vehicles impacting an unprotected end. If a grate is used, an open area shall be provided between the bars of 1.5 to 3.0 times the area of the culvert entrance.
    c. Shielded with a guard rail if the culvert is very large, cannot be extended, has a channel which cannot be safely traversed by a vehicle, has significant flooding hazard with a grate, or has headwalls which protrude 6" or higher above driving surface within the "clear zone".

5.2.4.6. *<u>Debris Control</u>*

Debris control should be designed using FHWA, Hydraulic Engineering Circular No. 9, *Debris-Control Structures* and shall be considered for the following conditions:

1. Where experience or physical evidence indicates the watercourse will transport a heavy volume of controllable debris, particularly in urbanized areas,
2. For culverts located in mountainous or steep regions,
3. For culverts that are under high fills, and
4. Where maintenance access is limited.

It is recommended, however, that increasing the pipe size be used as an alternative to grates (this does not apply to detention basin outlet pipes).

5.2.5. Culvert Material and Installation

5.2.5.1. *<u>Material Selection</u>*

The material selected for culverts should be based on service life, durability, structural strength, hydraulic efficiency, bedding conditions, abrasion and corrosion resistance, and joint tightness. Acceptable materials for culverts intended to be public are:

- Corrugated Metal Pipe (CMP).
- Helical Corrugated or Spiral Rib Metal Pipe per MAG Section 760.3.
- Rubber Gasket Reinforced Concrete Pipe (RGRCP) - bell or groove and spigot or tongue.

- Reinforced Concrete Box Culvert (RCBC) per Section 5.2.5.2.

All metal pipe culverts shall be a minimum of 14 gauge, Aluminized Steel Type 2 pipe. Thicker gauge pipe may be warranted with increases in fill height per manufacturers recommendations.

Joints on metal pipe culverts, if required, shall be either rivet lap joint construction (annular corrugations) or continuous lock or welded seam (helical corrugations) and wrapped with non-woven geotextile filter fabric or "O" ring gaskets.

5.2.5.2. *Reinforced Concrete Box Culverts*

Reinforced Concrete Box Culverts (RCBC) shall be constructed in accordance with the ADOT Reinforced Concrete Box Culvert Manual (31-019), 1994 as revised; the ADOT Division of Highways Standard Drawings; and the following additional criteria:

1. Precast box sections shall meet ASTM C850.

2. The minimum acceptable RCBC height is three (3) feet for public facilities, however, four (4) feet is preferred due to maintenance concerns.

3. All RCBC shall have inlet cut off walls and an appropriately designed outlet cut-off wall, four (4) feet minimum.

4. Joints shall be smooth, sealed with butyl rubber or asphaltic mastic, and wrapped with a minimum one (1) foot wide nonwoven geotextile fabric.

Non-standard box culvert sizes may require special structural design.

5.2.5.3. *Culvert Sizes and Shape*

Minimum diameter for public roadway culverts shall be eighteen (18) inches or equivalent acrh pipe. Twelve (12) inch diameter pipes are permissible for private residential driveway crossings when adequate cover cannot be maintained.

Selection of minimum pipe size should also account for potential blockage from debris and sediment deposition (this does not apply to detention facility outlet structures).

Circular cross-sections are preferred, however, the use of arch or oval shapes is permitted only if dictated by hydraulic limitations, site characteristics, structural criteria, or environmental concerns.

5.2.5.4. *Cover Requirements*

Both minimum and maximum cover limits must be considered in the design of roadway culverts. Minimum cover limits are established to ensure the culvert's structural stability under live and impact loads. Dead loads become the controlling factor with increases in fill height. Procedures for analyzing loads on buried conduit are outlined in the *Handbook of Steel Drainage and Highway Construction Products* and the *Concrete Pipe Design Manual*, latest editions respectively.

The minimum allowable cover for culverts 18 to 36 inches in diameter shall be one (1) foot from top of pipe to top of subgade or top of finish grade if no subgrade is present. For culverts greater than 36 inches in diameter, minimum cover should be 30% of the culvert diameter, if possible. The top of any culvert should never extend above the roadway subgrade into the roadway street section.

5.3. CULVERT DESIGN PROCEDURE

Culvert design shall be in accordance with the procedures given in the Federal Highway Administration (FHWA) Hydraulic Design Series Number 5 (HDS-5), *Hydraulic Design of Highway Culverts.* The nomographs used in HDS-5 are based on inlet control and are considered to be accurate to within about 10 percent in determining the required inlet control headwater (FHWA 1985). These nomographs were computed assuming a culvert slope of two (2) percent. For different culvert slopes, the nomographs are less accurate because inlet control headwater changes with slope. Computer programs used for culvert design and analysis should be fully HDS-5 compliant. The HY-8 computer program is also acceptable for culverts analysis.

Inlet or outlet control must be determined to properly design a culvert. Because Manning's Equation assumes uniform, steady flow, it should not be used to design culverts due to the rapidly varied flow conditions associated with culvert hydraulics.

The following procedure is based primarily on HDS-5 and uses the Culvert Design Form (the "Form") in Figure 5-5. This form is based on earlier versions published by the FHWA (1965) and the Arizona Department of Transportation (1973).

Step 1. Summarize the design discharge, tailwater height, drainage basin area, stream slope and general channel shape under the heading <u>Hydrologic Data</u> on the Form.

Step 2. Select a preliminary culvert shape, size, material, and entrance type under the heading <u>Culvert Description</u> on the Form. Enter the total design flow (Q) and flow per barrel (Q/N) in Rows 1 and 2 respectively.

Step 3. <u>Evaluate Inlet Control</u>:

a. Using the nomographs on Figures 5-6 through 5-9, locate the culvert size and flow rate on the appropriate scales. Nomagraphs for less common culvert shapes and materials can be found in FHWA HDS-5, HEC-5, HEC-10, or HEC-13 publications.

b. Extending a line from the culvert size through the design flow rate, determine the appropriate headwater/culvert-height (HW/D) based on entrance type from Scales 1, 2, & 3. Enter HW/D into Row 3 on the Form.

c. Multiply HW/D by the culvert height to obtain the required headwater (HW). If the approach velocity is negligible, or if it is intentionally disregarded by the designer, the headwater at the inlet (HW_i) equals the HW computed from the HW/D ratio. If approach velocity is considered, subtract the approach velocity head ($V_i^2 / 2g$). Enter HW_i in Row 4.

d. Evaluate if the inlet should be depressed (FALL) below the streambed in order to obtain the additional hydraulic head required to operate the culvert. Compute FALL using the formula given in Technical Footnote 3 on the Form. Note: When making this determination, the impacts of sedimentation must be considered, otherwise a larger size culvert may be required.

If FALL is negative or equals zero, set FALL equal to zero, and proceed to the next step. When FALL is positive and the culvert is under inlet control, the invert must be depressed below the streambed by this amount. If FALL is acceptable, enter FALL in Row 5 and proceed to the next step, however, if FALL is unacceptably large, select another culvert size and begin again at Step 3a.

e. Compute the invert elevation (EL_{hi}) of the inlet control section with the formula given in Technical Footnote 4. Enter this elevation into Row 6.

Step 4. Evaluate Outlet Control:

a. Determine the tailwater depth (TW) above the outlet invert by either normal depth for the design flow rate or backwater calculations (as appropriate) in the downstream channel and enter this depth in Row 7.

b. From Figures 5-10 through 5-12, determine the critical depth (d_c) of flow in the culvert. For other culvert shapes, refer to HDS-5. Enter d_c into Row 8.

NOTE: If $d_c > 0.9D$, consult the *Handbook of Hydraulics* (Brater and King) for a more accurate d_c determination, if needed, since the curves are truncated where they converge.

c. For tailwater elevations (TW) less than the crown of the culvert, calculate as $(d_c + D) / 2$, where D is the height of the culvert.

d. Determine the depth from the outlet invert to the hydraulic grade line (h_0) by selecting the larger of either TW or $[(d_c + D)/2]$. Enter this value in Row 10.

e. From Table 5-1 in this manual, obtain the appropriate entrance loss coefficient (K_e) for the proposed inlet configuration. Enter into Row 11.

f. Compute the head loss (H) using the formula in Technical Footnote 7 when downstream channel velocity is neglected. See Tables 5-3 and 5-4 for Mannings 'n' values. If the downstream channel velocity is included in the analysis, then use Equation 5.6.

$$\mathbf{H = [(V^2/2g) - (V_d^2/2g)] + [K_e + (29n^2L/R^{1.33})]\ V^2/2g} \qquad \mathbf{(5.6)}$$

where:

H	= head loss through culvert, feet,	
g	= acceleration due to gravity, 32.2 ft/sec^2,	
V	= average velocity of flow in culvert barrel, ft/s	
V_d^2	= channel velocity downstream of the culvert,	ft/s
K_e	= entrance loss coefficient (Table 5-1),	
n	= Manning's roughness coefficient,	
L	= barrel length, feet,	
R	= hydraulic radius of the full culvert barrel, feet,	
R	= A/P	
A	= full cross-sectional area of flow, sq. ft.,	
P	= wetted perimeter of the culvert barrel, feet.	

Enter headloss (H) in Row 12.

If the culvert has bends, junctions, or grates, Technical Footnote 7 or Equation 5.6 will not strictly apply. The engineer should refer to Chapter IV in HDS-5 for appropriate head loss parameters. Nomographs for evaluating headloss under outlet control can also be found in HDS-5.

g. Calculate the required outlet control headwater (EL_{ho}) which is defined in Technical Footnote 8. Enter into Row 13.

Step 5. Compare the headwater elevations computed for both inlet (HW_i) and outlet (EL_{ho}) control. The higher of the two values is designated as the controlling headwater elevation. If the controlling headwater elevation is higher than the design headwater previously established, the potential for using an improved inlet configuration should be considered if the culvert is under inlet control, giving consideration to sedimentation. Refer to Chapter IV in HDS-5 for additional information on improved inlets. However, under outlet control, an improved inlet should not be considered. Instead the engineer should consider increasing the culvert size or adding more barrels.

Step 6. Calculate the outlet velocity:

a. For inlet control, determine the normal depth and velocity in the culvert from Manning's Equation. The velocity at normal depth is assumed to be equal to the outlet velocity (see Figure 5-13a).

b. For outlet control, determine the area of flow at the outlet based on the culvert geometry and the following:

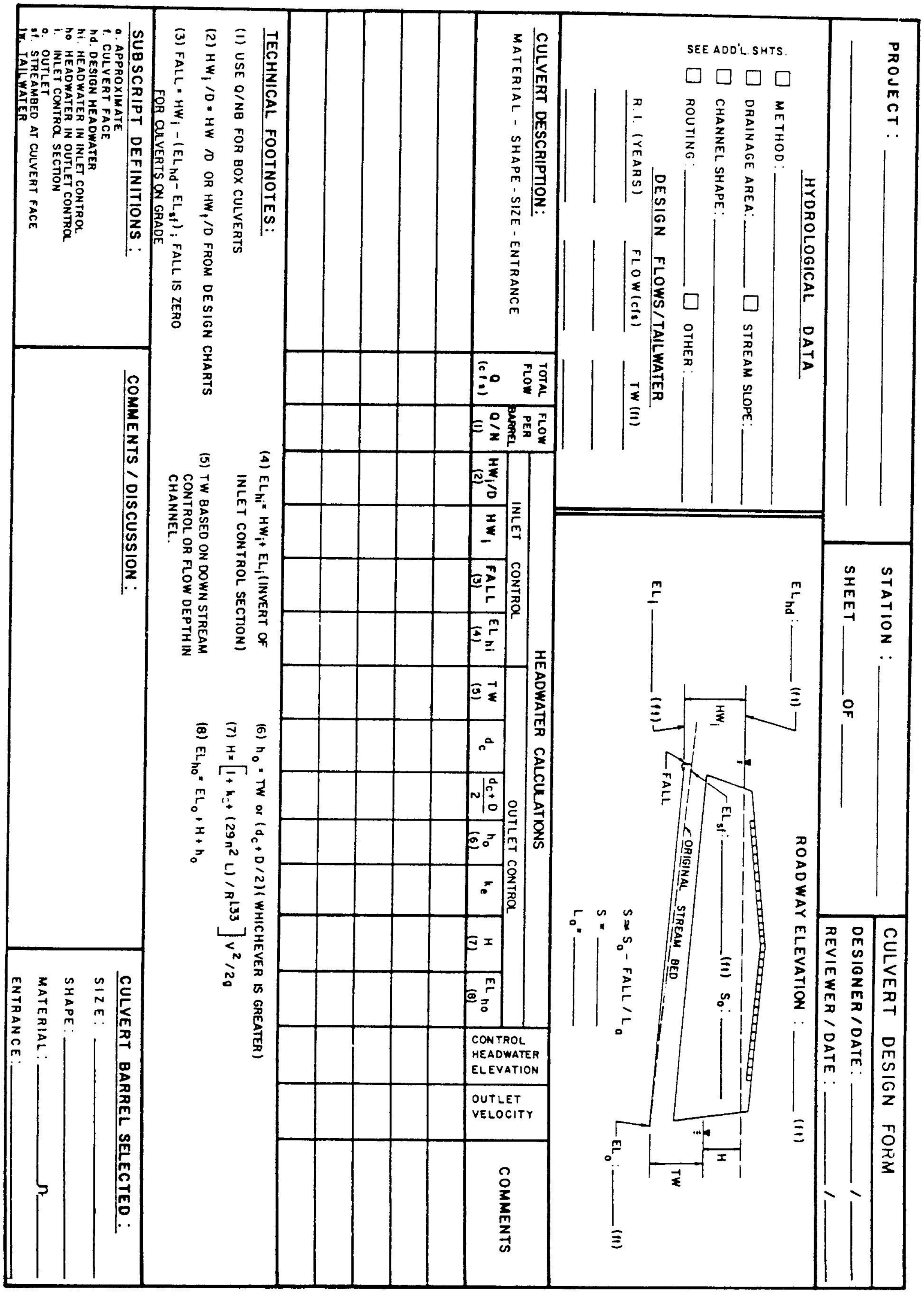
1. Critical depth, if the tailwater is below critical depth,
2. The tailwater depth, if the tailwater is between critical depth and the crown of the culvert outlet, and,
3. The culvert height, if the tailwater is above the crown of the culvert outlet.

Once the flow area at the outlet is determined, the outlet velocity can be calculated by dividing the discharge in the culvert by the computed flow area (see Figure 5-13b).

Compare the culvert outlet velocity with the existing channel velocity to determine if outlet protection measures are required (see Section 5.2.3.2 of this Chapter).

Step 7. Repeat the design process if necessary until an acceptable culvert configuration is determined. Once the culvert geometry is determined, it must fit into the roadway cross section. The culvert must have adequate cover; the culvert length should be close to the roadway right-of-way width; and the headwalls and wingwalls must be properly dimensioned.

If the selected culvert will not fit the site, return to the culvert design process and select another culvert and length. If neither improved inlets or flow routing are to be applied, document the design. An acceptable design should always be accompanied by a performance curve which depicts culvert behavior over a range of discharges.



PROJECT : ______

STATION : ______

SHEET ____ OF ____

CULVERT DESIGN FORM

DESIGNER / DATE : ______ / ______

REVIEWER / DATE : ______ / ______

HYDROLOGICAL DATA

☐ METHOD : ______

☐ DRAINAGE AREA : ______ ☐ STREAM SLOPE : ______

☐ CHANNEL SHAPE : ______

☐ ROUTING : ______ ☐ OTHER : ______

SEE ADD'L. SHTS.

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)

ROADWAY ELEVATION : ______ (ft)

CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS — INLET CONTROL: HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL: TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_i/D FROM DESIGN CHARTS

(3) FALL = $HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ or $(d_c + D/2)$ (WHICHEVER IS GREATER)

(7) $H = \left[1 + k_e + (29n^2 L)/R^{1.33}\right] V^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS :

a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

COMMENTS / DISCUSSION :

CULVERT BARREL SELECTED :

SIZE : ______

SHAPE : ______

MATERIAL : ______

ENTRANCE : ______

Figure 5-5: Culvert Design Form

TABLE 5-3: MANNING'S "n" VALUES FOR CULVERTS

TYPE OF CONDUIT	WALL/JOINT DESCRIPTION	MANNING'S 'n'
Concrete pipe	Good joints, smooth walls	0.013
	Good joints, rough walls	0.014 - 0.016
	Poor joints, rough walls	0.016 - 0.017
	Badly spalled	0.015 - 0.020
Concrete Box	Good joints, smooth finished walls	0.014 - 0.018
	Poor joints, rough, unfinished walls	0.014 - 0.018
Corrugated Metal Pipes	2-2/3 by 1/2 inch corrugations	0.024 - 0.027
and Boxes, Annular or	6 by 1 inch corrugations	0.022 - 0.025
Helical Corrugations	5 by 1 inch corrugations	0.025 - 0.026
	3 by 1 inch corrugations	0.027 - 0.028
	6 by 2 inch corrugations	0.033 - 0.035
	9 by 2-1/2 inch corrugations	0.033 - 0.037
Spiral Rib Metal Pipe	3/4 by 3/4 inch recesses at 12 inch spacing, good joints	0.013
High Density Polyetheylene Pipe (HDPE)		0.013

Source: USDOT, FHWA, 1985

TABLE 5-4: MANNING'S "n" FOR CORRUGATED METAL PIPE

TYPE OF PIPE	UNPAVED
Annular 2.67" X 1/2" (All diameters)	0.024
Helical 1.5" X 1/4":	
8 inch dia.	0.012
10 inch dia.	0.014
Helical 2.67" X 1/2":	
12 inch dia.	0.011
18 inch dia.	0.014
24 inch dia.	0.015
30 inch dia.	0.017
36 inch dia.	0.018
42 inch dia.	0.019
48 inch dia.	0.020
54 inch dia. or greater	0.021
Annular 3" X 1" (all diameters)	0.027
Helical 3" X 1":	
36 inch dia.	0.022
42 inch dia.	0.022
48 inch dia.	0.023
54 inch dia.	0.023
60 inch dia.	0.024
66 inch dia.	0.025
72 inch dia.	0.026
78 inch dia.	0.027
Corrugations 6" X 2":	
60 inch dia.	0.033
72 inch dia.	0.032
120 inch dia.	0.030
180 inch dia.	0.028

Source: AISI, 1994

Note: In general, it is recommended that the annular resistance factors be used for corrugated metal pipes with helical corrugations unless certain specific design criteria are met. These criteria include: the conduit flows full; the conduit is circular in shape; there is no erosion resistant sediment build-up in the conduit; the conduit is greater than 20 diameters long; and the conduit is unlined. In most cases, culverts will not meet all of this criteria. However, charts are provided in "*Hydraulic Flow Resistance Factors for Corrugated Metal Conduits,"* J.M. Norman, FHWA-TS-80-216, 1980. (Source: HDS-5)

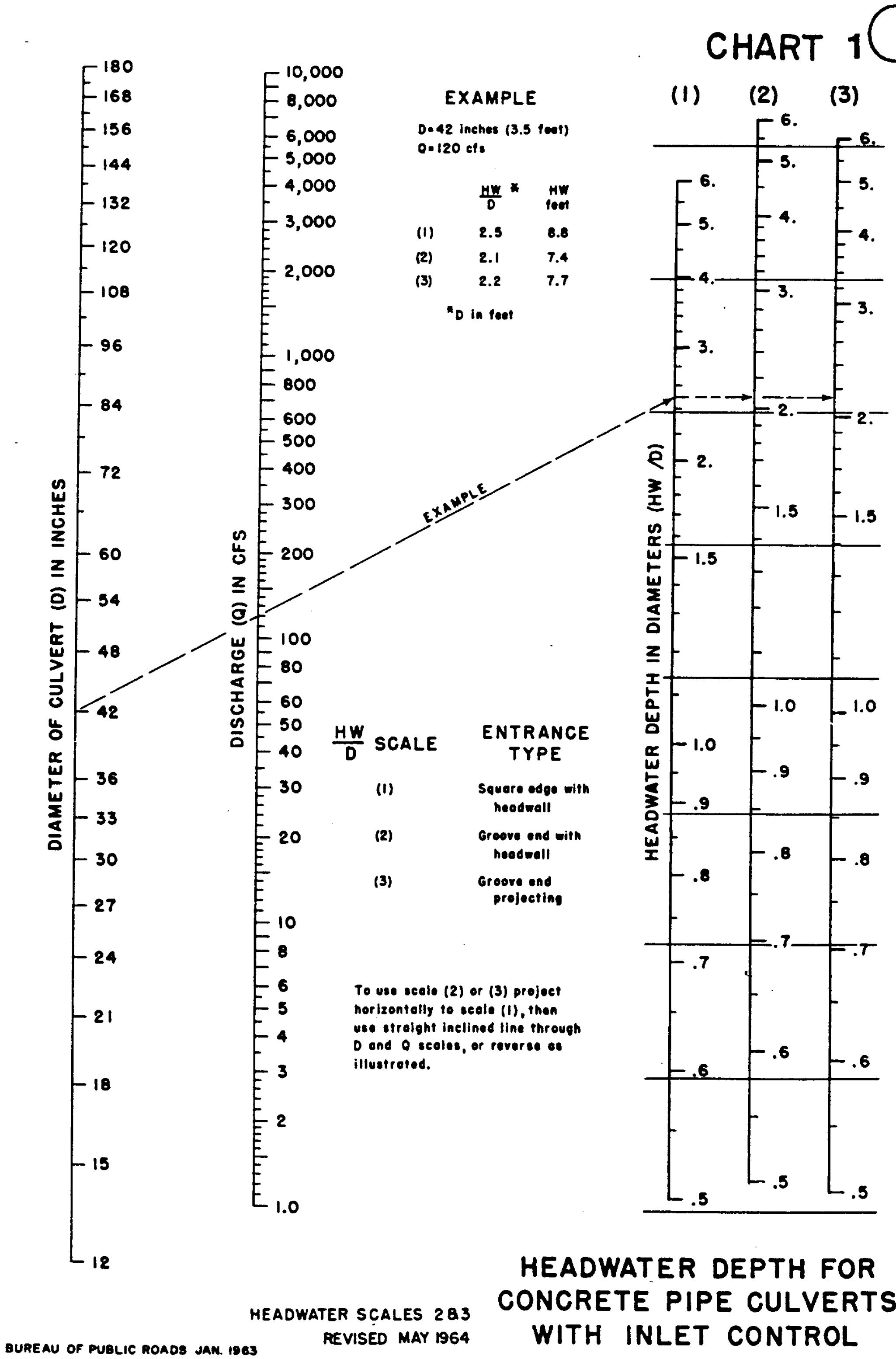


Figure 5-6
Source: FHWA, HDS-5, 1985

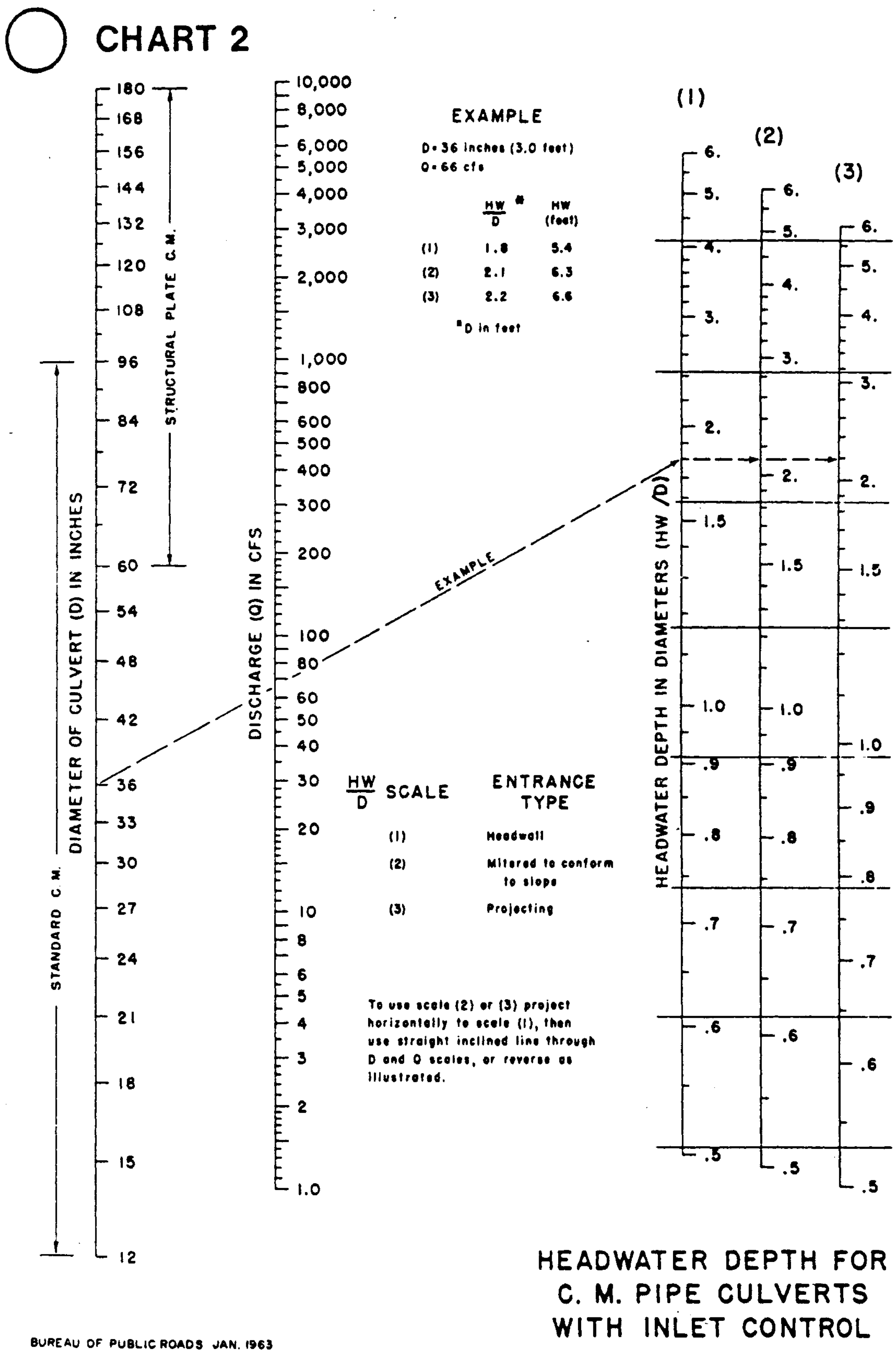


Figure 5-7
Source: FHWA, HDS-5, 1985

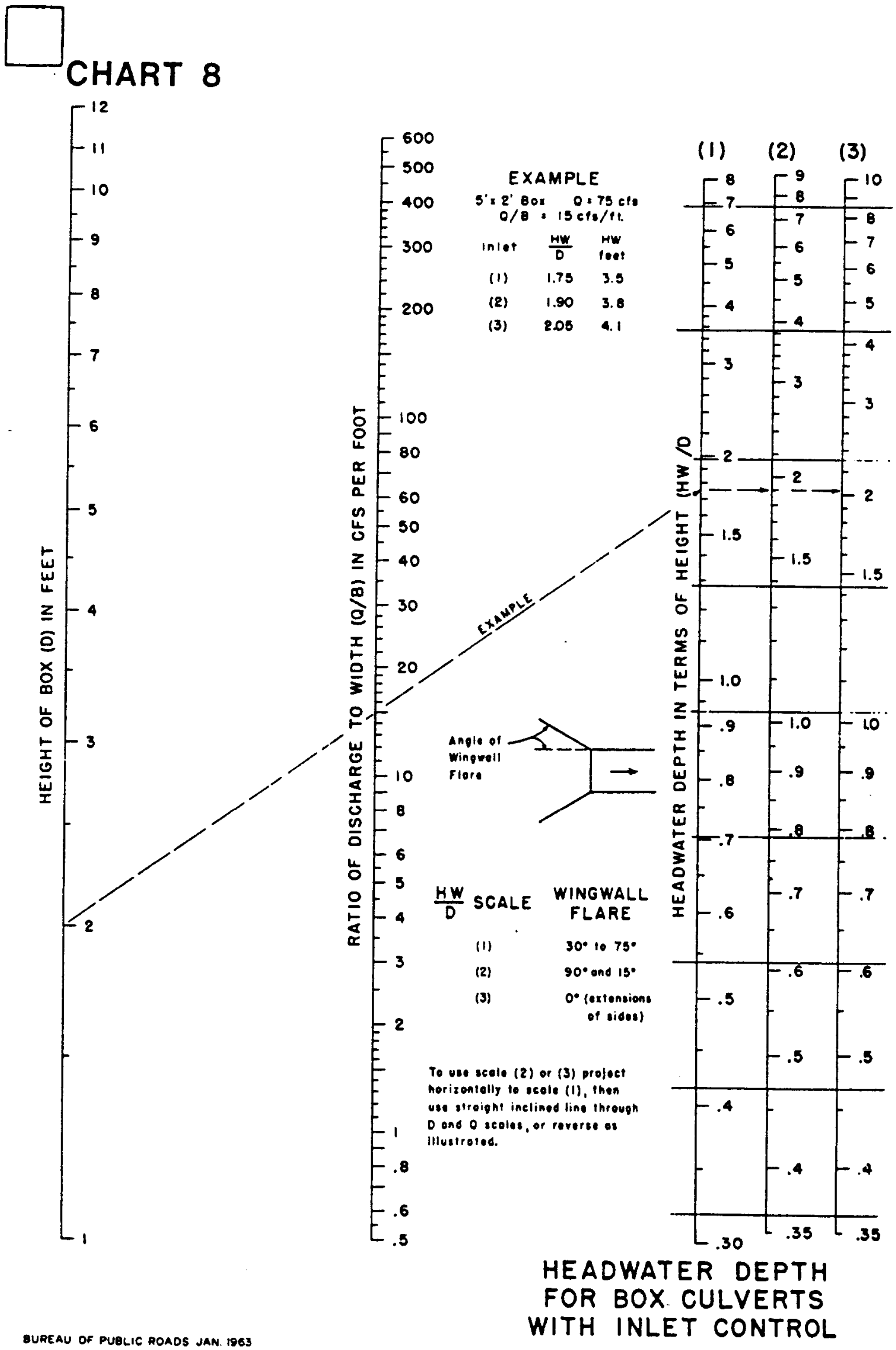


Figure 5-8
Source: FHWA, HDS-5, 1985

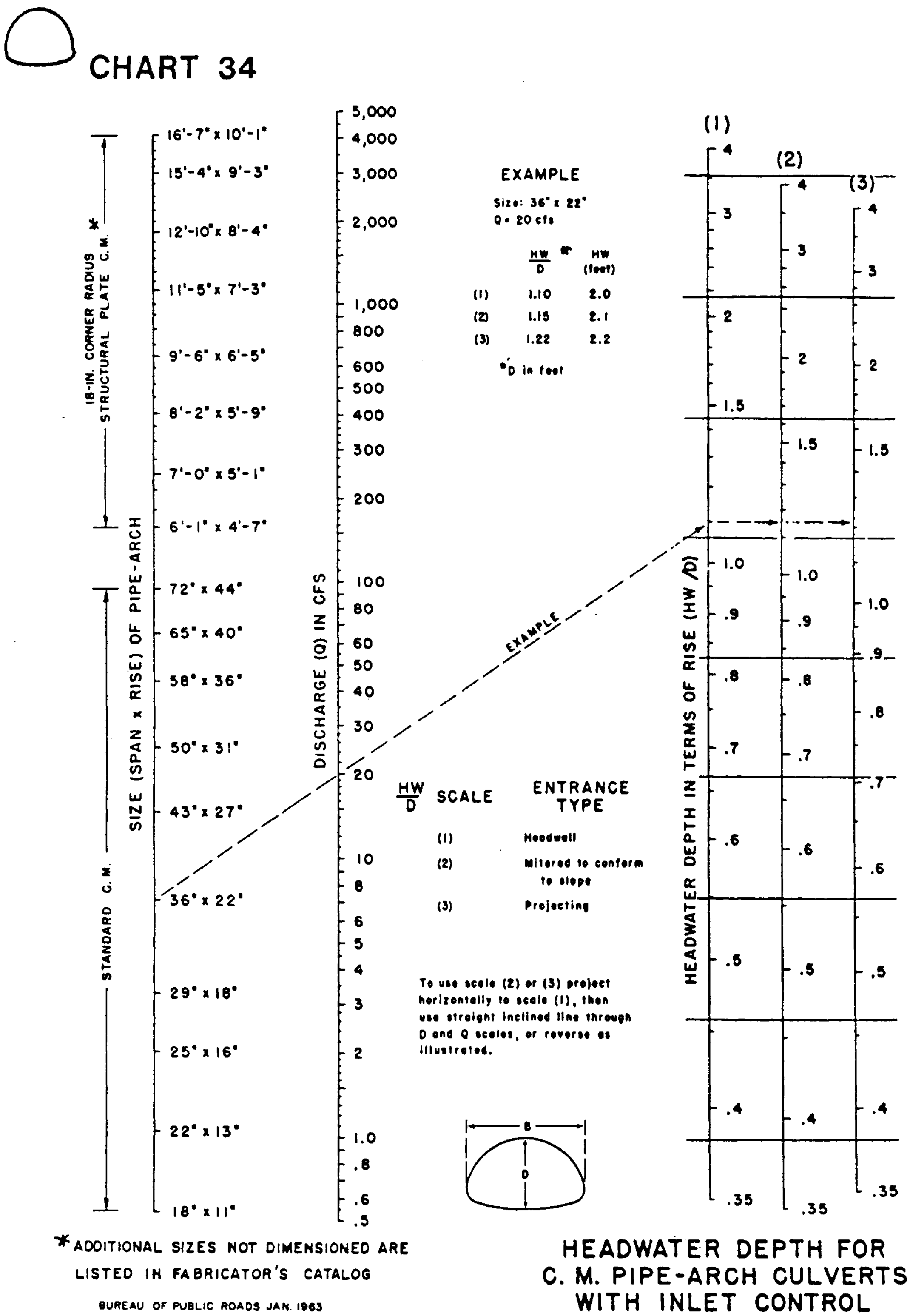


Figure 5-9
Source: FHWA, HDS-5, 1985

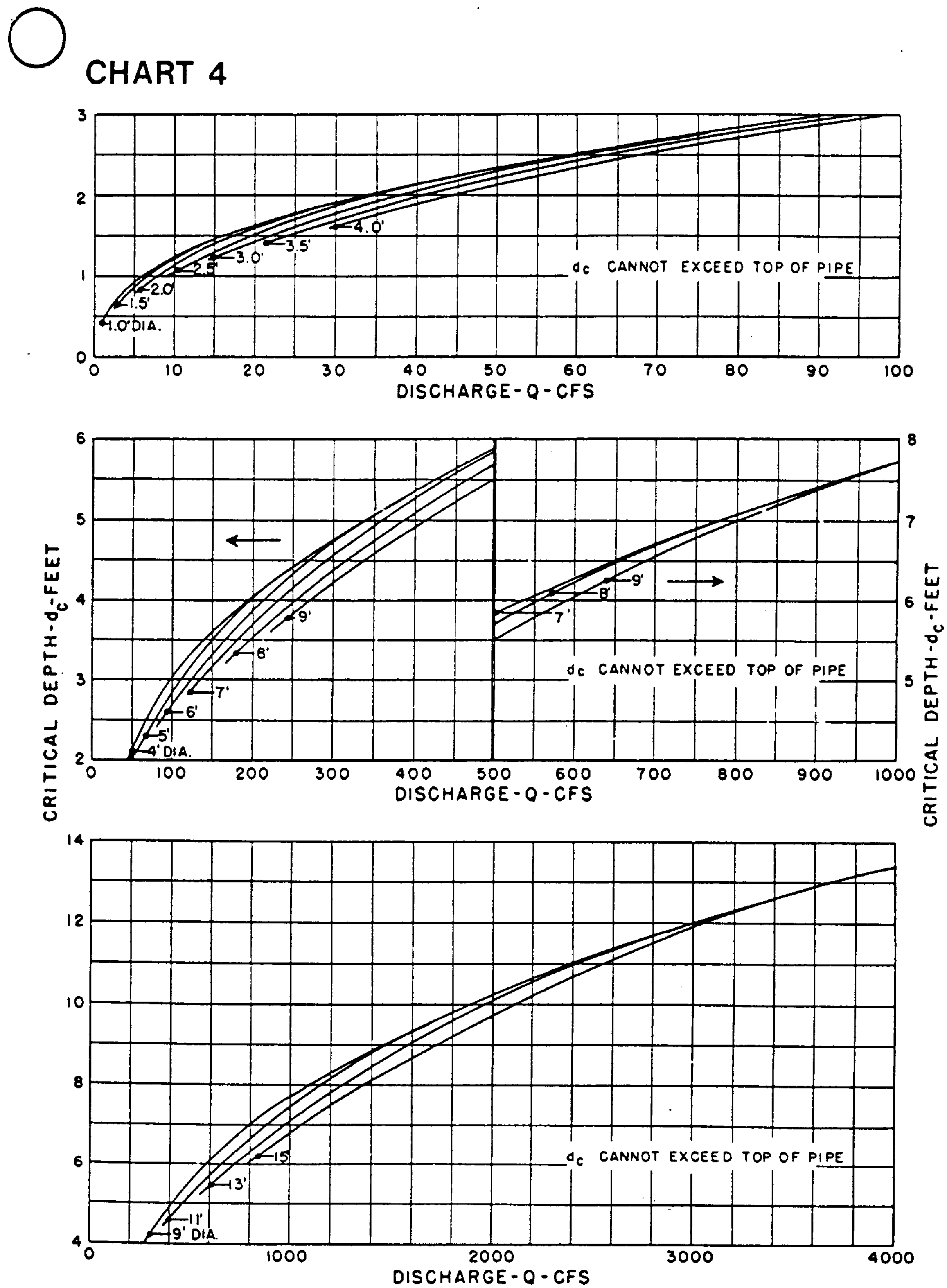


Figure 5-10: Critical Depth - Circular Pipe

Source: FHWA, HDS-5, 1985

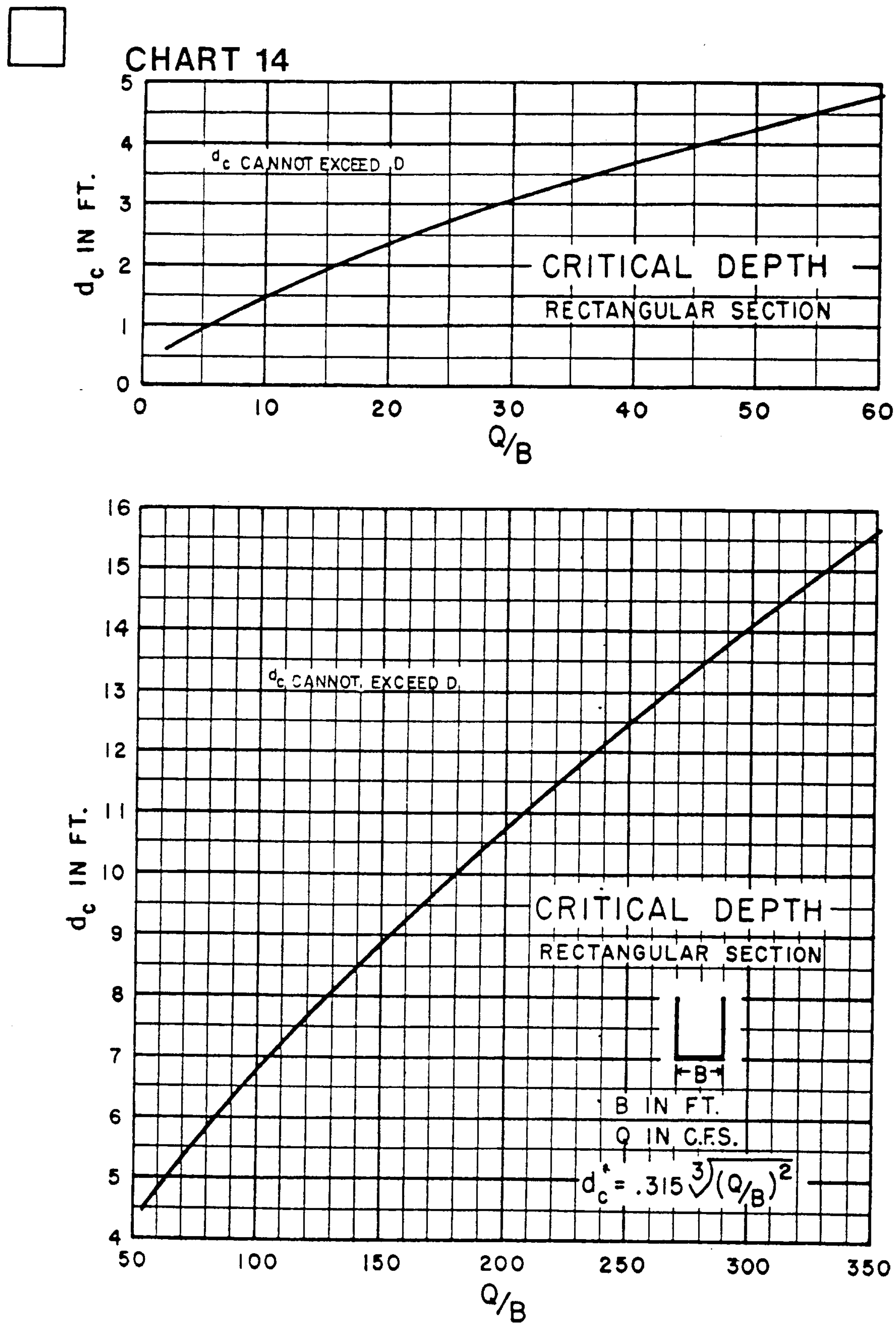


Figure 5-11: Critical Depth - Rectangular Section
Source: FHWA, HDS-5, 1985

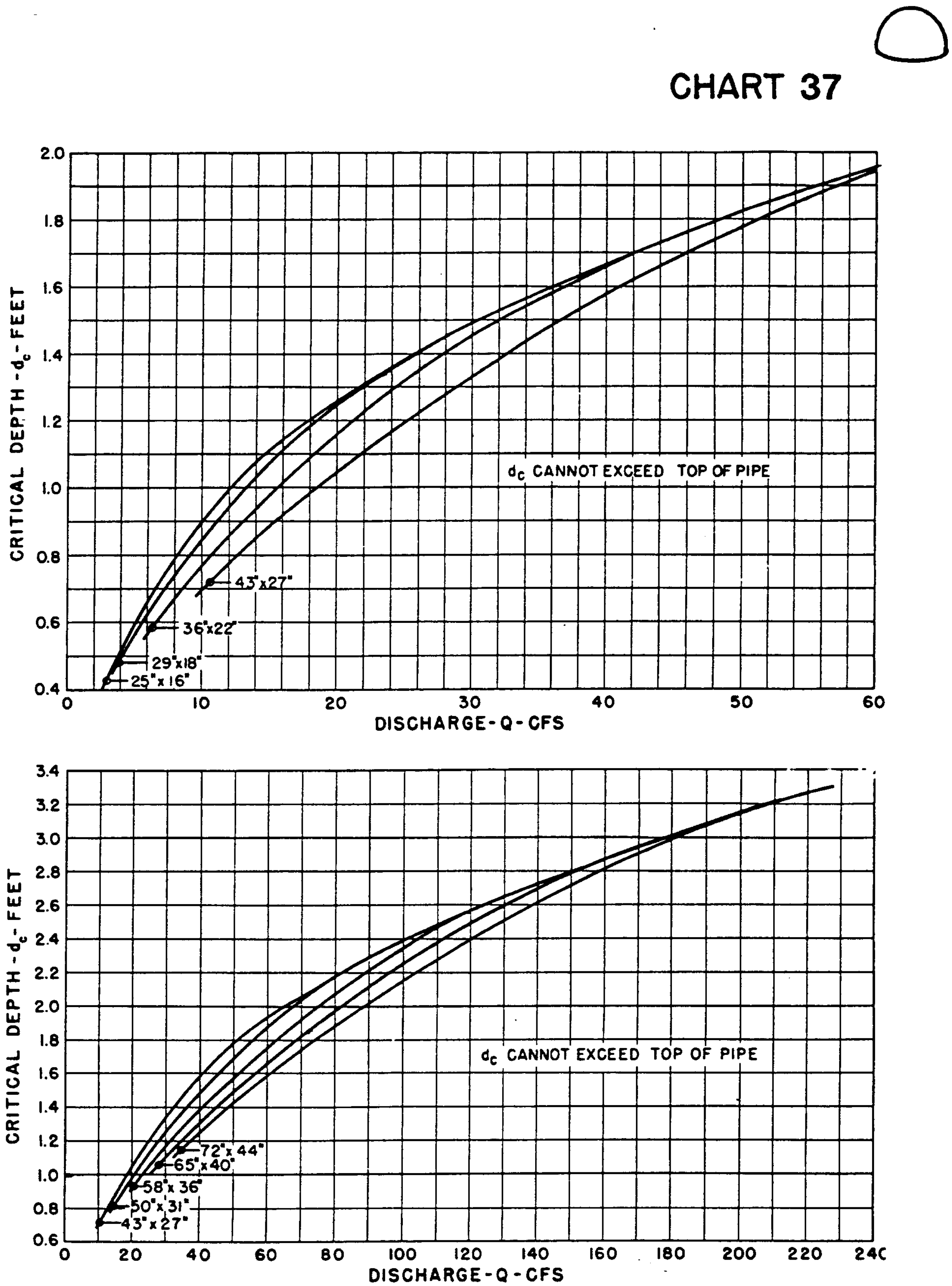


Figure 5-12: Critical Depth - Standard CMP Arch
Source: FHWA, HDS-5, 1985

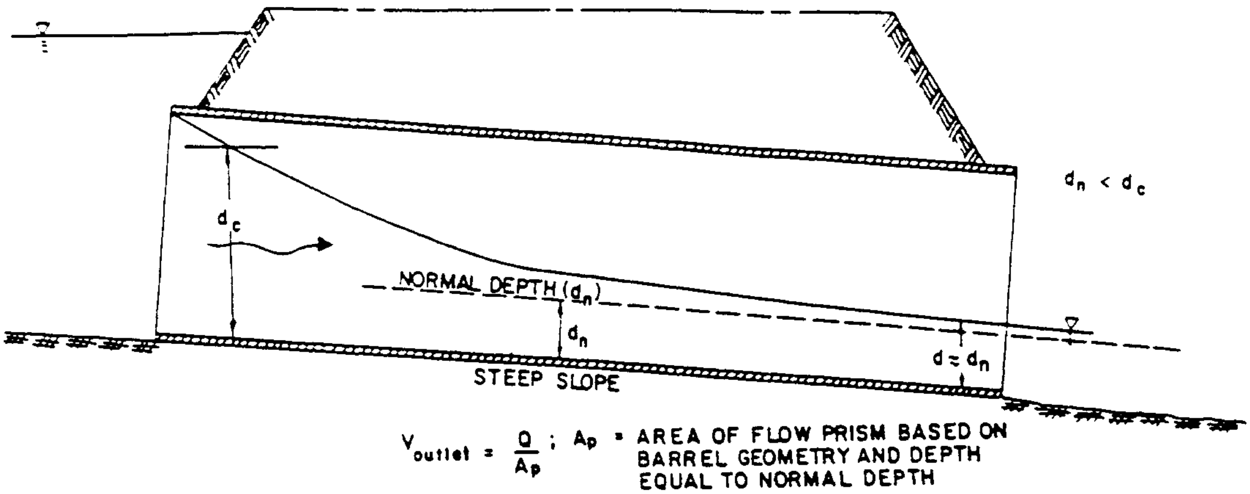


Figure 5-13a: Outlet Velocity - Inlet Control

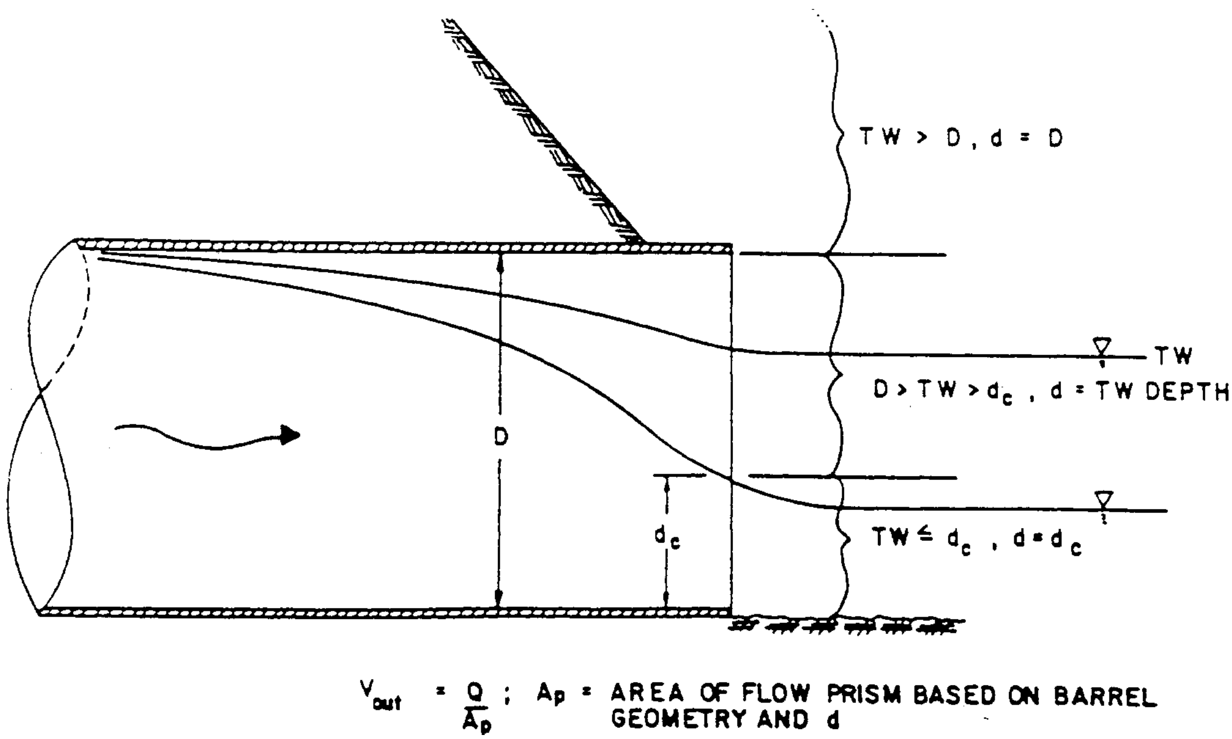


Figure 5-13b: Outlet Velocity - Outlet Control
Source: FHWA, HDS-5, 1985

5.4. BRIDGES

For the purposes of this manual, bridge is defined as:

- Structures that transport vehicular traffic over watercourses or other obstructions,
- Part of a stream crossing system that includes the approach roadway over the floodplain, relief openings, and the bridge structure, and
- Structures with a centerline span of twenty (20) feet or more.

5.4.1. General Policies

a. The final design selection shall consider the maximum backwater allowed by the NFIP/FEMA or local floodplain regulations unless exceeding the limit can be justified by special hydraulic conditions. Backwater shall not significantly increase flood damage to upstream property.

b. The final design shall not significantly alter the flow distribution in the floodplain and whenever possible, bridge structures should be designed so that there is little or no disturbance to the flow. Velocities through the structure shall not damage either the roadway facility or increase damages to adjacent property.

c. The "crest vertical curve profile" should be considered as the preferred roadway crossing profile when allowing for embankment overtopping at a lower discharge.

d. Degradation or aggregation of the watercourse as well as contraction and local scour shall be estimated, and appropriate positioning of the foundation, below the total scour depth if practicable, shall be included as part of the final design.

e. Bridges should be designed to minimize disruption of ecosystems and values unique to the floodplain and wash being crossed.

5.4.2. General Design Criteria

5.4.2.1. Hydraulic Analysis

Hydraulic analyses for both pre and post-bridge conditions shall be performed using step backwater computer models such as HEC-2 (U.S. Army Corps of Engineers, 1990), HEC-RAS (U.S. Army Corps of Engineers, 1995), or WSPRO (USDOT, FHWA, HY-7, 1988).

Recommended methodology for hydraulic analysis of bridge crossings can be found in *Hydraulics of Bridge Waterways,* (USDOT, FHWA, HDS-1, 1978).

5.4.2.2. Hydraulic Design

Risk Evaluation: The selection of hydraulic design criteria for determining the watercourse opening, road grade, scour potential, riprap and other features shall consider the potential impacts to:

- interruptions to traffic,
- adjacent property,
- the environment, and
- the infrastructure of the roadway.

The consideration of the potential impacts constitutes an assessment of risk for the specific site. The least total expected cost (LTEC) alternative should be developed in accordance with FHWA, HEC-17(3) where a need for this type of analysis is indicated by the risk assessment. This analysis provides a comparison between other alternatives developed in response to considerations such as environmental, regulatory, and political.

Design Floods: The design flood(s) for evaluation of backwater, clearance, flow distribution, and overtopping shall be established predicated on risk based assessment of local site conditions. They shall reflect consideration of traffic service, environmental impact, property damage, hazard to human life, and floodplain management requirements.

Backwater and Increases Over Existing Conditions: Designs shall conform to all applicable federal, state, and local floodplain regulations. The bridge structure shall not cause any significant increase in the existing water surface elevation.

Scour: Design for bridge foundation scour shall be for the design flood which generates the maximum scour depth. The design should use a geotechnical design factor of safety of 2.5. The resulting design should then be checked using the 500-year design discharge and a factor of safety of at least 1.0. Procedures for determining scour at bridges can be found in USDOT, FHWA, HEC-18, *Evaluating Scour at Bridges,* Edition 2, 1993.

Bridge piers, if necessary, must be oriented parallel to the direction of flow. Pier spacing and abutments should be designed to minimize flow disruption and potential scour.

Clearance: Where possible, a minimum freeboard of two (2) feet shall be provided between the approach design discharge water surface elevation and the low chord of the bridge. Structural design and clearance of the bridge structure shall also take into account passage of debris and ice impacting the bridge. When this two (2) foot clearance cannot be maintained, clearance shall be established by the engineer based on the type of watercourse and the level of protection desired, as approved by the Stormwater Manager.

Flow Distribution: The conveyance of the proposed crossing location shall be calculated to determine the flow distribution and to establish the location of bridge openings. The proposed structure shall not cause any significant change in the existing flow distribution. Relief openings in the approach roadway embankment shall be investigated if there is more than a ten (10) percent redistribution of flow.

<u>Deck Drainage</u>: Improperly drained bridge decks can cause problems such as corrosion, icing, and hydroplaning. Whenever possible, bridge decks should be watertight and all deck drainage should be carried to the ends of the bridge. Drains at the ends should have adequate capacity to carry all bridge drainage. Gutter flow from roadways must be intercepted before it reaches the bridge. Zero gradients and sump vertical curves should be avoided on bridges.

Where it is necessary to intercept deck drainage at intermediate points along the bridge, the design of such interceptors shall conform to the procedures presented in Chapter 6.

CHAPTER 6: PAVEMENT DRAINAGE

Effective drainage of urban streets is essential to the maintenance of roadway service levels and traffic safety. Water on streets can interrupt traffic, reduce skid resistance, increase the potential for hydroplaning, limit visibility due to splash and spray, cause difficulty in steering a vehicle, and cause a nuisance and possible hazard to pedestrian traffic.

Street drainage requires consideration of surface drainage, gutter flow, and drainage inlet capacity. The design of these components is dependent upon the design frequency and the allowable spread of stormwater on the pavement surface. This chapter presents design criteria and procedures obtained from FHWA, HEC-22, *Urban Drainage Design Manual,* November 1996. Additional procedures and design guidelines not included in this chapter can be found FHWA, HEC-22.

6.1. POLICIES

a. Street drainage and roadways shall be designed so as to maintain the natural drainage patterns existing prior to development, whenever possible.

b. The street section shall be designed to convey local runoff only and shall not be used as major stormwater carriers for contributing watersheds.

c. Drainage facilities shall be installed to convey runoff under streets or street grades shall be set so diversion of runoff or ponding will not occur on adjacent properties.

d. Street slopes (longitudinal and transverse) and curb heights shall not be increased to create more carrying capacity for runoff. Curb overtopping is not permitted for the specified design storm.

e. New inverted crown public streets are prohibited.

f. Existing alleys shall not be used to convey runoff unless the entire alley is designed and constructed to convey runoff to the nearest downstream street.

g. Drainage facilities shall be placed to intercept runoff from sources outside the street section to avoid significant concentrated flows onto and over sidewalks or curb and gutter.

h. In all cases, street drainage shall be confined to the public right-of-way. Runoff which leaves the right-of-way shall do so in a controlled manner and shall be contained in appropriate right-of-way or drainage easement.

6.2. STREET AND GUTTER DRAINAGE

6.2.1. Design Frequency and Allowable Spread

For local curbed street sections, runoff from the 10-year design storm must be contained between the curbs of the street and the 100-year flow within the right-of-way.

For collector and arterial curbed street sections, at least one twelve (12) foot travel lane in each direction must remain free from flooding for the 10-year design storm and the 100-year flow within the right-of-way.

If either of the above two criteria are exceeded, storm drains facilities (see Chapter 7) will be required. In all instances, the 10-year design storm must be contained within the combined street gutter and storm drain system.

A check storm (e.g., 100-year) should be used any time runoff could cause unacceptable flooding during less frequent storm events. Drainage inlets should also be evaluated for the check storm when a series of inlets terminates at a sump vertical curve where ponding to hazardous depths could occur.

6.2.2. Longitudinal and Transverse Slope

The minimum longitudinal gradient for all public street types shall be no less than 0.5 percent for both curbed and rural street sections. Minimum grades can be maintained in flat terrain by use of a rolling or "sawtooth" profile, or by warping the cross slope to achieve rolling gutter profiles.

When local streets intersect arterial or collector streets, the grade of the arterial/collector street shall be continued uninterrupted, whenever possible. When collector and arterial streets intersect, the grade of the arterial street shall be maintained as much as possible.

Pavement cross-slope for new streets shall be a maximum of 2.0 percent on street sections with a central crown. Sheet flow across a street shall be kept to an absolute minimum. Median areas shall not be drained across travel lanes, except with raised, narrow medians. Shoulders must be sloped to drain away from the pavement per the City's Engineering Standard Details.

For drainage purposes, valley gutters are not permitted to cross arterial or collector street. Valley gutters may be used on local streets at intersections, when a storm drain is not required, and when approved by the Stormwater Manager. In cases where a valley gutter is not permitted or is inadequate, the water must be removed from the street to storm drains or other approved method(s).

To provide adequate drainage in sump vertical curves, a slope of 0.4 percent should be maintained within fifty (50) feet of each side of the low point of the curve, with a minimum slope of 0.3 percent. To accomplish this, the length of the curve divided by the algebraic difference in grades should be equal to or less than 167.

Pipe under-drains may be required where the possibility of ground water or surface water seepage to the subbase soil exists.

6.2.3. Gutter Flow

For the purposes of this section, a pavement gutter is defined as a section of pavement adjacent to the roadway which conveys runoff during a given storm event. It may include a portion or all of the travel lane. Gutters may have a straight cross-slope forming a triangular shape or may have a composite cross-slope where the gutter slope varies from the pavement cross-slope.

Manning’s equation cannot be used to determine gutter capacity without modification because the hydraulic radius does not adequately describe the gutter cross-section. The following modified Manning's Equation should be used to evaluate theoretical gutter flow capacity:

$$\mathbf{Q = (0.56 / n)\, S_x^{1.67}\ S^{0.5}\ T^{2.67}} \qquad \mathbf{(6.1)}$$

where:
- Q = discharge, cfs
- n = Manning's roughness coefficient (see Table 6-1)
- S_x = pavement cross slope, ft/ft
- S = longitudinal cross slope, ft/ft
- T = width of flow or spread, ft

Utilizing $d = TS_x$, and $z = 1/S_x$, Equation 6.1 becomes:

$$\mathbf{Q = 0.56(z / n)\, S^{0.5}\ d^{2.67}} \qquad \mathbf{(6.2)}$$

Nomographs for solving Equation 6.1 for triangular and composite gutter sections are given in Figures 6-1 through 6-3. Table 6-1 provides recommended roughness coefficients for street or parking lot design. Consideration should also be given to actual flow conditions due to obstructions and discontinuity in the gutter flow caused by driveways, debris, sediment, parked cars, etc.

6.2.4. Curb and Gutter Terminations

Adequate erosion protection and conveyance measures are required where the curb and gutter section terminates to prevent undermining of the pavement/taper edge, prevent headcutting of roadway fill slopes, and prevent the gutter flow from entering private property. The gutter flow from the street section must be conveyed into a receiving channel, roadside swale, or other approved means. Concrete spillways or riprap protection shall be provided when velocities may cause headcutting at the curb termination or erode the receiving channel/swale.

6.2.5. Rural Street Drainage and Roadside Channels

Roadside ditches or channels for rural, uncurbed street sections shall be designed for the 25-year design storm. The runoff shall be contained within the roadside channels with the allowable water depth elevation below the roadway subgrade to avoid unnecessary saturation of the subgrade and/or aggregate base course shoulder. The minimum depth of a roadside channel shall be one (1) foot and shall be designed per the City's Engineering Standard Details.

The underlying soil conditions, depth of flows, flow velocities, and maintenance shall be considered in the roadside channel design. Long reaches of riprap lined roadside channels should be avoided, if possible, due to long term maintenance problems.

Trapezoidal cross-sections are permissible, however, the channel bottom shall be a minimum of four (4) feet wide for maintenance purposes.

Crown ditches or interceptor drains may be required at the top of the cut slope embankments with tributary drainage area above drains toward the cut and has a drainage path greater than forty (40) feet measured horizontally.

TABLE 6-1: MANNING'S 'n' FOR STREET AND PAVEMENT GUTTERS

TYPE OF GUTTER OR PAVEMENT	MANNINGS "n"
Asphalt Pavement:	
Smooth texture	0.013
Rough texture	0.016
Concrete Gutter, troweled finish	0.012
Concrete Gutter w/asphalt pavement:	
Smooth	0.013
Rough	0.015
Concrete pavement:	
float finish	0.014
broom finish	0.016
Brick or pavers	0.016
For gutters with small slope, where sediment, may accumulate increase above values of "n" by	0.002

Source: USDOT, FHWA, HDS-3, 1977

NOTE: A minimum value of 0.016 shall be used for all new paved street designs, regardless of theoretical values.

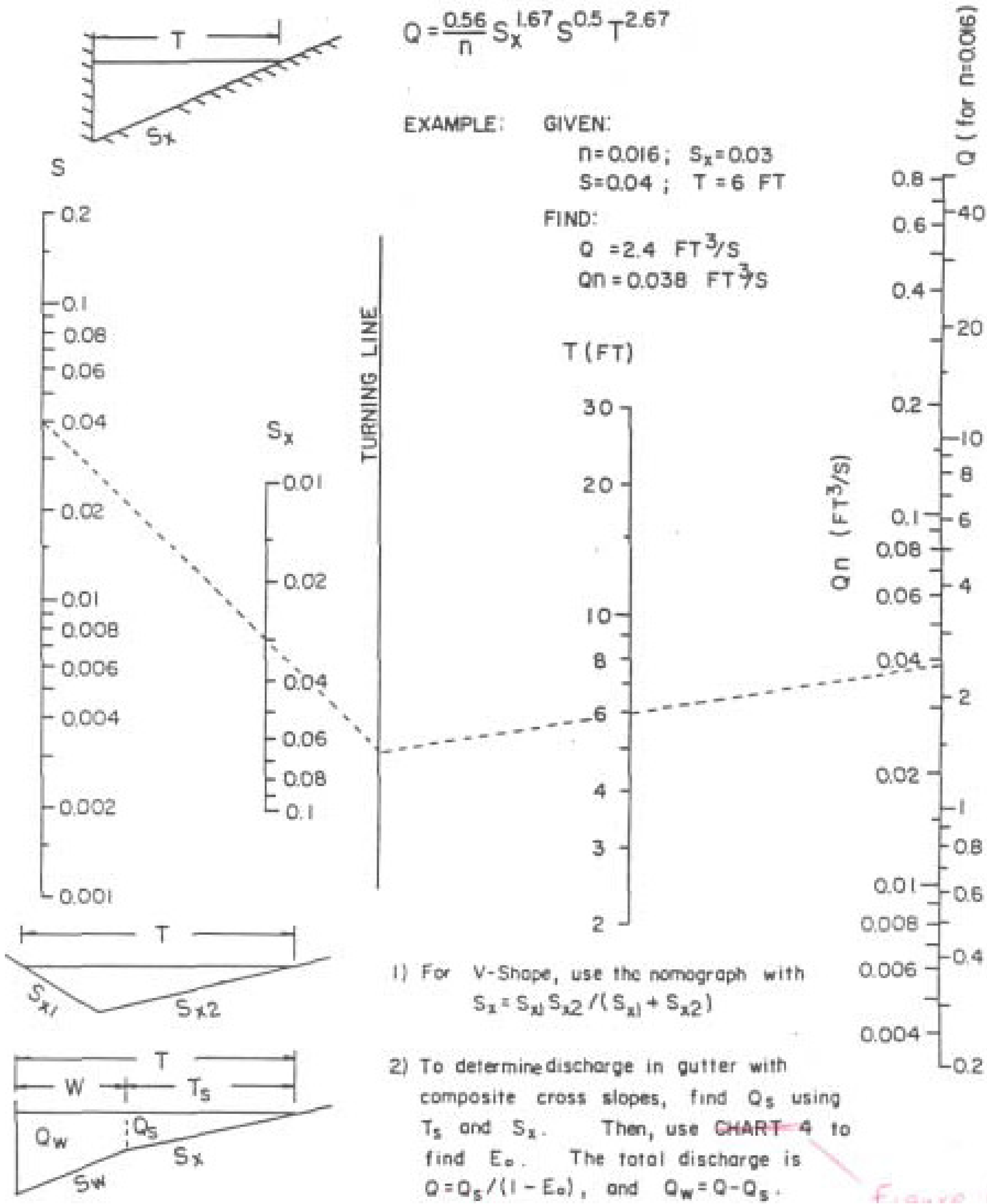


Figure 6-1: Flow In Triangular Gutter Sections
Source: FHWA, HEC-12, 1984

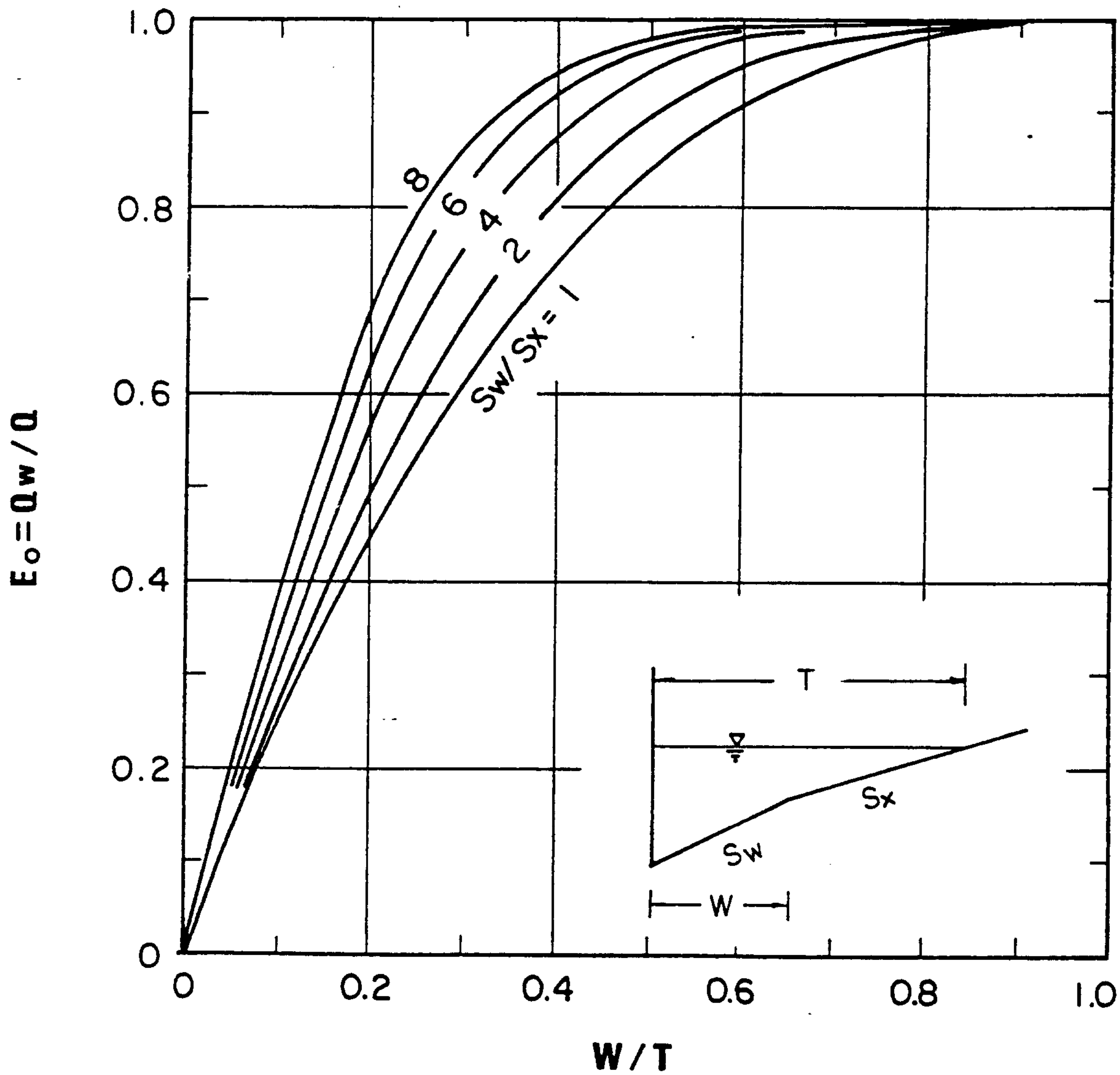


CHART 4

Figure 6-2: Ratio Of Frontal Flow To Total Gutter Flow
Source: FHWA, HEC-12, 1984

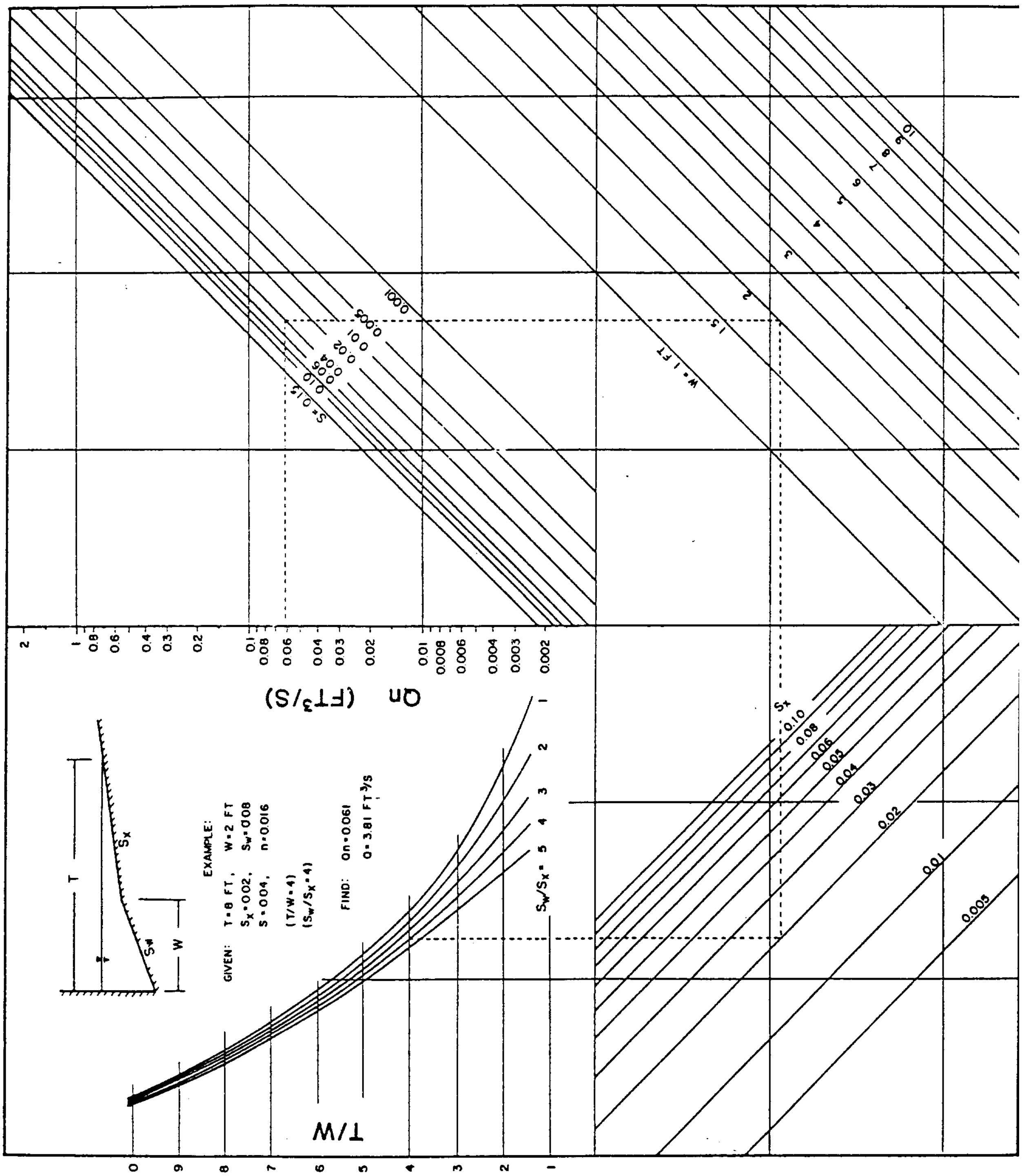


Figure 6-3: Flow In Composite Gutter Sections
Source: FHWA, HEC-12, 1984

6.3. <u>DRAINAGE INLETS</u>

Storm drain inlets (i.e., catch basins) are necessary to drain streets and convey runoff in excess of allowable pavement spread or flow depth criteria. Inlets are typically located in gutter sections, roadside channels, parking lots, or other sump locations. Inlet capacity is dependent upon the inlet geometry and the characteristics of the street and gutter flow. Inadequate inlet capacity and spacing may cause flooding on the roadway, private property, or result in a hazard to the traveling public.

6.3.1. Inlet Types

There are four classes of drainage inlets:

1. grate inlets,
2. curb-opening inlets,
3. combination inlets, and
4. slotted drains.

Grate inlets consist of a horizontal grated opening in the gutter, channel bottom, or other locations and may be placed either on a continuous grade or in a sump condition. Curb-opening inlets consist of a vertical opening in the curb face and are most effective on flat grades and in sumps. Combination inlets consist of both a curb-opening and a grate inlet placed in a side-by-side configuration or the curb-opening may be placed upstream of the grate (sweeper inlet). Slotted drains consist of an opening cut along the longitudinal axis of a conduit with bars perpendicular to the opening to maintain the slotted configuration. Figure 6-4 illustrates the four types of drainage inlets.

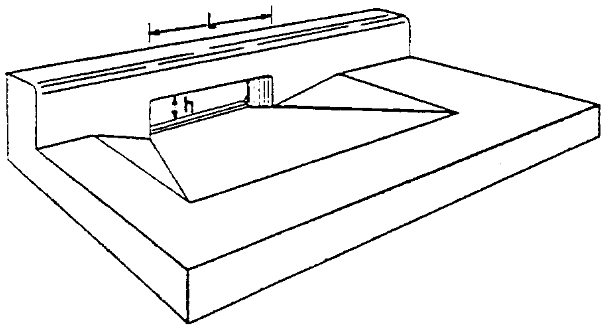
All catch basins shall be designed and constructed in accordance with current MAG Standard Details or other equivalent details as approved by the Stormwater Manager.

6.3.2. Allowable Spread and Flow Depth

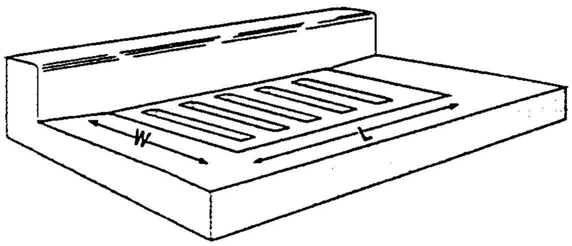
Inlet spacing must be designed to meet allowable spread limits (with carryover considered) and curb overtopping restrictions. It is recommended that inlet spacing not exceed 600 feet for collection of nuisance flows.

Catch basins and storm drains are required for collector and arterial streets when one twelve (12) foot travel lane in each direction cannot be maintained for the 10-year design storm. The allowable spread for streets with permanent parking lanes shall be limited to the parking lane width.

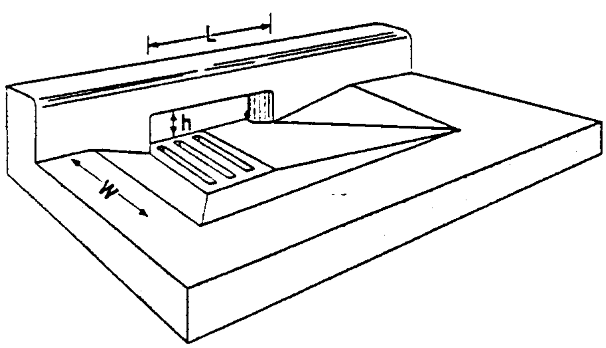
Storm drain inlets and storm drains are required for local streets when the curb is overtopped for the 10-year design storm.



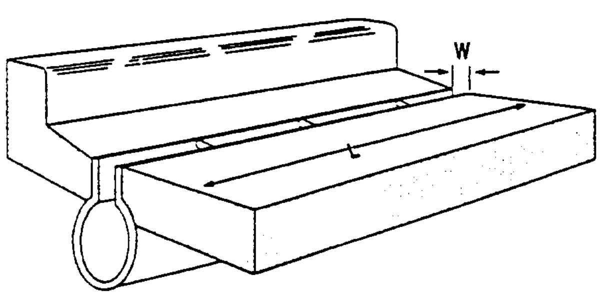
(a) Curb Opening Catch Basin Inlet



(b) Grated Catch Basin Inlet



(c) Combination Catch Basin Inlet



(d) Slotted Drain Catch Basin Inlet

Figure 6-4: Drainage Inlet Types

Inlets in all sump locations must be sized to prevent overtopping of the curb for local streets and maintain a twelve (12) foot travel lane free from accumulated runoff for collector and arterial streets. Caution should be used when placing an inlet at the end of a downhill cul-de-sac as this often allows excess water to bypass the inlet onto private property if the inlet becomes clogged.

6.3.3. Inlet Locations

One-hundred percent interception of the 10-year design storm is required for all inlets in sumps or other locations as required by the Stormwater Manager to prevent hazardous ponding conditions.

Flanking inlets may be required on new arterial street sump vertical curve locations to prevent hazardous ponding in the event the primary sump inlet is completely clogged (see Section 6.3.8).

Inlets shall be located such that concentrated flow or heavy sheet flow will not cross traffic lanes, whenever possible. Where pavement surfaces are warped (e.g., at cross streets or ramps) runoff shall be intercepted just before the change in cross slope, whenever possible. Inlets should be placed on arterial and collector streets upstream of locations where the pavement cross slope begins to superelevate to avoid concentrated flows crossing the roadway and reduce traffic hazards associated with icing.

Street runoff is not permitted to cross arterial streets at intersections, therefore inlets are required upgrade of pedestrian crossings and intersections, whenever possible. Consideration should also be given to avoid locating inlets within existing or proposed driveways locations, whenever possible.

Inlets shall be selected, sized and located to prevent silt and debris, carried in suspension, from being deposited on the traveled way where the longitudinal gradient is decreased.

6.3.3.1. Geometric Controls

The following information is typically required to determine locations of drainage inlets:

- plan sheet suitable for outlining drainage areas;
- road profiles;
- typical street cross-sections;
- grading cross-sections;
- superelevation diagrams; and
- contour maps.

Using the above information, geometric controls will determine a number of locations which require inlets with 100% interception, regardless of contributing drainage area, as identified below:

- Immediately upgrade of arterial intersections, if possible;
- At all sump locations in gutter grade;
- Immediately upgrade of bridges;
- Immediately upgrade of cross slope reversals, if possible;
- At the end of channels in cut sections; and

- Behind curbs, shoulders or sidewalks to drain low areas.

In addition to the areas identified above, runoff from areas draining towards the roadway shall be intercepted by roadside channels or inlets before it reaches the street. This applies to drainage from roadway cut slopes, side streets, and other areas outside of the roadway. In general, curbed street sections and pavement inlets are an inefficient means for handling extraneous drainage behind the curb.

6.3.4. Grate Inlets

Grate inlets may be utilized where clogging due to debris will not be a problem. Grate inlets generally lose capacity with increases in the longitudinal street grade, but to a lesser degree than do curb-opening inlets. Increasing the length of grate inlets does little to increase capacity, however, increasing the grate width greatly increases the capacity. Where street grades exceed three (3) percent, grate or combination inlets should be used instead of curb-opening inlets.

Where debris is anticipated to be a potential problem, consideration shall be given to debris handling efficiency ratings of the grate. Efficiency ratings and formulas for computing efficiencies for grates can be found in HEC-22 or inlet charts prepared by grate manufacturers. Table 6-2 outlines the results of debris handling efficiencies of several common grates. Curved vane and tilt bar grates typically have the highest debris handling efficiencies for weir flow conditions.

Grate inlets subject to vehicular traffic must have adequate load bearing capacity. All grated inlets must be bicycle safe. Where possible, curb opening inlets should be used if the street has a bike lane. Table 6-3 outlines rating efficiencies with respect to bicycle and pedestrian safety.

TABLE 6-2: GRATE DEBRIS HANDLING EFFICIENCIES

RANK	GRATE	Longitudinal Slope (0.005)	Longitudinal Slope (0.04)
1 (Best)	CV - 3-1/4 - 4-1/4	46	61
2	30 - 3-1/4 - 4	44	55
3	45 - 3-1/4 - 4	43	48
4	P - 1-7/8	32	32
5	P - 1-7/8 - 4	18	28
6	45 - 2-1/4 - 4	16	23
7	Reticuline	12	16
8	P - 1-1/8	9	20

Source: FHWA, HEC-22, 1996

TABLE 6-3: BICYCLE AND PEDESTRIAN SAFETY RANKINGS

Rank	Grate Style
1 (Best)	P - 1-7/8 - 4
2	Reticuline
3	P - 1-1/8
4	45 - 3-1/4 - 4
5	45 - 2-1/4 - 4
6	Curved Vane
7	30 - 3-1/4 - 4

Source: Burgi, 1978

6.3.4.1. Grate Inlets On Grade

A grate inlet on a continuous grade will intercept all of the frontal flow passing over the grate, unless the grate becomes clogged or splash-over occurs. Splash-over will occur if the velocity is high or the grate is short and only a portion of the frontal flow will be intercepted. The total capacity of a grated catch basin on grade is the sum of the frontal flow and the side flow, minus the splash-over flow.

The ratio of frontal flow to total gutter flow (E_o) for a straight cross-slope is expressed by:

$$\mathbf{E_o = Q_W/Q = 1 - (1 - W/T)^{2.67}} \quad \textbf{(6.3)}$$

where:
- Q_W = frontal flow at width W, cfs
- Q = total gutter flow, cfs
- W = width of grate, ft
- T = total spread of water at the gutter, ft

Figure 6-2 provides a graphical solution of E_o for either straight cross-slopes or depressed gutter sections.

The ratio of side flow (Q_S) to total gutter flow is expressed as:

$$\mathbf{Q_S/Q = 1 - Q_W/Q = 1 - E_o} \quad \textbf{(6.4)}$$

The ratio of frontal flow intercepted to total frontal flow (R_f) is expressed by:

$$\mathbf{R_f = 1 - 0.09\ (V - V_O)} \quad \textbf{(6.5)}$$

where:
- V = velocity of flow in the gutter, ft/s (see Fig. 6-5)
- V_O = gutter velocity where splash over first occurs, ft/s

This ratio is equivalent to the frontal flow interception efficiency. Figure 6-6 provides a graphical solution of Equation 6.5 which takes into account grate length, bar configuration, and gutter velocity at which splash-over occurs. The gutter velocity needed to use Figure 6-5 is the total gutter flow divided by the flow area.

The ratio of side flow intercepted to total side flow (R_s), or side flow interception efficiency is expressed by:

$$\mathbf{R_s = 1 / [1 + (0.15V^{1.8} / S_xL^{2.3})]} \qquad \textbf{(6.6)}$$

where: L = length of the grate (ft) and the other terms are previously defined

Figure 6-7 provides a solution to Equation 6.6.

The efficiency of a grate (E) is expressed as:

$$\mathbf{E = R_f\, E_o + R_s(1 - E_o)} \qquad \textbf{(6.7)}$$

The interception capacity of a grate inlet on a continuous grade is then equal to the efficiency of the grate (E) multiplied by the total gutter flow (Q):

$$\mathbf{Q_i = EQ = Q[R_f\, E_o + R_s(1 - E_o)]} \qquad \textbf{(6.8)}$$

6.3.4.2. *Grate Inlets In Sump*

The capacity of grate inlets in a sump is dependent upon the open area of the grate and the ponding depth. The ability to pass debris is therefore critical. A grate inlet in a sag operates as a weir up to about 0.4 feet above the top of the grate, for a standard gutter inlet. When the depth of water exceeds about 1.4 feet, the grate begins to operate as an orifice. Between 0.4 feet and 1.4 feet, a transition from weir to orifice flow occurs. The use of combination or curb-opening catch basins is recommended for curbed street sump locations.

The capacity of a grate operating as a weir is expressed as:

$$\mathbf{Q_i = 3.0\, P\, d^{1.5}} \qquad \textbf{(6.9)}$$

where: Q_i = rate of discharge into the grate opening, cfs
P = perimeter of grate excluding bar widths and the side against the curb, ft
d = depth of water above the grate, ft

The capacity of a grate operating as an orifice is expressed as:

$$\mathbf{Q_i = 0.67\, A\, (2gd)^{0.5}} \qquad \textbf{(6.10)}$$

where: A = clear opening area of the grate, ft^2

g $= 32.2$ ft/s^2
d = depth of water above the grate, ft

Figure 6-8 provides a plot of Equations 6.9 and 6.10 for various grates.

The transition from weir (approx. 0.4 feet) and orifice (approx. 1.4 feet) results in an interception capacity less than that computed by either the weir or the orifice equations. Using Figure 6-7, this capacity can be approximated by drawing in a curve between the lines representing the perimeter and net area of the grate to be used.

Tilt Bar or curved vane grates are not recommended for grate inlets in a sump which operate as an orifice.

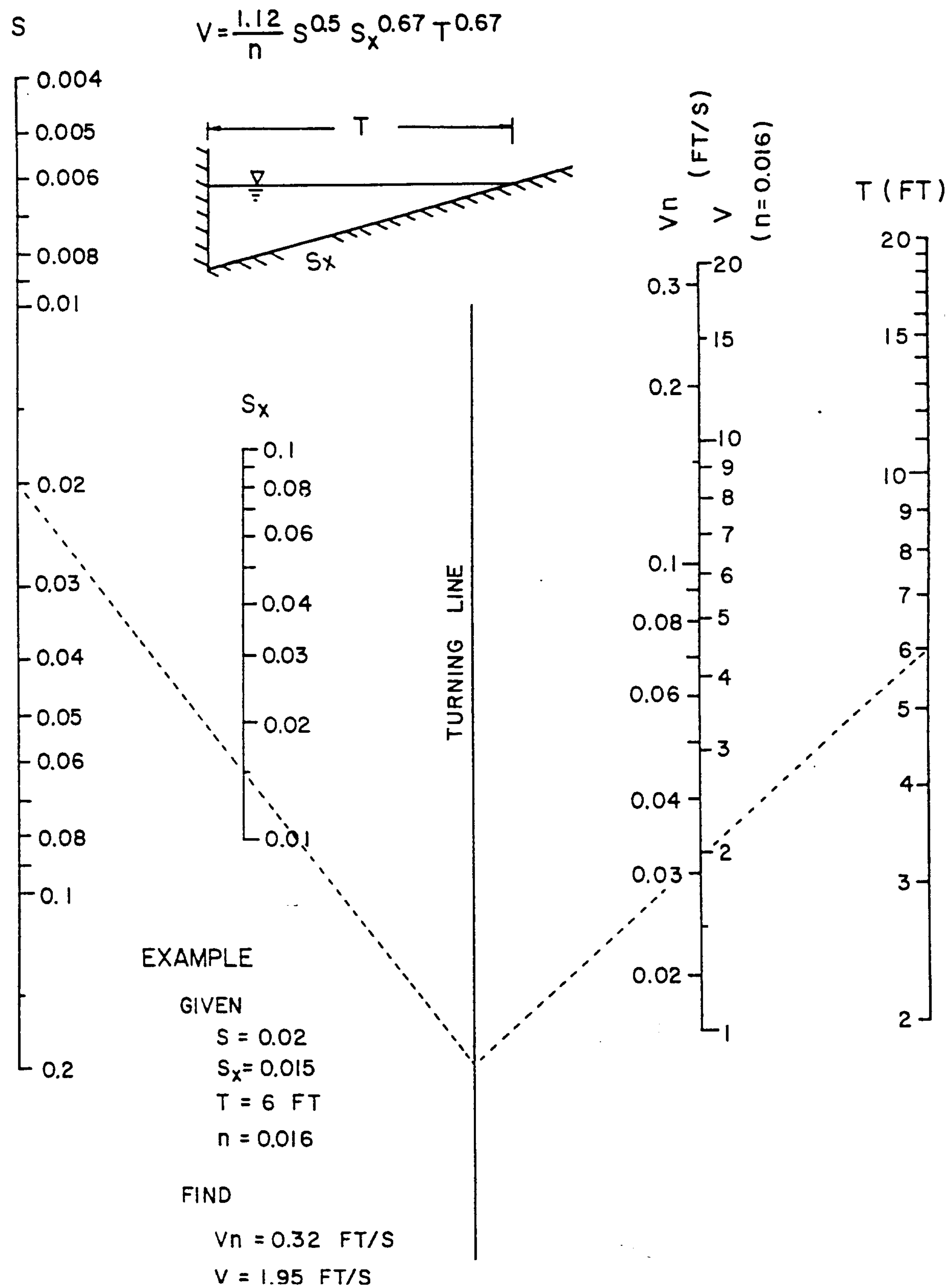


Figure 6-5: Velocity In Triangular Gutter Sections
Source: FHWA, HEC-12, 1984

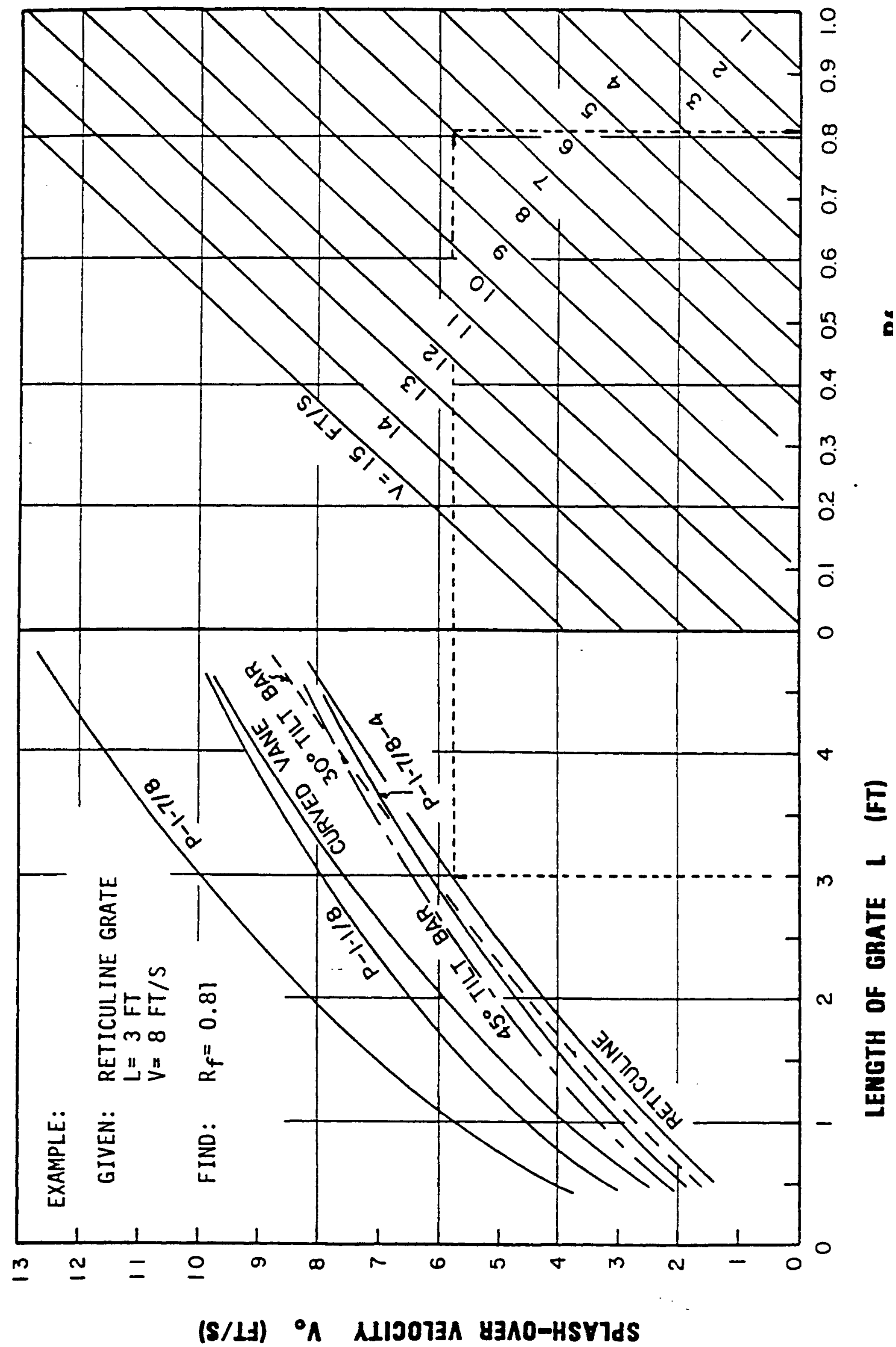


Figure 6-6: Grate Inlet Frontal Flow Interception Efficiency

Source: FHWA, HEC-12, 1984

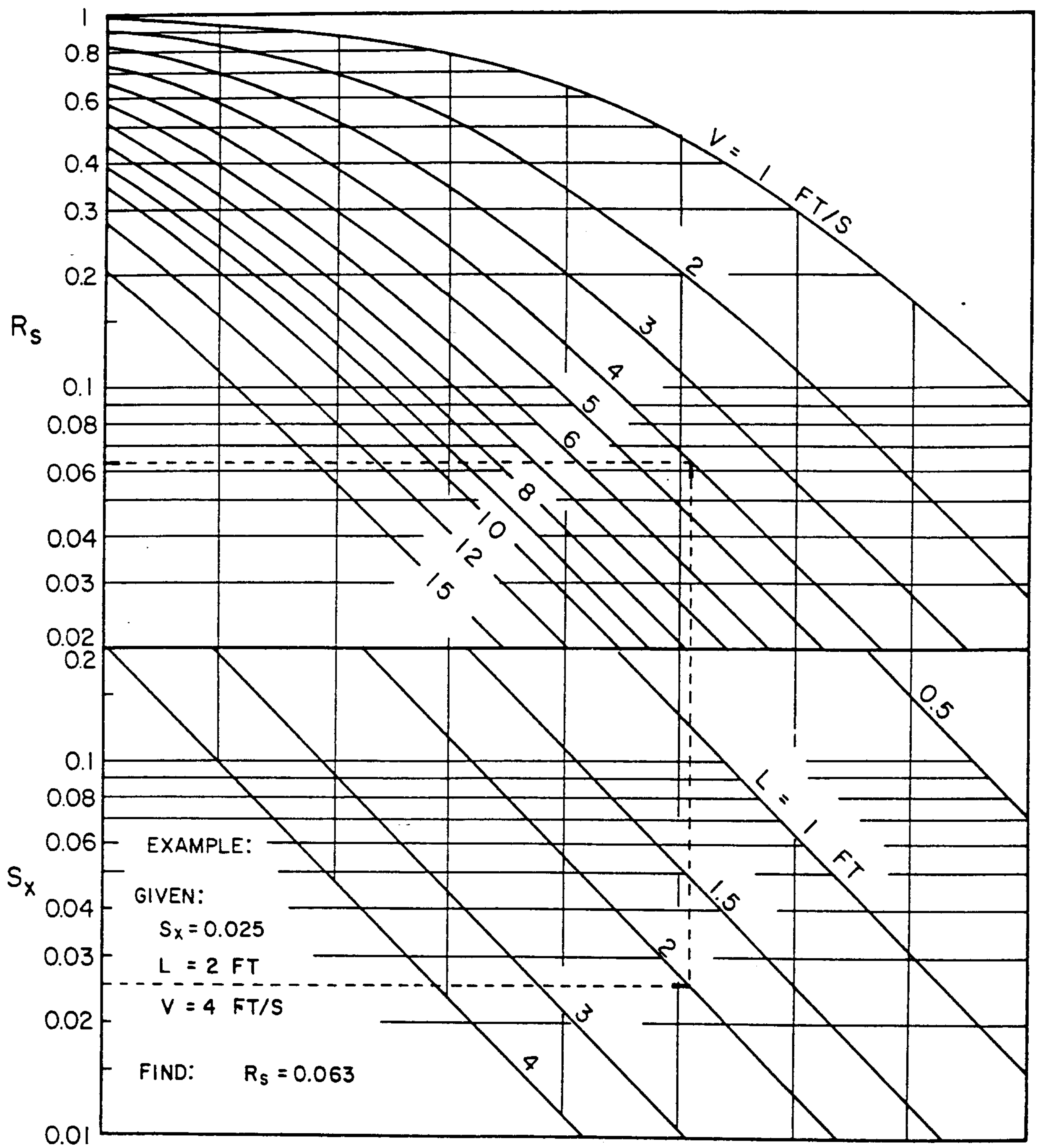


Figure 6-7: Grate Inlet Side Flow Interception Efficiency
Source: FHWA, HEC-12, 1984

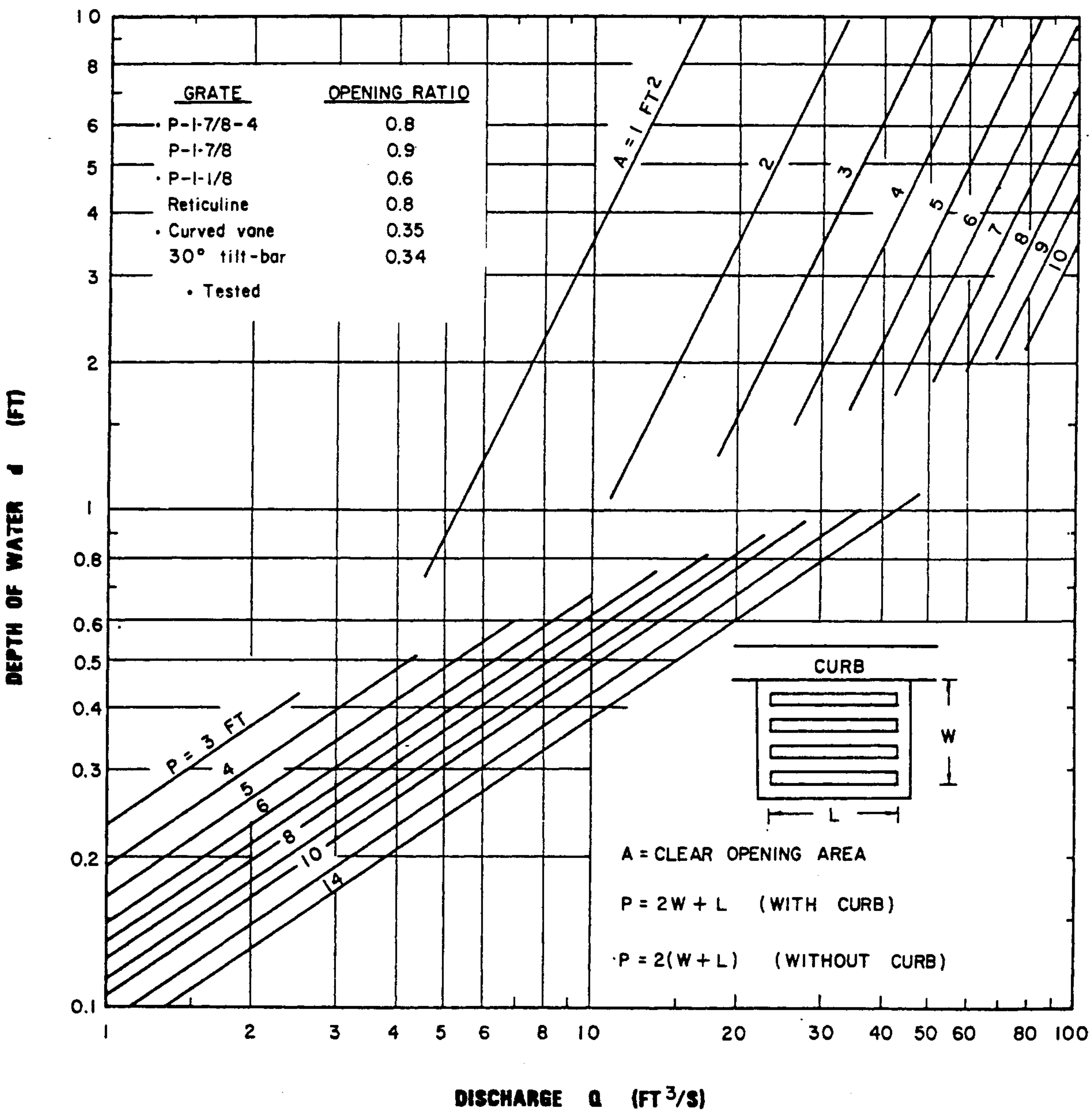


Figure 6-8: Grate Inlet Capacity In Sump Conditions
Source: FHWA, HEC-12, 1984

6.3.5. Curb Opening Inlets

Curb opening inlets are most effective on flatter slopes, in sump locations, with flows carrying significant amounts of debris, and where the flow depth at the curb is sufficient for the inlet to perform efficiently. The interception capacity of curb-opening inlets decreases as the longitudinal gutter grade increases.

Curb-opening inlets are recommended in curbed sump locations and on grades less than three (3.0) percent. In addition, curb-opening inlets less than five (5.0) feet in length should not be used on continuous grades due to inefficient interception. Curb-opening inlets on grade should also not be placed on the inside of a horizontal curve due to inefficient interception.

The curb opening shall not exceed six (6) inches in height, unless approval from the Stormwater Manager is obtained and the opening is equipped with cross bars for safety reasons.

6.3.5.1. Curb Opening Inlets on Grade

The length of a curb opening inlet for total interception of gutter flow on a pavement section with a straight cross slope (i.e., no gutter depression) is expressed in Equation 6.11.

$$\mathbf{L_T = 0.6\ Q^{0.42}\ S^{0.3}\ (1/nS_x)^{0.6}} \qquad \mathbf{(6.11)}$$

where:
- L_T = curb length required for 100% interception
- S = longitudinal gutter slope, ft/ft
- n = Manning's roughness coefficient
- S_x = pavement cross slope, ft/ft

Figure 6-10 is a nomograph for the solution of Equation 6.11.

The efficiency of curb inlets shorter than the length required for total interception is expressed by:

$$\mathbf{E = 1 - (1 - L_i/L_T)^{1.8}} \qquad \mathbf{(6.12)}$$

where:
- E = efficiency, ratio of intercepted discharge to total discharge
- L_i = curb inlet length, ft

Figure 6-11 provides a nomograph for the solution of Equation 6.12.

The length of an inlet required for total interception by a depressed curb inlet or curb inlets in depressed gutter sections can be found by use of an equivalent cross-slope (S_e) in place of S_x (in Equation 6.11), as expressed by:

$$\mathbf{S_e = S_x + S_w'\ E_o} \qquad \mathbf{(6.13)}$$

where: S_w' = the cross-slope of the gutter measured from the cross-slope of the pavement, which equals (a/12W), ft/ft

E_o = the ratio of flow in the depressed section to the total gutter flow

The length of curb opening required for total interception can be significantly reduced by increasing the cross-slope or equivalent cross-slope. The equivalent cross-slope can be increased by use of a continuously depressed gutter or a locally depressed gutter section.

Using the equivalent cross slope (S_e), Equation 6.11 becomes:

$$\mathbf{L_T = 0.6\ Q^{0.42} S^{0.3}\ (1/nS_e)^{0.6}} \qquad \textbf{(6.14)}$$

6.3.5.2. Curb Opening Inlets In Sumps

A curb opening inlet in a sump operates as a weir to depths equal to the curb opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transitional stage.

The equation for the interception capacity of a depressed curb opening inlet operating as a weir is:

$$\mathbf{Q_i = 2.3\ (L + 1.8W)\ d^{1.5}} \qquad \textbf{(6.15)}$$

where:
- L = length of curb opening, ft
- W = width of depression, ft
- d = depth of water at curb measured from the normal cross slope gutter flowline, ft

Figure 6-12 provides a solution for Equation 6.15. The weir equation is applicable to depths at the curb approximately equal to the height of the opening plus the depth of the depression. Therefore, the limitation on the use of Equation 6.15 for a depressed curb opening is:

$$\mathbf{d \leq h + (a/12)} \qquad \textbf{(6.16)}$$

The weir equation for curb opening inlets without a depression (i.e., W = 0) becomes:

$$\mathbf{Q_i = 3.0Ld^{1.5}} \qquad \textbf{(6.17)}$$

Equation 6.17 is applicable to depths at the curb which are approximately equal to the height of the opening plus the depth of the depression (i.e. ,$d \leq h$).

Curb opening inlets operate as orifices at depths greater than approximately 1.4h and the capacity is then computed as:

$$\mathbf{Q_i = 0.67A\ (2gd_0)^{0.5}} \qquad \textbf{(6.18)}$$

where:
- A = clear area of opening (h x L), ft^2

g = 32.2 ft/s²
d_0 = depth at lip of curb opening, ft
h = height of curb opening orifice, ft

Equation 6.18 is applicable to depressed and un-depressed curb inlets and the depth at the inlet includes any gutter depression. At a depth of 0.4 feet, the depressed curb opening inlet has about 70 percent more capacity than an undepressed inlet.

The height of the orifice in Equation 6.18 assumes a vertical opening. As shown in Figure 6-9, other orifice throat configurations can change the effective depth on the orifice and the dimension (d_i - $h/2$). A limited throat width could reduce the inlet capacity by causing the inlet to go into orifice flow at depths less than the height of the opening.

6.3.6. Combination Inlets

Combination inlets provide the advantages of both grate and curb opening inlets. Curb-openings are typically placed upstream of the grate and act as a "sweeper" or debris interceptor during the initial phases of the storm. Combination inlets in a sump location can also have a curb-opening on both sides of the grate or the curb opening can be equal in length to the grate.

For combination inlets on grade, the grate section shall be placed at the downstream end of the inlet structure and must also be bicycle and pedestrian safe.

6.3.6.1. Interception Capacities

The interception capacity of a combination inlet on a continuous grade consisting of a grate and curb-opening of equal length placed side-by-side is no greater than that of the grate alone. Therefore, the capacity is then determined by neglecting the curb opening.

A sweeper combination inlet on a continuous grade has the interception capacity equal to the sum of the grated catch basin and the portion of the curb opening inlet upstream of the grate.

The interception capacity of a combination inlet in a sump is essentially equal to that of the grate only for weir flow unless the grate becomes clogged. For orifice flow in a sump, the capacity is equal to the capacity of the grate plus the capacity of the curb opening.

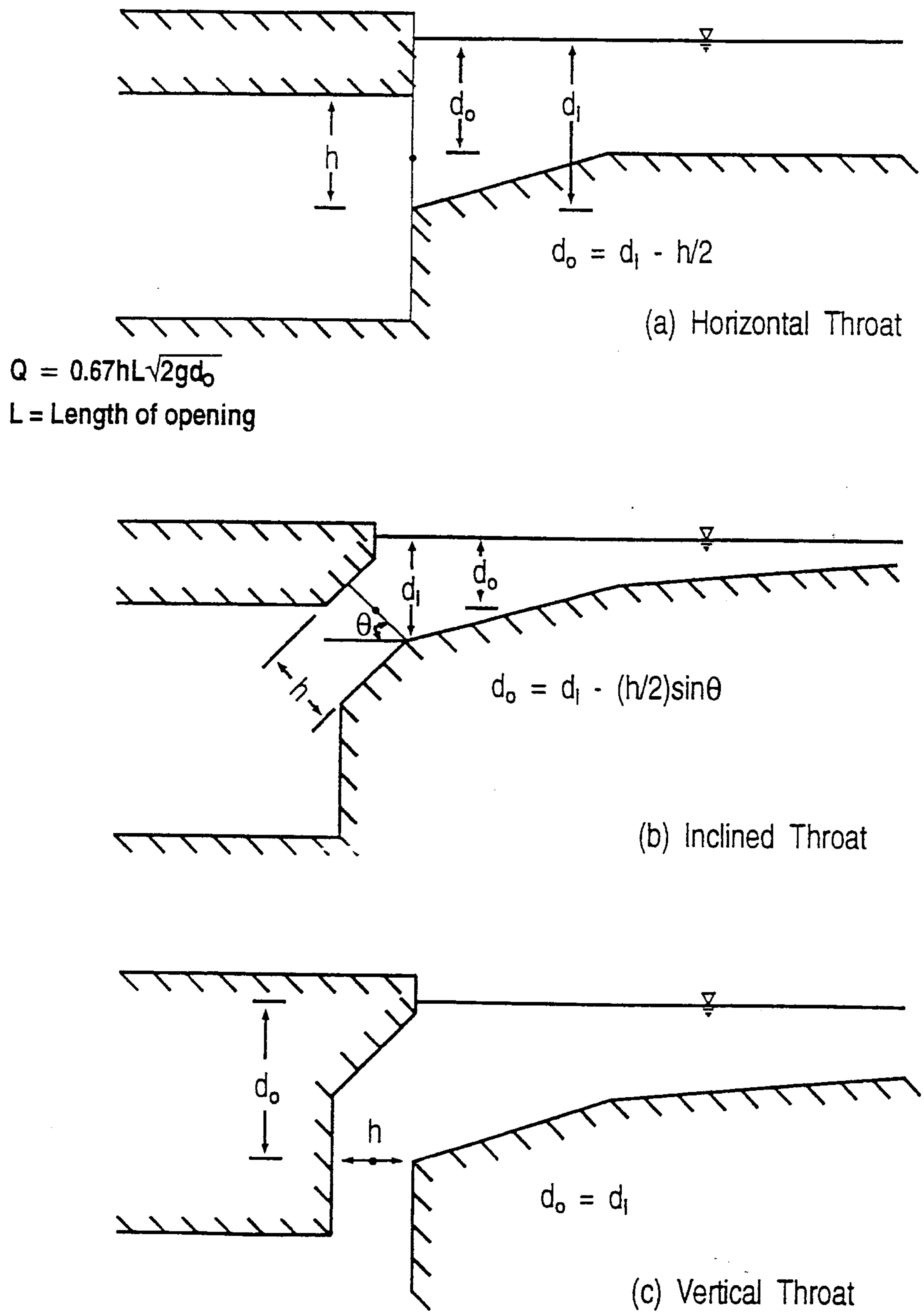


Figure 6-9: Curb Opening Catch Basin Inlets
Modified From: USDOT, FHWA, HEC-12, Fig. 21, 1984

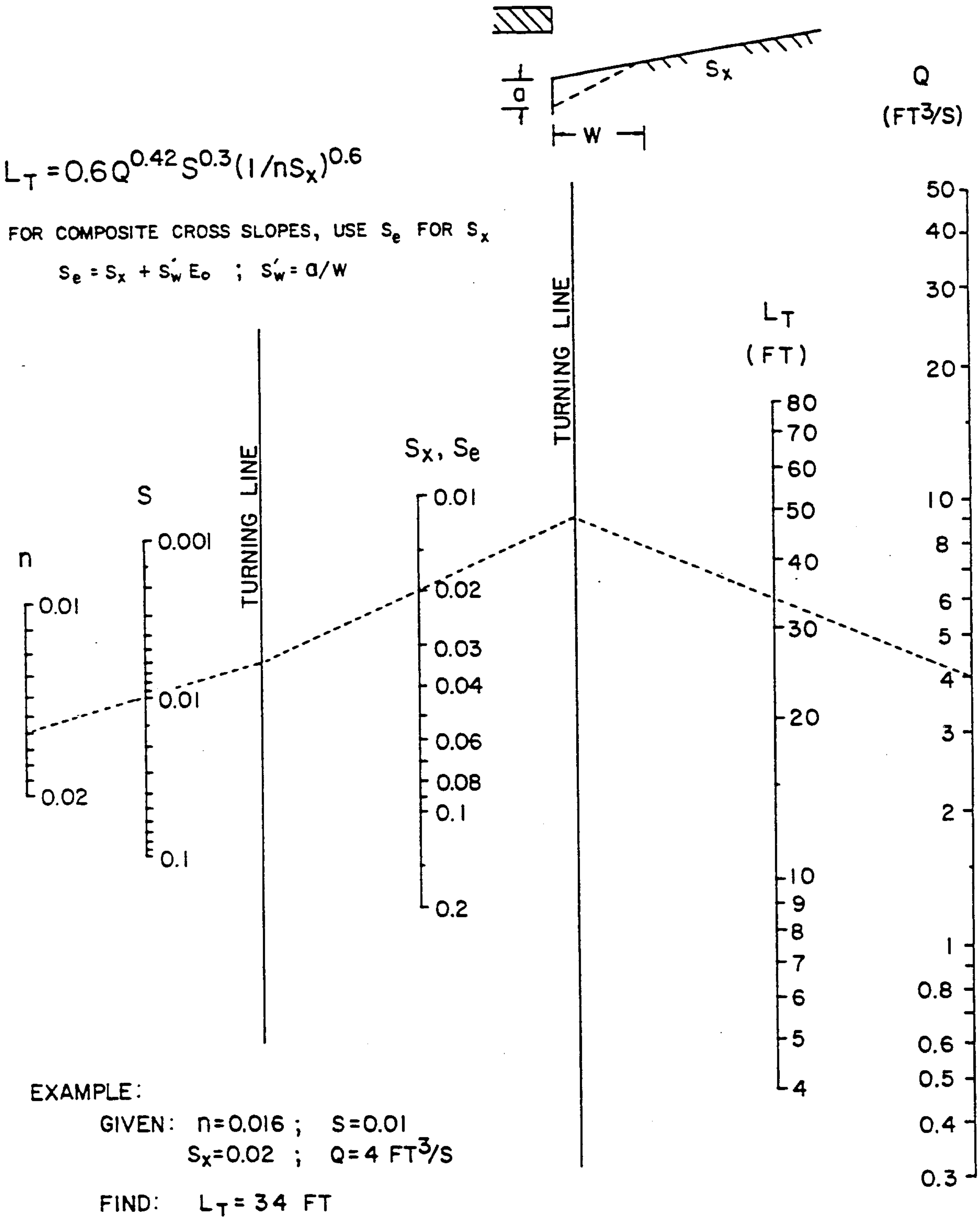


Figure 6-10: Curb-Opening And Slotted Drain Inlet Length For Total Interception

Source: FHWA, HEC-12, 1984

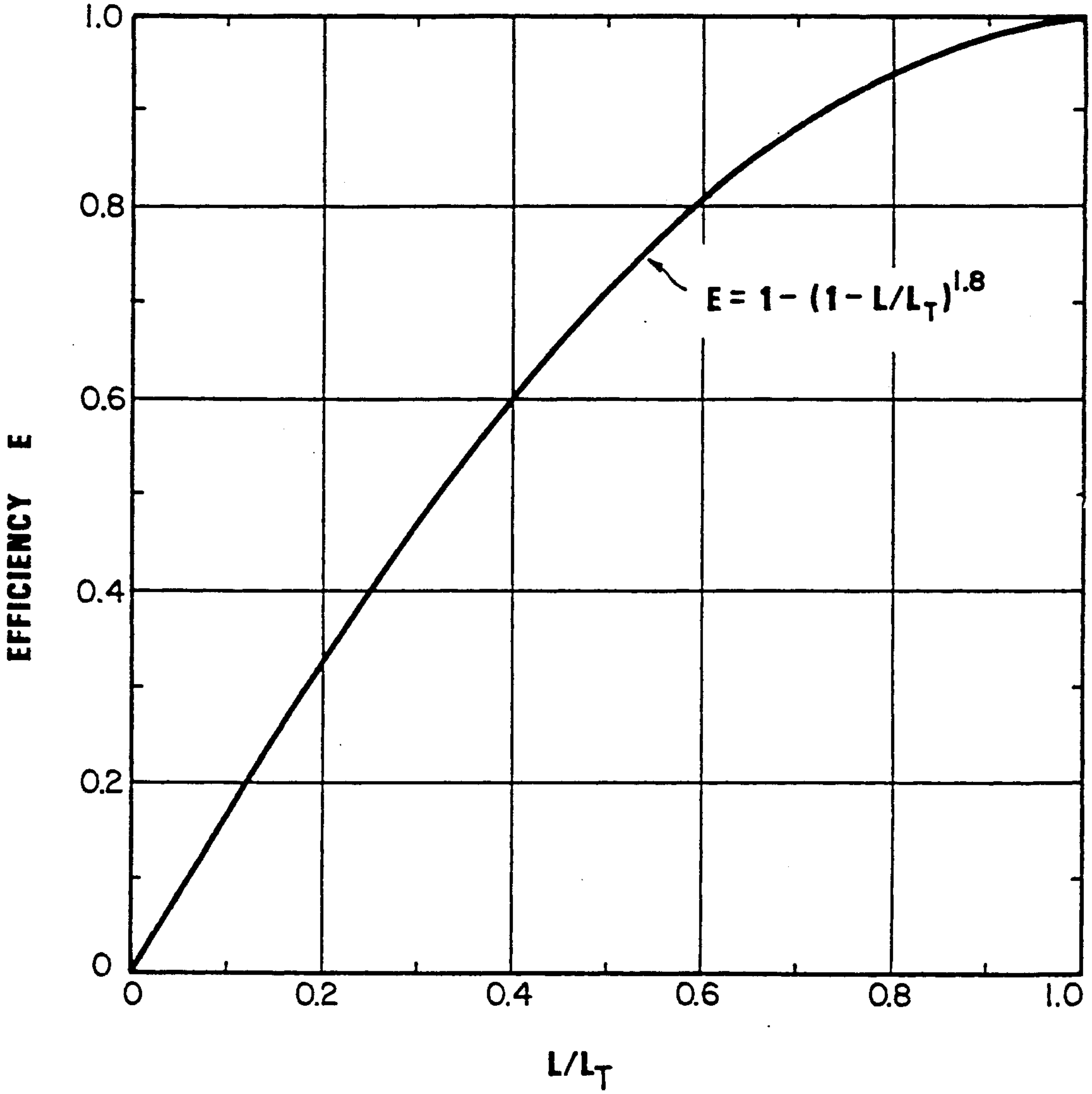


Figure 6-11: Curb-Opening and Slotted Drain Inlet Interception Efficiency
Source: FHWA, HEC-12, 1984

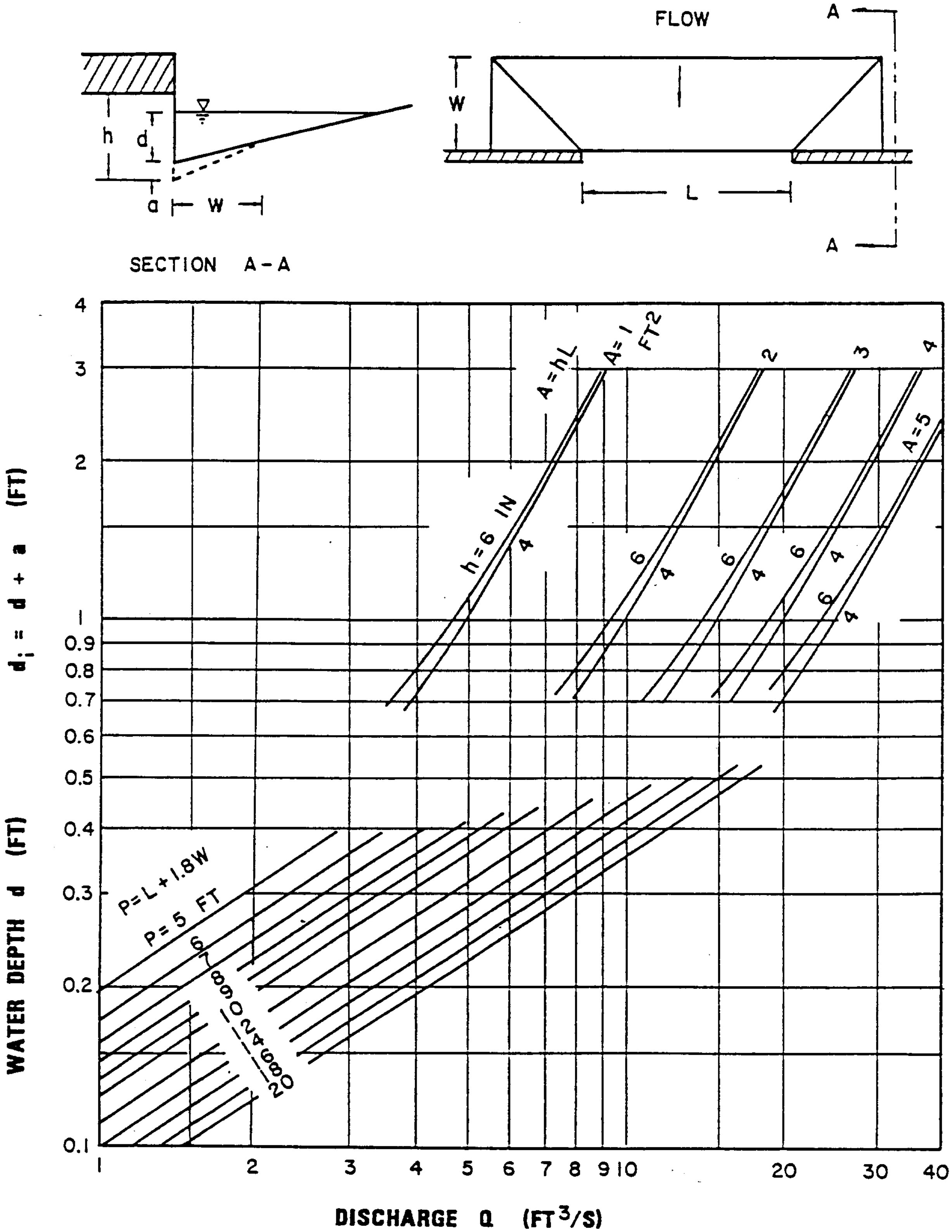


Figure 6-12: Depressed Curb-Opening Inlet Capacity In Sump Locations

Source: FHWA, HEC-12, 1984

6.3.7. Slotted Drain Inlets

Slotted drain inlets are only permitted for special circumstances, as approved by the Stormwater Manager, where standard drainage inlets will not suffice. Slotted drain inlets on a longitudinal grade function best when designed and constructed as a curb opening inlet and when debris is not a factor. Slotted inlets for public storm drains are not permitted in sump locations due to high clogging potential and sediment deposition problems in the pipe section.

Transverse slotted drains function best for shallow low velocity sheet flow (e.g. in parking lots) but are not recommended for more concentrated flow due to the small opening width and low splash over velocity threshold.

Slotted drain inlets, when used, shall be set in concrete for vehicular loading and to maintain constant grade. They should be accessible at both end of the pipe for maintenance/cleaning. This may be accomplished by extending the pipe ends beyond the area requiring the slotted drain.

6.3.8. Flanking Inlets

In addition to inlets required in sump locations, it is good engineering practice to place flanking inlets on each side of the low point when a depressed area has no outlet except through the storm drain system. The purpose of flanking inlets is to act as a relief for the inlet at the sump should it become clogged or if the design spread is exceeded.

Flanking inlets can be located so they will function before the allowable spread is exceeded at the sump and should be located so that they will capture all of the flow when the primary inlet at the sump is clogged.

If the flanking inlets are the same dimension as the primary inlet, each will intercept one-half of the design flow when they are located so that the depth of ponding at the flanking inlets is sixty-three percent (63%) of the depth of ponding at the low point. If the flanking inlets are not the same size as the primary inlet, it will be necessary to either develop a new factor or perform a trial and error solution using assumed depths with the weir equation to determine the capacity of the flanking inlet at the given depths.

Table 6-4 shows the spacing required for various depths at curb criteria and vertical curve lengths. The vertical curve length (K) is defined as:

$$\mathbf{K = L/(G_2 - G_1)} \qquad \mathbf{(6.19)}$$

where: L = the length of the vertical curve in feet,
G_1/G_2 = are the approach grades.

6.3.8.1. Flanking Inlet Location Example

Given: A 491.8 ft (L) sump vertical curve at an underpass on a 5-lane Type 1 roadway. Beginning and ending approach grades are -2.50% and +2.50% respectively. The spread at the design Q is not to exceed 10 ft.

Find: The location of the flanking inlets if located in relief of the inlet at the low point when it is clogged.

Solution: Step 1. Find the rate of vertical curvature, K.

$K = L/(S_{begin} - S_{end})$
$K = 491.8/(-2.50 - 2.50) = 98.4$ ft.

Step 2. Determine depth at design spread.

$d = S_x T = (0.02)(10.0\text{ ft.})$
$d = 0.22$ ft.

Step 3. Determine the depth for the flanker inlets.

$d = 0.22\text{ ft.}(0.63) = 0.14$ ft.

Step 4. For use with Table 6-4 or x; d = 0.22 - 0.14 = 0.07 ft.

Inlet spacing $= x = [200(0.07)98.4]^{0.5} = 37$ feet.

TABLE 6-4: DISTANCE TO FLANKING INLETS

K (Feet)

d(ft)	20	30	40	50	70	90	110	130	160	167
0.03	11	13	15	17	20	23	26	28	31	32
0.06	15	19	22	24	29	33	36	39	44	45
0.1	20	24	28	32	37	42	47	51	57	58
0.2	28	35	40	45	53	60	66	72	80	82
0.3	35	42	49	55	65	73	81	88	98	100
0.4	40	49	57	63	75	85	94	102	113	116
0.5	45	55	63	71	84	95	105	114	126	129
0.6	49	60	69	77	92	104	115	125	139	142
0.7	53	65	75	84	99	112	124	135	150	153
0.8	57	69	80	89	106	120	133	144	160	163

NOTES:
1) $x = (200dK)^{0.5}$, where x = distance from low point
2) d = depth at curb in feet (does not include sump depth)
3) Maximum k = 167

6.3.9. Inlet Clogging

The inlet reduction factors outlined in Table 6-5 shall be used to the increase the calculated inlet length or area to accommodate clogging from debris, pine needles, snow/ice, etc.

TABLE 6-5: INLET CLOGGING FACTORS

INLET TYPE	DESIGN CONDITION	CLOGGING FACTOR (%)
Grate	Sump	50
Grate	Continuous Grade	50
Curb Opening	Sump	20
Curb Opening	Continuous Grade	20
Combination, sweeper	Sump	35
Combination, equal length	Sump	50
Combination, equal length	Continuous Grade	50
Combination, sweeper	Continuous Grade	50 - grate; 20 - curb opening
Slotted Drain	Continuous Grade	20

6.4. INLET SPACING PROCEDURE ON CONTINUOUS GRADE

Allowable design spread is the governing criterion for locating storm drain inlets on a continuous grade. The interception of the upstream inlet will dictate the initial spread and as flow is contributed to the gutter section in the downstream direction, the spread increases. The next downstream inlet is located at the point where the actual spread equals the allowable spread. The spacing of the inlets is a function of the amount of upstream carryover flow, the intermediate tributary drainage area, and the gutter geometry.

After determining those locations where inlets are necessary as outlined in Section 6.3.3, inlet spacing can be computed by the following procedure and using the inlet computation sheet given in Figure 6-12:

Step 1 Complete the blanks at the top of the sheet to identify the job by project name, project #, street name, and designer's initials.

Step 2 Mark on the plan sheet(s) the location of inlets which are necessary for 100% interception as outlined in Section 6.3.3.

Step 3 Start at the high point, at one end of the job if possible, and work towards the low point. Begin at the next high point and work backwards toward the same low point.

Step 4 To begin the process, select a trial drainage area approximately 300 ft to 500 ft long below the high point and outline the area on the plan. Include any area which may drain over the curb, onto the street. Drainage from large areas behind the curb must be intercepted before it reaches the roadway.

Step 5 In Columns 1 & 2, record the proposed inlet by number and station. Identify the curb and gutter type in column 19 (remarks). A drawing of the street cross-section should be supplied.

Step 6 Compute the drainage area outlined in Step 4 and record in Column 3.

Step 7 A runoff coefficient of 0.90 will be used, enter in Column 4.

Step 8 Compute the time of concentration, T_c, for the first inlet and enter in Column 5. The minimum T_c is five minutes.

Step 9 Determine the rainfall intensity based on the time of concentration, T_c. Enter in Column 6.

Step 10 Compute the discharge in the gutter using $Q = CIA\ C_f$ and enter in Column 7.

Step 11 From the roadway profile, enter the longitudinal slope, S_L, at the inlet, into Column 8, taking into account any superelevation.

Step 12 From the street cross-section enter the cross slope, S_x, in Column 9 and the grate or gutter width, W, in Column 13.

Step 13 For the first inlet in the series, enter the value from Column 7 into Column 11, since there in no previous bypass flow. Also enter a 0 in Column 10 for the first inlet.

Step 14 Determine the spread, T, by using Equations 6.1 or Figure 6-1 and enter in Column 14. Also, determine the depth at the curb, d, by multiplying the spread by the appropriate cross-slope, and enter in Column 12. Compare the calculated spread to the allowable spread. Additionally, compare the depth at the curb with the actual curb height. If the calculated spread (Col. 14), is near the allowable spread and the computed depth is less than the curb height, continue on to Step 15. If not, expand or decrease the drainage area up to the first inlet to increase or decrease the spread as needed. Repeat Steps 6 through 14 as needed.

Step 15 Calculate W/T (Col. 13/Col. 14) and enter the value in Column 15.

Step 16 Select the inlet type and dimensions and enter in Column 16.

Step 17 Calculate the flow intercepted by the inlet (grate), Q_i, and enter in column 17. Approximately 75 percent of carryover flow should be intercepted for maximum design efficiency.

Step 18 Determine the bypass flow, Q_b, and enter in Column 18.

Step 19 Proceed to the next inlet downstream. The next downstream drainage area should be long and more or less of uniform width. Using an assumed time of concentration of five (5) minutes and a runoff coefficient of 0.90, compute the flow in the gutter and enter in Column 7. Using the drainage area from Step 5 which produces the allowable spread, divide this area by the average drainage area width to estimate the approximate length to next downstream inlet.

Step 20 Record the previous bypass value from Column 18 in Column 10. Determine the total gutter flow by adding Columns 7 and 10 and enter in Column 11.

Step 21 Determine the spread and depth at the curb as outlined in Step 14. Repeat Steps 18 through 21 until the spread and depth at the curb are within the design criteria.

Step 22 Select inlet type and enter in Column 16.

Step 23 Determine the intercepted flow by subtracting Column 17 from Column 11. This completes the spacing design for the inlet.

Step 24 Repeat Steps 19 through 23 for each subsequent inlet down to the low point.

INLET SPACING COMPUTATION SHEET

Date: ______ **Street** ______
Computed By: ______ **Sheet** ___ **of** ___

INLET		GUTTER DISCHARGE Design Frequency ______					GUTTER DISCHARGE Allowable Spread ______								INLET DISCHARGE			RMK
No.	Stat.	Drain. Area A	Run-off Coeff. C	Time of Conc. t_c	Rain. Inten. I	$Q = CIAC_f$	Long. Slope S_L	Cross Slope S_X or S_W	Prev. by-pass Flow	Total Gutter Flow	Depth d	Grate or Gutter Width W	Spread T	W/T	Inlet Type	Inter-cept Flow Q_i	by-pass Flow Q_b	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)

Figure 6-13: Inlet Spacing Computation Sheet

CHAPTER 7: STORM DRAINS

This chapter presents policies and criteria for the design and construction of public storm drain systems. Private storm drain systems should also be designed in accordance with this chapter to ensure continuity with public systems. Procedures for sizing storm drains and computing the energy losses and hydraulic grade line through a storm drain system are also presented.

Storm drains are generally that portion of a roadway drainage system that are designed to collect surface water through drainage inlets and convey the water through closed conduits to an outfall. The conduit system is comprised of different lengths, shapes, and sizes of storm drain pipes which are connected by appurtenant structures such as manholes, junctions, or other miscellaneous structures. A section of conduit connecting one inlet or appurtenant structure to another is termed a "segment". Storm drain system outfalls will typically be open channels, storm drain main lines, detention facilities, or other bodies of water.

7.1. POLICIES

a. All offsite runoff from whatever source must be taken into account in the design of a storm drain system if such runoff could affect the street that the storm drain is serving.

b. Storm drain systems serving collector and arterial streets must keep one twelve (12) foot lane of traffic open in each direction for the 10-year design storm and the 100-year storm within the right-of-way. Storm drain systems for local streets must keep the 10-year design flow between the curbs.

c. The minimum design frequency for all public storm drain facilities shall be the 10-year design storm. The 100-year design storm may govern in some downstream reaches of a system or for storm drains with larger contributing watersheds. In all cases, both design storms should be checked to determine which condition governs.

d. The minimum easement width for public storm drains thirty-six (36) inches in diameter or less shall be sixteen (16) feet. For multiple pipe installations or pipe diameter greater than 36 inches in diameter the easement width shall be the conduit width(s) plus eight (8) feet on each side of the conduit.

e. The minimum acceptable diameter for any public storm drain pipe is eighteen (18) inches or equivalent arch pipe. Main-line storm drains should be at least twenty-four (24) inches in diameter.

f. New storm drains and manholes shall not be located under existing or future curb and gutter or sidewalk, whenever possible.

g. When connecting into an existing storm drain system, the existing storm drain systems shall be analyzed to determine available capacity.

7.2. STORM DRAIN DESIGN CRITERIA

7.2.1. Design Velocity and Slope

In general, storm drain slopes and velocities should increase in the downstream direction progressively throughout the length of the system. The minimum allowable storm drain slope for any storm drain pipe shall be 0.5 percent or the slope which will produce a velocity of three (3) feet per second for the pipe flowing full, whichever is greater. Slopes less than 0.5% require special approval by the Stormwater Manager.

Desirable minimum velocity is five (5) feet per second, however all storm drains shall be designed such that the minimum self-cleaning velocity will be three (3) feet per second flowing full. This criteria results in a minimum flow velocity of two (2) feet per second at a flow depth equal to twenty-five (25) percent of the pipe diameter. The minimum slopes necessary to ensure a velocity of 3 ft/sec in storm drains can be calculated by Equation 7.1 below:

$$\mathbf{S = [(n\ V)^2] / [2.208\ R^{4/3}]} \qquad \mathbf{(7.1)}$$

where:
- S = the slope of the hydraulic grade line, ft/ft
- n = Manning's roughness coefficient
- V = the mean velocity (3 ft/sec)
- R = the hydraulic radius, ft.

The following relative flow conditions for different depths in a circular pipe should also be noted:

1. Peak flow occurs at 93 percent of the height of the pipe. This means that if a pipe is designed for full flow, the design will be slightly conservative.
2. The velocity in a pipe flowing half-full is the same as the velocity for full flow.
3. Flow velocities for flow depths greater than full flow are greater than velocities at full flow.
4. As the depth of flow drops below half-full, the flow velocity drops off rapidly.

7.2.2. Alignment

Storm drains shall be straight, with uniform slopes between manholes, whenever possible. Curved storm drains may be permitted when long radius curves are necessary to conform to street layout, however, storm drains smaller than four (4) feet in diameter should not be designed with curves. Long radius bends are available from many suppliers and are preferred as a means of changing direction in storm drains four (4) foot in diameter or larger, unless a manhole is required. The radius of curvature specified should coincide with standard curves available in the type of material utilized. The minimum radius shall not be less than 100 feet.

7.2.3. Storm Drain Conduit Size

The minimum pipe diameter for public storm drains shall be eighteen (18) inches in diameter. The use of elliptical or arched pipe for storm drains is not recommended and must be approved by the Stormwater Manager prior to use. Storm drain pipe sizes shall increase in the downstream direction. Decreasing the pipe size in the downstream direction is not permitted even for flow on a steeper slope or pressure profiles.

7.2.4. Storm Drain Conduit Material Selection

Factors such as life expectancy, durability, physical strength, depth of cover, joint tightness, hydraulic performance, ease of handling, installation costs, and maintenance should all be considered in the selection of storm drain materials to maximize performance and cost effectiveness. First cost shall not be used as the determining factor.

Permissible pipe materials for public storm drain systems are:

1. Corrugated Metal Pipe (CMP) - 14 Ga., Annular, Aluminized Steel Type 2 per MAG Section 760.
2. CMP - 14 Ga., Helical Corrugated, Aluminized Steel Type 2 per MAG Section 760.
3. Rubber Gasket Reinforced Concrete Pipe (RGRCP) per Subsection 7.2.4.1.
4. Spiral Rib Metal Pipe (SRP) per Subsection 7.2.4.2.
5. High Density Polyethylene Pipe (HDPE) per Subsection 7.2.4.3.
6. Reinforced Concrete Box Culvert (RCB) per Subsection 4.2.2.2.

Standard CMP joints shall be either rivet lap joint construction (annular corrugations) or continuous lock or welded seam (helical corrugations) and wrapped with non-woven geotextile filter fabric or "O" ring gaskets.

All storm drain conduit shall be of sufficient structural strength to withstand AASHTO HS-20-44 loading at a minimum. Special designs may be required depending on loading requirements and depths of backfill.

7.2.4.1. Reinforced Concrete Pipe (RCP

The service life of most RCP can be 75 years or longer, depending on factors such as corrosion, abrasion, and freeze-thaw. RCP is available in diameters ranging from 18 - 144 inches and in typical manufactured lengths of 7.5 - 8 feet.

Non-reinforced concrete pipe is not permitted for public storm drain systems.
All RCP shall be a minimum of Class III under public roadways and shall be manufactured in accordance with the following standards:

- MAG Section 735 - Reinforced Concrete Pipe.
- ASTM C76 - Reinforced Concrete Culvert, Storm Drain and Sewer Pipe.
- ASTM C443 - Reinforced Low-Head Concrete Pressure Pipe.
- ASTM C443 - Joints for Circular Concrete Sewer and Culvert Pipe, Using Rubber Gaskets
- ASTM C665 - Reinforced Concrete D-Load Culvert, Storm Drain and Sewer Pipe.

Material standards for concrete aggregate, steel reinforcing, Portland cement, and gaskets are also referenced in the above specifications. RCP should be designed for each individual project. Indirect design is typically used and is presented in the Concrete Pipe Handbook (SAMM or 3EB) prepared by the American Concrete Pipe Association.

The maximum allowable velocity for RCP shall be 20 ft/sec. Velocities greater than 20 ft/sec. may require increases in the compressive strength of the concrete, increases in specific hardness of the concrete aggregate, increased the cover over the reinforcing steel, or providing plastic lining.

The minimum allowable cover for RCP shall be one (1) foot from top of pipe to top of subgrade. The maximum allowable cover for Class III RCP installed in clay soils shall be 11 feet for 18 - 42 inch RCP; 12 feet for 48 -78 inch RCP; and 13 feet for 84 - 144 inch RCP.

Joints for RCP shall be bell and spigot ends with O-ring rubber gaskets conforming to MAG Section 765 to provide a watertight joint.

7.2.4.2. Spiral Rib Steel Pipe (SRP

A service life of SRP can be 50 years or longer, depending on corrosion and abrasion conditions. Available manufactured diameters are typically 18 - 102 inches and in laying lengths of 4 - 40 feet.

The minimum pipe thickness shall be 14 gauge for pipe diameters of 18 - 60 inches and 12 gauge for pipe diameters of 60 - 72 inches. SRP with diameters greater than 72 inches will require structural design to determine adequate gauge thickness.

Materials for SRP shall meet the following standards:

- AASHTO M274 - Steel Sheet, Aluminum Coated (Type 2) for Corrugated Steel Pipe.
- ASTM A819 - Steel Sheet, Aluminum Coated (Type 2) for Storm Sewer and Drainage Pipe.

Pipe shall be manufactured in accordance with the following standards:

- AASHTO M36 - Corrugated Steel Pipe, Metallic-Coated, for Sewers and Drains.
- ASTM A760 - Corrugated Steel Pipe, Metallic-Coated, for Sewers and Drains.
- MAG Section 760 - Coating Corrugated Metal Pipe and Arches.

SRP shall be designed in accordance with the following standards:

- AASHTO Standard Specification for Highway Bridges, Section 12 - Soil-Corrugated Metal Structure Interaction Systems.
- ASTM A796 - Structural Design of Corrugated Steel Pipe, Pipe Arches, and Arches for Storm and Sanitary Sewers and Other Buried Structures.

The minimum allowable cover for SRP, from top of pipe to top of subgrade, shall be one (1) foot for 18 - 48 inch diameter pipe; 1.5 feet for 54 - 72 inch pipe; and two (2) feet for over 72 inch diameter pipe. The maximum allowable cover shall be 30 feet for pipe diameters of 18 - 54 inches; 28 feet for 60 inch pipe; 26 feet for 66-inch; 24 feet for 72-inch; and 22 feet for 78-inch or greater.

Joints for SRP shall be coupling bands conforming to AASHTO M36 with O-ring gaskets to produce a watertight joint. Coupling bands shall be a minimum of 10.5 inches wide and shall be made from aluminized steel of the same thickness as the pipe. Hardware for coupling bands shall conform to AASHTO M36 and rubber gaskets shall meet the requirements of AASHTO M198.

7.2.4.3. High Density Polyethylene Pipe (HDPE

A service life of HDPE can be as high as 75 years, depending on such factors as photo degradation, oxidative degradation, and slow crack growth under tensile stressing. However, HDPE conduit does not have a long history of use for storm drain, particularly in the Flagstaff area.

Parameters for use of HDPE for public storm drains are as follows:

1. Pipe diameters between 18 and 36 inches.
2. The minimum allowable cover for HDPE shall be two (2) feet or one (1) pipe diameter, whichever is greater, from top of pipe to top of subgrade.
3. The maximum allowable cover shall be ten (10) feet.

HDPE pipe shall meet the following standards:

- MAG Sections 601, 603, and 738.
- ASTM F-894.

HDPE pipe shall be designed so that deflections are limited to five (5) percent. Deflections should be determined using the Modified Iowa Deflection Formula.

Joints for HDPE shall be bell and spigot type joints and elastomeric gaskets to provide a watertight joint. Split couplings shall not be used. Joints shall meet AASHTO M294 standards. Mandril testing may be required at the discretion of the Stormwater Manager.

7.2.5. Separation Requirements

Installation and backfill requirements for public storm drains shall be in accordance with City of Flagstaff Engineering Design & Construction Standards.

Vertical and horizontal separation requirements for storm drain conduit to waterlines shall be the same as for sewer pipes per the City of Flagstaff Engineering Design and Construction Standards & Specifications.

The minimum clearance between storm drains and all other dry underground utilities shall be twelve (12) inches and shall cross at angles greater than forty-five (45) degrees, if possible. If 12 inches of separation cannot be maintained, one of the pipes must be encased in concrete.

Crossings of open channels may require concrete encasement to minimize damage to the pipe if adequate cover (24" minimum) cannot be obtained or scour is anticipated.

7.2.6. Storm Drain Outfalls

All storm drain systems will have an outfall where the flow is discharged into either a natural watercourse, artificial channel, another storm drain system, or other drainage facility. Several aspects of storm drain outlet design must be given consideration, including but not limited to the invert of the storm drain outlet, tailwater elevation(s), type of receiving watercourse, orientation of the outlet, and local scour.

If the outfall is a wash or stream, it may be necessary to consider the coincidental probability of two hydrologic events occurring at the same time. There may be instances where excessive tailwater causes flow to back up in the storm drain system and possibly cause surcharging out inlets and manholes. The invert of the storm drain outlet shall be a minimum of one (1) foot above the channel invert at the same point, whenever possible.

The tailwater depth at the storm drain outlet must be considered carefully for purposes of evaluating the hydraulic grade line. See Section 7.7.3 for guidance on determining the proper tailwater elevation.

Storm drains that discharge into open channels shall be provided with an appropriate headwall/wingwall. Projecting outlets are not permitted. The orientation of a storm drain outlet into a wash or channel should be positioned so the discharge is pointed in the downstream direction. This will reduce turbulence and the potential for local scour. If the outlet is perpendicular to the direction of flow in the receiving channel, erosion of the opposite channel bank must be considered and a channel bank lining of riprap or other appropriate material will be required. An energy dissipator may be required if outlet velocities warrant.

7.3. MANHOLES AND JUNCTION STRUCTURES

The primary function of a storm drain manhole is to provide access to the storm drain system for inspection and maintenance. Manholes also serve as flow junctions and can provide ventilation or pressure relief for the storm drain system. Catch basins can also serve as access holes and can be used instead of manholes in some cases to provide the benefit of extra stormwater interception.

7.3.1. Location and Spacing

Manhole location and spacing criteria has been developed primarily for storm drain maintenance requirements. At a minimum, manholes are required for the following locations:

1. At junctions where two or more storm drains converge, excepting laterals from adjacent catch basins,
2. at vertical deflections,
3. changes in pipe size, and
4. at horizontal alignment changes as outlined below:

PIPE DIAMETER (INCHES)	DEFLECTION
18 - 42	$\geq$ 22½ degrees
42 and up	$\geq$ 45 degrees

5. Manholes may also be required by the Stormwater Manager at other locations to facilitate maintenance.

Manholes at vertical deflections shall be at or as close as practical to the point of deflection, with allowance for manufactured bends. If the manhole is not at the point of deflection, it shall be located immediately upstream of the deflection.

In addition to the above criteria, manholes will be required at intermediate points along long runs of storm drain in accordance with the criteria outlined in Table 7-1.

TABLE 7-1: Manhole Spacing Criteria

PIPE DIAMETER (INCHES)	MAXIMUM DISTANCE (FEET)
18 - 24	300
27 - 36	400
42 and up	500

If possible, manholes shall not be located in traffic lanes. However, if it is not possible to avoid locating a manhole in a traffic lane, every effort shall be made to avoid locating it within a street intersection and/or the vehicle wheel path.

7.3.2. Manhole Configurations

Typical manhole configurations are illustrated in Figure 7-1. Storm drain manholes shall be constructed in accordance with current adopted MAG Standard Details. Where storm drains are too large (60" and up) to reasonably accommodate MAG concrete manhole structures, a vertical riser or prefabricated "tee" to the storm drain may be used with prior approval from the Stormwater Manager and special design considerations.

A pressure manhole shaft and pressure frame and cover is required whenever the design hydraulic grade line elevation at the manhole is within twelve (12) inches of the adjacent ground elevation.

To differentiate storm drain manholes from sewer or communication conduits, the manhole cover shall have the words "STORM DRAIN" cast into the top surface of the cover in accordance with MAG Standard Detail 424 lettering requirements.

Manhole depths shall be determined by the storm drain profile and surface topography. Common depths range from five (5) to thirteen (13) feet. Manholes which are shallower or deeper may require special design considerations. Deep manholes (greater than 12 feet) shall be five (5) feet in diameter and must be designed to withstand soil pressures. If a manhole will extend below the water table, it must also be designed to withstand hydrostatic pressure and/or seepage. Proper access steps or built-in ladders must be provided for manholes deeper than two (2.0) feet.

Manhole shafts shall be sixty (60) inches in diameter for storm drain pipes thirty-six (36) inches in diameter or greater.

7.3.3. Manhole Shaping

Figure 7-2 depicts several types of manhole shaping configurations which have been found to be efficient in reducing head losses at junctions and bends. Flow channels and benches (see Figure 7-1) may also be used to decrease losses and reduce unnecessary turbulence in the manhole. Benching should only be used when the hydraulic grade line is relatively flat and there is no appreciable head available. Typically, the slopes of a storm drain system do not require the use of benches to hold the hydraulic grade line in the correct place.

A minimum drop of 0.10 foot is required through all storm drain manholes. A drop of 0.3 feet is required for a manhole with two contributing laterals, if possible. Where a storm drain changes direction through a manhole without increasing in size, a drop of 0.4 feet is required, if possible. For pipes flowing full, the crown of the outlet pipe should be set below the crown of the inlet pipe by the amount of the loss in the manhole. This is referred to as "hanging the pipe on the hydraulic grade line".

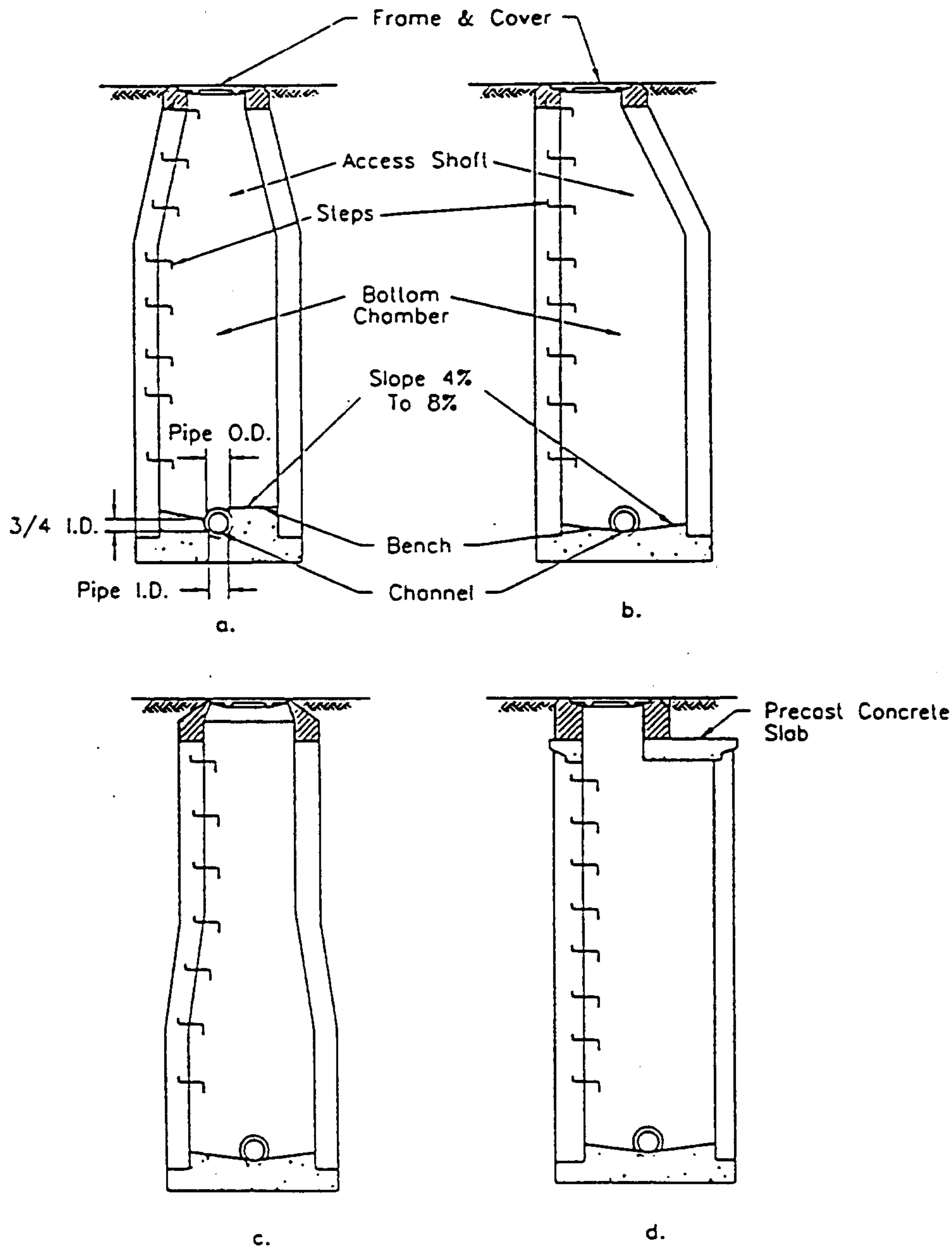


Figure 7-1: Typical Manhole Configurations

If possible, laterals entering a manhole shall not be aligned opposite one another. A deflection or lateral offset may be required to achieve this. Lateral inflow pipes entering a main line storm drain (without a manhole) shall not be aligned opposite each other, but should be separated laterally by at least two lateral pipe diameters.

7.4. MAINTENANCE CONSIDERATIONS

It is essential that maintenance be considered during both the design and construction of storm drain systems. Common maintenance problems associated with storm drains include debris, sedimentation, scour, piping, roadway or embankment settlement, and structural damage to the conduit. The likelihood of scour and abrasion inside the conduit should also be considered during design. Access for inspection and maintenance of storm drains as well as drainage inlets must also be considered.

Clearing accumulated debris and sediment from storm drain and inlets is a routine maintenance requirement for any facility owner, however this problem is often overlooked during construction and adequate sediment/erosion control precautions should be undertaken.

Piping, roadway or embankment settlement, and structural damage problems, when they occur, are usually attributed to poor construction practices and can be avoided through proper design, installation specifications, and inspections.

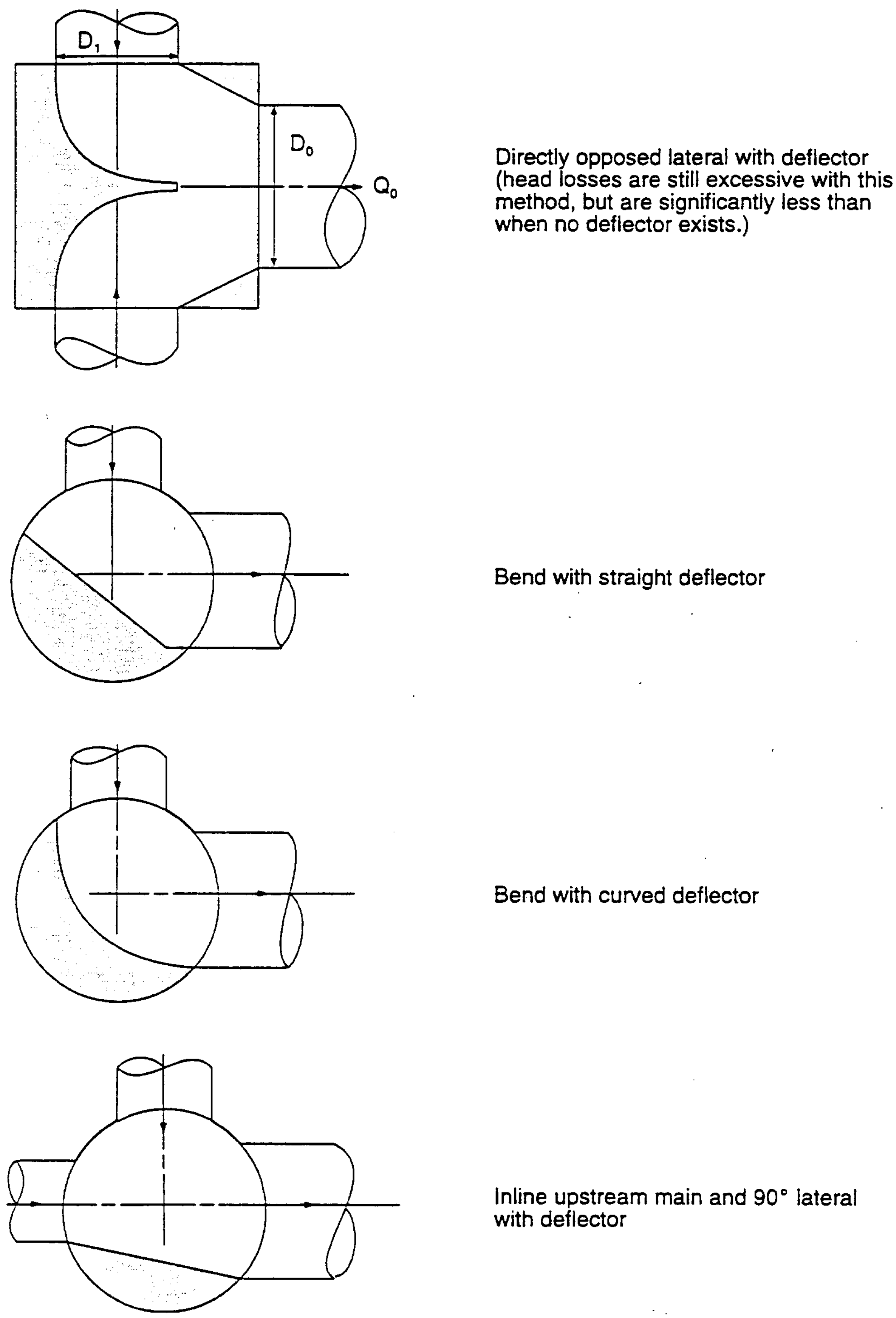


Figure 7-2: Efficient Manhole Shaping Configurations
Source: University of Missouri

7.5. STORM DRAIN HYDRAULICS

The design procedures presented in this section assume that flow within each storm drain segment is steady and uniform, meaning that the discharge and depth of flow in each segment are assumed to be constant with respect to time. The average velocity throughout each segment is also considered to be constant.

In actual storm drain systems, the flow at each inlet is variable and flow conditions are not truly steady or uniform. However, since the usual hydrologic methods used in storm drain design are based on computed peak discharges at the beginning of each segment, it is a conservative practice to design using the steady uniform flow assumption.

7.5.1. Open Channel vs. Pressure Flow

There are two design conditions for designing storm drains under the steady uniform flow assumption: (1) open channel or gravity flow, and (2) pressure flow.

For ordinary conditions, all public storm drains systems shall be designed and sized based on the assumption that they will flow full or practically full under the design discharge and will not be placed under a pressure head. This is a conservative approach since the peak flow actually occurs at 93% of full flow. Due to pipe size constraints, particularly at the upper end of a system, full flow design may not always be possible and open channel flow will occur.

7.5.1.1. Open Channel Flow

For open channel flow to occur, the storm drain must be sized so that the water surface within the pipe remains open to atmospheric pressure and the flow energy is derived from the flow velocity, depth, and elevation.

Designing a storm drainage system for open channel flow may result in conduit sizes larger than those of pressure flow. However, this design practice provides a factor of safety by providing additional flow capacity for increases in flow above the design discharge. This factor of safety is often desirable since the methods of peak discharge estimation are not exact, and once in place, storm drains are costly to upsize and replace.

7.5.1.2. Pressure Flow

Pressure flow design requires that the flow in the storm drain be at a pressure greater than atmospheric. The flow energy is also derived from flow velocity, depth, and elevation, however the pressure head will be above the top of the conduit and will not equal the depth of flow in the conduit. Under pressure flow, the pressure head rises to the level represented by the hydraulic grade line (see Section 7.7), which is the level to which water would rise in a vertical tube (e.g., manhole) at any

point along the storm drain.
There are situations where pressure flow design is desirable. For example, it may be necessary to use an existing system which must be placed under pressure flow to accommodate the proposed design flows. In some cases, the large grade changes typical in the Flagstaff area will create pressure flow situations. When pressure flow is justified, the design hydraulic grade line must be a minimum of twelve (12) inches below the finished grade, gutter flowline elevation, or grate invert elevation at inlets, whichever is lower, when all energy losses are considered. Caution should be taken to ensure that the pressure within the pipe does not exceed the manufacturer's maximum safe limits for joints and seals.

7.5.2. Hydraulic Capacity

The hydraulic capacity of storm drain conduit with circular cross sections flowing full can be computed with Manning's Equation as follows:

$$\mathbf{V = [1.486\, R^{2/3}\, S^{1/2}] / n} \tag{7.2}$$

where:

V	= mean velocity of flow, ft/s
R	= hydraulic radius (area/wetted perimeter)
S	= slope of the hydraulic grade line, ft/ft
n	= Manning's roughness coefficient

In terms of discharge, the above equation then becomes:

$$\mathbf{Q = [1.486\, A\, R^{2/3}\, S^{1/2}]\, n} \tag{7.3}$$

where:

Q	= rate of flow, cfs
A	= cross sectional area of flow, ft^2

For a conduit flowing full, the above equations become:

$$\mathbf{V = [0.590\, D^{2/3}\, S^{1/2}] / n} \tag{7.4}$$

$$\mathbf{Q = [0.463\, D^{8/3}\, S^{1/2}] / n} \tag{7.5}$$

where: D = diameter of the pipe, ft

Manning's Equation can also be written to determine friction losses for storm drain pipes as:

$$\mathbf{H_f = [2.87\, n^2\, V^2\, L] / [S^{4/3}]} \tag{7.6}$$

and,

$$H_f = [29\ n^2\ L\ V^2] / [R^{4/3}\ (2g)] \tag{7.7}$$

where: H_f = total head loss due to friction, ft (refer to Section 7.7.2)
L = length of pipe, ft, and
All other terms are as previously defined.

7.5.2.1. *Roughness Coefficient*

Manning's "n" values for closed conduit storm drains can be found in Tables 5-3 or 5-4 located in Chapter 5. These values are typically for new pipe based on laboratory testing. Therefore, it is recommended that the Manning's roughness coefficient used for storm drain design reflect the "aged" condition of the storm drain material or actual field conditions since in reality, sediments, dirt, debris, anti-skid materials, leaves, pine needles, and other materials are deposited into storm drain systems and deposit there.

7.5.2.2. *Storm Drain Shape*

The shape of a storm drain pipe also influences its capacity. For most applications, circular conduit will be utilized, however a significant increase in capacity can be realized by using an alternate shape. Table 7-2 provides a listing of the increase in capacity which can be achieved using alternate conduit shapes that have the same height as the original shape, but have a different cross sectional area. Although alternate shapes are typically more expensive than circular ones, their use can be justified in some cases based on their increased capacity or to provide required cover.

TABLE 7-2: ALTERNATE CONDUIT SHAPE CAPACITY INCREASES

	Area (% Increase)	Conveyance (% Increase)
Circular	--	--
Oval	63	87
Arch	57	78
Box (B = D)	27	27
Box (B = 2D)	154	208

Source: FHWA, HEC-22, 1996

7.6. STORM DRAIN DESIGN - RATIONAL METHOD

This section presents a simplified method of estimating the discharges required to size a storm drain system using the Rational Method. This method can only estimate the peak discharges for small areas which cannot be hydrologically routed.

The total design of a storm drain system is typically divided into the following operations:

1. Determination of inlet location and spacing as outlined in Chapter 6.
2. Preparation of a plan layout and profile of the storm drainage system.
3. Determining the contributing drainage area(s) and resultant peak discharge(s).
4. Computing the hydraulic capacity and hydraulic gradient of the storm drain system.

The three basic guidelines recommended to follow in laying out a storm drain profile are:

1. Keep the storm drain as close to the surface as minimum cover requirements and/or hydraulic design requirements allow to minimize excavation costs.
2. Design the system for full flow conditions. "Hang" the crown of the pipe on the hydraulic gradient for design conditions.
3. Design the system at the uppermost inlet and proceed downstream.

7.6.1. Time of Concentration and Discharge

The design discharge at any point in a storm drain system is not the sum of the flow rates of all inlets above the point of interest and will generally be less than this cumulative total.

In using the Rational Method for storm drain design, the time of concentration becomes very influential in determining the design discharge. The time of concentration is defined as the time required for water to travel from the hydraulically most distant point in the total contributing watershed to the design point. This typically consists of two components: (1) the time for overland and gutter flow to reach the inlet, and (2) the time to flow through the storm drain system to the next point of interest. The flow path having the longest time of concentration to the point of interest in the storm drain system will usually define the duration used in selecting the rainfall intensity used in the Rational Equation.

There are exceptions, however, to this application of the Rational Method. For example, a smaller, highly impervious subarea within a larger drainage area may have an independent discharge higher than that of the total drainage area. This is typically due to the higher C value and higher intensity associated with the short time of concentration and can occur when a highly impervious area exists at the most downstream portion of the total drainage area and the total drainage area flows through the lower impervious area. When this situation occurs, two separate calculations should be made and the largest value of discharge is used:

1. Calculate the runoff from the total drainage area with its weighted C value and intensity associated with the longest time of concentration.

2. Calculate the runoff from only the smaller impervious area using a higher C value and higher intensity associated with the shorter time of concentration.

7.6.2. Storm Drain Sizing Procedure

The actual storm drain design can be accomplished by the following procedure and using the Storm Drain Computation Sheet provided in Figure 7-3. This procedure assumes that each storm drain segment will be initially designed under a full flow condition.

Step 1. Prepare a working plan layout and profile of the proposed storm drain system which establishes the following:

a. location of storm drains,
b. direction of flow,
c. location of manholes or other structures,
d. numbering of all structures and storm drain segments, and
e. location of existing and proposed utilities (e.g., water, sewer, gas, electric, cable, etc.)

Step 2. Determine the following hydrologic parameters for the tributary drainage areas for each inlet on the storm drain system:

a. drainage areas,
b. runoff coefficients, and
c. travel time.

Step 3. Using the information generated in Steps 1 and 2, complete the following information on the design form for each segment of pipe starting at the upstream most storm drain segment:

a. Enter the FROM and TO stations in Columns 1 and 2.
b. Enter the length of the pipe segment in Col. 3 (LENGTH).
c. Enter the incremental drainage area in Col. 4 (INC. D.A.).

The incremental drainage is that area tributary to the inlet at the upstream end of the storm drain segment under consideration.

d. Enter runoff coefficient (C) in Col. 6 using the appropriate frequency factor

(1.1 for 25-year or 1.25 for 100-year).

This is the runoff coefficient for the drainage area tributary to the inlet at the upstream end of the storm drain segment under consideration. Refer to Section 3.1.6 for appropriate C values.

e. Compute and enter the inlet time of concentration in Col. 9 (T_c INLET).

This is the time required for water to travel from the hydrologically most distant point of the drainage area to the inlet of the upstream end of the storm drain segment under consideration.

f. Compute and enter system time of concentration in Col. 10 (T_c SYSTEM).

This is the time for water to travel from the most remote point in the storm drain system to the upstream end on the storm drain segment under consideration. For the beginning of the storm drain system this will be the same as the value in Col. 9. For all other pipe segments this value is computed by adding the SYSTEM T_c in Col. 10 to the SEGMENT TIME in Col. 17 from the previous segment to get the system T_c at the upstream end of the section under consideration.

Step 4. Using the information from Step 3, compute the following:

a. Add the incremental area in Col. 4 (INC. D.A.) to the previous section's total area and enter this value in Col. 5 (TOTAL D.A.).

b. Multiply the value in Col. 4 (INC. D.A.) by the runoff coefficient (C) in Col. 6 and enter the product, DA, in Col. 7 ("INC","AREA X "C")

c. Add the value in Col. 7 to the value in Col. 8 ("TOTAL", "AREA" X "C") for the previous storm drain segment and put this value in Col. 8.

d. Using the larger of the two T_c's in Col. 9 and 10, determine the rainfall intensity and place this value in Col. 11 (RAIN "I").

e. Calculate the discharge as the product of Columns 8 and 11. Enter this value in Col. 12 ("Q").

f. Size the pipe using the criteria presented in Section 7.5.2 to convey the discharge by varying the slope and pipe size as necessary. The storm drain should be sized as close as possible to full gravity flow with a minimum pipe diameter of 18". Enter pipe size in Col. 13 (PIPE DIA.).

g. Enter the pipe slope determined in Step 4f in Col. 21.

h. Place the full flow capacity of the pipe in Col. 14.

i. Compute the full flow and design flow velocities in the pipe and place these values in Columns 15 and 16, respectively.

j. Calculate the travel time in the pipe segment by dividing the pipe length in Col. 3 by the design flow velocity in Col. 16. Enter this value in Col. 17.

k. Calculate the approximate crown drop at the structure to off-set the potential structure energy losses using Equation 7.12 or 7.13. Enter this value in Col. 20.

l. Compute the pipe inverts at the upper (U/S) and lower (D/S) ends of this segment of pipe, including any pipe size changes that occurred along this segment.

Step 5. Repeat Steps 3 and 4 for all pipe segments to the storm drain outlet.

Step 6. Check the design by calculating the hydraulic grade line as described in Section 7.7.

STORM DRAIN COMPUTATION SHEET											Date: ____ Computed By: ____			Street/Storm Drain Line ____ Sheet ____ of ____						
STATION						"AREA" X "C"		TIME OF CONCENTRATION						VELOCITY			INVERT ELEVATION			
FROM	TO	PIPE LENGTH (ft)	INC. D.A. (ac)	TOTAL D.A. (ac)	"C" VALUE	INC. (ac)	TOTAL (ac)	T_c INLET (min)	T_c SYSTEM (min)	RAIN "I"	"Q" (ft^3/S)	PIPE DIA. (ft)	CAP. FULL (ft^3/S)	FULL (ft/s)	DESIGN (ft/s)	SEG. TIME (min)	U/S INVERT	D/S INVERT	CROWN DROP (ft)	SLOPE (ft/ft)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)

Figure 7-3: Storm Drain Computation Sheet

7.7. HYDRAULIC GRADE LINE EVALUATION

The final step in designing a storm drain system is to check the hydraulic grade line (HGL) to determine if the design is adequate. The HGL is used by the designer in determining the acceptability of a proposed storm drain design by establishing the elevation to which water will rise in manholes and inlets when the system is operating under the design discharge and condition.
The "isolated pipe" approach of storm drain design is not acceptable for determining the hydraulic grade line on multiple inlet, multiple pipe storm drain systems. This simplified approach entails breaking the storm drain system into isolated pipes that are not connected through loss producing structures such as manholes or junction boxes and applying Manning's formula to determine the flow depth in each segment. Although this approach may be suitable for a single inlet, single pipe system, it can lead to false results and undesirable consequences on longer storm drain systems, therefore running the hydraulic grade line computations through the system is required.

Hydraulic grade line computations are based on the Bernoulli equation and can be expressed as:

$$\mathbf{V_1^2/2g + D_{hg1} + S_oL = V_2^2/2g + D_{hg2} + S_fL + H_m} \qquad \textbf{(7.8)}$$

The terms used in Equation 7.16 are defined in Figure 7.4b. The minor losses (H_m), in feet, are included in Equation 7.16 due to their importance in computing the HGL. The HGL is equal to the total energy grade line minus the velocity head ($V^2/2g$) at any point along the storm drain.

In general terms, the HGL is calculated by starting with the controlling tailwater elevation (see Section 7.7.3) at the storm drain outfall. From that elevation, calculations proceed upstream from junction to junction or manhole to manhole. At the lower end of the of each junction, the pipe friction losses from the downstream section (expressed in feet of loss) are added to the downstream HGL elevation. At the upstream end of each junction, the minor losses through the junction are added. If the transition is to a ditch section, the losses added are the entrance losses to the pipe system.

A manual procedure for computing the hydraulic grade line is presented in Section 7.7.4. Computer programs are readily used to design and analyze storm drain systems, however, these programs may utilize different methods to determine the energy losses than those presented in this chapter. The designer should review the methods used by computer programs and have the program approved by the Stormwater Manager prior to use.

The maximum hydraulic gradient shall not produce a velocity that exceeds 20 feet per second.

7.7.1. Open Channel Flow

For true open channel flow conditions, the HGL is a line representing the free water surface elevation at any point. The pipe and friction slopes for storm drains are assumed to be equal. Therefore, it is not necessary to compute a hydraulic grade line for these storm drains if the soffits of connecting pipes of unequal size are set at the same elevation, and if minor head losses along the storm drain are minimal. The HGL should then be set equal to the normal flow depth in the pipe(s). Figure 7-4a defines the relationships of open channel flow in a closed circular conduit.

7.7.2. Energy Losses

All typical energy losses shall be accounted for, as applicable, in all storm drain designs. Energy (head) is required to overcome the changes in momentum or turbulence caused by losses at inlets, outlets, pipe bends, transitions, junctions, and manholes. It is necessary to estimate all energy losses through storm drain pipes and junctions prior to computing the hydraulic grade line. These typical energy losses are outlined below and are summarized in Figure 7-5.

7.7.2.1. Pipe Friction Losses

Pipe friction or boundary shear loss is typically the major head loss in a storm drain system and can be estimated by Equation 7.8:

$$\mathbf{H_f = S_f\, L} \qquad \textbf{(7.9)}$$

where: H_f = friction loss, ft
S_f = friction slope, ft/ft
L = length of pipe, ft.

The friction slope is also the slope of the hydraulic gradient for a particular pipe segment and is defined as:

$$\mathbf{S_f = [Qn / (1.486\, A\, R^{2/3})]^2} \qquad \textbf{(7.10)}$$

where: R = hydraulic radius

7.7.2.2. Entrance Losses

The following equation is used to determine entrance losses for beginning flows:

$$\mathbf{H_e = K_e\, (V^2 / 2g)} \qquad \textbf{(7.11)}$$

where: K_e = 0.50, assuming square-edge

7.7.2.3. Outlet Losses

The outlet loss from a storm drain is a function of the change in velocity at the pipe outlet. For sudden expansion such as at an endwall, the outlet loss is:

$$\mathbf{H_O = 1.0\ [(V_O^2/2g) - (V_d^2/2g)]} \qquad \mathbf{(7.12)}$$

where: V_O = average outlet velocity, and
V_d = channel velocity downstream of the outlet.

When the outlet is fully submerged, as in a reservoir, $V_d = 0$ and the outlet loss is equal to one times the velocity head. For partially full flow where the pipe outlets in a channel with moving water in the same direction as the outlet water, the outlet loss may be reduced to virtually zero.

7.7.2.4. Junction and Transition Losses

Junctions: A pipe junction is a connection of a lateral pipe to a larger mainline without the use of a manhole or other structure. The loss for a pipe junction (H_I) can be determined as follows:

$$\mathbf{H_j = (V_2^2/2g) - (V_1^2/2g) - [(A_3V_3^2 / A_2g)]} \qquad \mathbf{(7.13)}$$

Figure 7-6(A) depicts this type of junction.

For instances where the mainline pipe diameter increases immediately downstream of the junction, the loss can be determined by:

$$\mathbf{HG = [2 / (A_1 + A_2)]\ \ [(Q_2V_2 - Q_1V_1 - Q_3V_3Cos\Theta) / g]} \qquad \mathbf{(7.14)}$$

Figure 7-6(B) depicts this type of junction.

Transitions: Transition loss (H_t) for an expansion where flow velocities decrease in the downstream direction can be determined by:

$$\mathbf{H_t = 0.2\ [(V_1^2 / 2g) - (V_2^2 / 2g)]} \qquad \mathbf{(7.15)}$$

Figure 7-6(C) depicts this type of junction.

{PRIVATE }7.7.2.5 Manhole Losses{tc \l 4 "7.7.2.5 Manhole Losses"}

Straight Through Manhole: For manholes with no change in either pipe size or discharge, the loss can be determined by:

$$\mathbf{H_{mhs} = 0.05\ (V^2 / 2g)} \qquad \mathbf{(7.16)}$$

where: V = velocity of outlet pipe

<u>Manhole with a Bend</u>: For manholes with a bend where the flow changes direction and there is no change in pipe size or discharge, the loss can be determined by:

$$\mathbf{H_{mhb} = K_{mhb} (V^2 / 2g)} \qquad \mathbf{(7.17)}$$

where: K_{mhb} is determined using Figure 7-7.

In cases where the inflow pipe invert is above the water level in the manhole, the outflow pipe will function as a culvert, and the manhole loss and the manhole HGL must be computed using procedures presented in Chapter 5. If the outflow pipe is flowing full or partially full under outlet control, the manhole loss (due to flow contraction into the outflow pipe) can be determined by setting K in Equation 7.17 equal to K_e as defined in Table 5-1. If the outflow pipe is flowing under inlet control, the water depth in the manhole can be determined using the inlet control nomographs given in Chapter 5.

7.7.2.5. <u>Bend Losses</u>

For bends in a storm drain pipe not occurring in a manhole or other junction structure, the minor loss can be determined by:

$$\mathbf{H_b = K_b (V^2 / 2g)} \qquad \mathbf{(7.18)}$$

The bend loss coefficient, K_b, is dependent upon the angle and sharpness of the bend. Figure 7-7 can be used to determine K_b for angles not exceeding 90 degrees.

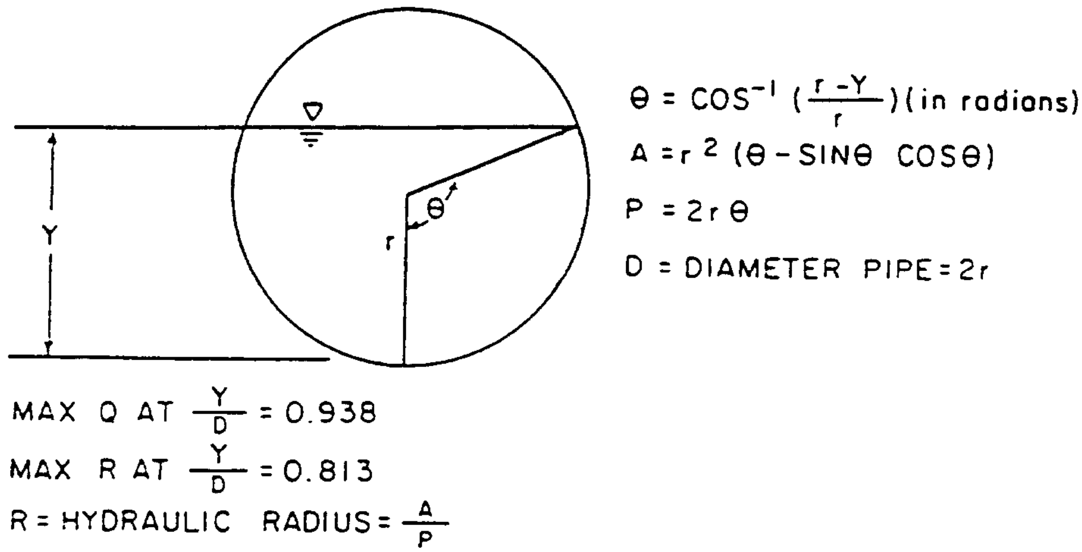


Figure 7-4a: Hydraulic Terms for Circular Conduit

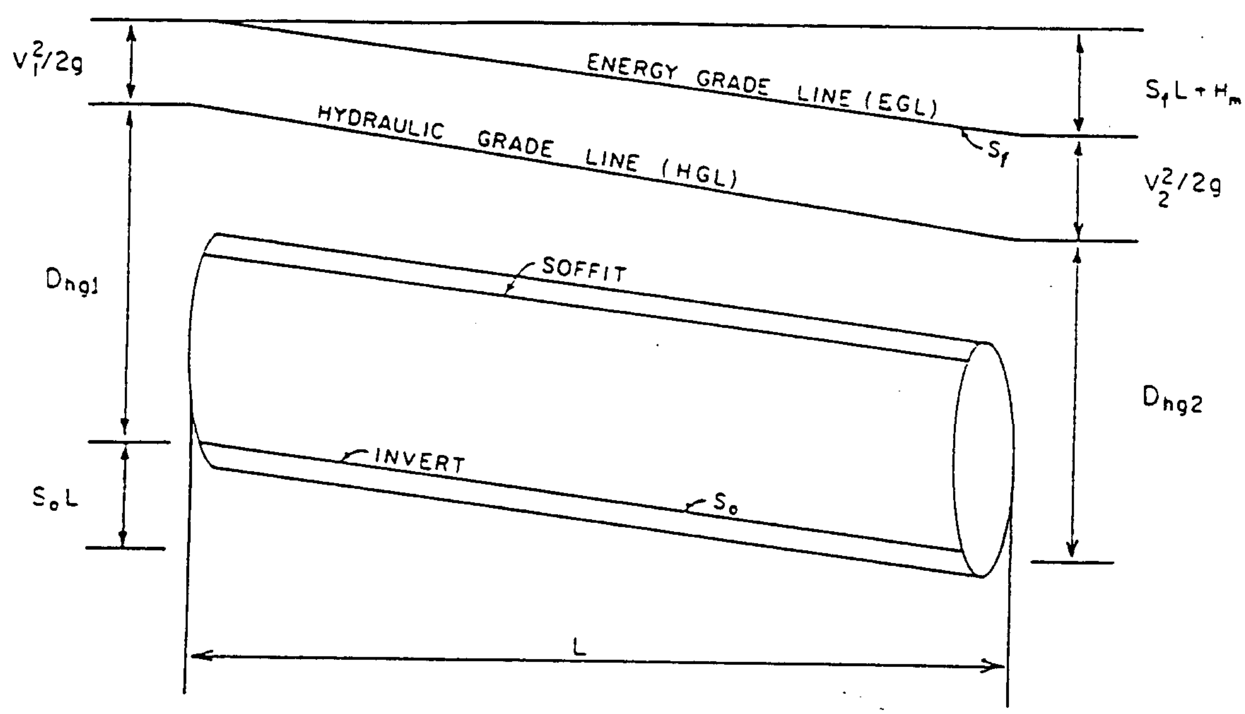


Figure 7-4b: Head Loss Terms for Storm Drains

SUMMARY OF ENERGY LOSSES

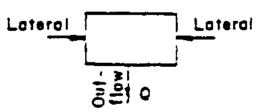
$$H_{tm} = \frac{V^2}{2g}$$

TERMINAL JUNCTION LOSSES
(at beginning of run)

Where g = gravitational constant, 32.2 feet per second per second.

$$H_e = 0.5\,\frac{V^2}{2g}$$

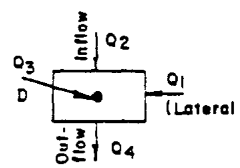
ENTRANCE LOSSES
(for structure at end of run)
Assuming square - edge



$$H_{j1} = \frac{V^2 (\text{Outflow})}{2g}$$

JUNCTION LOSSES

Use only where flows are identical to above, otherwise use H_{j2} Equation.



$$H_{j2} = \frac{Q_4V_4^2 - Q_1V_1^2 - Q_2V_2^2 + KQ_1V_1^2}{2gQ_4}$$

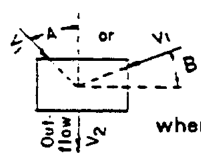
JUNCTION LOSSES
(After FHWA)

Total losses to include H_{j2} plus losses for changes in direction of less than 90° (H_b).

Where K = Bend loss factor

Q_3 = Vertical dropped-in flow from an inlet

V_3 = Assumed to be zero



where B = 90 - A

$$H_b = \frac{KV_1^2}{2g}$$

BEND LOSSES
(changes in direction of flow)

Where K	Degree of Turn (A) in Junction
0.19	15
0.35	30
0.47	45
0.56	60
0.64	75
0.70	90

FRICTION LOSS (H_f)

$$H_f = S_f \times L$$

Where H_f = friction head
S_f = friction slope
L = length of conduit

$$S_f = \left(\frac{Qn}{1.486\,AR^{\frac{2}{3}}}\right)^2$$

Where Q = discharge of conduit
n = Mannings coefficient of roughness (use 0.013 for R.C. Pipes)
A = area of conduit
R = hydraulic radius of conduit (D/4 for round pipe)

TOTAL ENERGY LOSSES AT EACH JUNCTION

$$H_T = H_{tm} + H_e + (H_{j1} \text{ or } H_{j2}) + H_b + H_f$$

Figure 7-5 Summary of Energy Losses

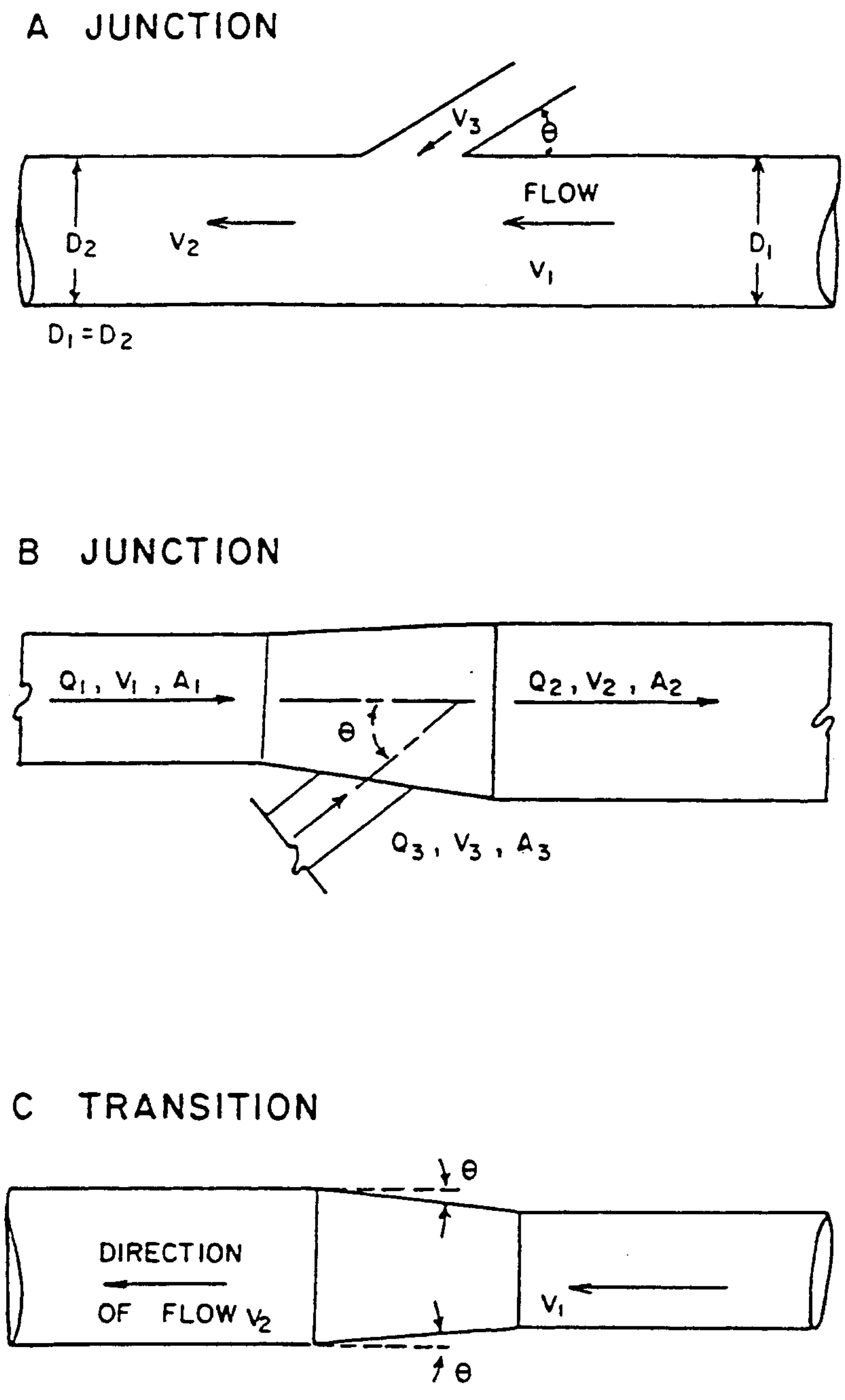


Figure 7-6 Junction and Transition Loss Configurations

Source: City of Tucson Drainage Standards Manual (1989)

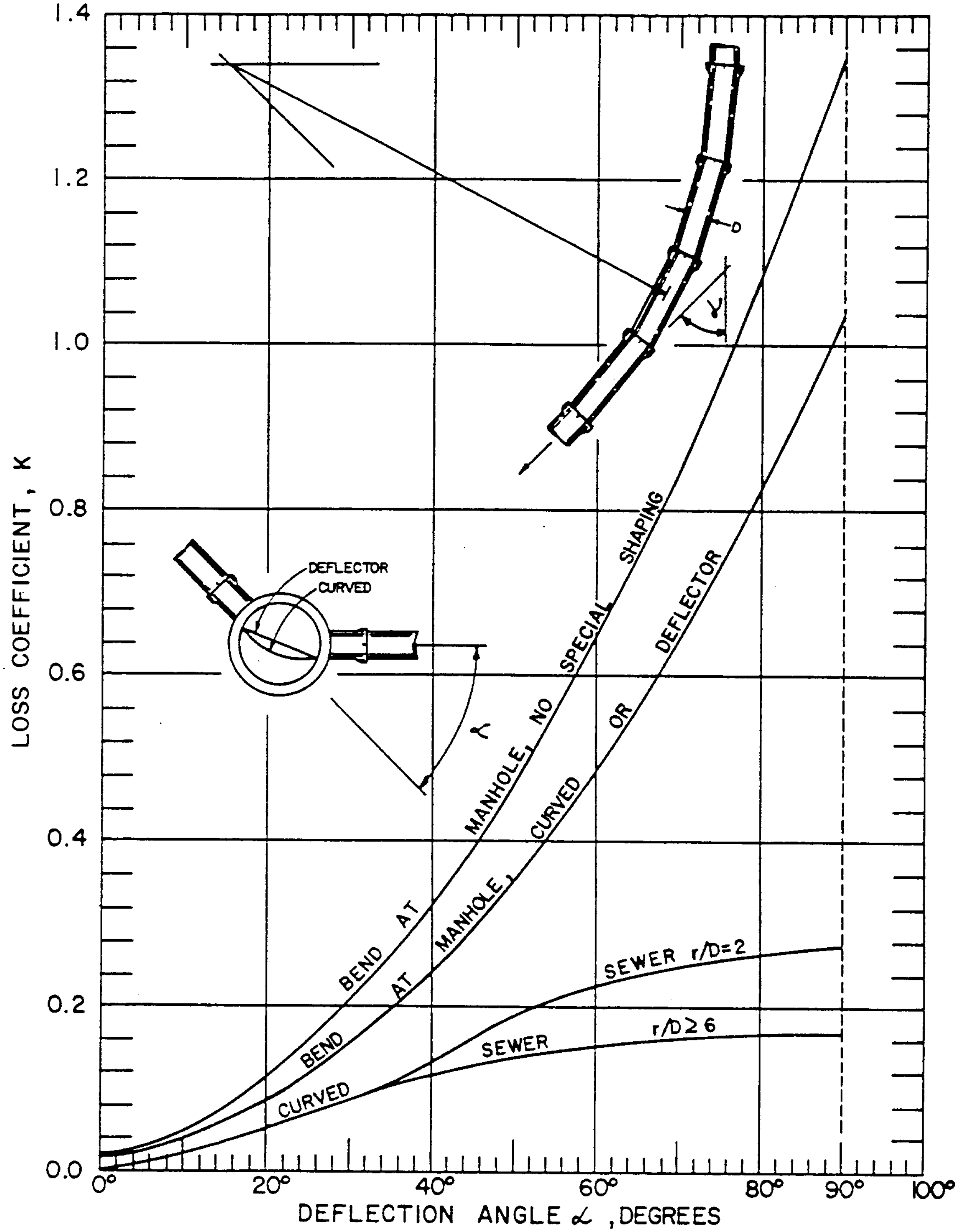


Figure 7-7 Manhole/Bend Headloss Coefficient

7.7.3. Controlling Water Surface Elevation

For most applications, the controlling water surface elevation or tailwater will either be above the crown of the outlet or can be considered to be between the crown and critical depth at the outlet. The tailwater may also occur between critical depth and the invert of the outlet and be a free outfall. The controlling tailwater for purposes of computing the hydraulic grade line shall be determined by one of the following criteria:

1. If the receiving body of water is a detention basin or lake, the tailwater shall either be the normal high water elevation in the lake or the high water elevation in the detention basin for the same design storm as that of the storm drain.

2. If the outfall is a wash, stream, or other open channel, the controlling tailwater shall be the water surface elevation in the channel for the same design storm as that of the storm drain or $(d_c + D)/2$, whichever is greater (where: d_c = critical depth and D = pipe diameter).

3. If the outfall is another storm drain, the controlling elevation shall be the highest hydraulic grade line elevation of the receiving storm drain immediately upstream or downstream of the junction for the same design storm.

4. For low tailwater conditions (TW depth $\leq$ D/3), the controlling tailwater shall be $(d_c + D)/2$ plus the invert of the outlet.

7.7.4. Hydraulic Grade Line Evaluation Procedure

This section provides a step-by-step procedure for manual computation of the hydraulic grade line (HGL) using a junction loss methodology. It is recommended that the designer perform and understand this procedure and the energy losses which can occur in a storm drain system to better interpret output from computer generated storm drain designs or analysis. Figure 7-8 illustrates proper and improper use of energy losses in developing a hypothetical storm drain system under pressure flow.

Most storm drain systems are designed to function in a subcritical flow regime where pipe and junction losses are summed to compute the upstream HGL level(s). If supercritical flow occurs, pipe and manhole losses are not carried upstream. Under supercritical flow, the designer should proceed to the next upstream pipe segment to determine its flow regime and continue the process until the storm drain returns to a subcritical flow regime.

The following HGL computation procedure for an outlet control utilizes the computation table given in Figure 7-9:

Step 1. Enter the station for the junction immediately upstream of the outflow pipe in Col. 1. The HGL computations begin at the outfall and are worked upstream taking each junction into consideration.

Step 2. Enter the outlet controlling water surface elevation based on the criteria given in Section 7.4.1.

Step 3. Enter the diameter of the outflow pipe (D_O) in Col. 3.

Step 4. Enter the design discharge (Q_O) for the outflow pipe in Col. 4.

Step 5. Enter the length of the outflow pipe (L_O) in Col. 5.

Step 6. Using Equation 7.9, compute and enter the friction slope (S_f) in ft/ft of the outflow pipe in Col. 6. This assumes full flow conditions.

Step 7. Multiply the friction slope (S_f) in Col. 6 by the length (L_O) in Col. 5 and enter the friction loss (H_f) in Col. 7. On curved alignments, calculate curve losses by using $H_C = 0.002(\phi)(V^2/2g)$, where ϕ = angle of curvature in degrees and add to the friction loss.

Step 8. Enter the velocity (V_O) of the outflow pipe in Col. 8.

Step 9. Enter the contraction loss (H_O) using $H_O = [0.25(V_o^2)] / 2g$ in Col. 9.

Step 10. Enter the design discharge (Q_i) for each pipe flowing into the junction in Col. 10. Neglect lateral pipes with inflows of less than ten percent of the mainline flow. Inflow must be adjusted to the mainline outflow duration time before a comparison is made.

Step 11. Enter the velocity of inflow (V_i) for each pipe flowing into the junction in Col. 11 (for exception, see Step 10).

Step 12. Enter the product of Q_i and V_i (Col. 10 and 11) for each inflowing pipe in Col. 12. When several pipes inflow into a junction, the line producing the greatest Q_iV_i value is the line which will produce the greatest expansion loss (H_i).

Step 13. Enter the controlling expansion loss (H_i) using $H_i = [0.35\ (V_i^2) / 2g]$ in Col. 13.

Step 14. Enter the skew angle of each inflowing pipe to the outflow pipe in Col. 14 (for exception in Step 10).

Step 15. Enter the greatest bend loss (H_B) using $H_B = [K(V_i^2) / 2g$, where K = the bend loss coefficient from Figure 7-7 corresponding to the various angles of skew of the inflowing pipes.

Step 16. Enter in Col. 16 the total head loss (H_T) by summing the values in Col's. 9 (H_O), 13 (H_i), and 15 (H_B).

Step 17 If the junction incorporates adjusted surface inflow of ten percent or more of the mainline outflow (i.e. drop inlet), increase H_T by 30 percent and enter the adjusted H_T in Col. 17.

Step 18. If the junction incorporates partial diameter inlet shaping, such as standard manholes, reduce the value of H_T by 50 percent and enter the adjusted value in Col. 18.

Step 19. Enter in Col. 19 the FINAL H (the sum of H_f and $H_{T\ adjusted}$).

Step 20. Enter in Col. 20 the sum of the elevation in Col. 2 and the FINAL H in Col. 19. This elevation is the potential water surface elevation for the junction under design conditions.

Step 21. Enter in Col. 21 the rim elevation, ground, top of grate, or gutter flow line, which ever is lowest, of the junction under consideration in Col. 20. If the potential HGL is within one foot of the rim, ground, or gutter elevation, adjustments are needed in the system to reduce the elevation of the HGL.

Step 22. Repeat the procedure starting with Step 1 for the next upstream junction.

7.7.4.1. *Lateral Connections*

A connecting lateral from a catch basin will act independently of the main line if the crown elevation of the lateral line, at the catch basin, is above the HGL or crown elevation of the main line, whichever is greater, by the amount of the friction loss (see Equation 7.8). The hydraulic gradient of the connecting lateral pipe is tied into the crown of the main line storm drain (or HGL if under pressure flow). The friction losses are then added to the crown (or HGL) elevation to determine the hydraulic gradient for the lateral.

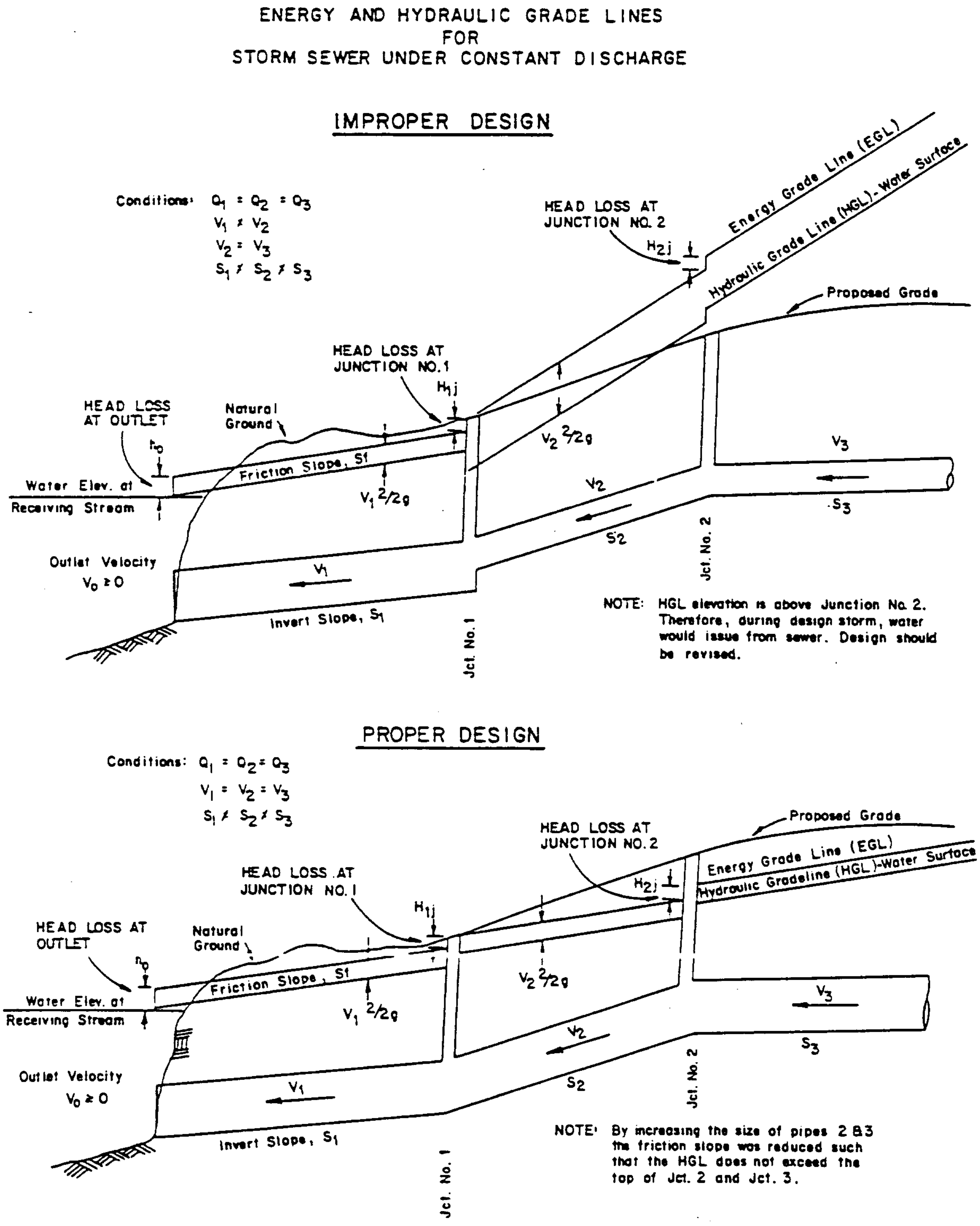


Figure 7-8: Use of Energy Losses In Developing A Storm Drain System

Source: AASHTO Model Drainage Manual

HYDRAULIC GRADE LINE COMPUTATION SHEET

Date: ____________ **Street/Storm Drain Line** ____________________
Computed By: ________________ **Sheet** _______ **of** ____________

							JUNCTION LOSS													
Sta.	OUTFALL WSE	D_o	Q_o	L_o	S_f	H_f	V_o	H_o	Q_i	V_i	$Q_i V_i$	H_i	Angle	H_B	H_T	$1.3H_T$	$0.5H_T$	FINAL H	WSE @ JUNC.	RIM ELEV.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)

Figure 7-9: Hydraulic Grade Line Computation Sheet

CHAPTER 8: STORAGE AND DETENTION FACILITIES

Urbanization and other land development activities, including construction of roads, changes natural pervious areas into impervious, altered surfaces. In addition, natural drainage systems are often replaced by lined channels, storm drains, and curbed streets. The result of such activities is an increase in the volume of runoff, peak discharge rates, erosion, and non-point source pollution due to the reduction in infiltration and natural vegetation.

In the absence of regional detention facilities and due to inadequate downstream capacities of existing streets, storm drain systems or channels, local on-site or sub-regional detention facilities are necessary to attenuate the increased runoff caused by development. Detention facilities can also serve a dual purpose by improving the quality of stormwater discharges. The temporary storage of stormwater runoff can reduce the extent of downstream flooding, soil erosion, sedimentation, and surface water pollution. Detention facilities can also be used to reduce the costs associated with large storm drain systems.

8.1. <u>POLICIES</u>

a. Stormwater detention is required for all new subdivisions, commercial and industrial developments, re-development of non-conforming sites (i.e., existing developed sites that do not have detention that have been razed and vacant for greater than six months), and other developments greater than 1/4 acre in size.

b. Detention requirements may be waived by the Stormwater Manager for the following:

   1. Single-family residential structure or lot (i.e., not associated with a new subdivision).
   2. Residential subdivisions with lot areas $\geq$ 1 acre in area, if it can be shown that such a waiver will not result in any adverse downstream effects, nor create any disturbance to the existing drainage patterns both within and adjacent to the subdivision.
   3. Developments less than 1/4 acre or increases in impervious area of $\leq$ 5,000 square feet. It must be demonstrated to the satisfaction of the Stormwater Manager that there will be no increase in the potential for damages to adjacent properties and adequate off-site or downstream drainage capacity is available.

c. Detention facility storage volume shall be adequate to attenuate the post-development peak discharge rates to pre-development discharge rates for the 2, 10, and 100-year design storms. Reservoir routing calculations must be used to demonstrate that the storage volume is adequate.

d. Detention facility outlet structure release rates shall be less than the pre-development peak runoff rates for the 2, 10, and 100-year storm events, with emergency overflow provisions. Design calculations are required to demonstrate that developed runoff from the 2, 10, and 100-year design storms are controlled.

e. The total combined post-development discharge from a development cannot exceed the total pre-development peak discharge for the 2, 10, and 100-year storms. Drainage flows of all frequencies shall enter and depart the property to be developed in substantially the same manner as under the pre-developed condition.

f. The same methodology shall be used for both pre-development and post-development analyses. The detention analysis shall be limited to the area of development only. Contributing drainage areas shall be analyzed separately for purposes of estimating the runoff which must be accepted and conveyed through the site.

g. The use of pavement parking lot storage as the primary detention facility is not permitted unless other sites or detention alternatives are not available.

h. Roof-top detention storage is not permitted for meeting City of Flagstaff detention requirements.

i. Individual on-lot storage systems within single family residential developments is not permitted for meeting City of Flagstaff detention requirements.

j. Developments which are phased shall prepare a master stormwater detention plan for the entire development. The master stormwater detention plan must either be implemented with the first phase, if possible, or detention must stand alone for each phase. Interim temporary detention facilities may be required for phasing.

k. The point or points at which a pre-development watercourse enters and leaves a site shall remain substantially the same after the property has been altered for the development. Drainage in general, including sheet flow, should leave the site as it did in the pre-developed condition. Developers must coordinate with downstream properties if the drainage exiting their property is to be changed (e.g., pre-development flow was sheet flow and the post-development discharge is a detention outlet pipe or point discharge).

l. Discharges from detention facilities shall be designed to enter established downstream drainage systems (e.g., drainage channels, natural watercourses, public streets, or storm drain systems) whenever possible. If flows are to be concentrated or ponded on the upstream or downstream side of the site, either a recorded drainage easement or written permission must be obtained from the affected property owner(s) prior to issuance of grading or building permits. Discharge velocities shall be reduced or dissipated to provide non-erosive flows and reduce damages to downstream properties.

m. The City of Flagstaff shall only accept large-scale regional detention basins for operation and maintenance. The City shall not accept small-scale, local on-site detention basins for operation, maintenance, or liability.

n. Maintenance of local on-site detention facilities shall be the responsibility of the property owner or homeowner's association. The City shall reserve the right to periodically inspect any detention facilities to verify that regular maintenance activities are being performed. Final Plats; Covenants, Conditions, and Restrictions (CC&R's); and/or Development Plans shall include a special note stating that (1) the owner(s) shall be solely responsible for the operation, maintenance, and liability for all detention facilities; and, (2) the City of Flagstaff may periodically inspect said detention facilities to verify that regular maintenance activities are being performed adequately.

o. The City of Flagstaff Parks and Recreation Division must review and approve proposed stormwater detention facilities designed within designated public areas or parks. Review and approval from the Parks and Recreation Commission may also be required.

p. No part of a private detention basin shall be constructed in a public right-of-way or public utility easement.

q. Site designs which consolidate detention areas into a limited number of larger facilities are preferred over designs which utilize a large number of small facilities. Designs which will demand considerable maintenance, will be difficult to maintain and access, or utilize numerous small facilities will not be permitted if other alternatives are physically possible.

8.2. SUBDIVISION REQUIREMENTS

All new subdivisions are required to provide detention for the entire subdivision, including the respective one-half of all abutting streets to the subdivision. Two or more subdivisions may join together to provide a common detention facility.

Preliminary Plat submittals shall be accompanied by a preliminary drainage report in accordance with Section 2.1.2 of this manual.

All Final Plat submittals require a final drainage report, in accordance with Chapter 2 of this manual, which technically demonstrates compliance with City of Flagstaff Floodplain Management Regulations, Stormwater Management requirements, and the drainage policies and design criteria set forth in this manual. This report must be submitted with the subdivision improvement plans and must be accepted prior to recording the final plat.

All subdivisions proposals (including proposals for manufactured home parks) greater than 50 lots or 5 acres, whichever is the lesser, located in FEMA designated Unnumbered Zone A areas must provide base flood elevation and floodway delineation data.

All subdivision proposals within riverine environments with a contributing watershed of 1/4 square mile or more must provide base flood elevation and floodway data in accordance with the criteria set forth on Arizona Department of Water Resources State Standards or other methods approved by the Stormwater Manager.

All final plats within FEMA or a City of Flagstaff designated 100-year floodplain must show the 100-year base flood elevations, floodplain/floodway limits, finish floor elevations a minimum of one (1) foot above the BFE, and appropriate erosion hazard setbacks.

8.3. DETENTION VOLUME ESTIMATION

8.3.1. Estimating Required Volume

The final design of any detention facility requires three items: 1) an inflow hydrograph, 2) a stage vs. storage curve, and 3) a stage vs. discharge curve. However, before the stage-storage and stage-discharge curves can be developed, a preliminary estimate of the required storage volume and the configuration of the detention facility are required. Reservoir-routing computations are then made to determine if the estimated storage volume is adequate and will provide the desired outflow hydrograph(s).

8.3.1.1. Triangular Hydrograph Method

A preliminary estimate of the storage volume required for peak flow attenuation can be estimated from a simplified design procedure that replaces the actual inflow and outflow hydrographs with standard triangular shapes. This method should not be applied if the hydrographs cannot be approximated by a triangular shape since this would introduce additional errors of the preliminary estimate of the required storage. This method will generally yield detention volumes that are low, but will provide a starting point for reservoir routing. The estimated storage volume is the area between the pre- and post-development hydrographs and can be expressed as:

$$\mathbf{V_s = 0.5\ T_i\ (Q_i - Q_o)} \qquad \mathbf{(8.1)}$$

where: V_s = storage volume estimate, ft^3
T_i = duration of storage facility inflow, seconds
Q_i = peak inflow rate, cfs
Q_o = peak outflow rate, cfs

Any consistent units may be used for Equation 8.1.

The duration of basin inflow should be derived from the estimated inflow hydrograph.

8.3.1.2. Modified Rational Method

The Modified Rational Method is permissible, but should be used with caution and limited to highly impervious developments under five (5) acres for normal detention design. For extended detention times, methods that can account for longer storms and actual rainfall distributions should be used.

For Modified Rational Method analysis, the duration of the storm shall be that which yields the highest storage requirement. The Modified Rational Graphical Hydrograph Method generally yields more realistic values when compared to fully routed applications. This method requires an iteration process to determine the duration that yields the greatest volume requirement.

The Modified Rational Graphical Hydrograph Method equation is:

$$\mathbf{V = 60\ [C_f CiAt - R(t_d+t_c)/2]} \qquad \mathbf{(8.2)}$$

where: V = the required volume of the pond, cubic feet
C = the post-development runoff coefficient
C_f = the antecedent precipitation factor
i = the rainfall intensity for t, in/hr
R = the allowable release rate, cfs
t_d = the storm duration to maximize the volume, min.
t_c = the post-development time of concentration, min.
A = drainage area, acres

8.3.2. Reservoir Routing

Proper detention facility design requires reservoir-routing to ensure the detention volume is adequate, determine the high water elevation(s), check for overtopping, and determine peak outflows. The most common method of routing an inflow hydrograph through a detention facility and determining the outflow hydrograph is the Storage Indication Method or the Modified Puls Method. This procedure requires an inflow hydrograph, a stage-discharge curve, and a stage-storage curve. This section will not present a procedure to perform reservoir-routing. It is recommended that the design engineer utilize commonly available computer programs.

8.3.3. Detention Design Procedure

A generalized procedure for designing detention facilities is presented below:

Step 1. Compute the inflow hydrographs for runoff from the 2, 10, and 100-year design storms using the procedures outlined in Chapter 3. Both pre and post-development hydrographs are required.

Step 2. Perform preliminary detention volume requirements for the hydrographs from Step 1 as outlined in Section 8.3.1 or other approved methods. If the storage requirements are satisfied for runoff from the 2, 10, and 100-year design storms, runoff from other intermediate storms are assumed to be controlled.

Step 3. Determine the physical dimensions necessary to hold the estimated volume from Step 2. The maximum storage requirement calculated from Step 2 should be used.

Step 4. Size the outlet structure(s). The estimated peak stage will occur for the estimated volume from Step 2. The outlet structure should be sized to convey the allowable discharge at this stage.

Step 5. Perform reservoir-routing calculations using inflow hydrographs from Step 1 to check the preliminary design. If the routed post-development peak discharges for the 2, 10, and 100-year design storms exceed the pre-development peak discharges, or if the peak stage varies significantly from the estimated peak stage from Step 4, then revise the estimated volume and return to Step 3.

Step 6. Design emergency overflow from runoff for the post-development 100-year peak discharge.

Step 7. Evaluate the downstream effects of detention primary outflow and overflow to ensure that the routed hydrographs and discharges do not cause downstream erosion and/or flooding problems.

Step 8. Evaluate the control structure outlet velocities and provide adequate local scour protection or energy dissipation if the velocity will have adverse downstream effects.

This procedure may involve numerous reservoir routing calculations to obtain the desired results. Generally, the 100-year design storm will determine the size of the basin and the 2-year design storm will determine the minimum size of the outlet structure(s). Typically, the 2-year outlet structure is sized and the depth of the storage when the 2-year storm is routed through the basin then normally becomes the elevation of the invert or weir for the 10 and/or 100-year storm outlet.

8.4. DETENTION FACILITY DESIGN

8.4.1. General Design Criteria

Detention facilities which incorporate basins below grade or embankments shall be designed and constructed as permanent drainage structures which are protected from long term erosion and are as maintenance free as possible. Detention basins and related facilities should be designed to minimize the following typical problems:

- weed growth,
- grass and vegetation maintenance,
- sedimentation control,
- bank deterioration,
- standing water or soggy surfaces,
- mosquito control,
- blockage of outlet structures,
- litter accumulation, and
- maintenance of fences and perimeter plantings.

No detention or retention facility shall retain standing water longer than thirty-six (36) hours unless the facility has been designed and constructed as a permanent body of water with appropriate health, safety, and water quality measures for such a body of water.

Basin inlet and outlet structures may be at or below grade or a combination of both. Drainage crossings or culverts shall be provided whenever runoff entering or exiting a basin crosses pedestrian paths or sidewalks.

Riprap aprons or other energy dissipating measures should be used at all inflow points (side slope and basin floor) to reduce velocities and encourage sedimentation.

Low flow channels are required on the bottom of basins which serve as multi-use areas and are recommended on all basins. Low flow channels should be designed with a minimum longitudinal slope of 0.5 percent. Concrete lined low flow channels can be designed with a minimum longitudinal slope of 0.2 percent.

If any portion of a detention basin is a retaining wall, design information for determining factors of safety against sliding and overturning is required. Provisions shall also be incorporated to prevent seepage through and under the retaining wall.

All detention facilities shall be incorporated into the Landscaping Plans.

All detention facilities will be reviewed for compliance with this manual and other applicable criteria.

The order of preference for detention facility design shall be:

1. The use of underground detention for all or part of the detention volume requirement.

2. The use of natural land-formed basins.

4. The use of structures, such as retaining walls or weir walls not to exceed four (4) feet in height that blend with the natural or built surroundings. Materials shall be limited to materials such as brick, native stone, rusticated masonry block, or form-lined colored concrete to blend with the natural or built surroundings.

8.4.2. Location and Configuration

In general, open detention facilities located adjacent to streets or buildings are discouraged. It is recommended to locate open basins in areas away from pedestrian traffic and public view, if possible. The use of underground detention is encouraged.

Detention basin configuration(s) shall be designed with surrounding land use, site land use, topography, unique site features, vegetation, visibility from adjoining streets or properties, and City of Flagstaff Land Development Code resource protection constraints taken into consideration.

Curvilinear or irregular shapes are required for open surface basins, whenever possible. The designer should vary the shape and side slopes of the basin and maximize the linear footage of the perimeter. Curvilinear contours at areas immediately adjacent to walls or structures is encouraged. Figure 8-1 illustrates a configuration concept.

The basin shape or floor should have a minimum length to width dimension of 4:1. Designs that incorporate basins in series with two or more stages or terraces are encouraged.

Slope and Depth Criteria

In general, basins should be kept to depths of three (3) feet or less with side slopes of 4H:1V or flatter. The following minimum slope and depth criteria are required for multi-use basins and basins that have unrestricted access:

a. A maximum of 2H:1V for protected side slopes and 3H:1V for unprotected side slopes where depths of ponding are less than three feet.

b. A maximum of 4:1 for side slopes where depths of ponding exceed three (3) feet.
c. Basins greater than six (6) feet in depth will require a benched configuration with bench widths at least three (3) times the height of the slope above it with a minimum width of six (6) feet.
d. Basins containing human activity areas shall incorporate access slopes of 8H:1V or flatter (12H:1V for ADA compliance) into the design for ingress and egress.

A minimum freeboard of 0.5 foot above the 100-year high water elevation is recommended for all detention facilities. Freeboard may include adjacent parking lot areas.

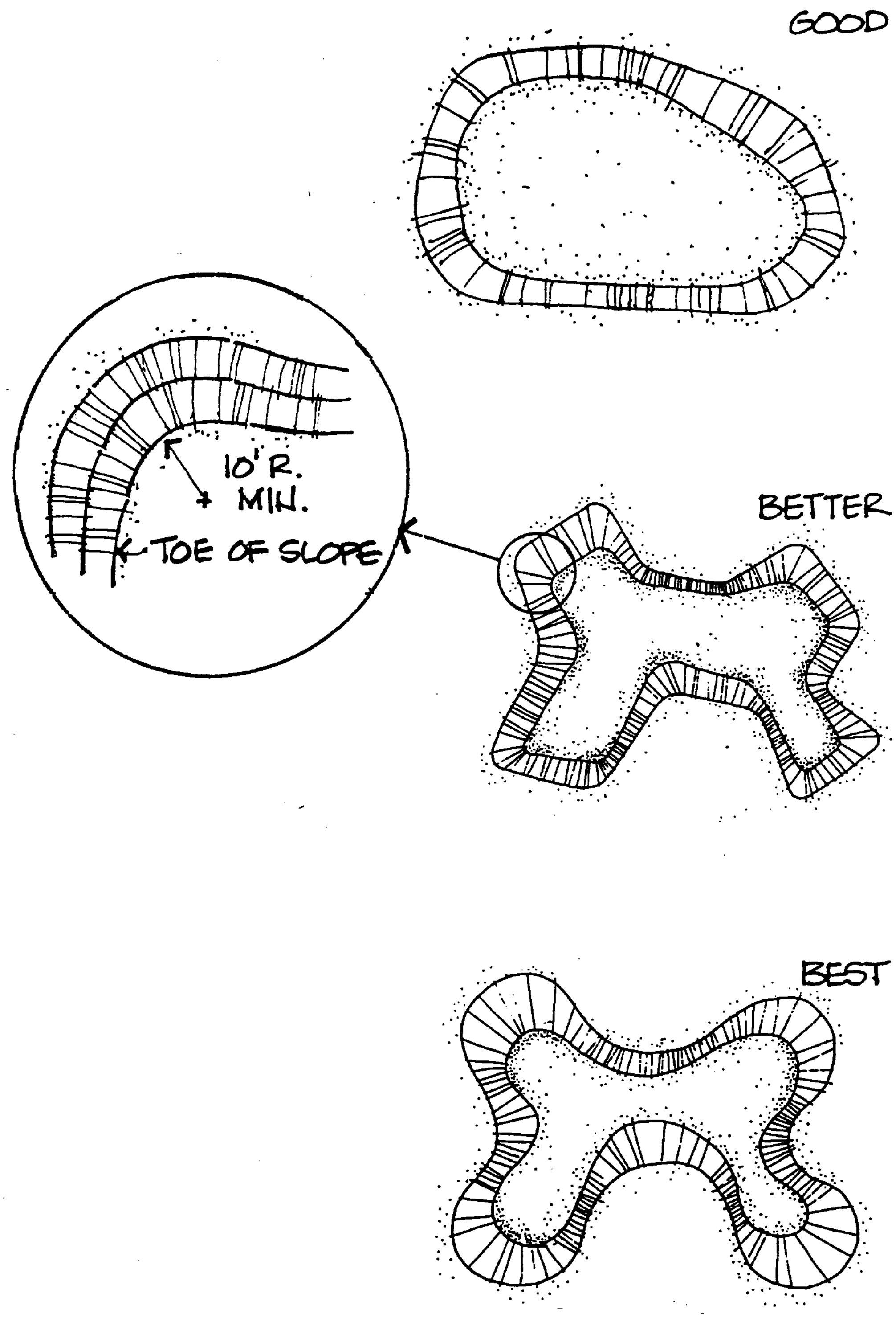


Figure 8-1: Detention Basin Configuration

The basin floor should be graded to drain toward the basin outlet. The minimum recommended slope is 0.2 percent. A meandering low flow channel across the basin bottom is recommended to accommodate frequent flows and prevent standing water.

8.4.3. Landscaping and Surface Treatment

All detention basins shall include appropriate surface treatment and erosion control measures including but not limited to landscaping, seeding, grass or sod, rock, gravel, or any combination thereof. Regardless of basin side slopes, seeding of the basin to promote vegetation shall be considered in the design to prevent rill and gully erosion.

Detention basin side slopes, embankments, and periphery shall be landscaped to mitigate and accentuate the basin. A minimum of two (2) plant units (as defined in the LDC) shall be required for every 100 feet of the perimeter of the basin, as measured at the top of the basin, in addition to LDC on-lot landscaping requirements. Underground facilities will not require landscaping.

Trees, shrubs, and other native vegetation may be used on basin side slopes and periphery. Most landscaping, except for grass/sod, should not be placed on the basin floor and shall not be planted in flow channels. Any landscaping placed on the basin floor shall not have a density such that it interferes with the stormwater storage function of the basin or regular maintenance activities. For basins located adjacent to streets, landscaping density shall be increased along the street frontage be screen the basin from the street.

The basin floor (if not using sod) and side slopes shall be seeded with a mix of native grasses (5 pounds/bOO square feet) . Prior to seeding, the ground surface must be scarified and cultivated to a depth of six (6) inches, and cleared of all debris and large stones (greater than 1-inch in any dimension). After clearing, grubbing, and cultivation, an organic fertilizer should be applied as specified by the landscape designer. An average of ten (10) pounds of fertilizer/bOO square feet is the desired treatment. When existing soil is unsuitable for growing native grass, incorporation of topsoil is recommended. A temporary irrigation system shall be installed to assist in the establishment of the seed and any other plant material shown on the landscaping plan.

Detention basins intended to accommodate passive or active recreation on the floor of the basin (e.g., playgrounds or sports fields) shall use turf grass on the basin floor. Prior to seeding or sodding, native material on the basin floor must be over-excavated to a depth of six (6) inches and planted with topsoil. A permanent irrigation system must be installed to maintain the turf grass.

Use of decomposed granite or small gradation rock shall be limited to basin side slopes of 3H:1V or flatter and is not recommended for use on basin floors, in flow channels, or at the basin inlets and outlets. Use of bio-mass filter landscaping treatment on basin floors is encouraged.

Over-excavation will be required to account for any volume displacement from landscaping islands, boulders, sod, or rock riprap placement within the basin and shall be noted on the Grading & Drainage and Landscape Plans.

8.4.4. Outlet Structures

Due to multi-frequency detention requirements, multi-stage outlet structures may be necessary in the design of many detention facilities. The outlet structure(s) should be designed to ensure complete basin drainage and can take the form of drop inlets, culverts, weirs, orifices, or any combination thereof. The use of weirs is encouraged. Figure 8-2 illustrates typical outlet structure configurations. Outlet structures that require manual operation to properly function are not permitted.
Outlet structures for detention basins shall be sized based on hydrologic reservoir-routing calculations and stage-discharge relationships. Inlet control should not always be assumed on detention facility outlet pipes, therefore outlet pipes should be evaluated using the FHWA, HDS-5 procedures outlined in Chapter 5.

Outlets structures shall be constructed such that they are physically opposite and the longest distance from the inlet structure(s), whenever possible. The distance between outlet structures and inflow points should be maximized to lengthen flow paths and detention times and provide effective settling of pollutants and sediment.

Metal outlet pipes projecting from a basin side slope or embankment are not permitted. Appropriate headwalls, commercial end sections, mitered sloped concrete protection, etc. are required for all outlet structures that incorporate only a pipe/culvert as the primary outlet structure.

The minimum recommended outlet culvert size for detention facilities is twelve (12) inches. Orifice plates are permitted provided the orifice plate is permanent, tamper-proof, and connects to a 12-inch minimum diameter outlet pipe. Outlet structures incorporating orifice plates shall include a trash rack to minimize clogging. Figure 8-3 illustrates a recommended method of orifice plate construction.

Outlet pipes less than twelve (12) inches in diameter may require adequate sediment and debris collection measures to be incorporated into the basin design to minimize clogging. Outlet structures shall not be oversized to account for debris blockage or clogging. Sediment traps should also be incorporated into the design of all multi-use detention facilities. Figure 8-4 illustrates a sediment trap concept.

Outlet pipes which present a hazard to children (e.g., outlet pipe connects directly to a large storm drain system) should be equipped with a bar/grate configuration in accordance with the Uniform Building Code safety requirements to prevent entrance of children into the outlet opening.

Local scour at culvert outlets as well as long term degradation downstream of the facility must be considered in the overall basin design. Erosion control measures such as energy dissipators, cut-off walls, riprap aprons or basins, and rock lined channels shall be provided to reduce outlet velocities and allow flows to return to natural conditions, to as great an extent as possible, prior to exiting onto the downstream property.

8.4.5. Detention Embankments

It is recommended that detention facilities be constructed with the storage volume located entirely below the natural ground or finished grade adjacent to the facility. Use of detention embankments or berms is discouraged due to the potential for downstream flood hazard in the event of failure, however, the following criteria must be incorporated into the detention embankment design, at a minimum:

a. Vegetated embankments shall be less than twenty (20) feet in height and shall have side slopes no steeper than 3:1 (horizontal:vertical). Embankments protected with riprap or other approved erosion control measure shall be no steeper than 2:1.
b. A geotechnical engineering study and slope stability analysis is required for embankments exceeding ten (10) feet in height or for embankment slopes exceeding those given above. It is recommended that ADWR, *Guidelines for the Design and Construction of Small, Low Hazard Potential Embankment Dams*, be utilized for design criteria.
c. Top width of the embankment shall be a minimum of 1/2 the height of the embankment.
d. Design of spillways shall also incorporate adequate erosion control and energy dissipating measures to ensure the stability of the embankment. The use of concrete spillways is preferred.
e. Seepage through the embankment and piping, particularly along the outside of the primary outlet conduit and spillway wall, must also be considered in the design of the embankment. Soil used for detention embankments should have a relatively low permeability after placement and shall be compacted to a minimum of 95%.

A minimum of one (1) foot of freeboard above the spillway elevation is required for public detention facilities that incorporate an embankment. The spillway elevation should be set equal to or above the 100-year water surface elevation.

Overflow or spiliway structures utilizing riprap protection shall incorporate an anchored biomass filter under and over the riprap layer. This shall then be covered with at least six (6) inches of topsoil and seeded with native grasses as described in Subsection 8.4.4 and this manual.

The use of concrete spillways is preferred for embankments exceeding ten (10) feet in height or when embankment stability could be compromised during overflow.

Public roadway fill sections may be used for detention public detention basins only. Special design considerations must be given to the stability of the fill slopes, seepage and piping, protection from erosion, overtopping, adequate drainage easements, and limits of ponding. All public roadway fill sections used as detention embankments or acting as a dam shall be designed in accordance with ADWR Dam Safety criteria.

8.4.5.1. Emergency Overflow

All detention facilities shall incorporate provisions for emergency overflow for instances where the primary outlet structure fails or storm events greater than the design capacity occur. The overflow shall be designed to pass the post-development 100-year peak discharge at a minimum. Design criteria shall incorporate the protection of the basin and the method of discharge. The overflow must be conveyed downstream in the same manner as it would have under pre-development, historic conditions.

Spillways for detention facilities that employ embankments must be designed to protect the embankment side slopes, particularly on the downstream side. It is recommended that the spillway be located to direct flow around the embankment on or through natural ground, as illustrated in Figure 8-5, versus placing the spillway directly over the primary outlet structure. The safety of downstream persons, properties, and consequences of embankment failure should be considered and evaluated.

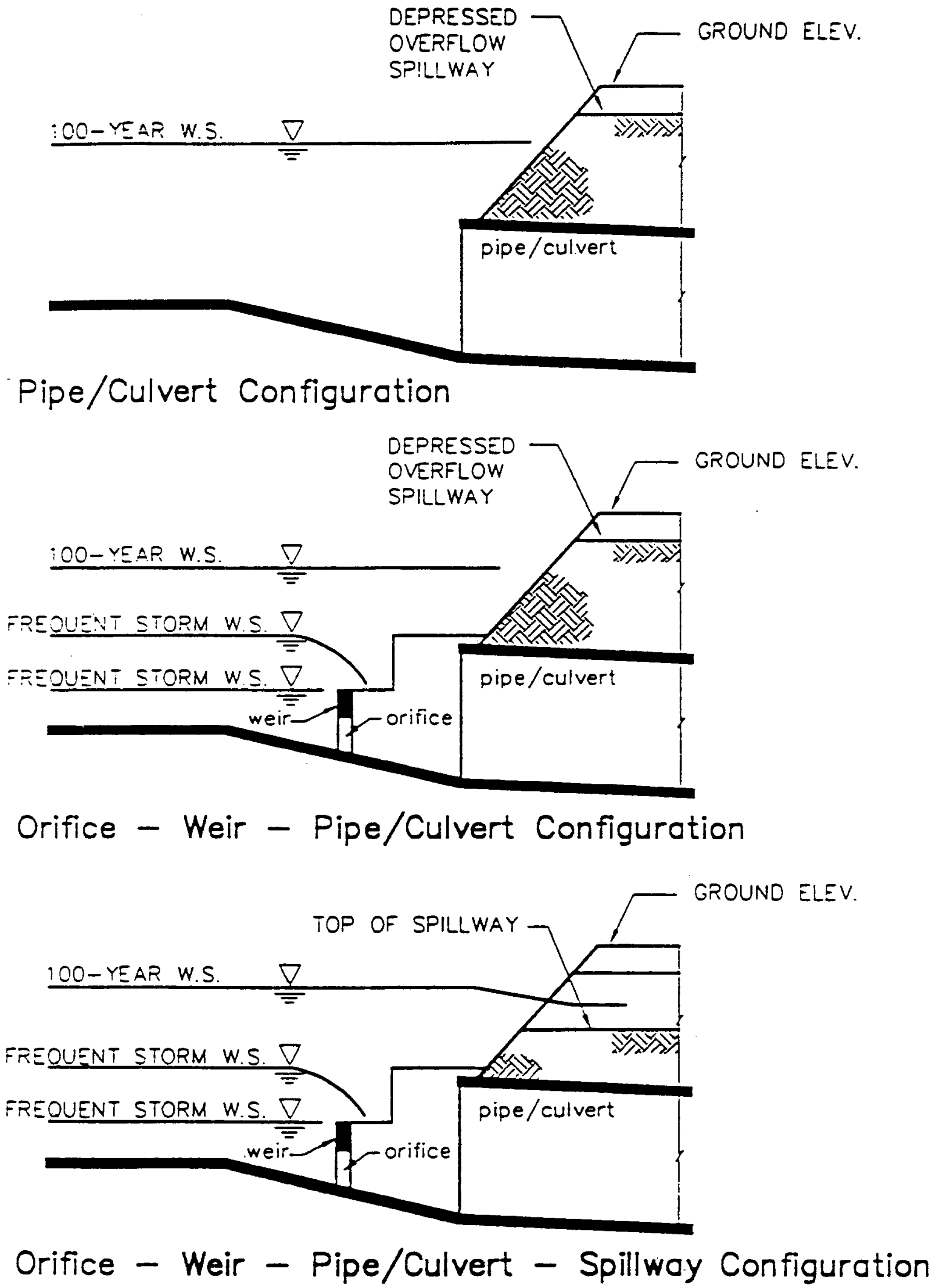
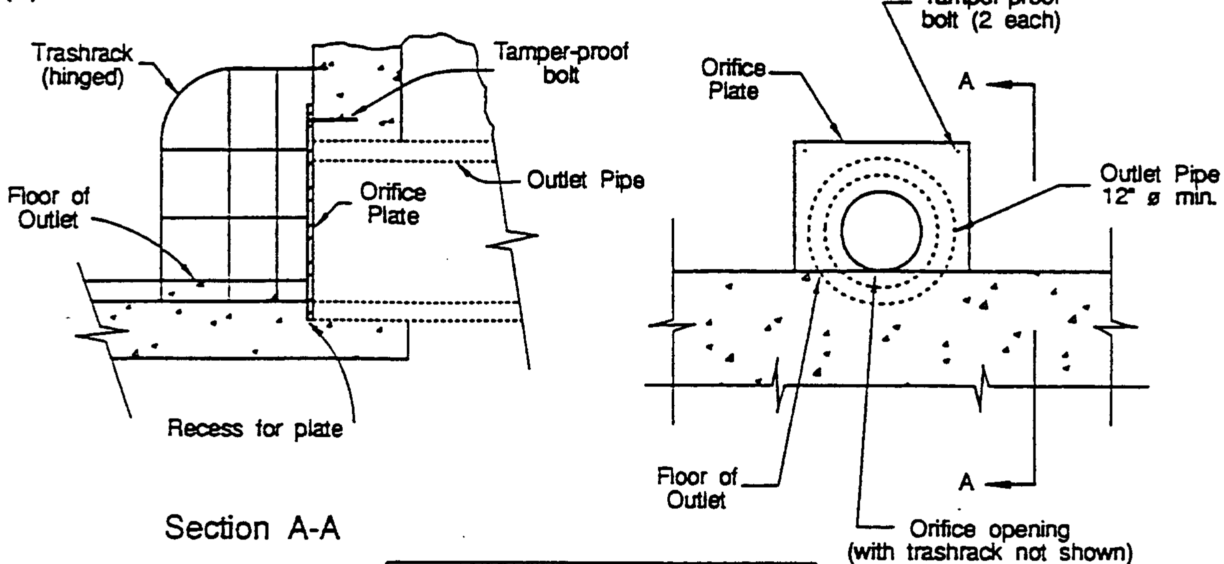


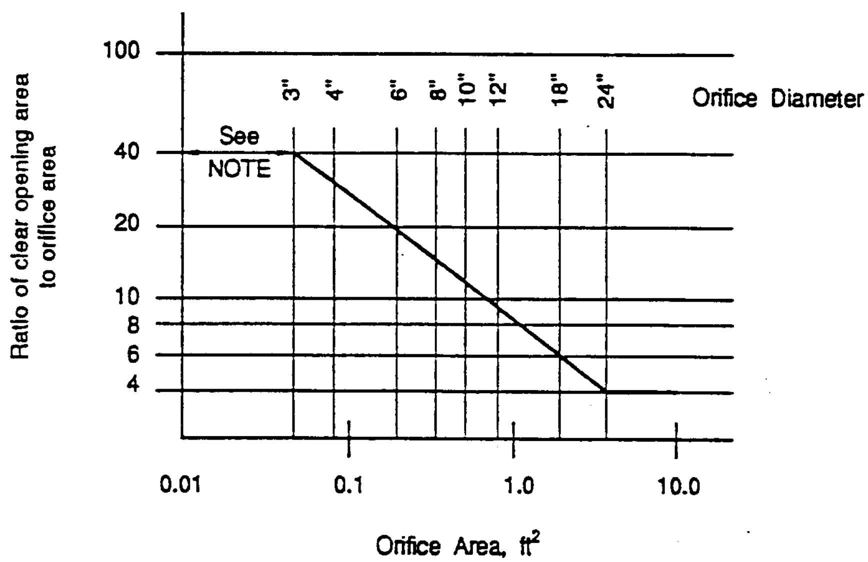
Figure 8-2: Detention Basin Outlet Structure Examples
Source: Pima County Dept. of Transportation and Flood Control District

(a) Orifice Plate Details



NOTE: Trashrack capacity to be 10 times orifice capacity.

(b) Trashrack Area Requirements



NOTE: For orifice diameter less than 3", use a minimum clear opening of 2 ft^2.

Figure 8-3: Orifice Plate Detail
Source: FCDMC Drainage Design Manual, Vol. II, 1996

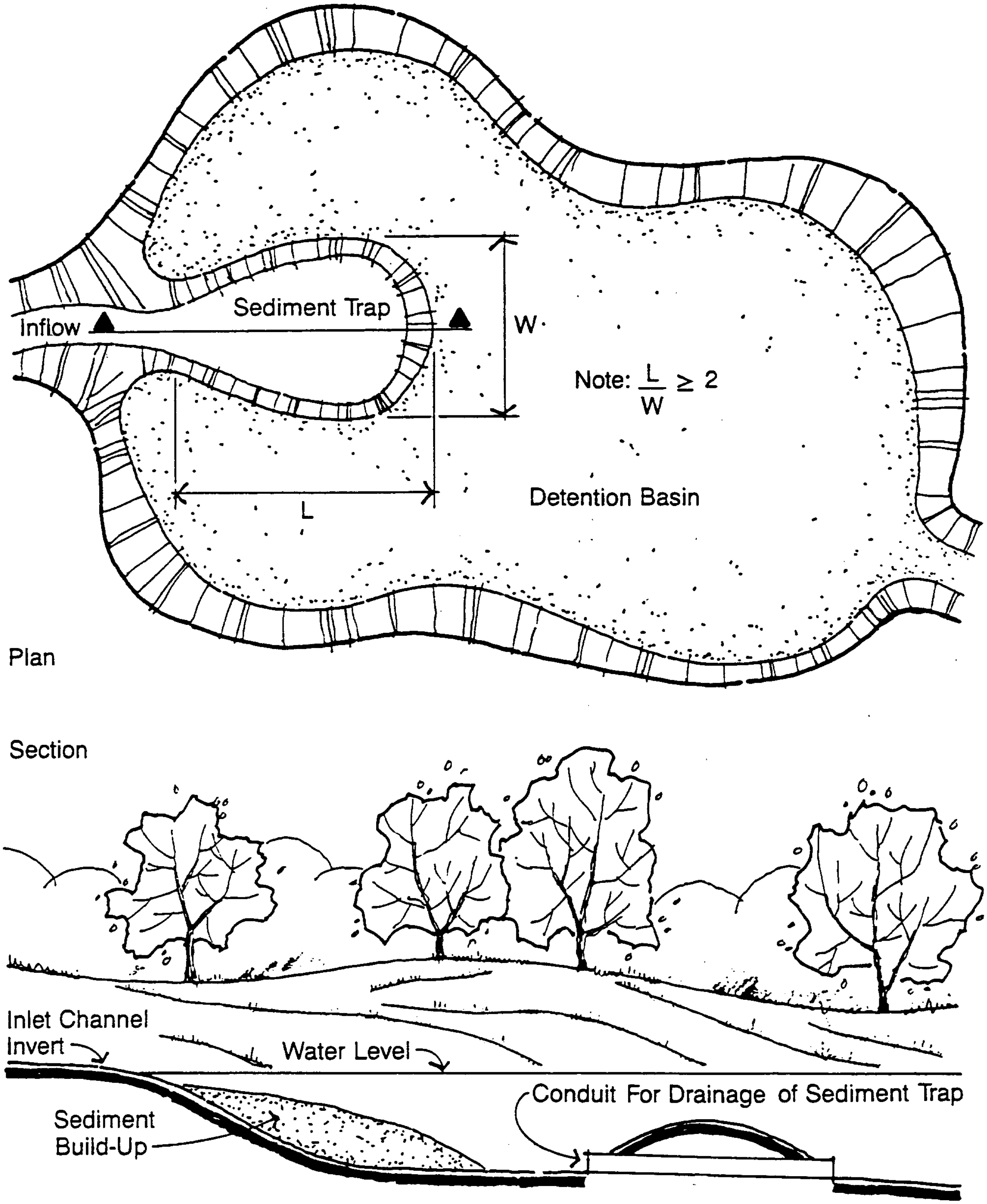


Figure 8-4: Sediment Trap Concept
Source: Pima County Dept. of Transportation and Flood Control District

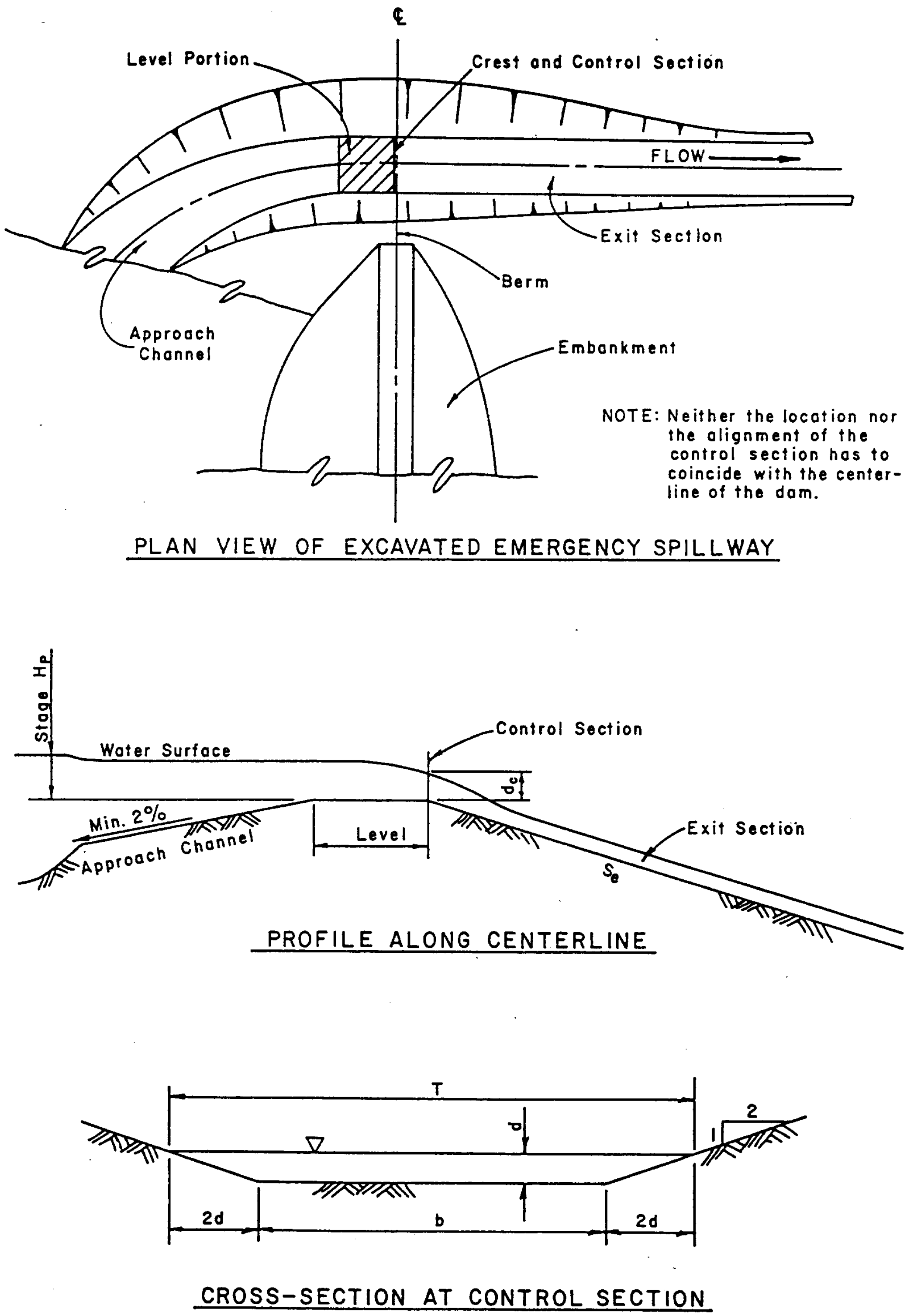


Figure 8-5: Emergency Spillway

8.4.6. State of Arizona Dam Safety Requirements

The Arizona Department of Water Resources (ADWR) has legal jurisdiction over all dams in the State of Arizona. ADWR must be contacted regarding dam safety jurisdiction and specific requirements in the design of any dam or embankment which may fall under state jurisdiction.

8.4.7. Safety

Detention facilities in general should be open, visible, and accessible so that persons can appreciate any hazard and guard themselves, and so that rescue efforts, if necessary, are not impeded.

To reduce the potential dangers associated with detention facilities the following recommendations are made in the design and construction:

- keep basin and water depths shallow.
- keep side slopes as flat (4H:1V or flatter) as possible to allow easier egress in the event of flooding.
- basin floor should consist of material or soil which provides firm footing when saturated. The ground surface side slopes around the facility should be made rough.
- break up ponding water with islands, boulders and mounds; put solid objects for holding onto in the ponding area such as large plants or rocks.
- avoid sudden constrictions and high velocities at outlets. Broad open rectangular weirs are preferable to anything involving the mouths of pipes and/or creating a hazardous vortex at the outlet.
- restricting public access and/or identifying potential hazards associated with detention basins, particularly in multi-use areas.

If detention facilities must be designed with steeper side slopes or vertical walls, adequate routes out of the basin should be provided.

Signage should be provided to inform the public of the basin purpose and potential safety hazard from stormwater. (e.g. , DANGER - This basin is designed to collect stormwater runoff DO NOT ENTER during rainy or threatening weather).

8.4.7.1. Security Barriers

Detention basins designed in accordance with Sections 8.4.3 and the previous section should preclude the need for security barriers to prevent unauthorized access. Security barriers, with maintenance access gates, are recommended along the top of all basin side slopes steeper than 3H:1V and where water depths exceed three (3) feet.

Security barriers may consist of masonry, wood, chain-link, or wrought-iron (pool-type) fencing and must be a minimum height of forty-two (42) inches. Fencing, if necessary, should not restrict the hydraulic capacity of inlet or outlet structures.

Appropriate hand railings shall be provided, per the Uniform Building Code, for all retaining walls or inlet/outlet structure headwalls and wingwalls over thirty (30) inches in height. Detail sections of proposed barriers shall be shown on the grading and drainage plans or development plans, as appropriate.

CHAPTER 9: LOW IMPACT DEVELOPMENT REQUIREMENTS FOR INFILTRATION AND REUSE OF STORMWATER

Volumetric increases to the city stormwater conveyance system, as the result of increased impervious surface have resulted in the need to upsize stormdrain systems in the City. While detention is effective in controlling peak discharges, the volumetric increase is not abated. Low Impact Development (LID) is an accepted standard in which stormwater is infiltrated and/or reused, resulting in the mitigation of these volume increases.

Low Impact Development will address water quality concerns that can arise when impervious area is added to previously undeveloped sites. Runoff from impervious areas often contains suspended solids and heavy metals along with other contaminants. Many of the Low Impact Development measures will remove some of these pollutants.

Water conservation is also a consideration when designing for the Low Impact Development requirements. Water can in many cases be reused or directed towards landscape areas in an effort to reduce water consumption.

The use of LID will help control volume increases and hence reduce the cost of upsizing downstream infrastructure as well as promote water conservation.

9.1. POLICIES

a. Stormwater LID is required for all new subdivisions, commercial and industrial developments, redevelopment of non-conforming sites (i.e., existing developed sites that do not have detention that have been razed and vacant for greater than six months), and other developments greater than 1/4 acre in size. LID shall be implemented according to the following schedule as measured from the effective date of Ordinance No. 2009-07:

   1. For the first year the program will be voluntary.

   2. In the second year, developments will be required to retain/infiltrate one half (1/2) inch of runoff from all additional impervious surfaces.

   3. In the third year, developments will be required to retain/infiltrate one (1) inch of runoff from all additional impervious surfaces. The requirement to detain for the 2-year storm will no longer be required once the 1-inch threshold is implemented.

b. LID requirements may be waived by the Stormwater Manager for the following:

1. Single-family residential structure or lot (i.e., not associated with a new subdivision).

2. Residential subdivisions with lot areas $\geq$ 1 acre in area, if it can be shown that such a waiver will not result in any adverse downstream effects, nor create any disturbance to the existing drainage patterns both within and adjacent to the subdivision.

3. Developments less than 1/4 acre or increases in impervious area of $\leq$ 5,000 square feet. It must be demonstrated to the satisfaction of the Stormwater Manager that there will be no increase in the potential for damages to adjacent properties and adequate off-site or downstream drainage capacity is available.

c. For developments requiring LID, all new impervious surfaces shall be infiltrated or reused in accordance with the current applicable standard.

d. The methodology for determining the required volume of stormwater to be infiltrated or reused is as follows:
Impervious Surface (square feet) X current requirement (ft.) (e.g. ½ inch or 1 inch) = volume requirement

e. In order to provide specific guidance for the design of these LID facilities, the City has developed the City of Flagstaff Low Impact Development (LID) Guidance Manual. The Manual is hereby adopted as part of these Requirements.

f. A waiver may be granted by the Stormwater Manager for deviations from the Integrated Management Practice designs in the LID Guidance Manual. The waiver request will be reviewed on a case-by-case basis and will require supporting analysis, documentation, etc. (as determined by the Stormwater Manager) be submitted and approved prior to granting of the waiver.

g. The City established a maximum ponding depth of (.3) three-tenths of (1) one foot without an engineered infiltration system.

h. Developments which are phased shall prepare a master stormwater LID plan for the entire development. The master stormwater LID plan must either be implemented with the first phase, if possible, or LID measures must stand alone for each phase. Interim temporary LID facilities may be required for phasing.

i. The City of Flagstaff shall only accept large-scale LID facilities for operation and maintenance. The City shall not accept small-scale, local on-site LID's for operation, maintenance, or liability.

j. Maintenance of local on-site LID facilities shall be the responsibility of the property owner or homeowner's association. The City shall reserve the right to periodically inspect any LID facilities to verify that regular maintenance activities are being performed. Final Plats;

Covenants, Conditions, and Restrictions (CC&R's); and/or Development Plans shall include a special note stating that (1) the owner(s) shall be solely responsible for the operation, maintenance, and liability for all LID facilities; and, (2) the City of Flagstaff may periodically inspect said LID facilities to verify that regular maintenance activities are being performed adequately.

k. The City of Flagstaff Parks and Recreation Division shall review and approve proposed stormwater LID facilities designed within designated public areas or parks. Review and approval from the Parks and Recreation Commission may also be required.

l. No part of a private LID shall be constructed in a public right-of-way or public utility easement.

9.2. SUBDIVISION REQUIREMENTS

All new subdivisions are required to provide LID for the entire subdivision, including the respective one-half of all abutting streets to the subdivision. Two or more subdivisions may join together to provide a common LID facility.

Preliminary Plat submittals shall be accompanied by a preliminary drainage report that identifies LID locations, types and sizes and a preliminary calculation of the required volume.

All Final Plat submittals require a final drainage report, in accordance with Chapter 2 of this manual, which technically demonstrates compliance with City of Flagstaff Floodplain Management Regulations, Stormwater Management requirements, and the drainage policies and design criteria set forth in this manual. This report must be submitted with the subdivision improvement plans and must be accepted prior to recording the final plat.
Urbanization and other land development activities, including construction of roads, changes natural pervious areas into impervious, altered surfaces. In addition, natural drainage systems are often replaced by lined channels, storm drains, and curbed streets. The result of such activities is an increase in the volume of runoff, peak discharge rates, erosion, and non-point source pollution due to the reduction in infiltration and natural vegetation.

In the absence of regional detention facilities and due to inadequate downstream capacities of existing streets, storm drain systems or channels, local on-site or sub-regional LID facilities are necessary to attenuate the increased runoff caused by development. LID facilities can also serve a dual purpose by improving the quality of stormwater discharges. The temporary storage of stormwater runoff can reduce the extent of downstream flooding, soil erosion, sedimentation, and surface water pollution. LID facilities can also be used to reduce the costs associated with large storm drain systems.

CHAPTER 10: EROSION HAZARD/BUILDING SETBACKS

In addition to the hazards associated with flooding, natural and unlined artificial channels can experience erosion of the channel banks over a long period of time or during a single storm event. This can present a particular danger to adjacent structures or other development as the channel banks erode or migrate.

The purpose of this chapter is to present erosion hazard and building setback policies to be applied to new development adjacent to natural watercourses or unlined engineered open channels in order to protect structures from potential damage. This requirement is in addition to other applicable setbacks in the City of Flagstaff Land Development Code and/or the Uniform Building Code.

10.1. POLICIES

a. All new buildings or structures constructed adjacent to a natural watercourse or an unprotected engineered channel located within City of Flagstaff shall be set back from the channel top of bank a sufficient distance in order to protect the structure from erosion of the channel banks and to allow for maintenance access and buffer.

b. The setback shall be measured from the top edge of the highest channel bank with the minimum setback being ten (10) feet to allow for access and maintenance considerations.

c. A slope stability analysis of the channel may be required by the Stormwater Manager if conditions exist which indicate that the channel bank may be unstable.

d. Exceptions or reductions of the building setback requirements may be granted by the Stormwater Manager if it can be demonstrated that adequate erosion and flow velocity protection can and will be constructed. All requests and plans for exception or reduction of setback requirements must be accompanied by a detailed soil-stability and/or sediment transport studies approved by the Stormwater Manager. Requests for reduction of the minimum building hazard setbacks must take access and maintenance into consideration.

e. Building setbacks from natural channels or unlined engineered channels may still be applicable even if it is demonstrated that the channel is stable under Policy 9.1.c. and d. above.

f. Natural vegetation is not an acceptable means of bank stabilization for the purpose of reducing building setbacks. Vegetation may be used along watercourses where flow velocities are less than five (5) feet per second during the 100-year discharge, provided that there is an acceptable seeding and maintenance program.

CHAPTER 11: EROSION AND SEDIMENT CONTROL

Erosion and sedimentation are natural or geologic processes by which soil material is detached and transported from one location and deposited in another, primarily by rainfall and stormwater runoff. Urban development tends to accelerate erosion and sedimentation, resulting in safety hazards, costly maintenance problems, unsightly conditions, instability of slopes, and disruption of ecosystems.

For these reasons, the total design process of any project (public or private) must be done with due consideration given to minimizing erosion and sedimentation, particularly during construction activities. This chapter will present general guidelines and criteria for erosion and sediment control measures or Best Management Practices (BMP's). Additional guidelines, policies, and criteria will be incorporated into subsequent revisions of this manual or as federal NPDES Phase II requirements mandate.

11.1. <u>POLICIES</u>

a. The effects of erosion shall be considered in the location and design stages of new development, since erosion can be controlled to a considerable degree by geometric design particularly relating to cross-section.

b. The level of effort by the engineer and use of erosion control measures shall be commensurate with the potential for erosion.

c. On-site erosion control measures shall be applied to reduce the gross erosion (e.g., gullies or rilling) and sediment transport from the site.

d. Sediment control shall be used whenever possible or necessary, to prevent offsite damage or sediment deposition on public streets.

e. Special consideration shall be given to the maintenance of existing and proposed vegetative cover on areas of high erosion potential such as erodible soils, steep slopes, drainageways, and banks of streams.

f. Storm Water Pollution Prevention Plans (SWPPP) are required for any land disturbing activity between one (1) and five (5) acres. Developers are responsible for complying with NPDES Phase I requirements for land disturbing activities greater than 5 acres. SWPPP's should be prepared by an engineer.

11.2. GENERAL GUIDELINES

The design of erosion and sediment control measures involves the application of common sense planning, scheduling, and the control activities which will minimize the adverse impacts of soil erosion, transport, and deposition. The following basic guidelines should govern the development and implementation of sound erosion and sediment control measures:

- The project should be planned to take advantage of topography, soil type(s), waterways, and natural vegetation on the site.
- The smallest practical area(s) should be exposed for the shortest possible time.
- Use of flat side slopes, rounded and blended with the natural terrain.
- Drainage channels designed with due regard to width, depth, slopes, alignment and protective treatment.
- Use of facilities for ground water interception.
- Use of berms, dikes, sediment traps and other protective devices.
- Use of protective ground covers, grassed swales, natural vegetation, and landscaping.
- A thorough maintenance and follow-up program should be implemented.

These basic guidelines should be tied together in the planning process that identifies potential erosion and sediment control problems before construction begins.

11.3. DESIGN CRITERIA

The following criteria represents the minimum requirements necessary for controlling erosion and sedimentation from construction activities. This criteria is intended to work in conjunction with individually developed plans and establish minimum standards of soil conservation practice which apply to all land disturbing activities.

11.3.1. Stabilization

The following criteria refers to stabilization of denuded areas and soil stockpiles:

1. Permanent or temporary soil stabilization should be applied to denuded areas within fifteen (15) days after final grade is reached on any portion of the site. Soil stabilization should also be applied within (15) days to denuded areas which may not be at final grade but will remain dormant for longer than sixty (60) days.
2. Applicable practices include but are not limited to temporary erosion control material, vegetative establishment, mulching, and the early application of a gravel base on areas to be paved.
3. Soil stockpiles should be stabilized or protected with sediment trapping measures to prevent soil loss.

11.3.2. Protection of Adjacent Property

Properties adjacent to the site of a land disturbance shall be protected from sediment deposition. This may be accomplished by completing necessary rough grading including detention facilities; by preserving a well-vegetated buffer strip around the lower perimeter of the land disturbance; by installing perimeter controls such as sediment barriers, silt fencing, temporary berms, sediment basins; or by a combination of such measures.

All temporary and permanent erosion and sediment control practices shall be maintained and repaired as needed to assure continued performance of their intended function. Damages which may occur to adjacent property during construction are the responsibility of the property owner and/or permittee.

11.3.3. Cut and Fill Slopes

Cut and fill slopes, including those for public roadways, shall be designed and constructed in a manner that will minimize erosion. Consideration shall be given to the length and steepness of the slope, the soil type, upslope drainage area, groundwater conditions, and other applicable factors. The following guidelines are provided to aid in developing adequate design:

a. Roughened soils surfaces are preferred to smooth surfaces on slopes.
b. Diversions shall be constructed at the top of long steep cut slopes which have significant (greater than 40 feet in length upslope) drainage area above the slope in accordance with Uniform Building Code, Appendix Chapter 33. Diversions or terraces may be utilized to reduce slope lengths.
c. Concentrated stormwater shall not be allowed to flow down a cut or fill slope unless contained within an adequate temporary or permanent channel flume or slope drain structure.
d. Wherever a slope face crosses a water seepage plane that endangers the stability of the slope, adequate subsurface drainage facilities or other protection measures shall be provided.

11.3.4. Stream Crossings

All construction related to crossing streams or washes shall be in accordance with U.S. Army Corps of Engineers Section 404 Permitting requirements, ADEQ Section 401 Water Quality Certification requirements, and requirements of the Stormwater Manager.

Construction vehicles shall be kept out of watercourses to the extent possible or in areas not identified on the approved plans. Where in-channel work is permitted, precautions shall be taken to stabilize the work area during construction, prevent sedimentation, and allow stormwater runoff through the work area. The channel bed and banks shall always be re-stabilized immediately after in-channel work is completed.

Where an active (wet) watercourse must be crossed by construction vehicles regularly, a temporary stream crossing shall be provided in accordance with criteria established by the Corps of Engineers 404 permitting program or the Stormwater Manager.

11.3.5. Vegetative Cover

Seed bed preparation shall be considered for all vegetative control measures on public roadway fill slopes. Soil characteristics such as depth of rock, pH, fertility, and moisture shall be evaluated during plant selection. Lime, fertilizer, and irrigation will often be required to establish vegetative cover.

Permanent seeding with perennial cover shall only be used for public roadway fill slopes not exceeding 2H:1V and six (6) feet in height, during acceptable growing seasons.

Sodding should be used in areas requiring additional protection from concentrated flow, such as grassed swales and waterways and storm drain inlets (if velocities are acceptable). Sodding may also be appropriate when an immediate aesthetic effect is desired.

Mulching shall be used on public roadway slopes with all seeding operations to provide temporary protection during adverse weather conditions. Typical mulching material includes straw, hay, and wood chips.

11.3.6. Storm Water Pollution Prevention Plans

Stormwater Pollution Prevention Plans (SWPPP), BMP's, erosion, and sediment control design methods and facilities should be in accordance with the Arizona Department of Transportation, <u>Erosion and Pollution Control Manual</u>, June 1995, as amended. The SWPPP can be a separate plan sheet or can be incorporated into the grading & drainage plans.

CHAPTER 12: ENERGY DISSIPATORS

12.1. GENERAL

High exit velocities and flow expansion turbulence at conduit outlets often result in local scour, channel degradation, and conduit failure. Typical rock riprap aprons may be appropriate where moderate outlet velocities exist, however, they are not suitable for outlet velocities exceeding ten (10) feet per second. Riprap basins or concrete energy dissipators may be required to reduce high velocity outlet flows to acceptable limits.

In general, an energy dissipators is any device designed to protect downstream areas from erosion by reducing the velocity of flow to acceptable limits. This Chapter presents information and design procedures which are based on FHWA, HEC-14, *Hydraulic Design of Energy Dissipators for Culverts and Channels.*

12.2. DISSIPATOR TYPE SELECTION

The dissipator type which is selected should be appropriate for the location. Generally there are two types of energy dissipators, internal and external. An internal dissipator is located within the conduit barrel and an external dissipator is located outside of the conduit barrel outlet.

12.2.1. Dissipator Types

Dissipator types are outlined as follows:

- Riprap Aprons (see Section 5.2.4.4)
- Riprap Scour Holes or Basins (see Section 11.3.4)
- Internal Dissipators
- External Dissipators
- Stilling Basins

Internal dissipators are used where right-of-way is limited, a scour hole at the outlet is unacceptable, debris is not a problem, and only moderate velocity reduction is needed. Internal dissipators are typically in the form of tumbling flow or increased resistance. Information and design procedures for internal dissipators can be found in HEC-14 and FHWA/OH-84/007, *Internal Energy Dissipators.*

External dissipators are used where the outlet scour is not acceptable, only a moderate amount of debris is present, and the conduit outlet velocity is moderate (Froude No. < 3). A design procedure for the USBR Type VI concrete impact basin is presented in Section 11.3.5. Design procedures and information on other dissipators such as Contra Costa, Hook Type, CSU Rigid Boundary, and Drop

Structures can be found in HEC-14.

Stilling basins are used where an outlet scour hole is not acceptable, debris is present, and the conduit outlet velocity is high (Froude No. > 3). Design procedures and information on stilling basins such as St. Anthony Falls can be found in HEC-14.

12.3. CONDUIT OUTLET STRUCTURES

12.3.1. Conduit Outlet Type

In choosing a dissipator, the selected conduit end treatment has the following implications:

- Conduit ends which are projecting or mitered to the fill slope offer no outlet protection from damage.
- Headwalls provide embankment stability, erosion protection, protection from buoyancy, and reduce damage to the conduit.
- Commercial end sections add little cost to the conduit, require less maintenance, retard embankment erosion, and incur less damage from maintenance.
- Aprons do not reduce outlet velocity, but if used shall extend at least one culvert diameter downstream and shall not protrude above the normal streambed elevation.
- Wingwalls are used where side slopes of the channel are unstable, where the culvert is skewed to the normal channel flow, to redirect outlet velocity, and to retain fill.

12.3.2. Design Considerations

The material selected for the dissipator should be based on a comparison of the total cost over the design life of alternate materials and shall not be made using first cost as the only criteria. This comparison shall consider replacement cost and the difficulty of construction as well as traffic delays (if applicable).

The design frequency and discharge used for the energy dissipator shall be equal to or greater than that of the conduit. The conduit shall be designed independently of the dissipator, with the exception of internal dissipators, which may require an iterative solution. The conduit design shall be completed before the outlet protection is designed and shall include computation of the outlet velocity.

The conduit exit velocity after dissipation shall be consistent with the maximum allowable velocity in the downstream channel. Downstream channel protection shall be designed concurrently with the dissipator design.

Most energy dissipators will normally be exposed to public access, therefore, designs for energy dissipators must address safety concerns as part of the design. Adequate signage, fencing or railings may be necessary to identify the potential hazard associated with high velocity flows or keep the public away from hazardous locations.

12.3.3. Riprap Scour Holes/Basins

The following riprap basin design procedure was adapted from HEC-14, which was based on laboratory data obtained from full scale prototypical models. The principal features of a riprap basin are:

- Preshaping and lining with riprap of median size (D_{50}).
- Constructing the floor at a depth of h_s below the invert, where h_s is the depth of scour that would occur in the pad of riprap size D_{50}.
- The overall length (L_b) of the basin is $15h_s$ or $4W_o$ which ever is greater, where W_o is the pipe diameter or barrel width.
- Sizing D_{50} so that $2 < H_s/D_{50} < 4$.
- Sizing the length of the dissipating pool to be $10(H_s)$ or $3(W_o)$ whichever is larger for a single barrel.

The depth (h_s), length (L_s) and width (W_o) of the scour hole are related to the characteristic size (D_{50}) of the riprap, the discharge (Q), the brink depth (y_o), and the tailwater (TW). The laboratory results for angular riprap were approximately the same for rounded material when rock size and other variables were the same.

When the ratio of tailwater depth to brink depth (TW/y_o) is less than 0.75 and the ratio of scour depth to riprap size (h_s/D_{50}) was greater than 2.0, the scour hole functions very efficiently as an energy dissipator.

For high tailwater basins ($TW/y_o > 0.75$) the high velocity core of water emerging from the culvert retains its jet-like character as it passes through the basin, and diffuses in a manner similar to a concentrated jet entering a large body of water. As a result, the scour hole is not as deep as with low tailwater and is generally longer. Therefore, riprap may be required for the channel downstream of the riprap basin.

12.3.3.1. Riprap Basin Design Procedure

The form given in Figure 11-1 can be used with this procedure.

Step 1 Determine Input Flow:

a. Calculate y_o, V_o, and Froude No. at the culvert outlet/brink.

where: y_o = normal flow depth at brink
V_o = normal velocity at brink

Step 2 Check the tailwater: Determine if $TW/d_o \leq 0.75$.

Step 3 Determine the D_{50}:

a. Using Figure 11-2, select D_{50}/y_e (where $0.25 < D_{50}/y_e < 0.45$).
b. Obtain h_s/y_e using the calculated Froude Number.
c. Check to see if $2 < h_s/D_{50} < 4$ and repeat until a D_{50} is found within the range.

Step 4 Size the basin as shown in Figure 11-3:

a. Determine the length (L_s) of the pool (where $L_s = 10h_s$ or $3W_o$ minimum).
b. Determine the length (L_B) of the basin (where $L_B = 15h_s$ or $4W_o$ minimum).
c. Determine the thickness of riprap:
Approach = $3D_{50}$ or $1.5D_{max}$
Remainder = dD_{50} or $1.5D_{max}$

Step 5 Determine V_B:

a. Basin exit depth (d_B) = critical depth at basin exit.
b. Basin exit velocity (V_B) = $Q/(W_B)d_B$.
c. Compare V_B with the average normal flow velocity in the natural channel (V_d).

High Tailwater Design:

Step 6 Design a basin for low tailwater conditions from Steps 1 - 5.

a. Compute equivalent circular diameter (D_E) for brink area from:
$A = [\pi(D_E)^2]/4 = d_o(W_o)$
b. Estimate centerline velocity at a series of downstream cross sections using Figure 11-4.
c. Size the riprap per Chapter 4.

Riprap Basin Design Example:

Given: *Box culvert - 8 ft by 6 ft*
Design discharge = 800 cfs
Supercritical flow in culvert
Normal flow depth = brink depth = 4 ft
Tailwater depth = 2.8 ft

Find: *Riprap basin dimensions*

Solution: *Low Tailwater Condition*

Step 1 - Determine Input Flow:

a. $y_o = d_E$ for rectangular section
$y_o = 4\ ft = D_E = 4\ ft$
$V_o = Q/A = 800/(4)8 = 25\ ft/s$
$Fr = V/(gd_E)^{0.5} = 25/[32.2)4]^{0.5} = 2.2 < 3.0$, OK

Step 2 - Check TW:

a. Determine if $TW/d_o \leq 0.75$:
$TW/d_o = 2.8/4 = 0.7 < 0.75$, OK

Step 3 - Determine d_{50}:

a. Using Figure 11-2:
Select $D_{50}/y_E = 0.45$
Thus, $D_{50} = 0.45(4) = 1.80\ ft$

b. Obtain h_s/y_E using Fr from Fig. 9-4
Thus, $h_s.y_E = 1.6$

c. Check if $2 < h_s/D_{50} < 4$
$h_s = 4(1.6) = 6.4\ ft$
$h_s/D_{50} = 6.4/1.8 = 3.6$
Thus, $2 < 3.6 < 4$. OK

Step 4 - Size Basin:

a. As shown on Fig. 11-3, determine length of pool (L_s)
$L_s = 10h_s = 10(6.4) = 64\ ft$
$min = 3W_o = 3(8) = 24\ ft$
Therefore, use $L_s = 64\ ft$

b. Determine length of basin (L_B)
$L_B = 15h_s = 15(6.4) = 96\ ft$
$min - 4W_o = 4(8) = 32\ ft$
Therefore, use 96 ft

c. Determine thickness of riprap:
Approach $= 3D_{50} = 3(1.8) = 5.4\ ft$
Remainder $= 2D_{50} = 2(1.8) = 3.6\ ft$

Step 5 - Determine V_B:

a. *d_B = critical depth at basin exit = 3.3 ft (assuming a rectangular cross section with width = 24 ft)*
b. *$V_B = Q/(W_B)d_B = 800/(24 \times 3.3) = 10$ ft/s*
c. *$V_B = 10$ ft/s $< V_d = 18$ ft/s*

<u>High Tailwater Condition</u>:

Data and Steps 1 - 5 are the same as above with the addition of the following:

Tailwater depth = 4.2 ft
$TW/d_o = 4.2/4 = 1.05 > 0.75$
Downstream channel can only tolerate 7 ft/s

Step 6 - High TW Design:

a. *Design a basin for low tailwater conditions from Steps 1 - 5.*
$D_{50} = 1.8$ ft, $h_s = 6.4$ ft, $L_s = 64$ ft, and $L_B = 96$ ft
b. *Compute equivalent circular diameter (D_E) for brink area from:*
$A = \pi D_E^2/4 = d_o(W_o) = 4(8) = 32$ ft^2
$D_E = [32(4)/\pi]_{0.5} = 6.4$ ft
$V_o = 25$ ft/s
c. *Estimate centerline velocity at a series of downstream cross sections using Fig. 11-4.*

L/D_E	L	V_L/V_o	V_L	D_{50}
10	64	0.59	14.7	1.4
15*	96	0.37	9.0	0.6
20	128	0.30	7.5	0.4
21	135	0.28	7.0	0.4

**is on a logarithmic scale so interpolations must be made logrithmically.*

d. *Size the riprap using Chapter 4. The channel can be lined with the same size rock used for the basin. Protection must extend at least 135 feet downstream.*

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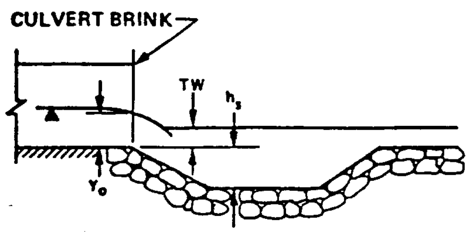

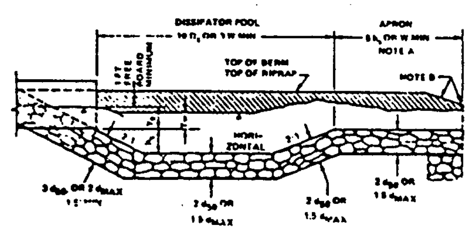

DESIGN VALUES	TRIALS		
CHART 11.3	1	2	3
D_{50}/d_e			
D_{50}			
F_r			
h_s/d_e			
h_s			
h_s/D_{50}			
$2 < h_s/D_{50} < 4$			

BASIN DIMENSIONS		FEET	
POOL LENGTH IS THE LARGER OF:	$10h_s$		
	$3W_o$		
BASIN LENGTH IS THE LARGER OF:	$10h_s$		
	$3W_o$		
THICKNESS APPROACH	$3D_{50}$		
THICKNESS APPROACH	$3D_{50}$		

TAILWATER CHECK	
TW	
d_e	
TW/d_e	
IF $TW/d_e > 0.75$ CALCULATE RIPRAP DOWNSTREAM USING CHART 11.4	
$D_e=(4A_c/\pi)^{.5}$	

DOWNSTREAM RIPRAP (Figure 9-5)				
L/D_e	L	V_L/V_o	V_L	D_{50}

Figure 12-1: Riprap Basin Design Chart
Source: AASHTO Model Drainage Manual, 1991

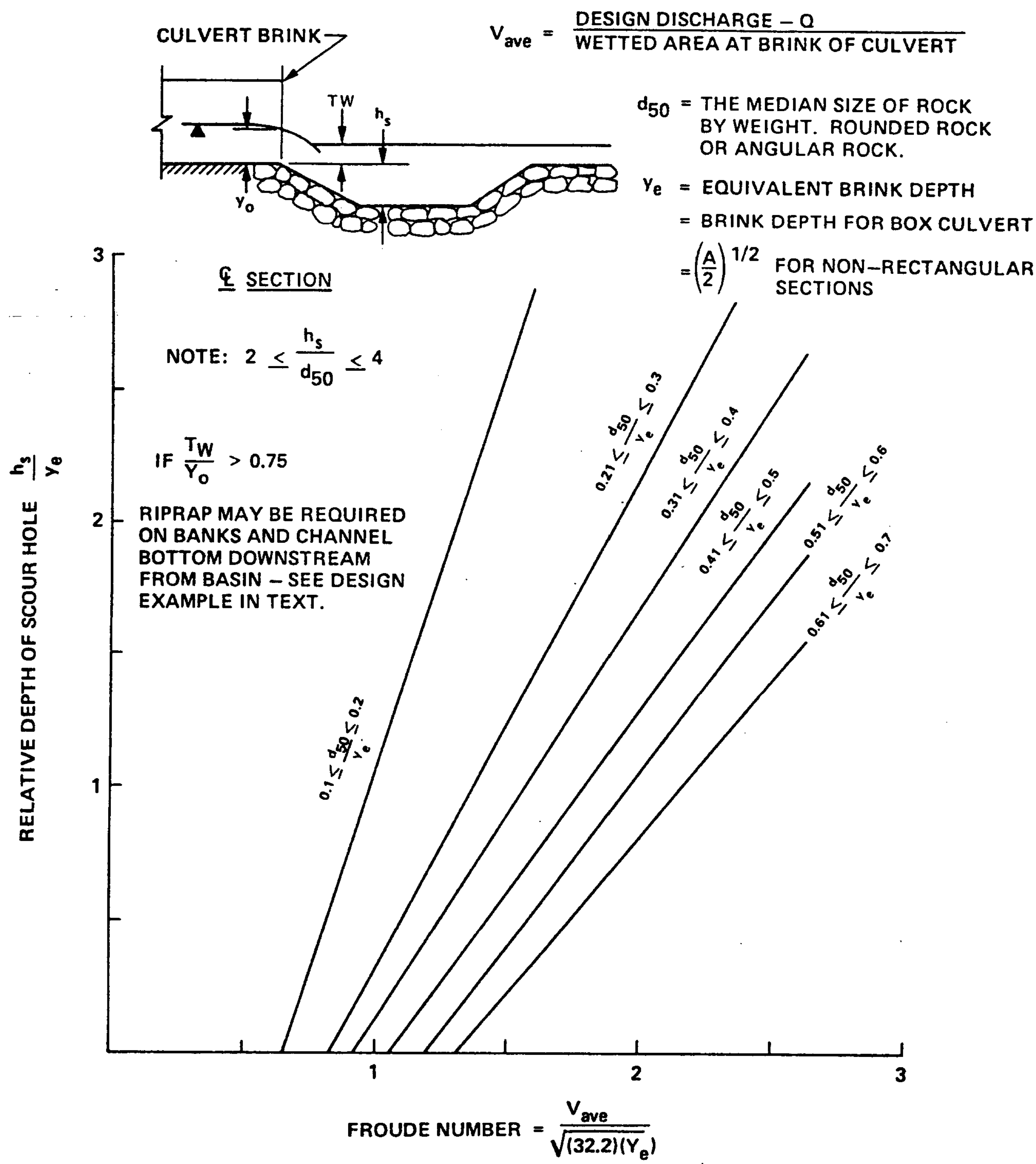

Figure 12-2: Relative Depth of Scour
Source: FHWA, HEC-14, 1983

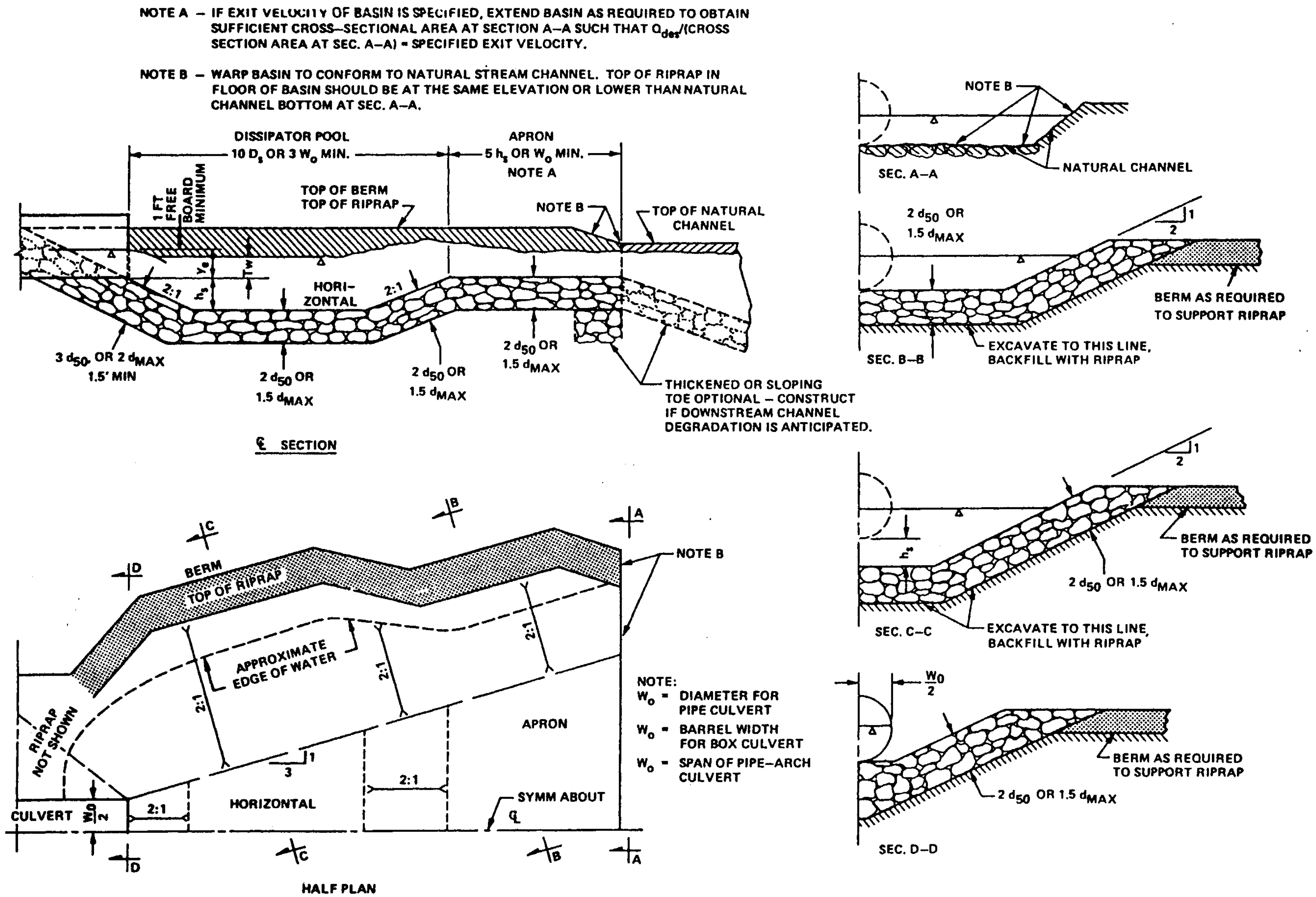

Figure 12-3: Details of Riprap Energy Dissipator
Source: FHWA, HEC-14, 1983

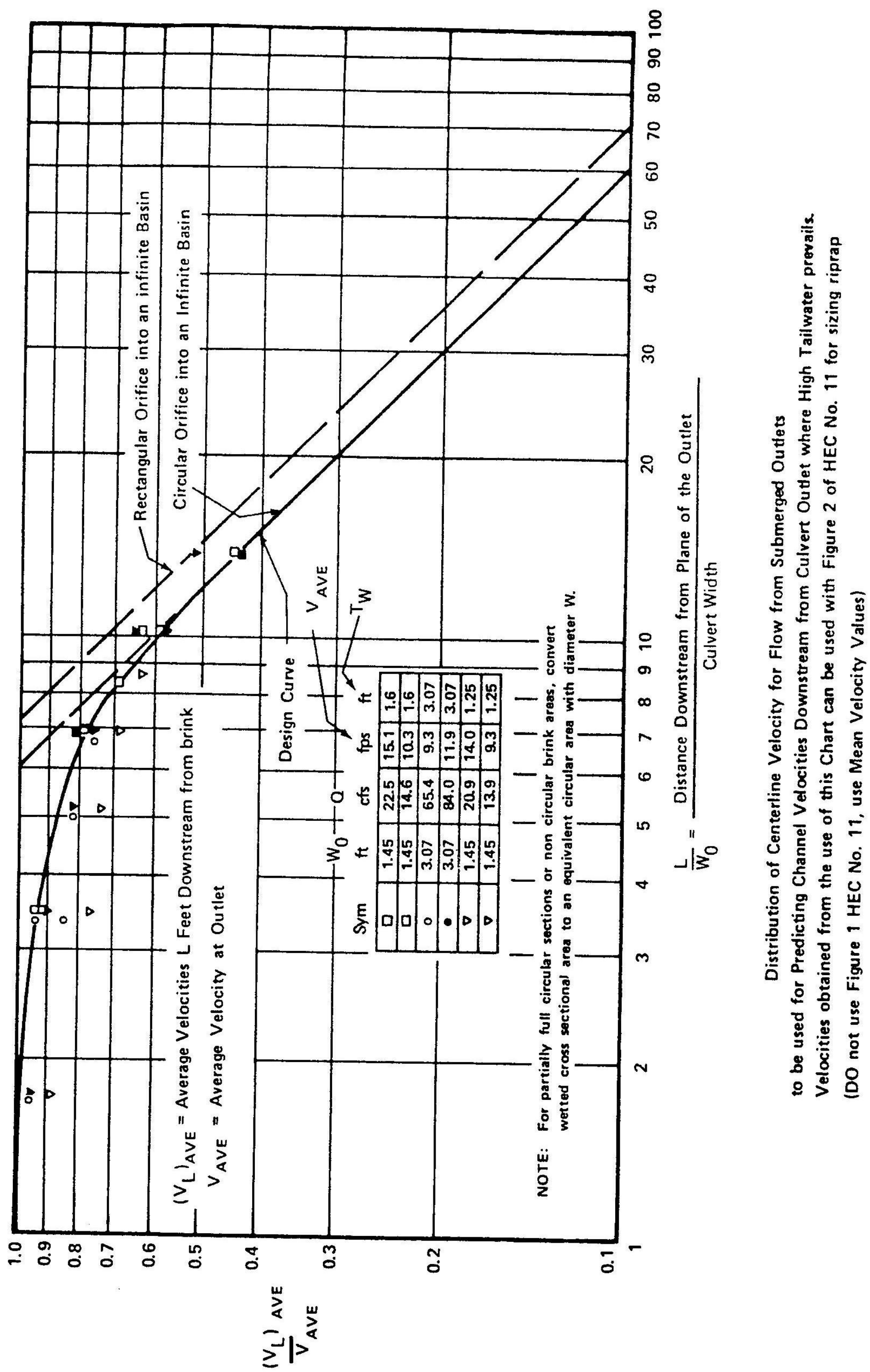

Figure 12-4: Velocity Distribution for High Tailwater

Source: FHWA, HEC-14, 1983

12.3.4. Concrete Outlet Structures

Appropriate reinforced concrete energy dissipation structures may be required to reduce erosive flow velocity (> 10 ft/s) to acceptable limits. These structures are suitable for a wide variety of site conditions and in some cases, they are more economical than riprap basins, particularly where long term operation and maintenance costs are considered. Figure 11-5 illustrates examples of concrete energy dissipation structures.

Initial design considerations for impact structures are:

- High energy efficiency is required,
- Low tailwater control is anticipated,
- Use of concrete is more economical due to structure size and long term maintenance, and
- Site conditions dictate the use of a concrete outlet structure due to presence of public use areas where plunge pools and standing water are unacceptable for safety reasons or right-of-way limits are small.

12.3.4.1. USBR Type VI Impact Basin

Design criteria and procedures for the USBR Type VI Impact Type Dissipator are provided in this section. SCS Technical Release No. 49, *Criteria For The Hydraulic Design Of Impact Basins Associated With Full Flow In Pipe Conduits* can also be used for full pipe flow and pipe diameters from 1.5 to 5.5 feet. Information and design procedures for other concrete structures can be found in HEC-14.

This structure is designed to operate continuously at the design flow rate, requires no tailwater to operate effectively, and may be used in open channels as well. However, it is not recommended where debris or ice buildup may cause substantial clogging.

Discharges of up to 400 cfs per barrel and velocities up to 50 ft/s (Froude No. < 9.0) can be used without subjecting the structure to cavitation damage. Design conditions should not exceed this criteria. For best results, the tailwater should be set so that the maximum tailwater does not exceed $h_3 + h_2/2$ (see Figure 11-6).

The basin should be constructed horizontal for all entrance conduits with slopes greater than 15°. A horizontal section of at least one pipe diameter long should be provided immediately upstream of the basin. For entrance pipes with slopes greater than 15°, provide a horizontal section at least two pipe diameters upstream.

For erosion reduction and protection from undermining, an end sill with a low flow drainage slot, 45° wingwalls, and a cut-off wall should be provided at the end of the basin. Riprap should be placed downstream of the basin for a length of least four times the conduit diameter. Figure 11-6 illustrates the USBR Type VI Impact Basin design configuration.

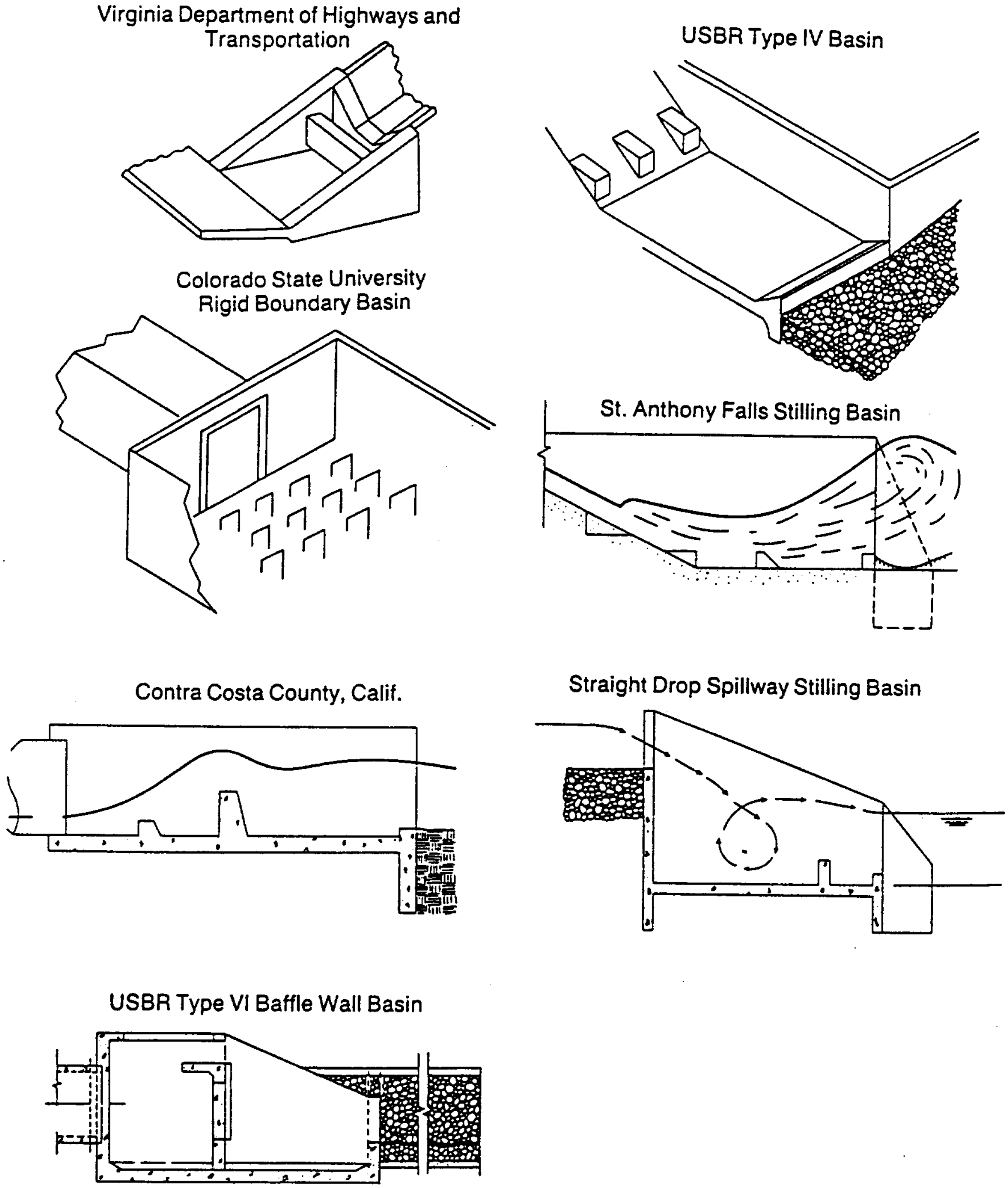

Figure 12-5: Energy Dissipation Structures
Source: North Carolina Erosion and Sediment Control Manual

12.3.4.2. USBR Type VI Impact Basin Design Procedure

The form given in Figure 11-7, can be used with this procedure.

Step 1 Calculate d_E:

a. For rectangular section, $d_E = D_o = y_o$.

b. For other sections, $d_E = (A/2)_{0.5}$.

Step 2 Determine Input Flow:

a. Calculate Froude No., $Fr = V_o/(g\ d_E)^{0.5}$.
b. Calculate specific energy, $H_o = d_E + V_o^2/2g$

Step 3 Determine Basin Width (W):

a. Using Figure 11-8, enter Fr and determine H_o/W
b. $W = H_o/(H_o/W)$

Step 4 Size Basin:

a. Using Table 11-1, obtain the dimensions of the basin as shown in Figure 11-6.

Step 5 Determine Energy Loss:

a. Using Figure 10-9, enter Fr and determine H_L/H_o.
b. $H_L = (H_L/H_o)H_o$.

Step 6 Determine Exit Velocity (V_B):

a. Exit energy $(H_E) = H_o - H_L$.
b. $H_E = d_B + V_B^2/2g$
$V_B = {}^*Q/W)/d_B$

USBR Type VI Design Example:

Given: *D = 48 dia. inch pipe (n = 0.15)*
Q = 300 cfs
$V_o = 40$ *ft/s*
$S_o = 0.15$ *ft/ft*
$d_o = 2.3$ *ft*

Find: *Basin dimensions, energy loss, and exit velocity.*
Solution:

Step 1 - Calculate d_E:

a. *Other sections other than rectangular, $d_E = (A/2)^{0.5}$*
$A = Q/V_o = 300/40 = 7.5\ ft^2$
$d_E = (7.5/2)^{0.5} = 1.94\ ft$

Step 2 - Determine Input Flow:

a. *Froude No.,* $Fr = V_o/(gd_E)^{0.5} = 40/[32.2(1.94))]^{0.5} = 5.05$
b. *Specific energy,* $H_o = d_E + V_o^2/2g = 1.94 + (40)^2/64.4 = 26.8$

Step 3 - Determine W:

a. *Using Figure 11-8 and Fr = 5.05,* $H_o/W = 1.68$
b. $W = H_o/(H_o/W) = 26.8/1.68 = 16\ ft$

Step 4 - Size basin:

a. *Using Table 11-1 and W, obtain remaining basin dimensions*

Step 5 - Determine energy loss (H_L):

a. *Using Figure 11-9 and Fr = 5.05,* $H_L/H_o = 0.67$
b. $H_L = (H_L/H_o)H_o = 0.67(26.8) = 18\ ft$

Step 6 - Calculate exit velocity (V_B):

a. *Exit energy* $*H_E) = H_o - H_L = 26.8 - 18 - 8.8\ ft$
b. $H_E = d_B + V_B^2/2g = 8.8$
$V_B = (Q/W)/d_B = (300/16)/d_B = 18.75/d_B$

d_B	V_B	$d_B + V_B^2/2g = 8.8$
$2.3 = d_o$	8.1	3.3
1.0	18.8	6.5
0.8	23.4	9.3, use
0.9	20.8	7.6

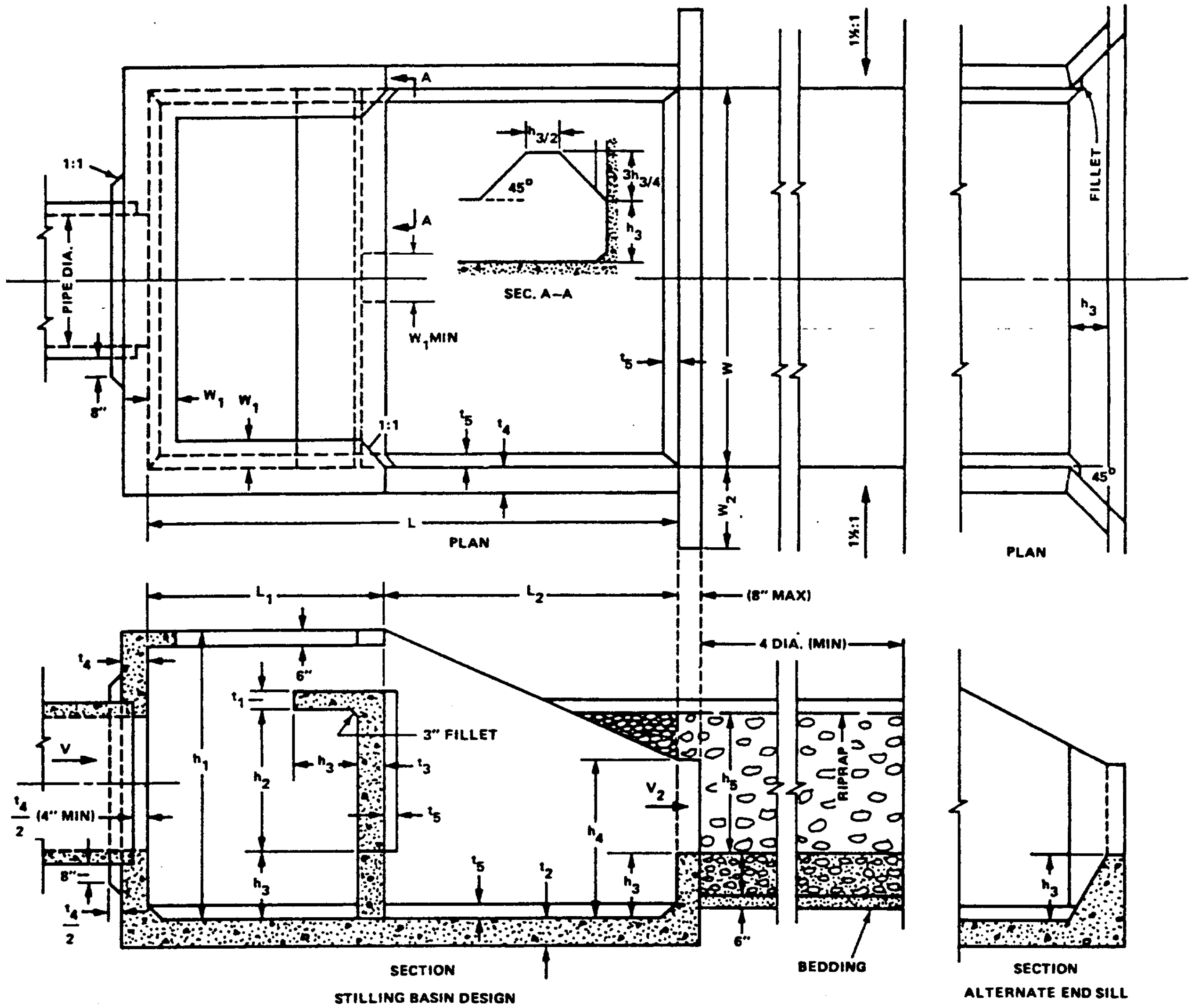

Figure 12-6: USBR Type VI Impact Basin
Source: AASHTO Model Drainage Manual, 1991

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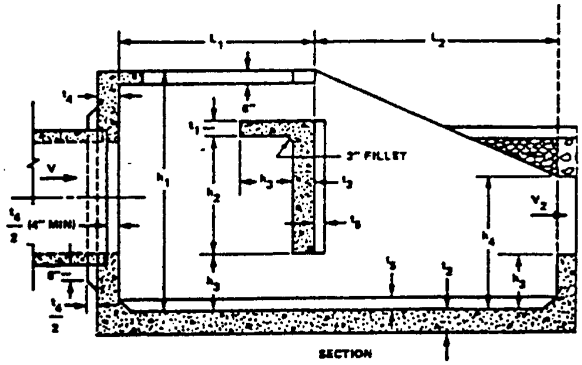

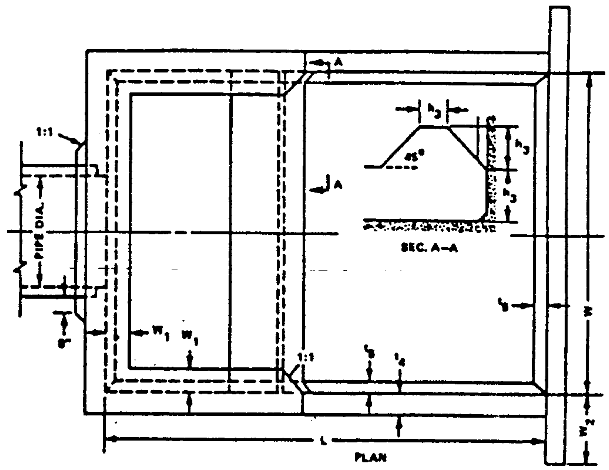

CHOOSE BASIN WIDTH (W) CHART 11.6	TRIALS		
	1	2	3
$d_e = y_e$			
V_o			
$H_o = d_e + V_o 2/2g$			
F_r			
H_o/W			
$W = H_o/(H_o/W)$			

CHECK OUTLET VELOCITY (V_B)			
H_L/H_o (CHART 11.7)			
$H_L = (H_L/H_o) H_o$			
$H_e = H_o - H_L$			
d_B			
$V_B = (Q/W)/d_B$			
$(H_e)_T = d_B + V_B 2/2g$			
IF $(H_e)_T \neq H_e$ CHOOSE ANOTHER D_B			

BASIN DIMENSIONS (FEET-INCHES) FROM TABLE 11.1							
W	h_1	h_2	h_3	h_4	L	L_1	L_2
W	W_1	W_2	t_1	t_2	t_3	t_4	t_5

Figure 12-7: USBR Type VI Impact Basin Design Chart
Source: AASHTO Model Drainage Manual, 1991

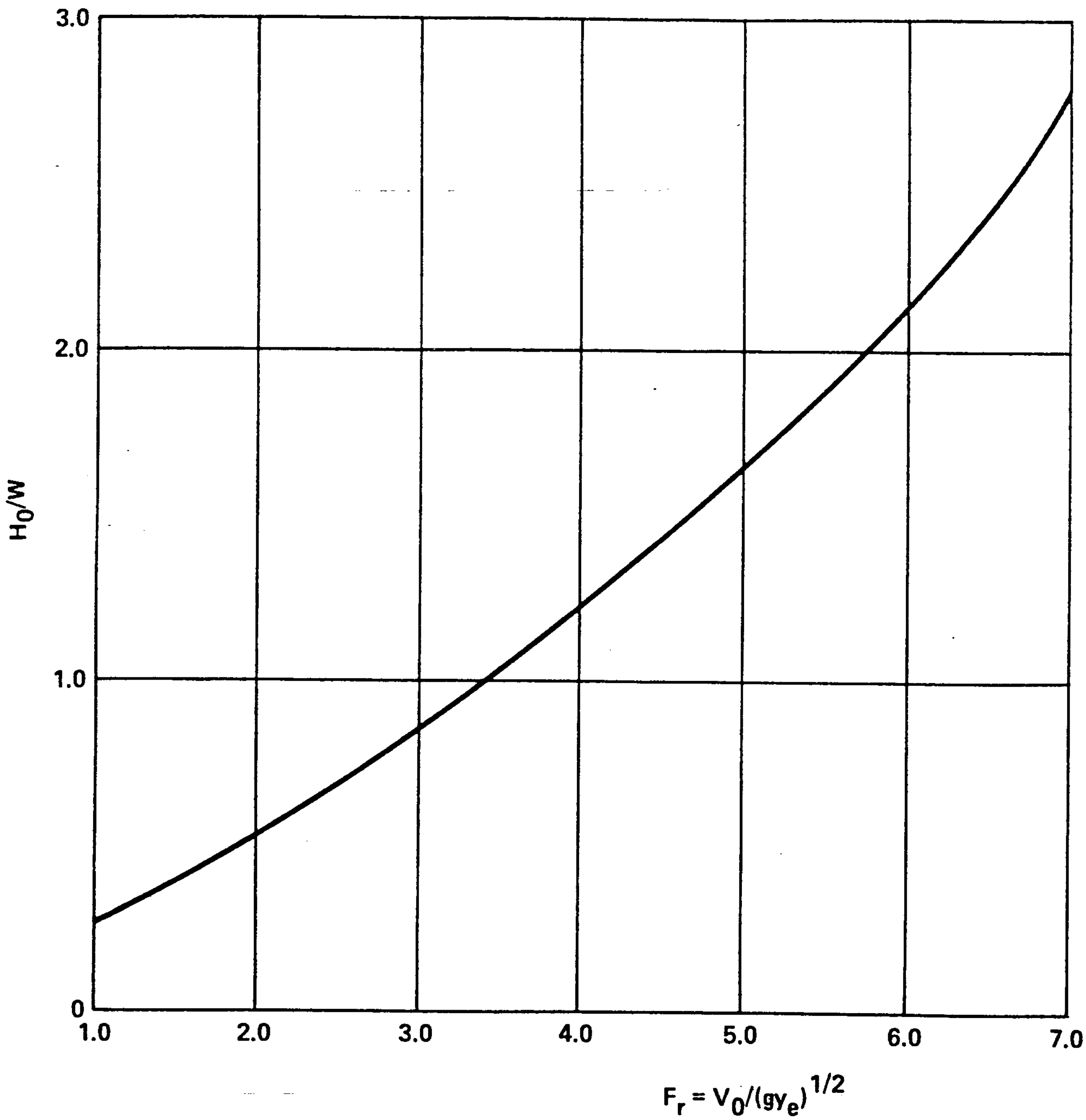

Figure 12-8: Design Curve For USBR Type VI Impact Basin
Source: AASHTO Model Drainage Manual, 1991

TABLE 12-1: USBR TYPE VI BASIN DIMENSIONS

W	h_1	h_2	h_3	h_4	L	L_1	L_2
4	3-1	1-6	0-8	1-8	5-5	2-4	3-1
5	3-10	1-11	0-10	2-1	6-8	2-11	3-10
6	4-7	2-3	1-0	2-6	8-0	3-5	4-7
7	5-5	2-7	1-2	2-11	9-5	4-0	5-5
8	6-2	3-0	1-4	3-4	10-8	4-7	6-2
9	6-11	3-5	1-6	3-9	12-0	5-2	6-11
10	7-8	3-9	1-8	4-2	13-5	5-9	7-8
11	8-5	4-2	1-10	4-7	14-7	6-4	8-5
12	9-2	4-6	2-0	5-0	16-0	6-10	9-2
13	10-2	4-11	2-2	5-5	17-4	7-5	10-0
14	10-9	5-3	2-4	5-10	18-8	8-0	10-9
15	11-6	5-7	2-6	6-3	20-0	8-6	11-6
16	12-3	6-0	2-8	6-8	21-4	9-1	12-3
17	13-0	6-4	2-10	7-1	21-6	9-8	13-0
18	13-9	6-8	3-0	7-6	23-11	10-3	13-9
19	14-7	7-1	3-2	7-11	25-4	10-10	14-7
20	15-4	7-6	3-4	8-4	26-7	11-5	15-4

W	W_1	W_2	t_1	t_2	t_3	t_4	t_5
4	0-4	1-1	0-6	0-6	0-6	0-6	0-3
5	0-5	1-5	0-6	0-6	0-6	0-6	0-3
6	0-6	1-8	0-6	0-6	0-6	0-6	0-3
7	0-6	1-11	0-6	0-6	0-6	0-6	0-3
8	0-7	2-2	0-6	0-7	0-7	0-6	0-3
9	0-8	2-6	0-7	0-7	0-8	0-7	0-3
10	0-9	2-9	0-8	0-8	0-9	0-8	0-3
11	0-10	3-0	0-8	0-9	0-9	0-8	0-4
12	0-11	3-0	0-8	0-10	0-10	0-9	0-4
13	1-0	3-0	0-8	0-11	0-10	0-10	0-4
14	1-1	3-0	0-8	1-0	0-11	0-11	0-5
15	1-2	3-0	0-8	1-0	1-0	1-0	0-5
16	1-3	3-0	0-9	1-0	1-0	1-0	0-6
17	1-4	3-0	0-9	1-1	1-0	1-0	0-6
18	1-4	3-0	0-9	1-1	1-1	1-1	0-7
19	1-5	3-0	0-10	1-2	1-1	1-1	0-7
20	1-6	3-0	0-10	1-2	1-2	1-2	0-8

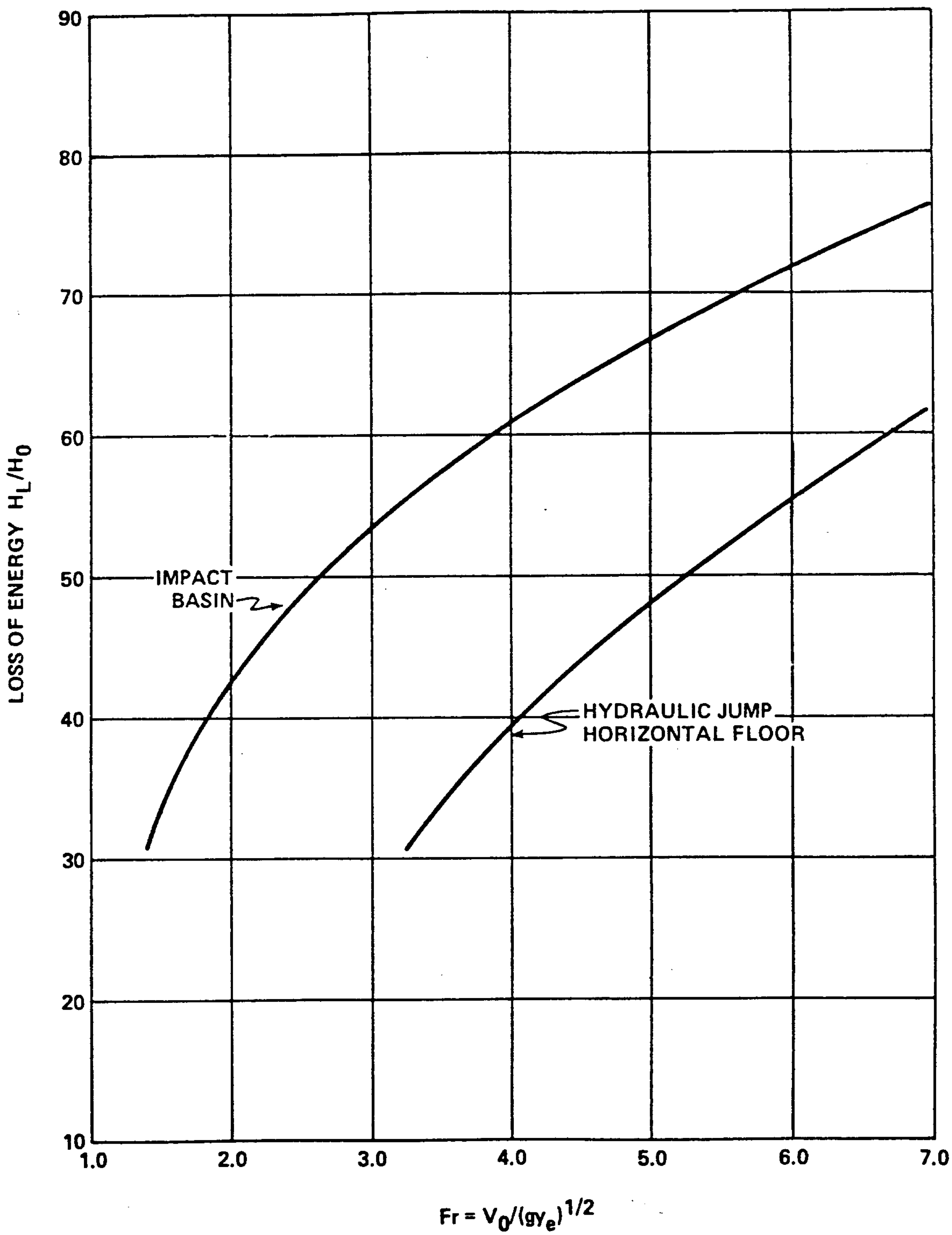

Figure 12-9: Energy Loss For USBR Type VI Impact Basin
Source: AASHTO Model Drainage Manual, 1991

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