



STATE OF LOUISIANA
DEPARTMENT OF TRANSPORTATION AND DEVELOPMENT
P.O. Box 94245
Baton Rouge, Louisiana 70804-9245
<http://www.dotd.la.gov/>

2011 HYDRAULICS MANUAL

Hydraulics
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PREFACE

The following text is a comprehensive documentation of LADOTD's Hydraulic Design Policies. This Manual is intended to be used in conjunction with LADOTD's Roadway Design Procedures and Details Manual. While design procedures must always be subject to refinement, the material presented herein represents current LADOTD policies.

The Manual is divided into categories based on topics with similar design criteria. The first part of a chapter gives LADOTD's design policies. Important theoretical information of applicable hydraulics for the region is highlighted in the additional parts of a chapter, such as Part A, Part B, etc. Included in the theoretical parts are lists of resource references and approved computer programs for the required hydraulic calculations. The theoretical portion of a chapter does not cover all information and should not be considered a substitute for the designer having a thorough working knowledge of hydraulics.

Although the writer of this Manual has strived for thoroughness, questions will undoubtedly arise which have not been covered. Such questions should be addressed to the Hydraulics Design Unit and amendments or revisions to this Manual may be made as the need arises.

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CHAPTER 1 GENERAL REQUIREMENTS

1.1 PURPOSE

One of the many problems in the design of highway projects is the satisfactory disposal of surface runoff. The water from the roadway, roadside channels and streams crossing the project must be disposed of properly without causing property or highway damage due to flooding or erosion. This Hydraulics Manual sets forth drainage design standards for Louisiana Department of Transportation & Development (LADOTD) projects.

1.1.1 Design Waivers

If unusual conditions warrant a deviation from the policies and procedures set in this manual, the Hydraulics Engineer Administrator should be consulted. A “design waiver” may be necessary. Further clarification of this procedure can be found in the Roadway Design Procedures and Details Manual.

1.2 PROJECT CLASSIFICATION

Projects are generally classified into one of two design types: urban or rural. However, some of the required hydraulics are the same for either case, such as the design of cross drain culverts or bridges.

1.2.1 Rural

Rural drainage design has open ditch facilities to catch the roadway runoff.

1.2.2 Urban

Urban drainage design usually consists of curb and gutter with storm drain pipe systems to drain runoff from the roadway and surrounding land.

1.3 DESIGN STORM FREQUENCY

Frequency is the number of times a flood of a given magnitude can be expected to occur on an average over a long period of time. Frequency is actually the probability that a flood of a given magnitude may be exceeded in one year. Frequency is an important design parameter in that it identifies the level of risk acceptance for the design of highway structures.

When a storm frequency is selected for a particular location, the designer is implying that the estimated effect of a larger storm on property, traffic and the environment does not justify constructing a larger structure at the time. Also, it should be noted that designing for small, frequent storms can result in traffic interruptions and can be costly since expensive repairs and damage to property can be high.

The general design storm frequency for a project will be determined as a function of the 20 year Projected Annual Average Daily Traffic (PAADT). The current AADT and PAADT should be requested through the Project Manager. Table 1.3-1 lists the frequencies for different drainage considerations. Frequencies for other conditions are discussed in their corresponding sections. If unusual conditions warrant, a “design waiver” to use a different frequency will be required.

Table 1.3-1 Design Storm Frequencies

DRAINAGE CLASSIFICATION	FREQUENCY
Interstates	50 years
Roadway Grade, Bridges, Cross Drains, or Side Drains under important side roads	25 or 50 years
Side Drains under private drives & average conditions	5 year
Median Drains	10 years
Storm Drains and Inlets	10 years
Roadside Channel	5 years
Detour Road Structures	1 year minimum

1.3.1 Determination of 25 or 50 Year Frequency

- a.) 25 year frequency is required when the PAADT is ≤ 3000 .
- b.) 50 year frequency is required when the PAADT is > 3000 .

1.3.2 Other Conditions to Consider a 50 Year Design Frequency

A 50 year frequency may be justified at individual sites on a project where:

- a.) The site is on a primary route for emergency vehicles or community evacuation.
- b.) The structure is considered a major drainage structure in a designated wetland area.
- c.) It is for urban arterial roads and streets.
- d.) The roads and streets have four or more lanes.
- e.) Ramps and approach roadways are within control of access boundaries of freeways.

1.4 PEAK DISCHARGE

1.4.1 General

Design peak rates of runoff are to be based upon the watershed conditions that are expected to exist 20 years in the future.

1.4.2 Gaged Stations

The United States Geological Survey (USGS) carries on a continuous program of stream gaging. This stream gaging data (gaging station locations, high water stages, and discharge records) should be used whenever it is applicable. For location of gaging stations and specific site data, refer to the USGS publications: “Magnitude and Frequency of Floods for Small Watersheds in Louisiana”, United States Geological Survey, 1979 and “Floods in Louisiana, Magnitude and Frequency”, 5th Edition, United States Geological Survey, 1998 which are available through LADOTD.

USGS has a yearly publication of the gage data collected. These should be used in conjunction with the above-mentioned references. For more information, contact the USGS office.

The U.S. Army Corps of Engineers also has gages around the state. For information concerning their gages, contact their New Orleans or Vicksburg office.

The methods for determining the peak discharge from gage data are described in the FHWA publication HDS-2 – Highway Hydrology. It is available on the FHWA website.

1.4.3 Ungaged Sites

When gage data is insufficient or unavailable, LADOTD uses three methods to estimate peak runoff rates essential to the hydraulic design of channels and structures. The methods vary according to the size of the area contributing runoff and to the structure application.

See Chapter 3 for the methods LADOTD uses for estimating peak rates of runoff in the design of highway drainage structures when insufficient or no observed data is available.

1.5 ROADWAY GRADE

When it is necessary to locate a highway within the limits of a floodplain, as is frequently the case in Louisiana, the presence of the highway can influence the ultimate flood stage and its impacts, while the flood stage can likewise have impact upon the highway. The impacts of the highway upon the flood and of the flood upon the highway are interdependent factors which must be balanced by design. A primary element of preliminary design is the selection of a proper roadway grade or elevation. The history of past floods, their effect on existing

drainage systems and their damage to property are of exceptional value in setting roadway grades. It is, therefore, essential that all aspects of grade selection be thoroughly studied and resolved very early in the preliminary design process.

1.5.1 Design Flood Stage

By definition, the road should not be overtopped by a flood of design frequency. Furthermore, it is generally desirable to maintain one (1) foot freeboard between the lowest elevation of the paved surface and the design flood stage.

The procedure for setting the minimum roadway grade is dependent upon the determination of the design flood stage for the roadway in question. Setting a roadway grade is based upon the road remaining open and usable for a flood of selected magnitude. A design storm frequency of either 25 years or 50 years should be utilized in setting the roadway grade, even for projects with storm drain systems or urban sections.

Proper evaluation of the flood potential factors in the planning stage of the project is essential, and may very well influence the choice of whether a rural or urban type section should be utilized.

1.5.2 Special Considerations for Urban Sections

Some will question why we would set grade based upon a 25 or 50 year frequency flood stage in an urban section, while designing the storm drain system for only a 10 year frequency. It is important to distinguish between the nuisance type surface flooding which may result from the drainage system being overburdened, and the complete inundation of the roadway by floodwaters. Placing the surface of the road above significant flood elevation and providing for surface drainage for lesser storms are two separate issues, both of which cannot always be achieved.

1.5.3 Raising the Roadway in an Urban Section

It may not be practical to raise the roadway above the design event if this is considerably higher than the existing roadway. Having to raise the roadway grade in urban sections may result in right-of-way acquisition problems, undesirable grades for driveways and other connections to the roadway, as well as storm water impoundment outside of the right-of-way.

1.5.4 Drainage Patterns in an Urban Section

For urban sections, the grade should not be set any lower than is necessary to permit drainage only of those areas which naturally slope toward the project. Sometimes, areas naturally draining toward the project are so large that, if allowed to drain over the curb, adequate surface drainage of the roadway becomes impractical or impossible. In such cases it may be necessary to set a grade which limits or prevents over-the-curb drainage and to intercept overland flow behind the curb. Other factors influencing grade in urban situations are

available right-of-way, utility conflicts and driveway connections. Setting low points at intersections and driveways should be avoided.

1.5.5 Flood Stage Investigation

The roadway grade determination is based on information gathered in conducting a flood stage investigation for the proposed alignment. The flood stage investigation must be performed before any culvert or bridge analysis can begin. The calculated headwater for minor structures should not be used to set roadway grades.

It is important that the flood stage investigation be conducted over the entire route of the proposed roadway to determine any areas of inadequacy. Several tools and methods to be used in the investigation are outlined below:

- a.) When gage data is available at a site, a statistical analysis, such as the Log-Pearson Type III analysis, should be performed to determine the stage for the design event. For a complete description of this design process, see the FHWA publication HDS-2 – Highway Hydrology.
- b.) In the absence of gage data, information gathered from persons having knowledge or records of past floods in the study area should be taken into consideration when determining the grade. This should include, but not be limited to: long-time residents, LADOTD district maintenance personnel, District Engineers and parish or city engineers. These interviews should be included in the hydraulics report with information on the person interviewed such as: length of residency, depth and length of time of roadway overtopping and approximate frequency of overtopping, etc. The designer should assign frequencies to reports of high water, and use sound engineering judgment to predict the stage for the design event.
- c.) Flood studies performed by the U. S. Corps of Engineers or USGS can be used if available.
- d.) FEMA Flood Insurance Rate Maps should be reviewed; however, these maps should not be used as the sole source of the flood stage investigation.

1.5.6 Impacts on Road vs. Impacts on Property

The concept is to select a minimum roadway elevation based upon the relative importance of keeping the road open and usable and the consequences of having the road closed or damaged by flood waters. An equally important consideration is the flood impact to adjacent property which is chargeable to the highway. This factor is especially important when raising the grade of a roadway which has a history of overtopping.

1.5.7 Traffic Considerations

Traffic interruptions due to flooding are always a serious occurrence. Interruptions and short delays due to floods may sometimes be warranted. However, the seriousness of the situation must be evaluated by considering all related factors including traffic volume, length of predicted delays, availability of alternate routes, overall importance of the route, etc. Under some circumstances, over-the-road flow may be designed for with the documented concurrence of the Chief Engineer.

1.6 FIELD SURVEY ESSENTIALS

For a complete guideline of LADOTD's survey criteria, consult the LADOTD Location & Survey Manual. Below is a list of the minimum information needed to perform an adequate hydraulic analysis to determine the required cross drain (culvert or bridge), its size, and proper location. The following data should be secured at the time normal field survey is in progress:

- A.) Drainage areas in acres contributing flow to streams crossing the project.
- B.) Type of terrain and land use.
- C.) Ridge lines.
- D.) Size, angle of crossing and flow lines of existing drainage structures.
- E.) Size of adjacent structures, upstream and downstream.
- F.) High water elevations for streams crossing the project, both upstream and downstream. (Show source of information.)
- G.) The low tide, high tide, and average tide elevations on streams affected by tidal movements.
- H.) Adequate elevation information for the designer to determine the general cross section and slope of any ditches or streams crossing the project. For streams which may require bridges or large culverts, a traverse, a profile, and cross sections for a distance far enough upstream and downstream to obtain a true representation of the floodplain should be provided.

1.7 DRAINAGE DESIGN FOR DETOURS

Where feasible, traffic should be diverted around the construction over state routes. When this is impractical, detour roads adjacent to the construction must be provided.

1.7.1 Detour Types

LADOTD recommends three different types of detours, namely B, C and D. These detours are discussed in detail and shown in chapters 4 and 8 of the Roadway Design Procedures and Details Manual.

For low profile runarounds, used on off-system projects only with a current ADT of 250 or less, the contractor shall be responsible for the drainage requirements. In the event the detours are damaged by flood waters due to insufficient hydraulic capacity, the contractor is responsible for the costs of restoring the detours to their original condition.

1.7.2 Hydraulic Capacity

Drainage structure openings for Types B, C, and D detours will be determined by the designer and shown on the detour layout. LADOTD will assume the responsibility for the hydraulic capacity of these detours, and, in the event the detours are damaged by flood waters due to insufficient hydraulic capacity, will reimburse the contractor for the costs of restoring the detours to their original condition. Drainage structure openings for Type B, C and D detours are designed according to the following design criteria.

- a.) The design storm frequency should be calculated in accordance with the AASHTO Model Drainage Manual, Chapter 19 - "Construction", Appendix 19A - "Temporary Hydraulic Facilities". The minimum design storm frequency for the detour is two (2) years.
- b.) The design discharge is estimated by methods previously described in this chapter using the computed design storm frequency.
- c.) The allowable headwater shall be set at ½ ft (6 in.) below finished grade of the detour, unless other controls govern the situation, such as surrounding development or predicted excessive outlet erosion.
- d.) The detour should be lower than the main road in order to provide relief in case of a large storm event occurring while the detour is in place.

1.7.3 Bridge Detours

When a bridge is necessary for the detour structure, 20' spans with bulkheads should be used for hydraulic computations. The length should be based on the type of detour or must clear the channel, (bulkheads should not be placed in the channel).

1.8 EROSION CONTROL

1.8.1 Temporary Erosion Control

Temporary erosion control products are used in sediment control from construction sites. Their placement and measures for this use in the Storm Water Prevention Pollution Plan are described in the publication “LADOTD Erosion Control Guidelines”. A copy of this publication is available on LADOTD’s website.

1.8.2 Permanent Erosion Control

The measurement and inclusion of permanent erosion controls on design plans are described in the publication “LADOTD Erosion Control Guidelines”. When to use permanent erosion controls and how to design for them are described elsewhere in this manual in the applicable chapters.

1.9 REFERENCES

- A.) Model Drainage Manual, Third Edition, AASHTO, 2005

CHAPTER 2 REPORT AND PLAN PREPARATION

2.1 GENERAL INFORMATION

The Roadway Design Procedures and Details Manual covers roadway plan preparation in Chapter 8. What is listed below is not intended to replace this procedure but instead highlight and emphasize parts pertaining to drainage structures. The Hydraulics Manual and the Roadway Design Procedures and Details Manual are independently written and updated. If there is a discrepancy between the two manuals, then both the Hydraulics Unit and the Project Manager should be consulted as to what should be required for a specific project.

2.2 HYDRAULICS REPORT

The submittal of preliminary plans to the Hydraulics Unit for review should be accompanied with one bound report that is properly indexed, typed and neatly arranged. Pages are to be numbered for referencing purposes when reviews are sent back to the designer. Also include in the report, the name of the firm and designer along with a phone number to contact the designer during normal business hours should questions arise about the design. The Hydraulics Unit will review the report for adherence to LADOTD's current policy. However, the hydraulic design is the responsibility of the designer. **All submitted reports must have the stamp, date and signature of the Professional Engineer in charge.**

2.2.1 Report Contents

Included in the report should be:

- a.) Design Classification:
Design classification of the project should be noted in the transmittal letter or on the title sheet when a project is submitted to the Hydraulics Unit.
- b.) Site Conditions:
Brief commentary should be included describing the conditions of the site, the reasons for the proposed major structure(s) and what kind of effect these structure(s) will have on the site.
- c.) Design Assumptions:
Thorough documentation of all design assumptions and design decisions is critical. All factors, especially judgmental factors, governing the selection of design parameters such as allowable backwater, allowable headwater, permissible velocity, outfall stage, etc., must be documented by the designer. However, the basis for the selection of the limiting

factors must be defensible by sound engineering principle. Any decisions reached between the designer and the Project Manager affecting the hydraulic design should also be explained in the document.

d.) Calculations:

All calculations contributing to the design of the proposed hydraulic structures (i.e., how the tailwater or stage elevation was determined, the discharge calculations and the sizing of any structures, etc.) are to be included in the report. The designer should ensure that the numbers and values shown on the corresponding plans match the calculations submitted.

e.) Photographs:

When there is a major stream crossing or special design requirements on a site, color photographs showing the conditions should be included in the report.

2.2.2 Bridge Hydraulic Reports

The requirement for a bridge hydraulic report is discussed in Chapter 11.

2.3 PRELIMINARY PLANS

2.3.1 Sheets Required

Preliminary plans are to include:

- a.) Existing Drainage Map
- b.) Design Drainage Map
- c.) Plan/Profile sheets

2.3.2 Plan Revisions

When plans are returned after being reviewed by the Hydraulics Unit, any changes, additions, or deletions are to be incorporated in the plans where appropriate. It is important that the calculations always match what is shown on the plans. Calculations should be revised when necessary and those changes incorporated into the plans. In most cases, the revised plans and calculations should be resubmitted to the Hydraulics Unit for another review.

2.3.3 pH & Resistivity

During the early stages of preliminary planning, pH, resistivity, and channel probings should be requested through the Pavement and Geotechnical Services Section by the Project Manager. The District Laboratory will furnish the Pavement and Geotechnical Services Section with a soil report. This report will include channel probings (when required), soil and water pH and resistivity values along with notes on abnormal situations such as high iron ore concentrations, industrial runoff, etc.

2.3.4 Detours

When a detour is specified, the recommended size and type drainage structure needed is detailed on the plans. The finished grade elevation of the detour road should also be shown.

2.4 PLAN-IN-HAND

All hydraulics related comments and recommendations should be forwarded to the Hydraulics Unit for further study.

2.4.1 Existing Conditions

The existing structures as shown on the plans should be verified. Significant physical characteristics of the site should be indicated on the plans such as:

- erosion
- evidence of flooding
- recent property development.

2.4.2 Proposed Design

At the plan-in-hand inspection, each structure site should be examined and a judgment made as to whether the design information used for the structure is appropriate. For larger culverts, the design headwater as shown on the prints should be verified with site conditions. Comments concerning topography, nature of channel, necessary channel changes, etc. should be recorded on the prints.

2.4.3 Culvert Extensions

When a culvert is to be extended, the following information needs to be verified in the field:

- size
- number of openings
- angle of crossing
- material type
- distance to left and right of the centerline

- type of end treatment (headwall, etc.)

For multi-barrel reinforced concrete boxes, the interior wall(s) thickness also needs to be verified.

2.5 FINAL PLAN PREPARATION

2.5.1 Sheets Required

Final plans are to include:

- a.) Summary of Estimated Quantities sheets
- b.) Summary of Drainage Structures sheets
- c.) Standard Plans/Special Details required for the project.

2.5.2 Plan Submittal

Upon completion of final plans, the plans are submitted to the Hydraulics Unit for final review. The drainage, including locations, descriptions, structure sizes, lengths, flow lines, Standard Plans/Special Details, allowable materials, quantities and field inspection notes are reviewed and checked thoroughly during final hydraulics review. Any changes from plan-in-hand regarding the structures should be incorporated into the final plans.

2.6 DRAINAGE SHEETS REQUIRED IN THE PLANS

The following sheets are required for an adequate check and comprehension of a drainage design.

2.6.1 Existing Drainage Map

A map showing existing drainage is furnished as part of the survey information for most projects. The primary purpose of this map is to show:

- a.) Drainage Areas: Size, shape, and direction of flow of all drainage areas that will affect the proposed roadway drainage, including existing cross drains.
- b.) Existing Drainage Structures: Size of all existing drainage structures under existing roadways and railroads in the vicinity.
- c.) Documented high water elevations at the existing cross drains.
- d.) Other pertinent information, such as areas where flooding of the existing roadway occurs.

Sample Existing Drainage Maps for rural and urban projects are shown in Figures 8-41 and 8-42 in the Road Design Procedures and Details Manual.

2.6.2 Design Drainage Map

Preparation of a comprehensive Design Drainage Map is fundamental to a good drainage design. The Design Drainage Map (DDM) serves to document design information and procedures, and may serve as a valuable reference long after the project has been built. The elements of this map include:

a.) pH and Resistivity:

The pH and resistivity table and probings (when required) are located on the Design Drainage Map (DDM).

b.) Cross Drain Structures:

Required cross drain structures are plotted at their actual location and are numbered consecutively with structure numbers increasing towards the end of the project. Their corresponding watershed boundaries and flow directions are also plotted. When cross drains are required on a project, a Hydrologic Summary Table is required to be on the DDM giving the appropriate information for each cross drain. (See Chapter 6 for a sample Hydrologic Summary Table.)

c.) Rural Drainage (Open Ditch Design):

For a rural design, the DDM is a composite of the existing drainage and contours from a USGS quadrangle map for the area. This is for the purpose of determining watershed areas and their corresponding slopes and hydraulic lengths. Repeating information about the existing structures is not shown unless the existing structure is part of the new design. The proposed centerline of the project and the accompanying station notations including beginning and ending station notes are included on the DDM. See Figure 8-43 in the Road Design Procedures and Details Manual.

d.) Urban Drainage (Storm Drain Systems or Curb and Gutter Design):

For urban projects the DDM is used to show basic design data for the storm drain system. The proposed geometric layout for the project is plotted along with intersecting streets in the vicinity. All proposed storm drain structures, including all pipes, catch basins, manholes, junction boxes, etc., are identified by a structure number. Structures are numbered consecutively with structure numbers increasing in the direction of flow. Drainage areas entering each individual inlet structure are plotted. The drainage area (acres), runoff coefficient (C), and the computed design discharge (cfs) for each pipe are written next to the individual structure or may be presented in a tabular form instead for clarity. See Figure 8-44 in the Road Design Procedures and Details Manual.

Other information required to initiate the inlet spacing design includes overland slope and length of strip for the various general areas and the longitudinal slope of the roadway. These items may be included on the DDM, but are not required.

2.6.3 Plan / Profile Sheets

a.) Grades:

A profile of adjacent ground elevations plotted on the plan-profile sheets is an essential design tool which gives immediate visual evidence of whether gutter (or swale ditch) grades are low enough to provide for drainage, as well as illustrating where grade cuts for over-the-curb drainage are excessive. Therefore, profiles of ground elevations at or near the right-of-way line are shown on the 60% Preliminary Plans submitted to the Hydraulics Unit for drainage review, as well as on 60% Final Plans for all projects, or portions of projects, where storm drains systems are used. The profiles are not shown on Final Contract Plans.

b.) Channel Grades:

Channel grades are shown only if a channel is required to be cut to an elevation different from the standard depth of channel shown on the typical section sheet.

c.) Side Drain Structures:

For side drain structures, the station, size and type are noted in the plan view. Side Drain pipes under intersecting state highways or other paved roads are classified as cross drains for the intersecting highway.

d.) Existing Structures:

In most cases the existing structures will be removed and are noted as such. However, in situations where any existing drainage structures will remain, they are properly noted on the plans and are accounted for in the design calculations.

e.) Proposed Cross Drain Structures:

The structures are identified in both the plan and profile views with the same number used in the Design Drainage Map. The station, structure type (CDP, RCB, SD, etc.), number of openings, size, length and angle of crossing if not at 90° are also included in the notation for each cross drain structure. Any erosion protection needed at the inlet or outlet of a cross drain structure is also noted.

- f.) Culvert Extensions:
Where the plans call for the extension of a culvert, pipe or box, the size, number of openings, angle of crossing, distance to left and right of the centerline are shown on the plans. The culvert is to be extended with the same material. Therefore, on the plans specify the material of the pipe or box.
- g.) Storm Drain Structures:
The structures are identified in both the plan and profile views with the same number used in the Design Drainage Map. The station, size, length, percent grade and flow lines are also included in the profile for each drainage structure. When there is a trunk line on each side of the roadway, a profile for each trunk line is shown.
- h.) Flow Line Elevations:
Structure lengths and flow line (invert) elevations given should permit a mathematical check of the slope for each drainage structure.
- i.) Fittings and Connections:
Fabricated pipe fittings are noted on Plan-Profile sheets and are measured per each fitting. When yard drain pipes are stubbed into the trunk line, each stub-in is noted on Plan-Profile sheets.
- j.) Sanitary/Utility Lines:
Any existing or proposed sanitary/utility lines which will affect the placement of a drainage structure are drawn in the profile with vertical clearances shown.
- k.) Servitudes:
Drainage servitude or construction servitude is occasionally required to: improve lateral channels for the proper function and maintenance of the highway, to improve the outfall for proper function of a storm drain system, or to align meandering channels crossing the highway. The Hydraulics Unit should be consulted on the type of required servitude.

2.6.4 Storm Drain Plan / Profile Sheets

The Roadway Design Procedures and Details Manual defines four cases when it is desirable to show proposed storm drain system notes on sheets other than the plan-profile sheets. These sheets will serve either as a supplement to or in lieu of the proposed drainage notes on the plan-profile sheets.

2.6.5 General Bridge Plan Sheet

The cross section of the bridge opening is plotted on the General Bridge Plan Sheet. The length, finished grade elevation, design water surface elevation and predicted scour elevation/scour line are also shown. The Hydraulic Data Table along with pertinent notes is required to be on the General Bridge Plan Sheet.

2.6.6 Summary of Drainage Structures Sheet

Summary of Drainage Structures is a part of final plan preparation and shows a detailed description of all required drainage structures listed in tabular form by stations in consecutive order. An example is shown in the Roadway Design Procedures and Details Manual, Figure 8-45.

a.) Column Headings:

Each Summary of Drainage Structure Sheet has column headings for:

- Structure Number,
- Station,
- Remarks (description or structure),
- Plan (standard plan/special detail name),
- Type (type of structure CDP, SD, SDP, etc.), and
- Quantity columns headed by type, material reference notation, type of joint, size, unit of measure

b.) Material Reference Notations:

For each structure type, a separate note giving the allowable materials is listed on the last page of the Summary of Drainage Structures Sheet.

c.) Gage Data:

When metal pipes are specified, gage and coating requirements (based on design service life, pH, resistivity and fill height) are shown on the Summary of Drainage Structures Sheet for each type of pipe.

d.) Different Classes of Reinforced Concrete Pipe:

When various classes of reinforced concrete pipe are used, they should appear under separate column headings on the Summary of Drainage Structures sheets.

e.) Jack and Bored Pipe:

When a pipe is to be bored or jacked, it is noted under a separate column heading on the Summary of Drainage Structures sheets.

f.) Column Totals:

Each quantity column is subtotaled on each sheet and the sum total for the project is shown on the last sheet of Summary of Drainage Structures. When two or more projects are included in one contract, a total for each project followed by a sum total for all the projects is shown.

2.6.7 Summary of Estimated Quantities Sheet

A single pay item number shall be used for each combination of RCP/PCP and CMP sizes. Pay item descriptions shown in the Summary of Estimated Quantities will not show gage, coating, joint or class requirements. Such information will be shown in the Summary of Drainage Structures and in the specifications.

When different classes of reinforced concrete pipe are used, they should appear under separate pay items on the Summary of Estimated Quantities sheets.

2.6.8 Standard Plans

LADOTD has a large inventory of Standard Plans for reinforced concrete box culverts, catch basins, and miscellaneous hydraulic structures. On any drawing which is used as a Standard Plan by LADOTD, all of the information which is necessary and sufficient to build the structure is shown, in either detail or in reference to another Standard Plan. Standard drainage structures require no detailing on the plans. Simply list the appropriate Standard Plan in the index on the title sheet and on the Summary of Drainage Structures sheets. When a Standard Plan for a RCB is required, the Special Detail/Standard Plan (PRCB-01) for Precast Boxes should also be included in the plans unless the design calls for cast-in-place only.

A book entitled "Standard Plans" may be purchased through LADOTD's General Files Section. It contains a listing and 1/2 size copy of the most commonly used LADOTD standard plans from all design departments (road, hydraulics, bridge, etc.).

2.6.9 Special Details

Situations often arise where a structure is required for which there is no Standard Plan. In such situations, a special detail must be prepared. Special details are required to be as thorough in information provided as are the Standard Plans. Standard plans used by LADOTD show all sections, details, information, and quantities necessary to construct typical drainage structures. The Special Details should be similar to the Standard Plans in style, and all structural design shall be in accordance with the latest AASHTO specifications. In cases where a Standard Plan is referenced for a portion of the structure, the nomenclature for dimensions and reinforcing steel should be the same on the special detail as on the Standard Plan so that there is continuity between the two drawings. Except in cases where the structure is to be paid for as a unit in place, a bill of reinforcing steel and total quantities are required with all special details.

LADOTD has some special details that can be used in many cases such as Pipe Headwalls and Pipe Toewalls.

2.6.10 Reinforced Concrete Box Extension

Many times extending the existing box culvert is sufficient for a project. Whether a special detail is required to be designed for the extension depends on the following criteria:

- a.) If the existing box culvert has the same cross-sectional dimensions as LADOTD's current Standard Plan, the box may be extended by specifying the appropriate box culvert Standard Plan and Standard Plan RCB-EXTENSION, which details the construction procedure for extending reinforced concrete boxes. No special detail is required.
- b.) If the existing box culvert is of a size no longer being built under LADOTD's current Standard Plans or the interior wall thickness varies from the current Standard Plan in the case of multiple barrels, a special detail is required.

CHAPTER 3 PEAK DISCHARGE

3.1 PURPOSE

This chapter contains the procedures and information such as rainfall charts, maps and tables that are needed to determine the peak discharge or runoff a drainage structure must handle.

3.2 RUNOFF DETERMINATION

The quantity of surface runoff reaching a culvert or bridge is determined by hydrologic analysis of the regional rainfall events and characteristics of the watershed. A variation of rate of runoff with time is defined as a runoff hydrograph. A runoff hydrograph is normally used when upstream storage volume is to be considered in the design of culverts and a reservoir routing routine is employed. Generally, however, only peak runoff rate is used as a design parameter in design of culverts and bridges. Methods of estimating peak runoff rate by LADOTD are described in the following sections.

3.2.1 Peak Rates of Discharge

Design peak rates of runoff are to be based upon the watershed conditions that are expected to exist 20 years in the future.

3.3 GAGED STATIONS

The United States Geological Survey (USGS) carries on a continuous program of stream gaging. This stream gaging data (gaging station locations, high water stages, and discharge records) should be used whenever it is applicable. For location of gaging stations and specific site data refer to the USGS publications: "Magnitude and Frequency of Floods for Small Watersheds in Louisiana", United States Geological Survey, 1979 and "Floods in Louisiana, Magnitude and Frequency", 5th Edition, United States Geological Survey, 1998 which are available through LADOTD.

USGS has a yearly publication of the gage data collected. These should be used in conjunction with the above-mentioned references. For more information, contact the USGS office.

The U.S. Army Corps of Engineers also has gages around the state. For information concerning their gages, contact their New Orleans or Vicksburg office.

3.4 UNGAGED SITES

When gage data is insufficient or unavailable, LADOTD uses three methods to estimate peak runoff rates essential to the hydraulic design of channels and structures. The methods vary according to the size of the area contributing runoff and to the structure application.

Table 3.4-1 lists the methods LADOTD uses for estimating peak rates of runoff in the design of highway drainage structures when insufficient or no observed data is available. The corresponding section listed for each method contains theoretical design information.

Table 3.4-1 Peak Discharge Determination for Ungaged Sites

DRAINAGE AREA	METHOD AND SOURCE	SECTION	LOCATION USE
2000 acres to 3000 mi ²	United States Geological Survey (USGS)	Part A	Bridges, Cross Drains
2000 acres or less	Natural Resources Conservation Service (NRCS)	Part B	Bridges, Cross Drains
200 acres or less	Rational Method	Part C	Storm Drains Median Drains

3.4.1 USGS Method

LADOTD uses an isohyetal line map for its annual rainfall data. The map was first published in 1974. LTRC reevaluated the map in the early 1990s and found no significant changes. Figure 3.4-1 is used to determine annual rainfall such as in the USGS Method. See Part A of this chapter for more information about the USGS Method.

3.4.2 NRCS Method

Table 3.4-2 gives rainfall depths for various return intervals to be used in the NRCS method. They correspond to the regions presented in Figure 3.4-2. See Part B of this chapter for more information about the NRCS Method.

3.4.3 Rational Method

Graphs of rainfall intensity vs. duration for various frequencies have been prepared for storms with return periods of 2, 10, 25, 50 and 100 years. Figures 3.4-3 through 3.4-5 were prepared from data presented in publication LADOTD 24-Hour Rainfall Frequency Maps and I-D-F Curves by Dr. Babak Naghavi, Louisiana Transportation Research Center, 1991. They are based on the three regions as shown on the map in Figure 3.4-2. See Part C of this chapter for more information about the Rational Method.

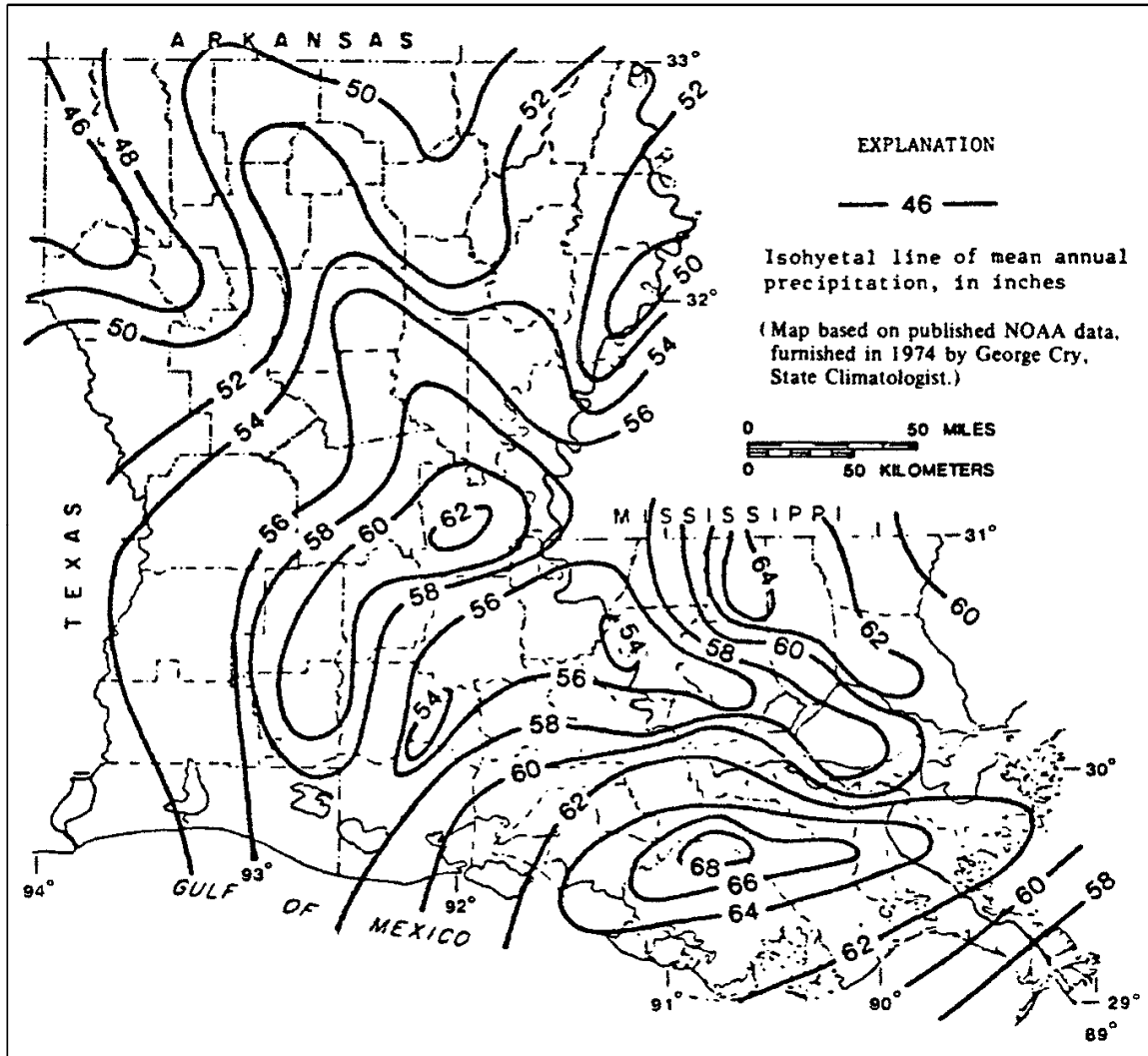


Figure 3.4-1 Mean Annual Precipitation for the USGS Method

Table 3.4-2 Louisiana Rainfall Depths (inches) for NRCS Method

RETURN PERIOD (Years)	DURATION (Hour)	REGION 1	REGION 2	REGION 3
2	6	3.5	3.0	2.8
	12	4.1	3.6	3.2
	24	4.8	4.0	3.6
5	6	4.6	4.0	3.7
	12	5.6	4.8	4.3
	24	6.5	5.4	4.9
10	6	5.5	4.8	4.4
	12	6.7	5.7	5.1
	24	7.8	6.5	5.8
25	6	6.6	5.9	5.3
	12	8.2	7.0	6.2
	24	9.6	8.0	7.0
50	6	7.6	6.9	6.0
	12	9.5	8.1	7.0
	24	11.1	9.2	8.0
100	6	8.6	7.9	6.8
	12	10.9	9.3	7.9
	24	12.6	10.5	9.0

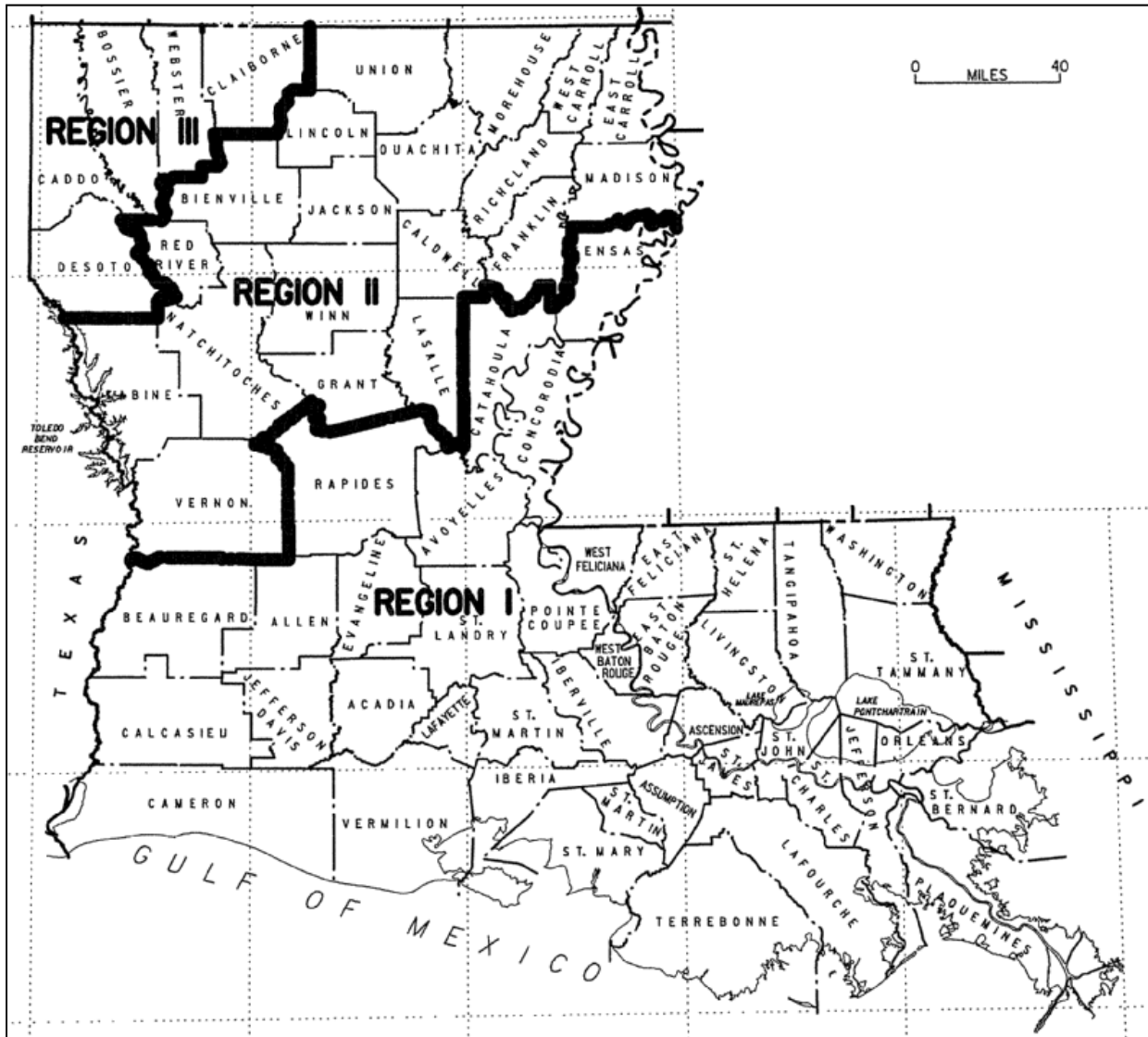


Figure 3.4-2 Louisiana Rainfall Regions

This map was developed by the Louisiana Transportation Research Center (LTRC). Use this map when determining the peak runoff by the NRCS and Rational Methods.

REGION 1

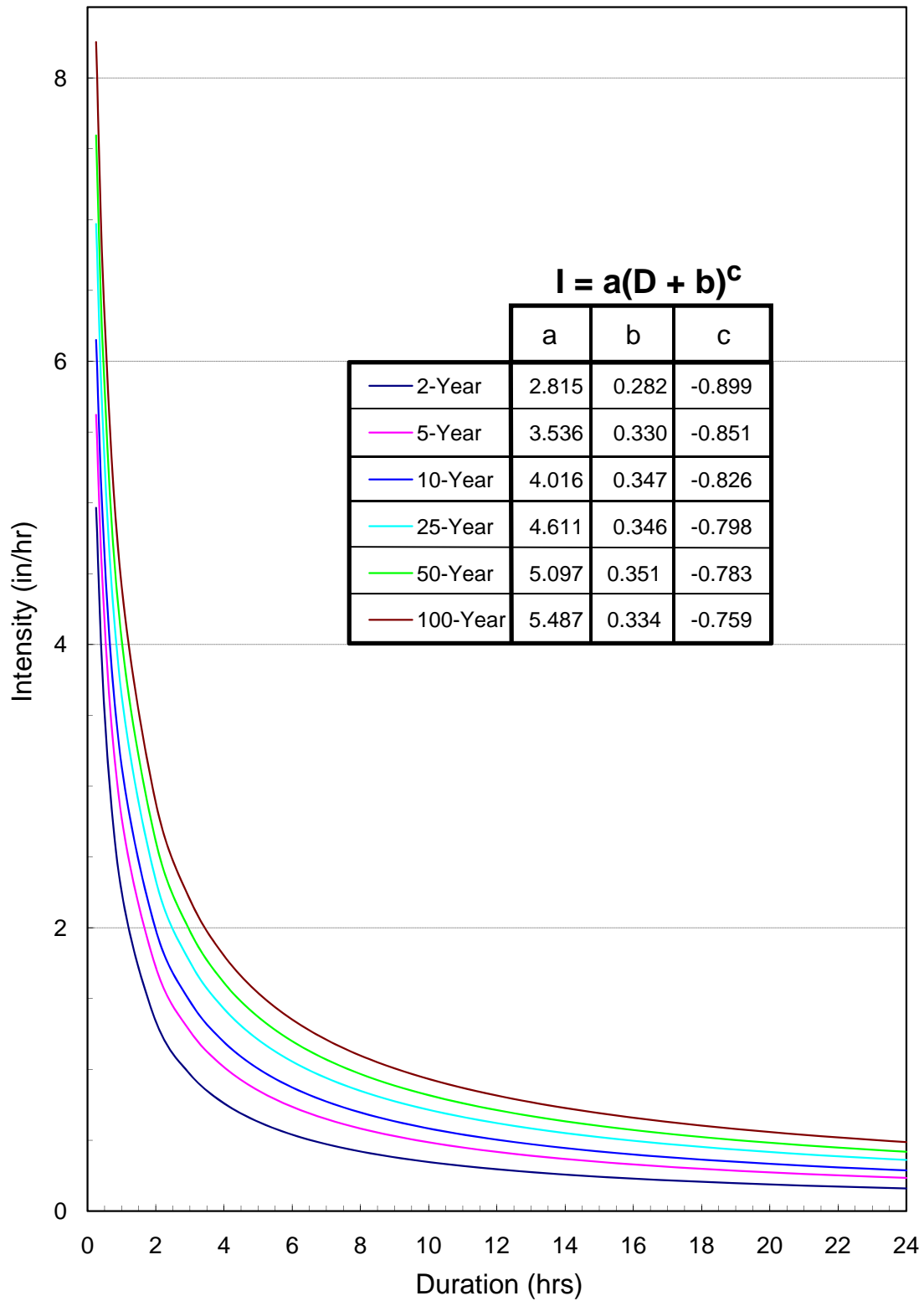


Figure 3.4-3 Region 1 Rainfall Intensity Curve (Rational Method)

REGION 2

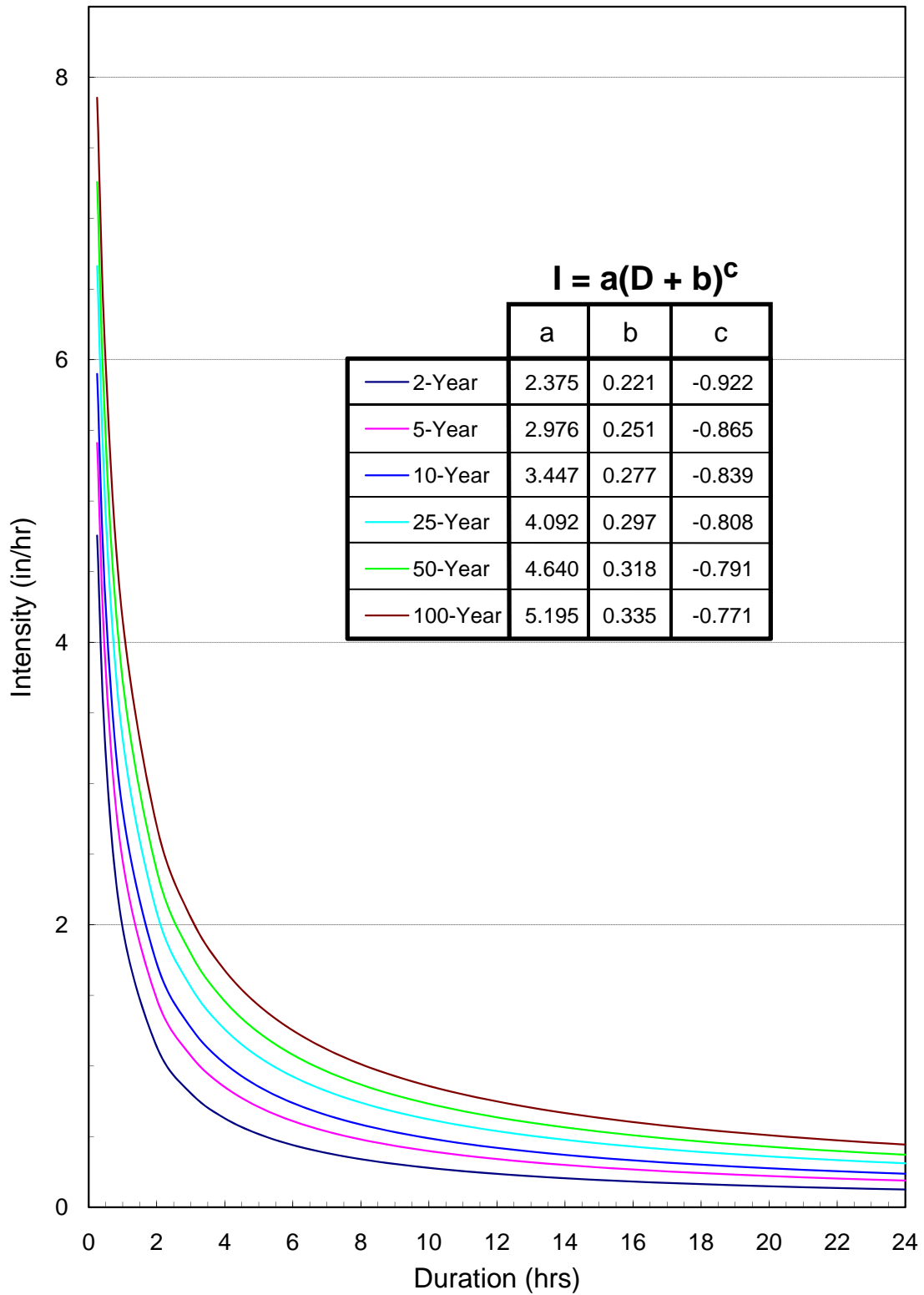


Figure 3.4-4 Region 2 Rainfall Intensity Curve (Rational Method)

REGION 3

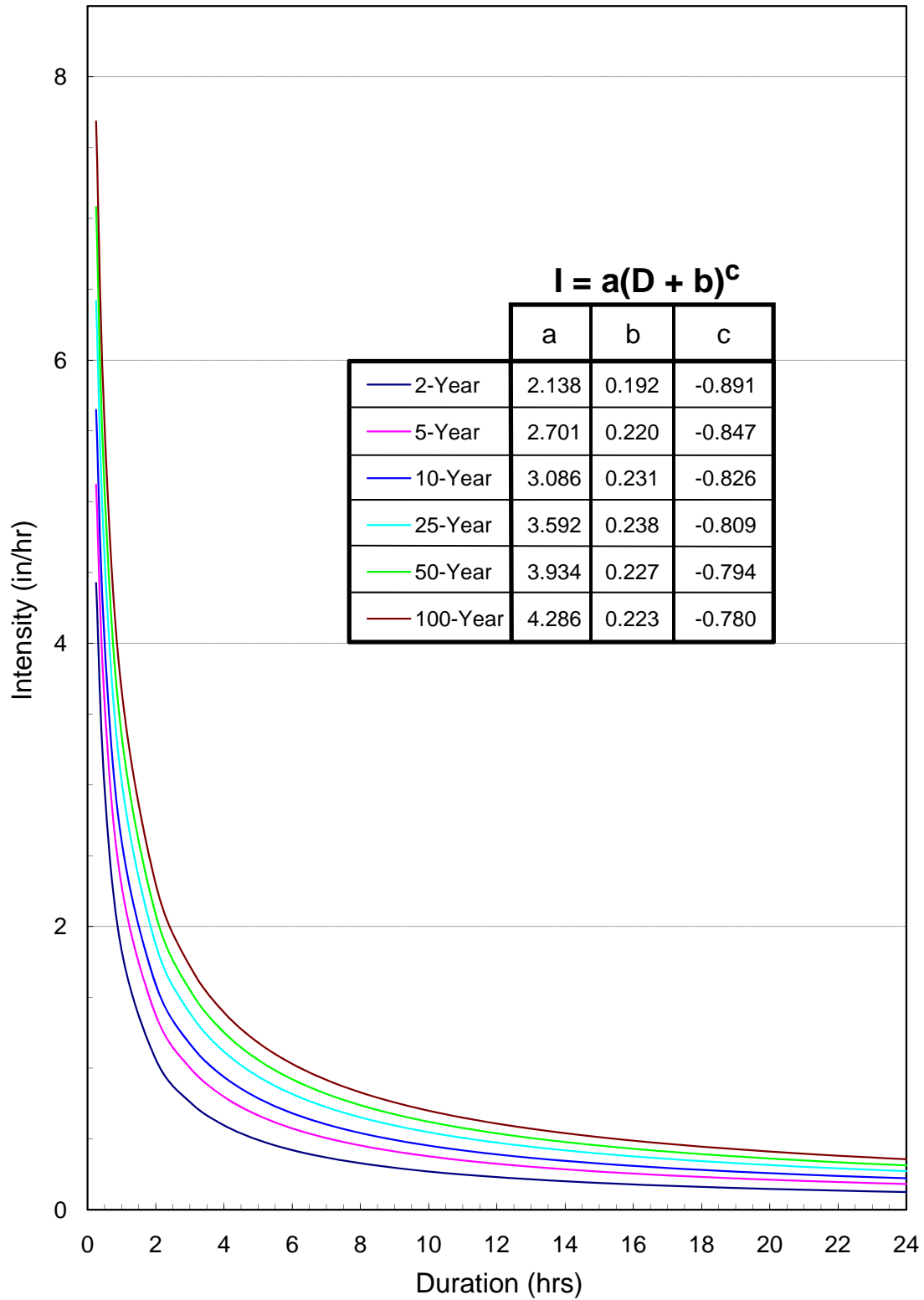


Figure 3.4-5 Region 3 Rainfall Intensity Curve (Rational Method)

CHAPTER 3 – PART A

UNITED STATES GEOLOGICAL SURVEY (USGS) METHOD

3-A.1 INTRODUCTION

For more information on the USGS Procedure, refer to these references or the latest publications/websites from the following agencies:

- A.) Floods In Louisiana, Magnitude and Frequency – 5th Edition, Technical Report No. 60, United States Department of the Interior, Geological Survey, Baton Rouge, LA, 5th Edition, 1998.
- B.) Highway Hydrology, Hydraulic Design Series No. 2, FHWA, Washington, D.C., 2nd Edition, October 2002.
- C.) Sauer, V. B., W. O. Thomas, Jr., V. A. Stricker, and K. V. Wilson, Flood Characteristics of Urban Watersheds in the United States, USGS Water-Supply Paper 2207, United States Department of the Interior, Geological Survey, 1984.

3-A.2 COMPUTER PROGRAMS

The following programs have been approved for use in the design of hydraulic structures for LADOTD highways:

- A.) The LADOTD computer program HYDR1130: “Peak Runoff, USGS” is based on the procedures outlined in this chapter.

3-A.3 USGS GAGE LOCATIONS

For locations of gaging stations and specific site data refer to Floods in Louisiana, Magnitude and Frequency and the USGS website.

3-A.4 UNGAGED LOCATIONS

The 1998 USGS computer modeling program uses what is called the Region of Influence Regression Model (RIRM). This method develops a new set of equations each time the program is run. Based on the data for the ungaged site the user inputs (drainage area, main channel slope and mean annual precipitation), the computer program selects from its data base stream flow gaging stations that have similar basin and climatic characteristics and uses them in the regression analysis.

3-A.5 INPUT REQUIREMENTS

- A.) Drainage Area (A), mi²
- B.) Mean Annual Precipitation (P), in. (See Figure 3.4-1)
- C.) Slope of the Main Channel (S), ft/mi.

The slope used in this procedure is measured between two points along the main channel. One point at 10 percent of the channel length and another point at 85 percent of the channel length, measured upstream from the point under consideration.

3-A.6 LIMITATIONS

The 1998 USGS RIRM computer modeling program is based on annual maximum discharge data for natural, unaltered streams.

3-A.6.1 Locations

The RIRM method is for use in rural areas of Louisiana, but not for urban areas. Section 3-A.7 describes the procedure to use for calculating the peak discharge in urban areas.

3-A.6.2 Altered Features

Do not use RIRM where dams, flood-detention structures, or other man-made works will have a substantial effect on annual maximum discharge. The RIRM modeling is also not suitable for channels that have been dredged or are affected by backwater. Under such conditions, stream-system studies, which involve reservoir and open-channel routing, may be required to properly evaluate flood frequency.

3-A.6.3 Basin and Climatic Characteristics

Table 3-A.6-1 gives the allowable ranges of the different basin and climatic characteristics for use in the 1998 USGS RIRM computer modeling program. If an ungaged site's basin and climatic characteristics fall outside the range of the data, the RIRM will print a warning.

Table 3-A.6-1 RIRM Ranges

BASIN & CLIMATIC CHARACTERISTICS	ALLOWABLE RANGE
Drainage Area	0.009 to 9,329 mi ²
Main Channel Slope	0.4 to 247 ft/mi
Mean Annual Precipitation	42 to 67 in.

3-A.7 URBANIZATION MODIFICATION

Basic regression equations presented in the above procedure estimate the peak discharge for ungaged rural sites. However, urbanization commonly increases the peak discharge. The following equations are used to adjust the peak discharges obtained from the RIRM regression equations:

$$UQ_2 = 13.2A^{0.21} (13 - BDF)^{-0.43} RQ_2^{0.73} \quad \text{Eq. 3-A.7-1}$$

$$UQ_5 = 10.6A^{0.17} (13 - BDF)^{-0.39} RQ_5^{0.78} \quad \text{Eq. 3-A.7-2}$$

$$UQ_{10} = 9.51A^{0.16} (13 - BDF)^{-0.36} RQ_{10}^{0.79} \quad \text{Eq. 3-A.7-3}$$

$$UQ_{25} = 8.68A^{0.15} (13 - BDF)^{-0.34} RQ_{25}^{0.80} \quad \text{Eq. 3-A.7-4}$$

$$UQ_{50} = 8.04A^{0.15} (13 - BDF)^{-0.32} RQ_{50}^{0.81} \quad \text{Eq. 3-A.7-5}$$

$$UQ_{100} = 7.70A^{0.15} (13 - BDF)^{-0.32} RQ_{100}^{0.82} \quad \text{Eq. 3-A.7-6}$$

$$UQ_{500} = 7.47A^{0.16} (13 - BDF)^{-0.30} RQ_{500}^{0.82} \quad \text{Eq. 3-A.7-7}$$

- Where:
- UQ_X = urban peak discharge for X recurrence intervals, (ft³/s)
 - A = contributing drainage area; in urban areas, drainage systems sometimes cross topographic divides. Such drainage changes should be accounted for when computing A, (mi²)
 - RQ_X = peak discharge for an equivalent rural drainage basin in the same hydrologic area as the urban basin, for a recurrence interval of X years; equivalent rural peak discharges are computed from the rural equations, (ft³/s)
 - BDF = basin development factor, an index of the prevalence of the urban drainage improvements; See Section 3-A.8 for more details.

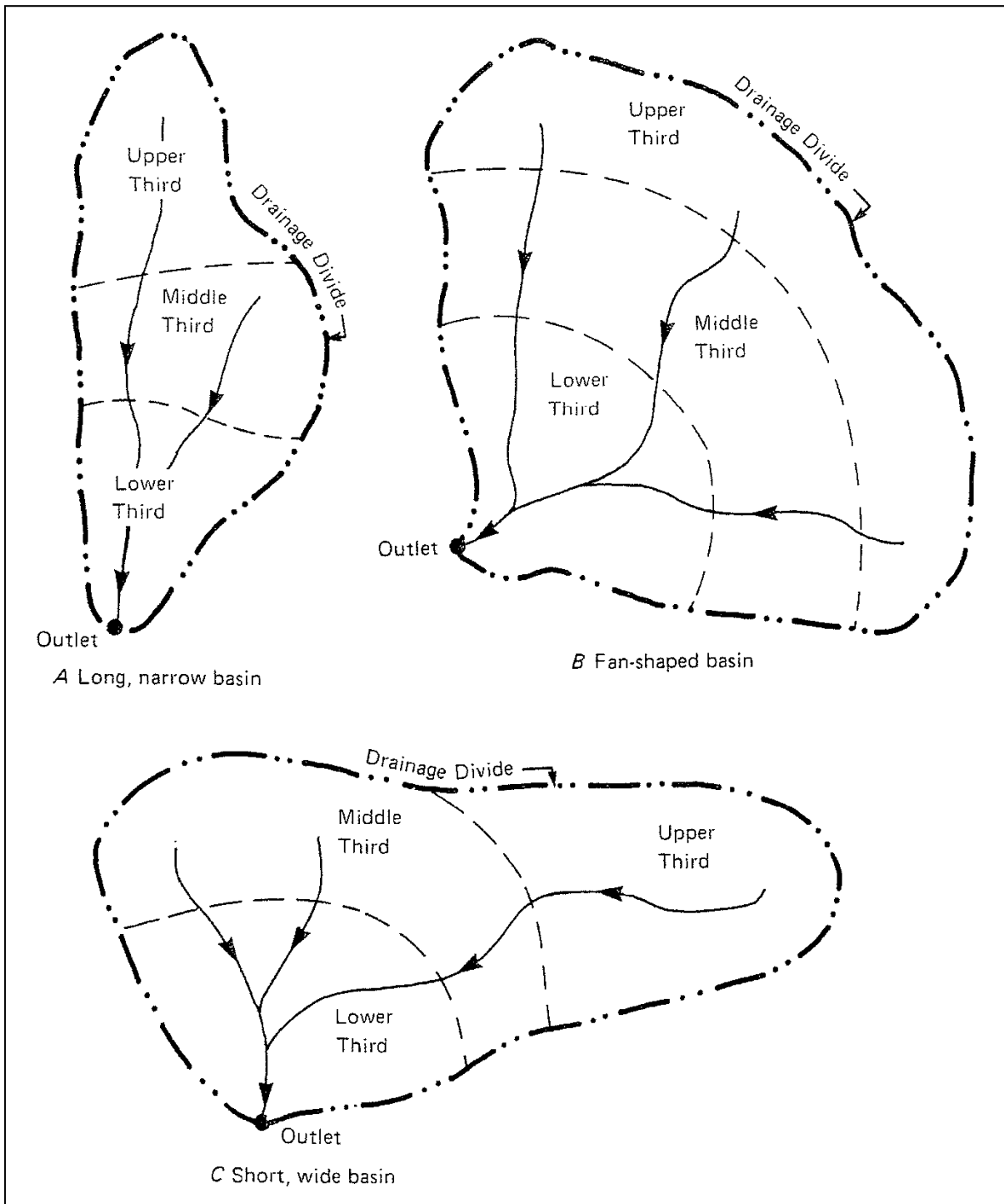


Figure 3-A.8-1 Typical Drainage Basin Shapes and Subdivision into Basin Thirds

3-A.8 BASIN DEVELOPMENT FACTOR (BDF)

The basin development factor is statistically very significant and offers a simple and effective way of accounting for drainage development and runoff response in urban areas. It can easily be determined from drainage maps and field inspections of the drainage basin.

3-A.8.1 Division of Drainage Area Basins

The basin is first divided into upper, middle, and lower thirds on a drainage map. Each third should contain about one-third of the contributing drainage area. Within each third, the travel distance of two or more streams should be equal. Schematic examples of three typical basin shapes and their division into thirds are shown in Figure 3-A.8-1 – A, B, C. All the thirds of the basin do not have to have equal travel distance with respect to each other. For instance, in Figure 3-A.8-1 – C the stream distances of the lower third are all about equal, but are longer than those in the middle third. Precise measurement of these basin dividing lines is not considered necessary. Engineering judgment should be used for subdividing more complex basin shapes and drainage patterns.

3-A.9 BDF DIVISION CODES

These guidelines for determining the various drainage-system codes are not intended to be precise measurements. A certain amount of subjectivity will necessarily be involved. Field checking should be performed to obtain the best estimate. The 50 percent guideline will usually not be difficult to evaluate because many urban areas tend to use the same design criteria, and therefore, have similar drainage aspects throughout.

Within each third of the drainage basin as described in Section 3-A.8.1, four aspects of the drainage system are evaluated and each assigned a code as follows.

3-A.9.1 Channel Modifications

When channel modifications such as straightening, enlarging, deepening, and clearing are prevalent for the main drainage channels and principal tributaries (those that drain directly into the main channel), then a code of one is assigned. Any or all of these improvements would qualify for a code of one. To be considered prevalent, at least 50 percent of the main drainage channels and principal tributaries must be improved to some degree over natural conditions. If channel improvements are not prevalent, then a code of zero is assigned.

3-A.9.2 Channel Linings

If more than 50 percent of the length of the main drainage channels and principal tributaries has been lined with an impervious material, such as concrete, then a code of one is assigned to this aspect. If less than 50 percent of these channels are lined, then a code of zero is assigned. It is assumed that the presence of channel linings is an indication of channel

improvements as well. Therefore, this is an added factor and indicates a more highly developed drainage system.

3-A.9.3 Storm Drains

Storm drains are defined as enclosed drainage structures (usually pipes), frequently used on the secondary tributaries where the drainage is received directly from streets or parking lots. Many of these drains empty into open channels; however, in some basins they empty into channels enclosed as box or pipe culverts. When more than 50 percent of the secondary tributaries within a subarea (third) consist of storm drains, then a code of one is assigned to this aspect; if less than 50 percent of the secondary tributaries consists of storm drains, then a code of zero is assigned. It should be noted that if 50 percent or more of the main drainage channels and principal tributaries are enclosed, then the aspects of channel improvements and channel linings would also be assigned a code of one.

3-A.9.4 Curb and Gutter Streets

If more than 50 percent of a subarea (third) is urbanized (covered by residential, commercial, and/or industrial development), and if more than 50 percent of the streets and highways in the subarea are constructed with curbs and gutters, then a code of one would be assigned to this aspect. Otherwise, it would receive a code of zero. Drainage from curb and gutter streets frequently empties into storm drains.

3-A.9.5 Overall BDF Number

The basin development factor (BDF) is the sum of the assigned codes. Therefore, with three subareas (thirds) per basin, and four drainage aspects to which codes are assigned in each subarea, the maximum urban effects on peaks for a fully developed drainage system would give a value of 12. Conversely, if the drainage system were totally undeveloped, then a BDF of zero would result. Such a condition does not necessarily mean that the basin is unaffected by urbanization. In fact, a basin could be partially urbanized, have some impervious area, have some improvement of secondary tributaries, and still have an assigned BDF of zero. This can still cause peak discharges to increase.

The BDF is a fairly easy index to estimate for an existing urban basin. The BDF is also convenient for projecting future development. Projections of full development or intermediate stages of development can usually be obtained from city engineers.

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CHAPTER 3 – PART B NATURAL RESOURCES CONSERVATION SERVICE (NRCS) (Formally the SCS Method)

3-B.1 INTRODUCTION

For more information on the NRCS Procedure, refer to these references or the latest publications/websites from the following agencies:

- A.) Urban Hydrology For Small Watersheds, Technical Release No. 55, USDA Natural Resources Conservation Service, June 1986.
- B.) Highway Hydrology, Hydraulic Design Series No. 2, FHWA, Washington, D.C., 2nd Edition, October 2002.

3-B.2 COMPUTER PROGRAMS

The following program has been approved for use in the design of hydraulic structures for LADOTD highways:

- A.) The LADOTD computer program HYDR1130: “Peak Runoff” is based on the procedures outlined in this section.

3-B.3 DESIGN CRITERIA

The NRCS method is used to estimate peak rates of runoff for ungaged rural and urban watersheds of 2000 acres or less. The rate and volume of runoff is influenced by the rainfall depth, duration, distribution over the duration period, and antecedent rainfall events.

3-B.4 WATERSHED FACTORS

The watershed size, shape, slope, soil type and land use also influence the peak rate of runoff. These factors are considered in the development of the equations for time of concentration and runoff.

3-B.5 TIME OF CONCENTRATION

Time of concentration is defined as the flow time from the hydraulically most remote point in the drainage area to the point under consideration.

3-B.6 HYDROLOGIC SOIL GROUPS

Soil properties influence the process of generation of runoff from rainfall and they must be considered in runoff estimation. Soil series for the watershed being studied should be determined using parish soil survey maps. Soil survey maps for all Louisiana parishes may be obtained from the United States Department of Agriculture, Natural Resources Conservation Service. The soils have been classified into four hydrologic soil groups as shown in Table 3-B.6-1. Table 3-B.6-2 lists the soils found in Louisiana and their corresponding hydrologic soil group.

Table 3-B.6-1 Soil Group Definitions

SOIL GROUP	DESCRIPTION
A	Soils having low runoff potential and high infiltration rates even when thoroughly wetted (low runoff potential). These consist chiefly of deep, well to excessively drained sands or gravels. These soils have a high rate of water transmission in that water readily passes through them (> 0.30 in/hr).
B	Soils having moderate infiltration rates when thoroughly wetted. These consist chiefly 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 water transmission (0.15 to 0.30 in/hr).
C	Soils having low infiltration rates when thoroughly wetted. These consist chiefly of soils with a layer that impedes downward movement of water or soils with moderately fine to fine texture. These soils have a slow rate of water transmission (0.05 to 0.15 in/hr).
D	Soils having high runoff potential. They have very low infiltration rates when thoroughly wetted (high runoff potential). These consist chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very low rate of water transmission (0 to 0.05 in/hr).

3-B.7 CURVE NUMBER

The hydrologic soil group of the watershed soil is used in conjunction with land use to determine a runoff curve number (CN) for the watershed. Runoff curve number (CN) values for various hydrologic classifications of soils and land use are also presented in Table 3-B.7-1. The selected runoff curve number (CN) should represent conditions which may be expected to exist 20 years in the future. When the watershed consists of several classes of soils, a weighted runoff curve number should be used. Values of runoff curve number given in Table 3.B.7-1 are for average antecedent moisture condition (AMC-II). For estimations with lower limit (AMC-I) and upper limit (AMC-III) of moisture conditions, refer to the NRCS website.

Table 3-B.6-2 Hydrologic Classification of Soils in Louisiana

Series Name	Hydrologic* Group	Series Name	Hydrologic* Group	Series Name	Hydrologic* Group
Acadia	D	Carroll	D	Gallion	B
Acy	C	Cascilla	B	Galvez	C
Alaga	A	Caspiana	B	Gentilly	D
Allemands	D	Chastian	D	Gilead	C
Alligator	D	Collins	C	Glenmora	C
Amagon	D	Commerce	C	Goldman	C
Amite	B	Convent	C	Gore	D
Anacoco	D	Coteau	C	Grenada	C
Andry	D	Coushatta	B	Guyan	D
Angie	C	Crevasse	A	Hannahatchee	C
Armistead	C	Crowley	D	Harris	D
Baldwin	D	Cuthbert	C	Hebert	C
Barbary	D	Cypremort	C	Henry	D
Basile	D	Darco	A	Hollywood	D
Beaumont	D	Deerfield	D	Hortman	D
Beauregard	C	Delcomb	D	Houston	D
Bennedale	B	Dexter	B	Huckabee	A
Bernaldo	B	Dossman	B	Iberia	D
Bibb	B/D	Dundee	C	Ijam	D
Bienville	A	Duralde	C	Independence	A
Blaney	B	Elysian	B	Iuka	C
Bonn	D	Essen	C	Izagora	C
Boswell	D	Eustis	A	Jeanerette	D
Bowie	B	Eutaw	D	Jena	B
Bruin	B	Evangeline	C	Johnston	B/D
Bruno	A	Falaya	D	Kalmia	B
Bude	C	Falkner	C	Kaufman	D
Buxin	D	Fausse	D	Kenner	D
Caddo	D	Foley	D	Kenny	A
Cadeville	D	Forestdale	D	Keo	B
Cahaba	B	Fountain	D	Kirvin	C
Calhoun	D	Fred	C	Kisatchie	D
Calloway	C	Freeland	C	Kolin	C
Cane	C	Frizzell	C	Kullit	B
Carlin	D	Frost	D	Lafe	D

Table 3-B.6-2 continued Hydrologic Classification of Soils in Louisiana

Series Name	Hydrologic* Group	Series Name	Hydrologic* Group	Series Name	Hydrologic* Group
Lafitte	D	Natchitoches	D	Sawyer	C
Lakeland	A	Newellton	D	Scatlake	D
Latanier	D	Norfolk	B	Severn	B
Leaf	D	Norwood	B	Sharkey	D
Lexington	B	Nugent	A	Shatta	C
Libuse	C	Ochlockonee	B	Shubuta	C
Lintonia	B	Okenee	D	Smithdale	B
Loreauville	C	Oktibbeha	D	Springfield	D
Loring	C	Oliver	C	Sterlington	B
Lucy	A	Ora	C	Stough	C
Lufkin	D	Orangeburg	B	Summerfield	C
Luverne	C	Osier	B/D	Sumter	C
Malbis	B	Palm Beach	A	Susquehanna	D
Mamou	C	Patoutville	C	Tenot	C
Mantachie	C	Pelham	B/D	Tensas	D
Mashulaville	B/D	Perry	D	Tilden	B
Maurepas	D	Pheba	C	Trebloc	D
Mayhew	D	Placedo	D	Tunica	D
Mckamie	D	Pledger	D	Una	D
Mclaurin	B	Portland	D	Urbo	D
Memphis	B	Prentiss	C	Vacherie	C
Mer rouge	B	Providence	C	Vaiden	D
Messer	C	Ragley	D	Vacluse	C
Meth	C	Red bay	B	Verdun	D
Mhoon	D	Rexor	A	Verrett	D
Midland	D	Richland	C	Vicksburg	B
Miller	D	Rilla	B	Vidrine	C
Moreland	D	Robinsonville	B	Waller	B/D
Morey	D	Roebuck	D	Waverly	B/D
Morse	D	Rosebloom	D	Woodtell	D
Mowata	D	Roxana	B	Wrightsville	D
Muskogee	C	Ruston	B	Yahola	B
Myatt	B/D	Sacul	C	Zachary	D
Nacogdoches	B	Savannah	C		

* Dual hydrologic soil groups are given for certain wet soils that can be adequately drained. The first letter applies to the drained condition and the second to the undrained from a surface drainage standpoint.

Table 3-B.7-1 Runoff Curve Number (CN) for Selected Agricultural, Suburban, and Urban Land Uses (Antecedent Moisture Condition II)¹

LAND USE	HYDROLOGIC SOIL GROUP			
	A	B	C	D
Woods or forest land	37	61	74	80
Pasture or range land	52	70	80	85
Cultivated land	67	76	83	86
Open spaces, lawns, parks, golf courses, cemeteries, etc.: ²				
Good condition: grass cover on 75% or more of the area	39	61	74	80
Fair condition: grass cover on 50% to 75% of the area	49	69	79	84
Poor condition: grass cover less than 50% of the area	68	79	86	89
Residential:				
Average lot size	Average % impervious ³			
1/8 acre or less	65	77	85	90
1/4 acre	38	61	75	83
1/3 acre	30	57	72	81
1/2 acre	25	54	70	80
1 acre	20	51	68	79
2 acre	12	46	65	77
Commercial and business area (85% impervious)	89	92	94	95
Industrial districts (72% impervious)	81	88	91	93
Paved parking lots, roofs, driveways, etc. (excluding right of way)	98	98	98	98
Paved streets and roads:				
Streets with curbs and storm drains (excluding right of way)	98	98	98	98
Roads with open ditches. (including right of way)	83	89	92	93
Gravel (including right of way)	76	85	89	91
Dirt (including right-of-way)	72	82	87	89

1 – Average runoff condition, and $I_a = 0.2S$

2 – CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space type.

3 – The average % impervious area shown was used to develop the composite CNs. Other assumptions are as follows: Impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition.

3-B.8 DESIGN INPUT

The following input criteria are needed to estimate peak rates of runoff for ungaged watersheds. It is assumed that the topography is such that you have approximately uniform surface flow into channels, drains, and streams. The peak rates of runoff are approximately the same as those obtained by preparing flood hydrographs and valley flood routing to the point of design.

3-B.8.1 Drainage Basin

The following information should be determined for the drainage basin:

- a.) Drainage Area (A), acres
- b.) Hydraulic Length of Watershed, ft.
The hydraulic length of the watershed is the distance the runoff must traverse from the most hydraulically distant portion of the watershed to the point under consideration.
- c.) Average watershed land slope (%)
The weighted average slope should be calculated using elevations along the hydraulic length of watershed.
- d.) Soil type(s) and hydrologic soil group(s) forming the drainage area.
The soil type can be determined from Soil Conservation Service parish soil survey maps. Hydrologic grouping for the various soil types are included in Table 3-B.6-2.

3-B.8.2 Runoff Curve Number

Determine the runoff curve number (CN) to be assigned to each drainage area by using the information in Section 3-B.8.1 and the curve number guide (see Table 3-B.7-1). A weighted curve number (CN) may be required for watersheds having different soil types and/or land use. It should be based on predicted land use for 20 years in the future.

3-B.8.3 Rainfall

The rainfall region is determined from Figure 3.4-2. Then use the 24-hour rainfall duration from Table 3.4-2 to determine the amount of rainfall (inches) associated with the design frequency storm.

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CHAPTER 3 – PART C RATIONAL METHOD

3-C.1 INTRODUCTION

The Rational Method computes only peak runoff values, neglecting flow variations with time and routing of the flow through the watershed and the collection system.

For a more detailed discussion of the theory and use of the Rational Method, refer to these references or the latest publications/websites from the following agencies:

- A.) ASCE Manual of Engineering Practice No. 37, Chapter IV, Design and Construction of Sanitary and Storm Sewers, 1976.
- B.) Highway Hydrology, Hydraulic Design Series No. 2, FHWA, Washington, D.C., 2nd Edition, October 2002.

3-C.2 DESIGN ASSUMPTIONS

Assumptions basic to the Rational Method are:

- A.) Limited to drainage areas less than 200 acres
- B.) The maximum runoff rate to any location is a function of the average rate of rainfall during the time of concentration.
- C.) The maximum rate of rainfall occurs during the time of concentration.
- D.) The variability of the storm pattern is not taken into consideration.

3-C.3 HYDROLOGIC ANALYSIS

The Rational Method Equation is:

$$Q=CiA \qquad \text{Eq. 3-C.3-1}$$

Where: Q = Peak runoff rate (ft³/s)
C = Runoff coefficient (Section 3-C.3.1)
i = Average rainfall intensity at the time of concentration (in./hr)
(Section 3-C.3.2 and Section 3.4.3)
A = Drainage area (acres)

3-C.3.1 Runoff Coefficient, C

The runoff coefficient, C, in the Rational Equation represents the fraction of rainfall on a given area which may be expected to become runoff.

Table 3-C.3-1 from Sewer Design and Construction - ASCE Manual No. 37, provides general guidelines for the selection of runoff coefficient factors.

Table 3-C.3-1 Runoff Coefficients

DESCRIPTION OF AREA	RUNOFF COEFFICIENTS
Business: Downtown areas Neighborhood areas	0.70 to 0.95 0.50 to 0.70
Residential: Single-family areas Multi-units, detached Multi-units, attached	0.30 to 0.50 0.40 to 0.60 0.60 to 0.75
Residential (suburban)	0.25 to 0.40
Apartment dwelling areas	0.50 to 0.70
Industrial: Light areas Heavy areas	0.50 to 0.80 0.60 to 0.90
Parks, cemeteries	0.10 to 0.25
Playgrounds	0.20 to 0.35
Railroad yard areas	0.20 to 0.40
Unimproved areas	0.10 to 0.30

3-C.3.2 Time of Concentration

Time of concentration (TC) is defined as the flow time from the most remote point in the drainage area to the point under consideration. Usually it is considered to be composed of time of concentration for drainage inlets plus time of flow in pipes. Figure 3-C.3-1 (adapted from Elwyn E. Seelye, "Databook for Civil Engineers", Volume 1 – Design, Second Edition, New York: John Wiley and sons inc., 1951) is provided to assist in estimating the overland flow time, which will be considered the time of concentration for drainage inlets.

Time of concentration for drainage inlets shall not be less than 5 minutes.

Note: The charts in Figures 3.4-3 to 3.4-5 use the variable “Duration” in hours to determine the rainfall intensity. Durations is the time of concentration as determined in Section 3-C.3-2 converted into hours.

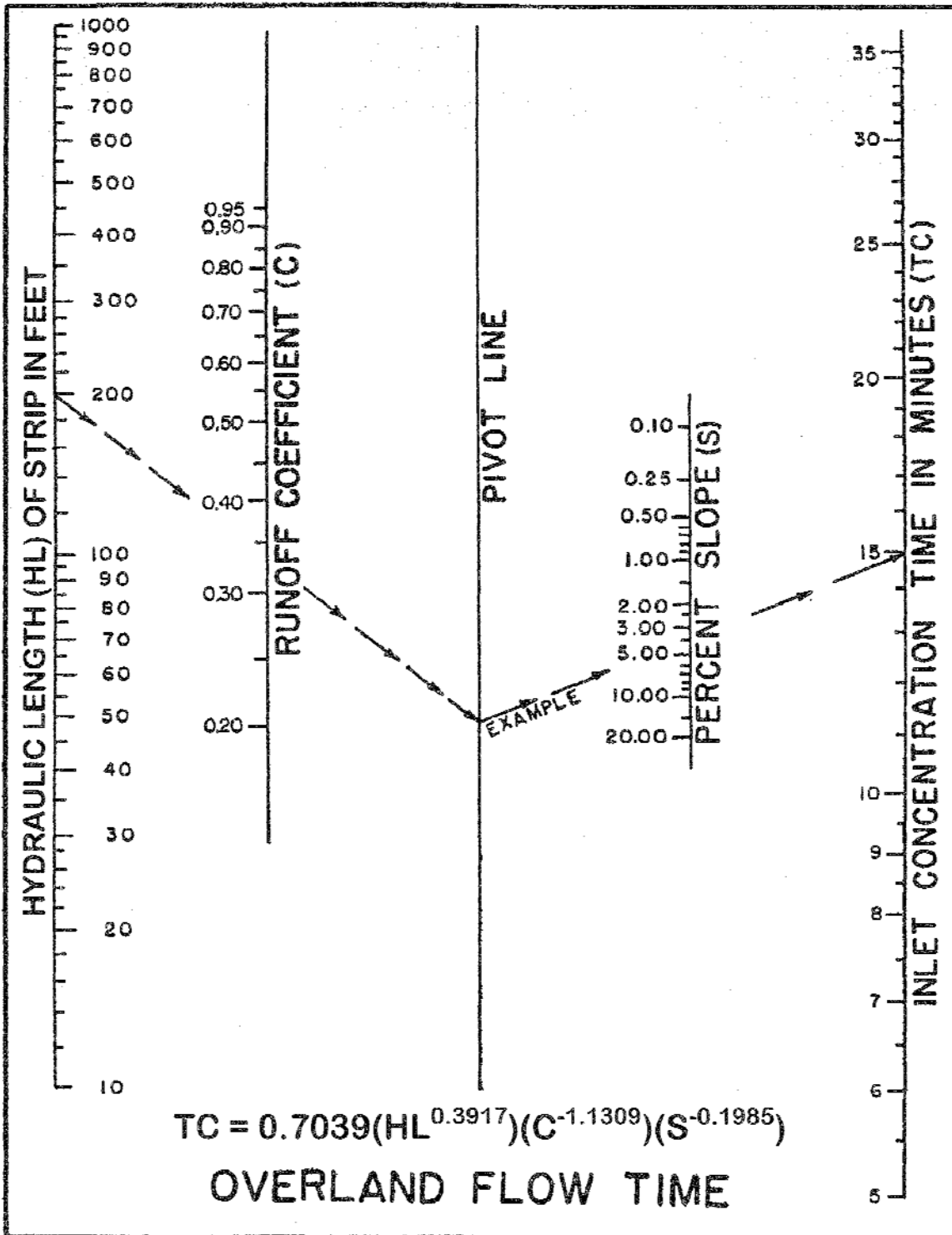


Figure 3-C.3-1 Time of Concentration

CHAPTER 4 OPEN CHANNELS

4.1 PURPOSE

This chapter presents standards for the design of artificial or manmade roadside channels, including roadside ditches, median ditches and interceptor ditches, outfalls and canals. Open channels shall be sized according to Manning's equation to accommodate the design flow from the area draining into the channel. For detailed information on open channel hydraulics, refer to Part A at the end of this chapter. Erosion control measures for open channels will be based on the policies presented in this chapter.

4.2 DESIGN STORM FREQUENCY

Roadside channels will be designed according to the criteria presented in Chapter 1 unless the channel is providing an outfall or is the major approach channel for a cross drain, in which case the design storm for the roadside channel will be the same as for a cross drain.

4.3 PEAK DISCHARGE

The methods for determining peak discharge are described in Chapter 1.

4.3.1 Lateral Channels

Lateral channels are designed to convey the design discharge of the culvert or culverts which they serve.

4.4 SHAPE AND SIDE SLOPES

4.4.1 Roadside Channels

Trapezoidal sections are normally used with side slopes not to be steeper than 3:1. Where erosive soils are present, consideration should be given to using flatter slopes.

4.4.2 Lateral Channels

Trapezoidal sections are normally used with side slopes to be no steeper than 2:1.

When lateral excavations are necessary, a channel cross section and profile, as well as a plan layout should be included in the plans.

4.5 ROADSIDE CHANNEL GRADE

A minimum channel grade of 0.1% shall be maintained.

4.5.1 Driveways

During preliminary design, careful attention should be paid to the ditches. In many cases, a ditch grade can be established so that a driveway is at the high point in the ditch grade, thus eliminating the need for a side drain structure. Such driveways are referred to as dry ramps.

4.6 LATERAL CHANNELS

Occasionally, the need arises to improve an existing channel, to change a channel alignment, or to design a new channel altogether. The primary reason for such channel improvements is generally economics.

Improvement of laterals is not routinely done and must be justified. Improvement or construction of a lateral channel is normally justified by a need to lower the design headwater at a culvert site in order to prevent or relieve flooding of the roadway or upstream property. It should be noted that the owner of the channel is the responsible party.

4.6.1 Drainage Servitude

Parameters for determining required widths of drainage servitude are channel top width, bank width needed for construction and maintenance, and amount of spoil.

4.6.2 Spoil Banks

Where spoil banks will be used, servitude width should be sufficient to allow a minimum distance of 10 feet from the toe of the spoil bank to the bank of the channel. In all cases, a minimum bank width (on one side) of 10 feet will be required to allow equipment usage. A minimum bank width of 5 feet on the opposite side will be sufficient. Spoil banks should be broken to allow for local drainage.

4.7 EQUALIZERS

On long continuous grades which are unbroken by lateral outfalls, "cross drain equalizers" shall be used at intervals of approximately 1000 feet – 1500 feet. The purpose of equalizers is to distribute the flow between the roadside channels. Equalizers shall be 24-inch diameter pipes or round equivalent pipe arches.

4.8 CHANNEL BLOCKS

Where a roadside channel discharges sharply into a lateral ditch or channel or into a larger crossing waterway and the difference in channel bottom elevations exceeds two feet, then a channel block is the preferred method of erosion prevention, as paved channels have a high failure rate. Channel liner material upstream of a channel block will be determined by the procedures given in this chapter. Figure 4.8-1 provides installation details for a channel block, i.e., Side Drain (Erosion). The availability of long joints, pipe flexibility, and joint strength are factors which dictate the use of corrugated metal pipe or plastic pipe for this special installation. Refer to EDSM II.2.1.1 Side Drain (Erosion) for allowable materials.

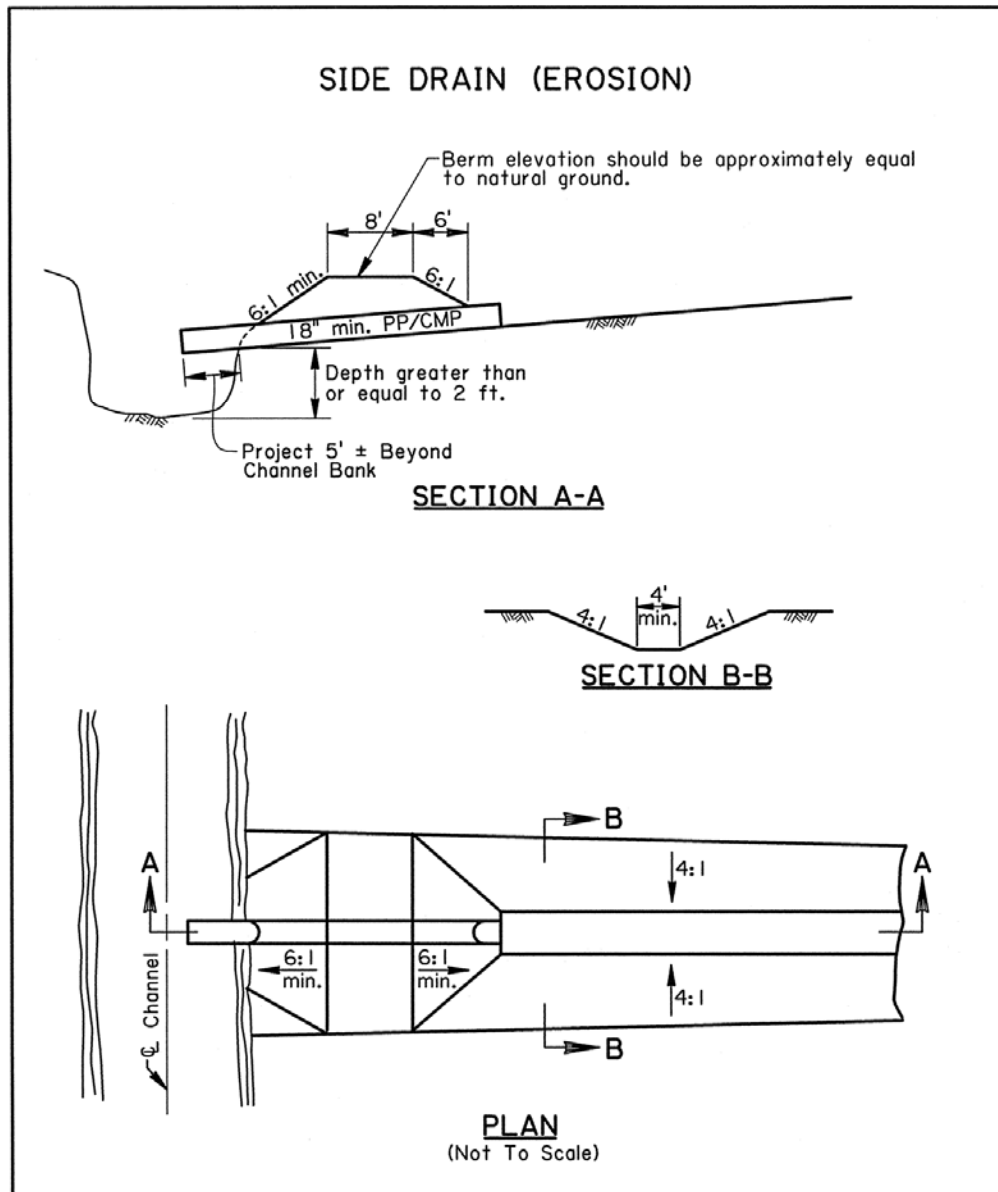


Figure 4.8-1 Channel Block - Side Drain (Erosion) Example

4.9 CHANNEL LININGS

After channel grades have been established, the hydraulic adequacy of the proposed channel section and the need for and station limits for any required erosion control measure should be determined.

Erosion in channels can become severe even at fairly mild highway grades. Therefore, channels often require stabilization against erosion. Factors that should be considered when selecting a channel lining are:

- velocity
- lining limitations
- construction and maintenance considerations
- safety

Channel linings can be rigid or flexible.

4.9.1 Rigid Linings

Advantages:

- Rigid channel linings can sustain a much higher magnitude of erosive force than flexible linings. They also can provide a much higher capacity and in some cases may be the only alternative.

Disadvantages:

- Rigid linings tend to fail when a portion of the lining is damaged

4.9.2 Flexible Linings

Advantages:

- The primary advantage of flexible linings from an erosion control standpoint is that flexible linings can sustain some change in channel shape while maintaining the overall integrity of the channel lining.
- Flexible linings permit infiltration and exfiltration, are less expensive than rigid linings, and have a natural appearance.

Disadvantages:

- Flexible linings have the disadvantage of being limited in the magnitude of erosive force they can sustain without damage to either the channel or the lining. In some cases, flexible linings may provide only temporary protection against erosion while allowing vegetation to be established. The vegetation will then provide permanent

erosion control in the channel. However, vegetative channel linings are not suited to sustained flow conditions or long periods of submergence.

4.10 EROSION CONTROL PRODUCTS

LADOTD has a classification system for flexible erosion control products to use for slope protection and channel linings. They are listed in the latest specification book under Erosion Control Systems and currently include Types A through F. The types are based on the shear stress the material can withstand, increasing in order of needed protection. The LADOTD Material Lab's QPL (Qualified Products List) 72 shows the specific products a contractor may use for the protection type called for on the plans.

As new products are developed and approved, QPL 72 is updated on LADOTD's website under the construction/materials lab/qualified products list section.

4.10.1 Roadside Ditch Lining Selection

The Hydraulics Unit has adapted LADOTD's Erosion Control Systems for use on roadside ditches based on the ditch slope. It is described in Table 4.10-1.

Table 4.10-1 is only for roadside ditches for the following criteria:

- the contributing drainage area is less than or equal to 10 acres,
- bottom slope does not exceed 10%,
- a depth of flow is not expected to exceed 1'.

4.10.2 Lateral Channels, Outfall Channels and Larger Roadside Ditches

For any channel that does not fit the criteria presented in Section 4.10.1, the methods as outlined in FHWA Publication, HEC 15 should be followed to determine the appropriate erosion protection.

**TABLE 4.10-1 – Erosion Control Protection for Roadside Ditches
(Refer to LADOTD’s QPL for specific applicable products)**

Range of Ditch Slope (%)	Type of Protection	Shear Stress Range	Maximum Sediment Loss	Minimum Vegetation Density (%)
0.10 to 0.50	vegetative mulch	–	–	–
0.51 to 2.50	A	–	0.06 lb/yd ² (0.034 kg/m ²)	80
2.51 to 3.00	C	0 – 2 psf (0 – 96 Pa)	0.45" (11.5 mm)	70
3.01 to 6.00	D	0 – 4 psf (0 – 192 Pa)	0.40" (10.0 mm)	70
6.01 to 8.00	E	0 – 6 psf (0 – 287 Pa)	0.40" (10.0 mm)	70
8.01 to 10.00	F	0 – 8 psf (0 – 383 Pa)	0.30" (8.0 mm)	70

Paved ditch alternates will not be allowed as a substitute for Erosion Control System Flexible Channel Liners.

CHAPTER 4 – PART A OPEN CHANNEL HYDRAULICS

4-A.1 INTRODUCTION

Open channels (roadside and lateral ditches and outfalls) shall be sized or analyzed according to the Manning's equation to accommodate the design flow from the area draining into the channel.

For more information on the design of open channels, refer to these references or the latest publications from the following agencies:

- A.) Urban Hydrology For Small Watersheds, Technical Release No. 55, USDA Natural Resources Conservation Service, June 1986.
- B.) SCS National Engineering Handbook, Section 4 (Hydrology), U.S. Soil Conservation Service, 1972.
- C.) Design of Roadside Channels with Flexible Linings, Hydraulic Engineering Circular 15, FHWA, Washington, D.C., April 1988
- D.) Design of Riprap Revetment, Hydraulic Engineering Circular 11, FHWA, Washington, D.C., 1989.

4-A.2 COMPUTER PROGRAMS

The following programs have been approved for use in the design of small open channels for LADOTD highways:

- A.) The LADOTD computer program HYDR1110: "Normal Water Surface" is a computer program which computes the normal water surface elevation, area of opening, and average velocity for a given irregular shaped channel cross-section as outlined in Section 4-A.5.
- B.) The LADOTD computer program HYDR1140: "Open Channel Flow" is a computer program which computes the normal depth of water, bottom width or design discharge for a given uniform shaped cross-section as outlined in this Section 4-A.4.

4-A.3 MANNING'S COEFFICIENT

Table 4-A.3-1 lists Manning's roughness coefficients for flood plains and for channels with vegetative linings and artificial linings. The probable condition of the channel when the design event is anticipated shall be considered when a Manning's "n" value is selected.

Table 4-A.3-1 Manning’s Roughness Coefficient for Channel Linings

Type of Open Channel	n
1. Open Channels, Lined Concrete Asphalt Fiberglass Roving	0.015 0.015 0.021
2. Open Channel, Excavated Earth, fairly uniform section: no vegetation, bare soil grass, some weeds dense weeds Channels not maintained: Dense weeds, high as flow depth Dense brush	0.020 0.025 0.030 0.100 0.120
3. Highway Channels and Swales With Maintained Vegetation Mowed to 2" (50 mm) Length 4" – 6" (100 mm – 150 mm) Length 6" – 12" (150 mm – 300 mm)	0.040 0.050 0.060
4. Natural Stream Channels and Flood Plains Major streams: Surface width at flood stage > 100' (30.5 m) Minor streams: some grass and weeds, little or no brush dense growth of weeds some weeds, heavy brush on banks	0.030 0.035 0.050 0.060
5. Flood Plains Adjacent to Natural Streams Pasture, no brush: Short grass high grass Cultivated areas: no crops crops Heavy weeds, scattered brush Light brush and trees Medium to dense brush	0.035 0.050 0.035 0.050 0.060 0.080 0.160

4-A.4 UNIFORM GEOMETRIC SHAPED CHANNEL

The depth of flow for an open channel with a uniform geometric shape, including roadside ditches and lateral channels, is determined by balancing Equations 4-A.4-1 and 4-A.4-2.

$$V = \frac{1.486 R^{2/3} S^{1/2}}{n} \quad \text{Eq. 4-A.4-1}$$

Where:

- V = Average velocity (ft/s)
- n = Manning's roughness coefficient (Table 4-A.3-1)
- R = Hydraulic radius (area of flow divided by wetted perimeter) (ft)
- S = Channel slope (ft/ft)

$$Q = VA \quad \text{Eq. 4-A.4-2}$$

Where:

- Q = Design capacity of the channel (ft³/s)
- V = Average velocity (ft/s)
- A = Area of flow (ft²)

4-A.4.1 Trapezoidal Sections

For commonly used trapezoidal sections, such as the one shown in Figure 4-A.4-1, the area of flow (A) and hydraulic radius (R) are:

$$A = \frac{D[2B + D(Z_1 + Z_2)]}{2} \quad \text{Eq. 4-A.4-3}$$

$$R = \frac{D[2B + D(Z_1 + Z_2)]}{2B + 2D\left[\left(1 + Z_1^2\right)^{0.5} + \left(1 + Z_2^2\right)^{0.5}\right]} \quad \text{Eq. 4-A.4-4}$$

Where:

- A = Area of flow (ft²)
- D = Depth of flow (ft)
- B = Bottom width of channel (ft)
- Z_{1,2} = Side slope for each side
- R = Hydraulic radius (area of flow divided by wetted perimeter) (ft)

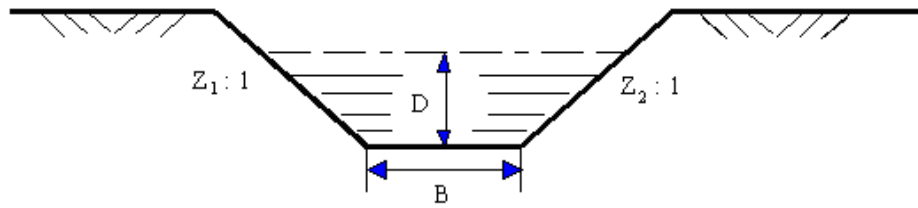


Figure 4-A.4-1 Typical Trapezoidal Cross Section of an Open Channel

4-A.4.2 Symmetrical Trapezoidal Sections

When $Z = Z_1 = Z_2$, the equations for the area and hydraulic radius simplify to:

$$A = D(ZD + B) \quad \text{Eq. 4-A.4-5}$$

$$R = \frac{D(ZD + B)}{B + 2D(1 + Z^2)^{0.5}} \quad \text{Eq. 4-A.4-6}$$

Where:

- A = Area of flow (ft²)
- D = Depth of flow (ft)
- B = Bottom width of channel (ft)
- Z = Side slope of channel
- R = Hydraulic radius (ft)

4-A.5 IRREGULAR SHAPED CHANNEL

In the majority of cross drain or storm drain design problems, the channel in which the cross drain lies or to which the storm drain pipe outfalls is an irregular shape. A depth of flow or tailwater for a particular storm can be calculated with practical accuracy by Manning's formula:

$$Q = A \left(\frac{1.486}{n} \right) R^{2/3} S^{1/2} \quad \text{Eq. 4-A.5-1}$$

Where:

- Q = Design discharge (ft³/s)
- A = Waterway area of flow across section (ft²)
- R = Hydraulic radius (ft)
- S = Water surface slope (assumed to be parallel to stream bed slope) (ft/ft)
- n = Manning's coefficient of roughness for the channel section (Table 4-A.3-1)

From a single cross section of the outfall channel, Equation 4-A.5-1 can be used to compute discharges for selected water surface elevations. A stage-discharge curve is then plotted.

Note that discharges are computed with the assumption that the total discharge of flow is equal to the sum of the discharges of the subdivided areas. Areas with the same coefficient of roughness should be grouped together.

From the stage-discharge curve, the depth of flow or tailwater elevation in the outfall channel can be found for the design discharge. An example of this type of problem can be seen below.

4-A.5.1 Example Problem

Determine the stage elevation for the channel cross-section shown in Figure 4-A.5-1(a) with a slope of 0.005 ft/ft and a discharge of 2200 cfs.

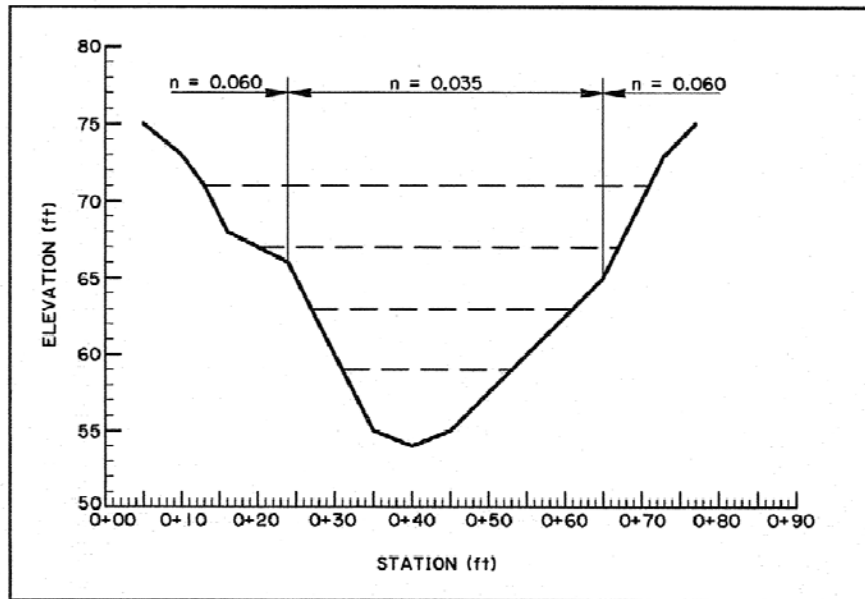


Figure 4-A.5-1(a) Channel Cross Section for Example Problem 4-A.5.1

a.) Choose different elevations and calculate the corresponding Q:

- at elevation 59.00 ft:

Area (A) = 69 ft²

Wetted perimeter (p) = 24.8'

Manning's coefficient (n) = 0.035

$$Q = (69) \left(\frac{1.486}{0.035} \right) \left(\frac{69}{24.8} \right)^{2/3} (0.005)^{1/2} = 410 \text{ cfs}$$

- at elevation 63.00 ft:

Area (A) = 181 ft²
Wetted perimeter (p) = 39.4 ft
Manning's coefficient (n) = 0.035

$$Q = (181) \left(\frac{1.486}{0.035} \right) \left(\frac{181}{39.4} \right)^{2/3} (0.005)^{1/2} = 1502 \text{ cfs}$$

- at elevation 67.00 ft:

$A_1 = 4.0 \text{ ft}^2$	$A_2 = 336.5 \text{ ft}^2$	$A_3 = 2.0 \text{ ft}^2$
$p_1 = 4.1 \text{ ft}$	$p_2 = 48.1 \text{ ft}$	$p_3 = 2.8 \text{ ft}$
$n_1 = 0.060$	$n_2 = 0.035$	$n_3 = 0.060$

$$Q = Q_1 + Q_2 + Q_3$$

$$Q = \left[\begin{aligned} & (4.0) \left(\frac{1.486}{0.060} \right) \left(\frac{4.0}{4.1} \right)^{2/3} \\ & + (336.5) \left(\frac{1.486}{0.035} \right) \left(\frac{336.5}{48.1} \right)^{2/3} \\ & + (2.0) \left(\frac{1.486}{0.060} \right) \left(\frac{2.0}{2.8} \right)^{2/3} \end{aligned} \right] (0.005)^{1/2}$$

$$Q = 3700 \text{ cfs}$$

- at elevation 71.00 ft:

$A_1 = 36.5 \text{ ft}^2$	$A_2 = 500.5 \text{ ft}^2$	$A_3 = 18.0 \text{ ft}^2$
$p_1 = 12.5 \text{ ft}$	$p_2 = 48.1 \text{ ft}$	$p_3 = 8.5 \text{ ft}$
$n_1 = 0.060$	$n_2 = 0.035$	$n_3 = 0.060$

$$Q = Q_1 + Q_2 + Q_3$$

$$Q = \left[\begin{aligned} & (36.5) \left(\frac{1.486}{0.060} \right) \left(\frac{36.5}{12.5} \right)^{2/3} \\ & + (500.5) \left(\frac{1.486}{0.035} \right) \left(\frac{500.5}{48.1} \right)^{2/3} \\ & + (18) \left(\frac{1.486}{0.060} \right) \left(\frac{18}{8.5} \right)^{2/3} \end{aligned} \right] (0.005)^{1/2}$$

$$Q = 7344 \text{ cfs}$$

- b.) Plot a stage-discharge curve with these points
- c.) The stage at the desired discharge of 2200 cfs is found to be 64.9 ft.
(See Figure 4-A.5-1(b))

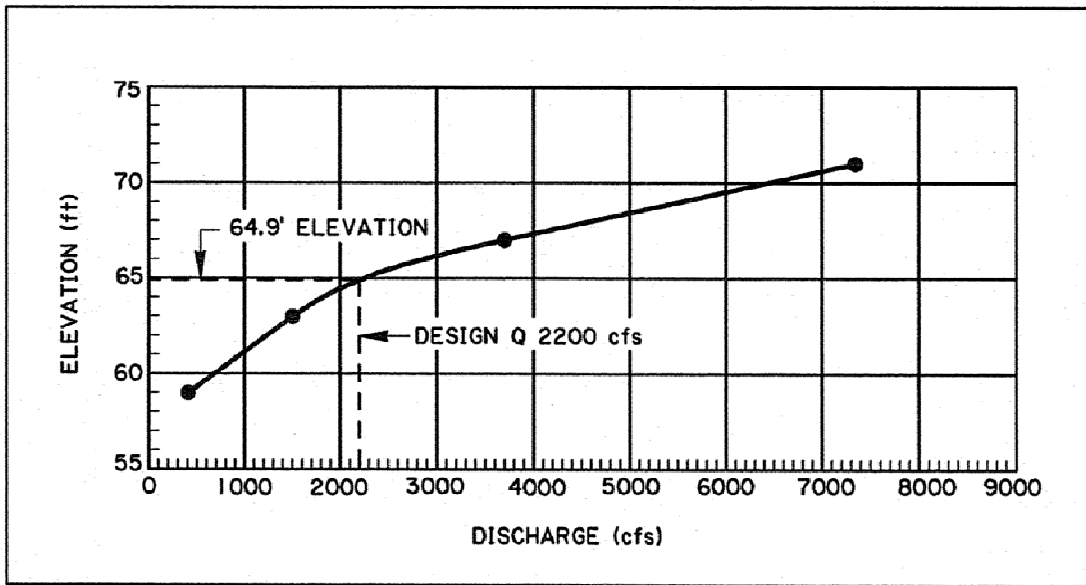


Figure 4-A.5-1(b) Stage-Discharge Curve for Example Problem 4-A.5.1

4-A.6 EROSION CONTROL PROTECTION

4-A.6.1 Open Channels and Roadside Ditches

One method commonly applied to determine if a channel is stable is the permissible tractive force (shear stress) method. The hydrodynamic force of water flowing in a channel is known as the tractive force. The basis for stable channel design is that flow-induced tractive force should not exceed the permissible shear stress of the selected erosion control level.

The design procedure for determining the shear stress a channel must sustain is outlined in FHWA Publication, HEC 15. (Reference C in Section 4-A.1)

4-A.6.2 Lateral Channels

A similar procedure as noted in Section 4-A.6.1 may be followed for lateral channels except that the design discharge of the culvert or culverts they serve should be used to evaluate the level of permanent erosion protection.

CHAPTER 5 CULVERT MATERIALS AND INSTALLATION

5.1 PURPOSE

This chapter presents standards and practices for specifying culvert materials and installation of all culverts used in cross drains, side drains, median drains and storm drains.

5.2 PIPE MATERIAL

It is LADOTD's policy to allow alternate materials and options for pipe culverts. The LADOTD pipe material policy is covered by the following sections of LADOTD's Engineering Directives and Standards Manual (EDSM):

- A.) **II.2.1.1** – "Pipe Material Selection Policy for Cross Drains, Side Drains and Storm Drains"
- B.) **II.2.1.6** – "Procedure for Determining Coating and Gage (Thickness) Requirements for Metal Pipe"
- C.) **II.2.1.13** – "Procedure for Quality Assurance, Fill Heights and Installation Requirements for Plastic Pipe Used in Cross Drains, Side Drains and Storm Drains"

The current approved copy may be found on the LADOTD website:

<http://webmail.dotd.louisiana.gov/ppmemos.nsf>

5.2.1 Pipes under Railroads

When it is necessary to place or replace a cross drain (pipe or pipe arch) under a railroad, the designer should contact the specific Railroad Company to determine the type of pipe material they require under their railroad tracks.

5.3 DESIGN SERVICE LIFE FOR CULVERTS

The design service life is defined as the estimated expectancy of how many years the pipe material will last before failure. The required design service life for different pipe applications is defined in EDSM II.2.1.1.

5.4 GAGE & COATING REQUIREMENTS

Estimated service life for metal pipe depends on soil and water conditions in the field. Gage (thickness) and coating for corrugated metal pipes will be selected in accordance to the latest EDSM II.2.1.6 and Standard Plan SAM-1.

5.5 JOINT TYPES

The type of joints required for each structure classification, (Cross Drain Pipe, Storm Drain Pipe or Side Drain), shall be in accordance to the latest EDSM II.2.1.1.

5.6 ALLOWABLE FILL HEIGHTS

Allowable fill height is the distance in feet from the top of the culvert to the top of the paved surface.

5.6.1 Corrugated Metal Pipe Allowable Fill Heights

Allowable fill heights for various metal gages for corrugated steel and corrugated aluminum pipes (or pipe arches) are given in Standard Plan SAM-1.

5.6.2 Plastic Pipe Allowable Fill Heights

Allowable fill heights for plastic pipes shall be in accordance to the latest EDSM II.2.1.13.

5.6.3 Reinforced Concrete Pipe Allowable Fill Heights

Generally, Class III reinforced concrete pipe is used unless the allowable fill height for Class III reinforced concrete pipe is exceeded. Recommended allowable fill heights are:

- a.) 18' for Class III reinforced concrete pipe,
- b.) 26' for Class IV reinforced concrete pipe, or
- c.) 40' for Class V reinforced concrete pipe.

5.6.4 Reinforced Concrete Box Culverts Allowable Fill Heights

Fill heights for cast-in-place boxes are in accordance with the standard plans. If a greater fill height is needed, contact the Hydraulics Unit.

5.7 MULTIPLE PIPES

A multiple pipe installation is the placement of two or more pipes or pipe arches in a single trench or embankment condition. Table 5.7-1 provides minimum spacing between multiple pipes and pipe arches. The dimensions shown are minimum distances between multiple lines of pipes or pipe arches. The maximum number of pipes allowed in a multiple pipe installation is four (4).

Table 5.7-1 Minimum Spacing for Multiple Lines of Pipes or Arch Pipes

Diameter / Round Equivalent Diameter (inches)	Round Pipes	Arch Pipes
	Minimum Spacing (ft - inches)	Minimum Spacing (ft - inches)
≤ 24"	1' - 0"	1' - 0"
30"	1' - 3"	1' - 0"
36"	1' - 6"	1' - 3"
42"	1' - 9"	1' - 5"
48"	2' - 0"	1' - 8"
54"	2' - 3"	1' - 10"
60"	2' - 6"	2' - 1"
66"	2' - 9"	2' - 3"
72"	3' - 0"	2' - 6"
78"	3' - 0"	2' - 8"
84"	3' - 0"	2' - 10"
90"	3' - 0"	3' - 0"
96"	3' - 0"	3' - 0"
102" or larger	3' - 0"	3' - 0"

5.7.1 Skewed Multiple Pipe

The distance between two or more pipes or pipe arches is measured perpendicular to the pipe or pipe arch even when they are to be installed on a skew.

5.8 CULVERT BEDDINGS

Bedding is the earth or other material on which a culvert is supported. An important function of the bedding is to assure uniform support along the barrel of each culvert section. Standard Plan BM-01 gives general guidelines of placement of the bedding. LADOTD's Pavement and Geotechnical Services Section should be consulted so that proper bedding for any specified culvert may be designed, especially in situations where unstable or weak bedding conditions are likely.

5.9 CULVERT INSTALLATIONS

There are two major classes of installation: trench installation and embankment installation. Figure 5.9-1 illustrates various types of pipe installations.

5.9.1 Embankment Installation

The most common type of installation used for highway culverts is the embankment installation where embankment fill is placed over the pipe and above existing ground surface. Embankment installation is further broken down into positive and negative projecting. The positive projecting type covers those installations where the pipe is bedded with its top anywhere from existing ground level to 90% of the nominal pipe diameter above existing ground. The negative projecting embankment type covers those installations where the pipe is bedded in a trench with its top below the existing ground surface.

5.9.2 Trench Installation

Trench installation covers those installations where the pipe is placed in a trench with no fill above existing ground surface. Trench installation is the type most commonly used in urban subsurface systems.

5.10 PIPES TO BE BORED OR JACKED

Occasionally construction limitations require pipes to be bored or jacked. These are relatively expensive procedures when compared to open cutting the roadway and should be avoided whenever possible. In general, pipes 30" in diameter and greater shall be jacked, and pipes less than 30" in diameter shall be bored.

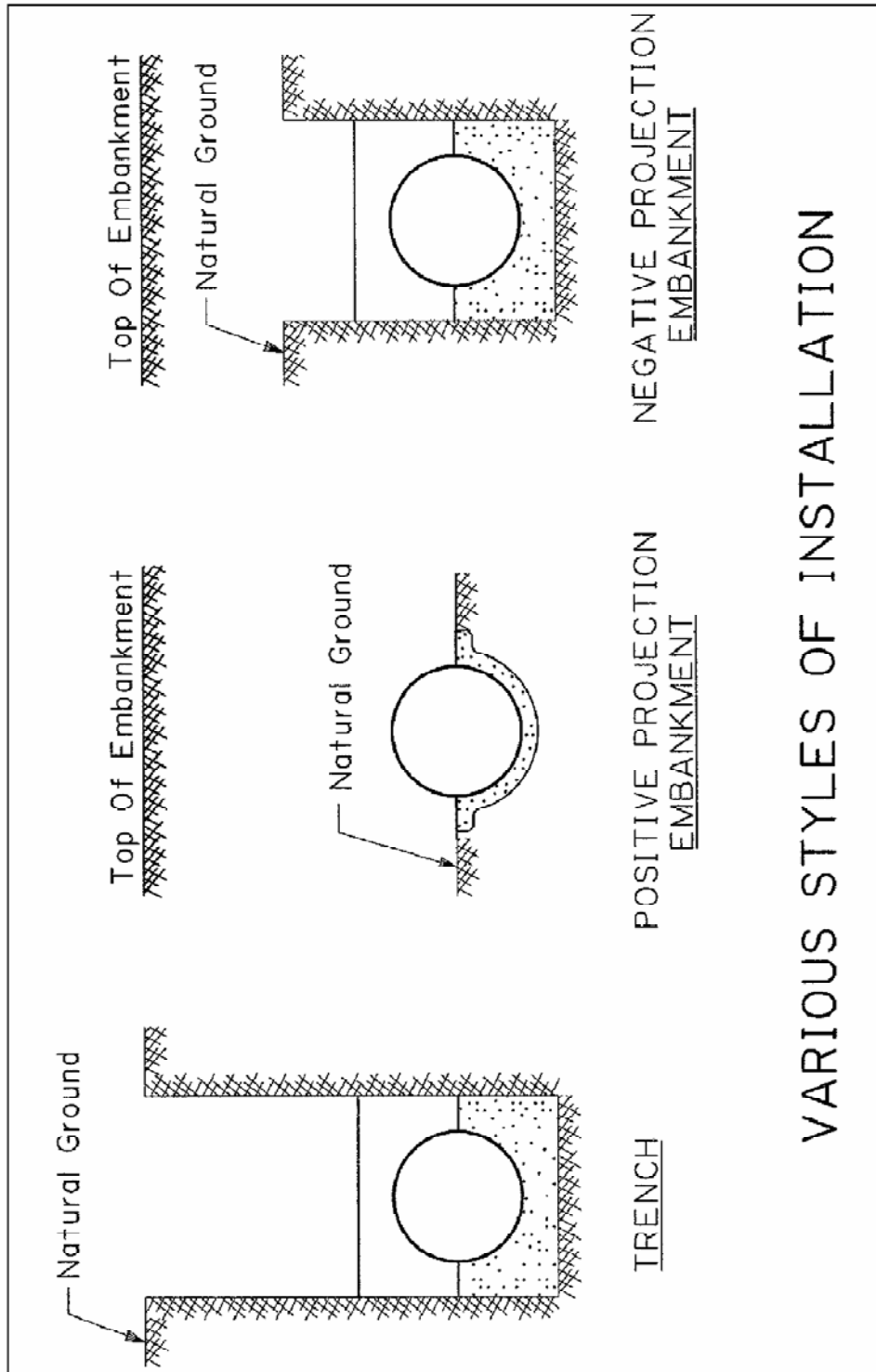


Figure 5.9-1 Typical Culvert Installations

5.11 SPECIAL INSTALLATIONS

Special installation conditions may warrant special design features, or they may justify the specification of one material type without alternate.

5.11.1 Limited Construction Time

Occasionally, the speed of construction may overshadow importance of economics. In such instances, pipes or precast reinforced concrete box culverts may be chosen over cast-in-place reinforced concrete box culverts.

CHAPTER 6

CROSS DRAIN CULVERTS

6.1 PURPOSE

This chapter presents standards and procedures for the design of culverts for adequate hydraulic capacity to carry the natural flow underneath a roadway. The hydraulic design of a cross drain consists of an analysis of the performance of the culvert in conveying flow from one side of the roadway to the other. To meet this conveyance function adequately, the design must include consideration of variables such as design discharge, tailwater elevation, headwater elevation, proper alignment, outlet velocity and maintenance problems. For detailed information of the hydraulic theory used in cross drain culvert design, refer to Part A of this chapter. A sample Hydrologic Summary Table is provided at the end of this chapter. It is useful in the design of culverts as well as being required on the plans.

6.2 DESIGN STORM FREQUENCY

The selection of design flood and headwater elevations should be based on the following risk conditions:

- Damage to adjacent properties
- Damage to the culvert and the roadway
- Traffic interruption
- Hazard to human life
- Damage to stream and floodplain environment.

The design storm frequency will be determined as a function of the Projected Annual Average Daily Traffic (PAADT). For further details on design frequency, see Chapter 1.

6.3 PEAK DISCHARGE

The methods for determining peak discharge are described in Chapter 3.

6.4 RUNOFF HYDROGRAPH

A variation of rate of runoff with time is defined as a runoff hydrograph. A runoff hydrograph is normally used when upstream storage volume is to be considered in the design of culverts and a reservoir routing routine is employed. Generally, however, only the peak runoff rate is used as a design parameter in the design of culverts.

6.5 CULVERT SELECTION

Conventional culverts include those commonly installed, such as circular pipes: reinforced concrete pipe (RCP), plastic pipe (PP) and corrugated metal pipe (CMP); arch pipes: reinforced concrete pipe arch (RCPA) and corrugated metal pipe arch (CMPA); and reinforced concrete box culverts (RCB). All such conventional culverts have a uniform barrel cross section throughout. The culvert inlet may consist of the culvert barrel projected from the roadway fill, mitered to the embankment slope, or it may have headwalls.

Whenever possible, the selected culvert should be the most economical alternative which meets the hydraulic, structural, and service life requirements. Generally, pipes may be installed more economically than reinforced concrete boxes; a single line of pipe is less expensive than double or multiple lines of smaller pipe; round pipe is less expensive than arch pipe.

In some circumstances, there are factors other than material cost which may govern the selection of the culvert. For these factors influencing culvert material selection including installation procedures, refer to Chapter 5 – Culvert Materials and Installation.

6.5.1 Use of Reinforced Concrete Box Culverts instead of Pipe

Conditions which may warrant the exclusive use of Reinforced Concrete Boxes:

- a.) In low headroom situations requiring multiple openings, reinforced concrete boxes have a geometric advantage in that more area of opening can be concentrated in a given width. This may result in not only an economic advantage, but also an operational advantage.
- b.) In urban areas, where the channel is paved or is to be paved, reinforced concrete boxes may “fit” the channel better.

6.6 SIZE OF STRUCTURES

Structures shall be sized based on their calculated hydraulic capacity as determined by an approved hydraulic analysis procedure. Culverts should be sized to fit the opening of the stream as much as possible, both vertically and horizontally. Where necessary, multiple lines of pipe may be used. See Chapter 5 for installation details.

6.6.1 Sizes of Reinforced Concrete Box Culverts

The Department has many Standard Plans for reinforced concrete boxes. They are included in the Department’s Standard Plans Booklet which is available through the General Files Section.

6.6.2 Minimum Culvert Size

Table 6.6-1 shows the minimum size structures used by LADOTD for cross drains. The minimum round equivalent size for pipe arch is the same as for round pipe.

Table 6.6-1 Minimum Culvert Size

LOCATION	STRUCTURE	MINIMUM SIZE
Cross Drains	Cross Drain Pipe (Arch)	24" diameter or round equivalent arch
	Reinforced Concrete Box	4' × 4'

6.6.3 Sizing Pipes for Alternate Materials

For those projects requiring Cross Drain Pipe items (RCP/PP or CMP), a different diameter for CMP may be required due to differences in hydraulic performance.

In general, plastic pipes are the same size as concrete pipes, whereas metal pipes are at least one size larger in order to achieve the same hydraulic performance. That is, for diameters up to 60" in diameter, metal pipes will be 6" larger and for diameters 60" and greater, metal pipes will be 12" larger.

The same number of lines should be specified for reinforced concrete pipe/plastic pipe and corrugated metal pipe when Cross Drain Pipe or Cross Drain Pipe Arch items are used.

6.7 CONSTRUCTION CLEARANCE

In order for a pipe not to be damaged during construction, **a minimum 9" cover is required between the top of the pipe and the lowest part of the subgrade.** Minimum cover should be increased to 12" for pipes greater than 84" in diameter.

There is no construction clearance required for the department's standard reinforced concrete box culverts.

6.7.1 Limited Headroom

In cases of limited headroom, it is often necessary to specify arch pipe instead of round pipe in order to achieve the necessary hydraulic capacity.

Round and arch pipes may not be mixed within a conduit item. Each cross drain pipe site requires either round pipe or arch pipe.

6.7.2 Burying Culverts

In order to accommodate possible future development, or in cases of limited headroom, the flow line of the structure may be buried below the channel bottom by a distance of up to:

- a.) one-third ($\frac{1}{3}$) the diameter of the round pipe,
- b.) one-fourth ($\frac{1}{4}$) the rise of the pipe arch,
- c.) one-third ($\frac{1}{3}$) the box height

The maximum depth any culvert may be buried is one foot.

6.8 CULVERT SLOPE

Culverts are generally installed on the same slope as the natural streambed slope. Modified culvert slopes or slopes other than that of the natural stream should be given special attention to ensure that detrimental effects do not result from the change.

6.8.1 Flow Line (FL)

Flow line is defined as the elevation of the bottom of the inside of a culvert. Flow lines should be set to match the natural stream bed and slope, or to match design stream bed in cases where channel improvements are being made. Invert elevations will usually be the same for all alternates.

6.9 BACKWATER

Table 6.9-1 summarizes the choice of the correct allowable headwater (AHW) and/or differential head (ΔH). Below are the definitions and when to incorporate each one in the design.

6.9.1 Differential Head (ΔH)

Differential head is the difference between the headwater and tailwater. At conditions of flat gradients and greater existing or potential development, differential head is the most practical design parameter.

Where Differential Head (ΔH) governs, it should not be greater than one foot.

In areas that are “flood sensitive” (such as urban areas in low spots that flood frequently), ΔH should be kept to a minimal increase without exceeding 0.5 ft.

It should be remembered that the horizontal extent of ponding for a given differential head will be much greater in flat areas than in hilly areas.

6.9.2 Allowable Headwater (AHW) & Allowable Headwater Elevation (AHWE)

In general, when the gradient of the upstream land is appreciable, and the existing development is minimal, then the Allowable Headwater (AHW) will be more important than the Differential Head (ΔH). However, the Differential Head should never exceed 1 foot.

Allowable Headwater Elevation for the design event should be at least one foot below the lowest elevation of the paved surface (excluding the shoulder), but may not exceed three (3) feet above the top opening elevation of the pipe. See Figure 6.9-1

6.9.3 Future Development

Existing and potential development (houses, commercial establishments, etc.) must be considered. The differential head (ΔH) is especially important in locations that have greater potential for development. In all cases, headwater elevations should be set at reasonable distances below existing slab elevations or reasonable forecasts of future slab elevations.

**Table 6.9-1 Guidelines for Design Parameters
Allowable Headwater (AHW) and Allowable Differential Head (ADH)**
(Refer to Section 6.9 for further criteria and restrictions)

RURAL – Nonagricultural, No Development
<p>Topography: Rural, rough, steep grades, pine hills, little or no existing potential development, nonagricultural, suited to timber farming. Terrain representative of north LA.</p> <p>Allowable Headwater (AHW): In some cases, potential for erosion at the outlet may control. Many structures will be in inlet control.</p> <p>Allowable Differential Head (ADH): Not significant</p>
RURAL – Agricultural, Limited Development
<p>Topography: Rural, rolling land, agricultural, subject to limited development.</p> <p>Allowable Headwater (AHW): Minimum clearance below finish grade will probably govern, but more consideration should be given to development possibilities.</p> <p>Allowable Differential Head (ADH): Usually not the most significant design parameter</p>
RURAL – Outskirts of Town, Some Development
<p>Topography: Rural, but on the outskirts of a town with some development existing and more expected, relatively flat land. Terrain representative of south Louisiana.</p> <p>Allowable Headwater (AHW): Usually not the most significant design parameter. Consideration should be given to HW as it may indicate a potential roadway flooding problem which could require corrective action, such as outfall improvement or centerline grade adjustment.</p> <p>Allowable Differential Head (ADH): The most significant design parameter. Limits are from 0.5' to about 1.0', depending on the terrain. Generally, the flatter the surrounding upstream land, the smaller the allowable differential head (ADH).</p>
URBAN
<p>Topography: Urban, within city limits with usual city type development.</p> <p>Allowable Headwater (AHW): ADH should be the controlling parameter. Consideration should be given to HW as it may indicate a potential roadway flooding problem which could require corrective action, such as outfall improvement or centerline grade adjustment.</p> <p>Allowable Differential Head (ADH): The most significant design parameter. Limits are from 0.5' to about 1.0'.</p>

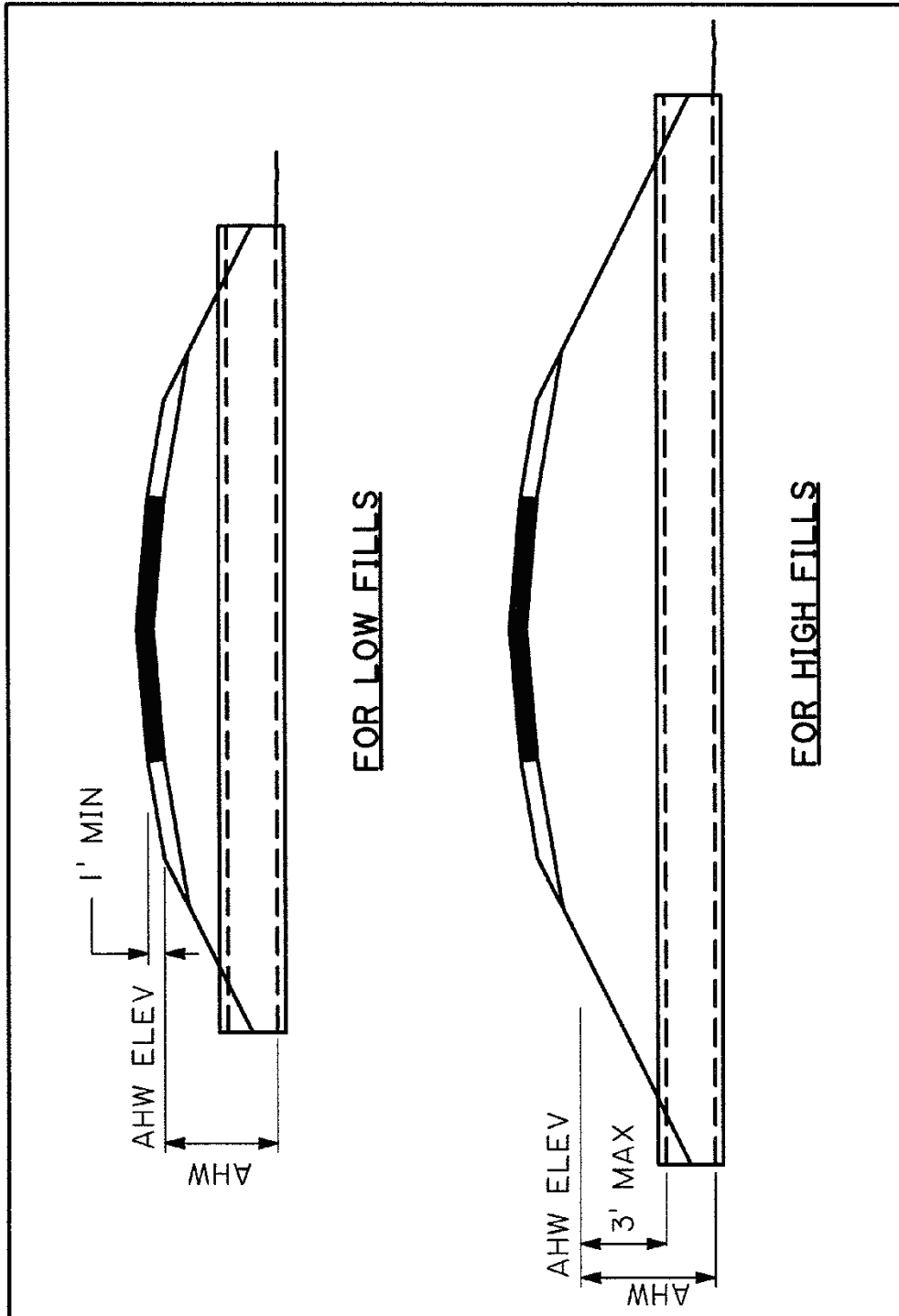


Figure 6.9-1 Allowable Headwater Elevation

6.10 OUTLET VELOCITY

Another factor influencing the performance of a culvert is the outlet velocity. The outlet velocity is the velocity measured at the downstream end of the culvert and it is usually higher than the maximum natural stream velocity. This higher velocity can cause streambed scour and bank erosion at the culvert outlet. (See Section 6.11) The slope and the roughness of the culvert barrel are the principle factors affecting outlet velocity.

6.11 SCOUR PROTECTION AT CULVERT OUTLETS

Scour occurs at every unprotected culvert outlet during major storms. In most cases, it is not of sufficient magnitude to affect the embankment or the structure. However, in some cases, culvert outlet scour may be very damaging. In fact, an entire embankment may fail due to a repetitive action consisting of scour hole formation followed by end joint loss. Culvert outlet scour problems are most often associated with structures that have velocities of approximately 9 ft/s or greater. It is assumed that maximum tailwater (tailwater equal to or greater than one-half the depth of flow at the culvert outlet) occurs most of the time in Louisiana.

6.11.1 Scour Protection

When the predicted depth of scour is greater than the limits listed in Table 6.11-1, scour protection will be required. Such protection could include headwalls, toewalls or riprap. The Hydraulics Unit has a listing of their headwall and toewall special details.

For conditions in Louisiana, increasing structure sizes in an effort to decrease scour to permissible values is usually an uneconomical solution when compared to using outlet scour protection.

Table 6.11-1 Maximum depths of scour which will be allowed without protection

Type of Structure	Maximum Depth of Scour for Unprotected Outlets
Reinforced Concrete Pipe without Headwalls	3.0 ft
Reinforced Concrete Pipe with Headwalls or Corrugated Metal Pipe, Plastic Pipe with last joint \geq 20 ft in length	4.5 ft
Reinforced Concrete Box	6.0 ft

6.12 LENGTH OF CULVERTS

Lengths of all pipe and pipe arch culverts will be specified in multiples of 2 ft. Lengths of all reinforced concrete box culverts are rounded off to the whole foot. The barrel length of a box culvert is measured from inside to inside of the headwall.

6.12.1 Calculation of Cross Drain Pipe Lengths

Lengths of cross drain structures are calculated by procedures described in Figures 6.12-1 and 6.12-2. Lengths of cross drain pipes without headwalls will be calculated based on the average elevation of the pipe centerlines (spring line elevation for pipe arches) of the alternates.

6.12.2 Use of Pipe Headwalls

The use of headwalls seldom reduces the length of a pipe culvert enough to offset the cost of the headwall. For this reason, pipe headwalls will generally not be used except for the following reasons:

- a.) When a cost estimate based upon latest unit prices indicates that the culvert will be less expensive with headwalls than without.
- b.) When necessary to keep the length of the culvert within the right-of-way or other physical constraints.
- c.) For special hydraulic requirements, such as the need for retaining walls, etc.
- d.) When required for erosion protection.

End structures used to separate storm water from irrigation water (rice flumes) are a special case of pipe headwalls, and the above discussion does not apply to those structures.

6.12.3 Reinforced Concrete Box Extension

Sometimes if an existing box culvert is extended, a Special Detail is required.
(See Chapter 2)

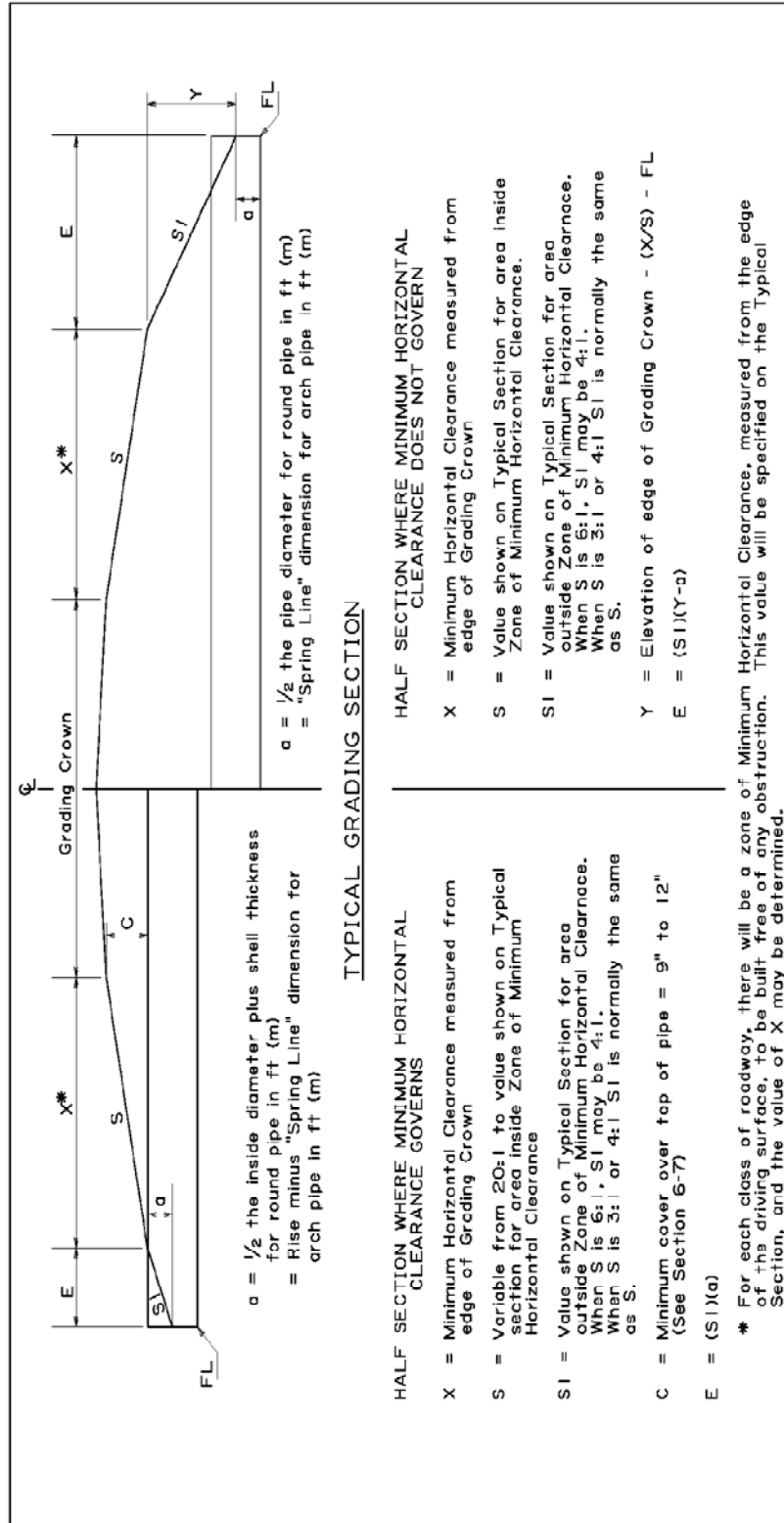


Figure 6.12-1 Computing Lengths for Pipes (Arches) without Headwalls

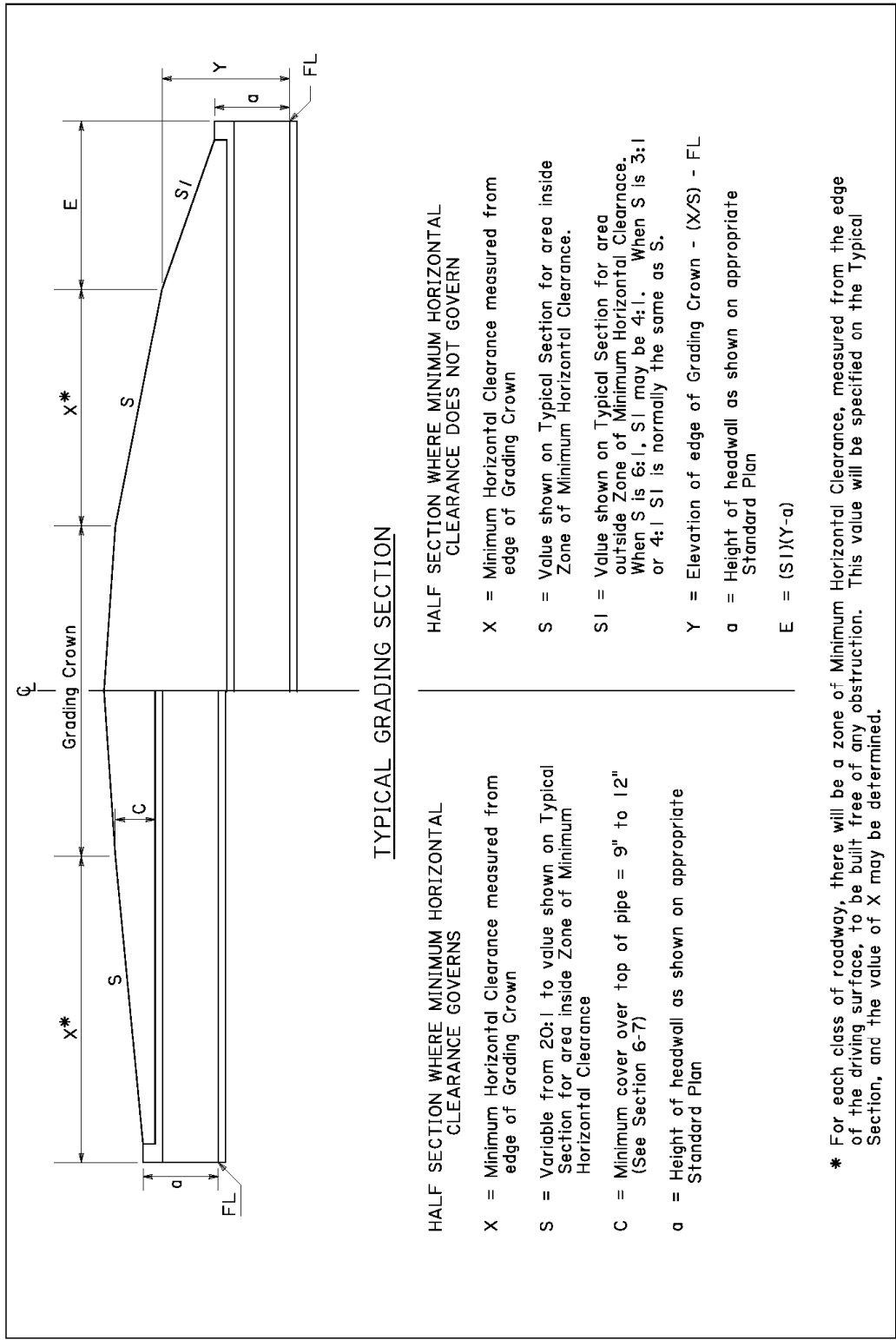


Figure 6.12-2 Computing Lengths for RCB and Pipes (Arches) with Headwalls

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CHAPTER 6 – PART A CULVERT HYDRAULICS

6-A.1 INTRODUCTION

There are two major conditions of culvert flow: (1) flow with **inlet control** and, (2) flow with **outlet control**.

To find more information on the theory of culvert hydraulics, refer to the following reference:

- A.) Hydraulic Design of Highway Culverts, Hydraulic Design Series No. 5, FHWA, Washington, D.C., September 2001, revised May 2005.

6-A.2 COMPUTER PROGRAMS

The following programs have been approved for use in the design of hydraulic structures for LADOTD highways:

- A.) The LADOTD computer program HYDR1120: “Hydraulic Analysis of Culverts” is based on the procedures outlined in this section.
- B.) HY 8 – FHWA computer program for hydraulic analysis of culverts.

6-A.3 TERMINOLOGY

- A.) Differential Head (DH or ΔH): The difference between headwater and tailwater.
- B.) Flow Line (FL): Flow line is defined as the elevation of the bottom of the inside of a culvert.
- C.) Headwater (HW): Any culvert which constricts the natural stream flow will cause a rise in upstream water surface to some extent. Headwater (HW) is defined as the total flow depth of the upstream water surface measured from the culvert inlet flow line.
- D.) Tailwater (TW): Tailwater (TW) is defined as the flow depth of water downstream of the culvert measured from the flow line of the culvert outlet. Tailwater can be an important factor in culvert hydraulic design because a submerged outlet may cause culverts to flow full rather than partially full.

6-A.4 MANNING'S COEFFICIENT

Table 6-A.4-1 lists the commonly used Manning's roughness coefficients used in the analysis of the hydraulic capacity of culverts.

Table 6-A.4-1 Manning's Roughness Coefficient, n, for Closed Conduits.

Type of Structure	n
Concrete Pipe	0.012
Concrete Box	0.012
Corrugated Metal Pipe:	
2 2/3" × 1/2" (68 × 13 mm) corrugations	0.024
3" × 1" (75 × 25 mm) corrugations	0.027
Plastic Pipe:	
Smooth flow	0.009
Corrugated	0.020

6-A.5 DESIGN HEADWATER

Design headwater depth is determined by computing headwater depths for both inlet control and outlet control and then using the higher value.

6-A.6 INLET CONTROL

A culvert operates with inlet control when the culvert barrel is capable of conveying more flow than the inlet will accept. The flow capacity is controlled at the entrance by the depth of headwater and entrance geometry, including the barrel shape, cross-sectional area and the inlet edge. Figure 6-A.6-1 shows different examples of inlet control.

In inlet control, roughness and length of the culvert barrel and outlet conditions (including tailwater) are not factors in determining culvert hydraulic performance. An increase in barrel slope reduces the headwater (HW) to a small degree, and any correction for slope may be neglected for conventional or commonly used culverts.

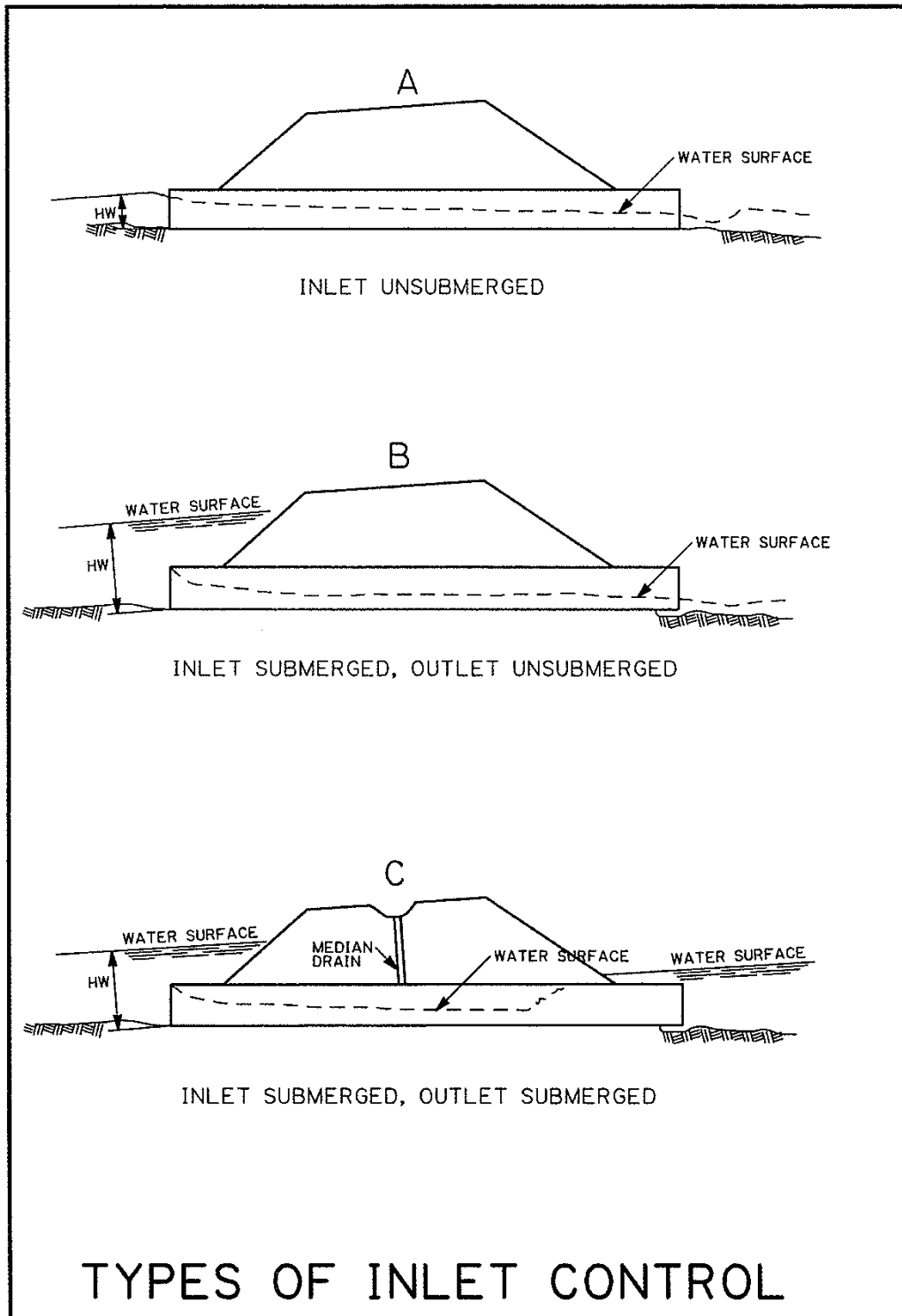


Figure 6-A.6-1 Examples of Inlet Control

6-A.6.1 Headwater Computation for Inlet Control Conditions

For culverts operating with inlet control, headwater (HW) is computed by the following equation:

$$HW = D \left(J + KX + LX^2 + MX^3 + NX^4 + PX^5 \right) \quad \text{Eq. 6-A.6-1}$$

Where: HW = Headwater (ft)
D = Height of culvert (ft)
J, K, L, M, N, P = Coefficients determined by inlet geometry.
(See Table 6-A.6-1)

$$X = \frac{Q}{BD^{1.5}}$$

Where: Q = Discharge (ft³/s)
B = Width of culvert (ft)

6-A.6.2 Depth of Flow Computation for Inlet Control Conditions

For culverts operating under inlet control, depth of flow at the outlet is the normal depth of flow (dn). Normal depth of flow is obtained by balancing the following equation:

$$\frac{Qn}{1.486S^{1/2}} = \frac{A^{5/3}}{p^{2/3}} \quad \text{Eq. 6-A.6-2}$$

Where: Q = Discharge (ft³/s)
n = Manning's roughness coefficient (Table 6-A.4-1)
S = Culvert slope (ft/ft)
A = Area of flow at any depth of flow (ft²)
p = Wetted perimeter of flow at any depth of flow (ft)

6-A.6.3 Outlet Velocity Computation for Inlet Control Conditions

Outlet velocity for culverts operating under inlet control is obtained from Manning's equation:

$$V = \left(\frac{1.486}{n} \right) \left(R^{2/3} \right) \left(S^{1/2} \right) \quad \text{Eq. 6-A.6-3}$$

Where: n = Manning's roughness coefficient (Table 6-A.4-1)
R = Hydraulic radius = A/p (ft)
A = Area of flow at normal depth of flow (ft²)
p = Wetted perimeter of flow at normal depth of flow (ft)
S = Culvert slope (ft/ft)

Table 6-A.6-1 Inlet Control Headwater Coefficients

CULVERT TYPE	INLET TYPE	J	K	L	M	N	P
REINFORCED CONCRETE PIPE	SOCKET-END PROJECTING	0.108786	0.662381	-0.233801	0.0579585	-0.0055789	0.00020505
	SOCKET-END WITH HEADWALL	0.114099	0.653562	-0.233615	0.0597723	-0.0061634	0.00024283
CORRUGATED METAL PIPE	PROJECTING	0.187321	0.567719	-0.156544	0.0447052	-0.003436	0.00008966
	HEADWALL	0.167433	0.538595	-0.149374	0.0391543	-0.0034397	0.00011588
	MITERED	0.107137	0.757789	-0.361462	0.1233932	-0.0160642	0.00076739
REINFORCED CONCRETE PIPE ARCH	SOCKET-END PROJECTING	0.09618	0.52593	-0.13504	0.03394	-0.00325	0.00013
	SOCKET-END WITH HEADWALL	0.1301	0.43477	-0.07911	0.01764	-0.00114	0.00002
CORRUGATED METAL PIPE ARCH	PROJECTING	0.089053	0.712545	-0.270921	0.0792502	-0.0079805	0.00029321
	HEADWALL	0.111281	0.610579	-0.194937	0.0512893	-0.0048054	0.00016855
	MITERED	0.083301	0.795145	-0.434075	0.163774	-0.0249139	0.00141066
REINFORCED CONCRETE BOX	SQUARE-EDGE HEADWALL	0.122117	0.505435	-0.10856	0.020781	-0.0013676	0.00003456
	BEVEL-EDGE HEADWALL	0.1566086	0.3989353	-0.0640392	0.01120135	-0.0006449	0.000014566

6-A.7 OUTLET CONTROL

Outlet control flow occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. The control section for outlet control flow in a culvert is located at the barrel exit or further downstream. In outlet control, the culvert hydraulic performance is determined by the following factors: depth of headwater, entrance geometry, including the barrel shape, cross-sectional area and the inlet edge, plus the controlling water surface elevation at the outlet and the slope, length, and roughness of the culvert barrel.

A culvert operating in outlet control may flow full or partly full, depending on various combinations of the above factors.

Figure 6-A.7-1 shows different examples of outlet control.

6-A.7.1 Headwater Computation for Outlet Control Conditions

Headwater depth (HW) for culverts operating under outlet control and flowing full can be determined by the following equation:

$$HW = H + H_o - (L \times S) \quad \text{Eq. 6-A.7-1}$$

Where:

- HW = Headwater depth (ft)
- H = Head for full flow (ft) (See Section 6-A.7.2)
- L = Length of the culvert (ft)
- S = Slope of the culvert (ft/ft)

H_o = Distance to the hydraulic grade line from the outlet invert (ft)
= D when $d_c \geq D$ and $D > TW$
= $(d_c + D)/2$ when $d_c < D$ and $(d_c + D)/2 > TW$
= TW when $TW > D$ or $TW > (d_c + D)/2$

Where:

- TW = Tailwater (ft)
- D = Height of culvert (ft)
- d_c = Critical depth of flow in culvert (ft)
(See Section 6-A.7.3)

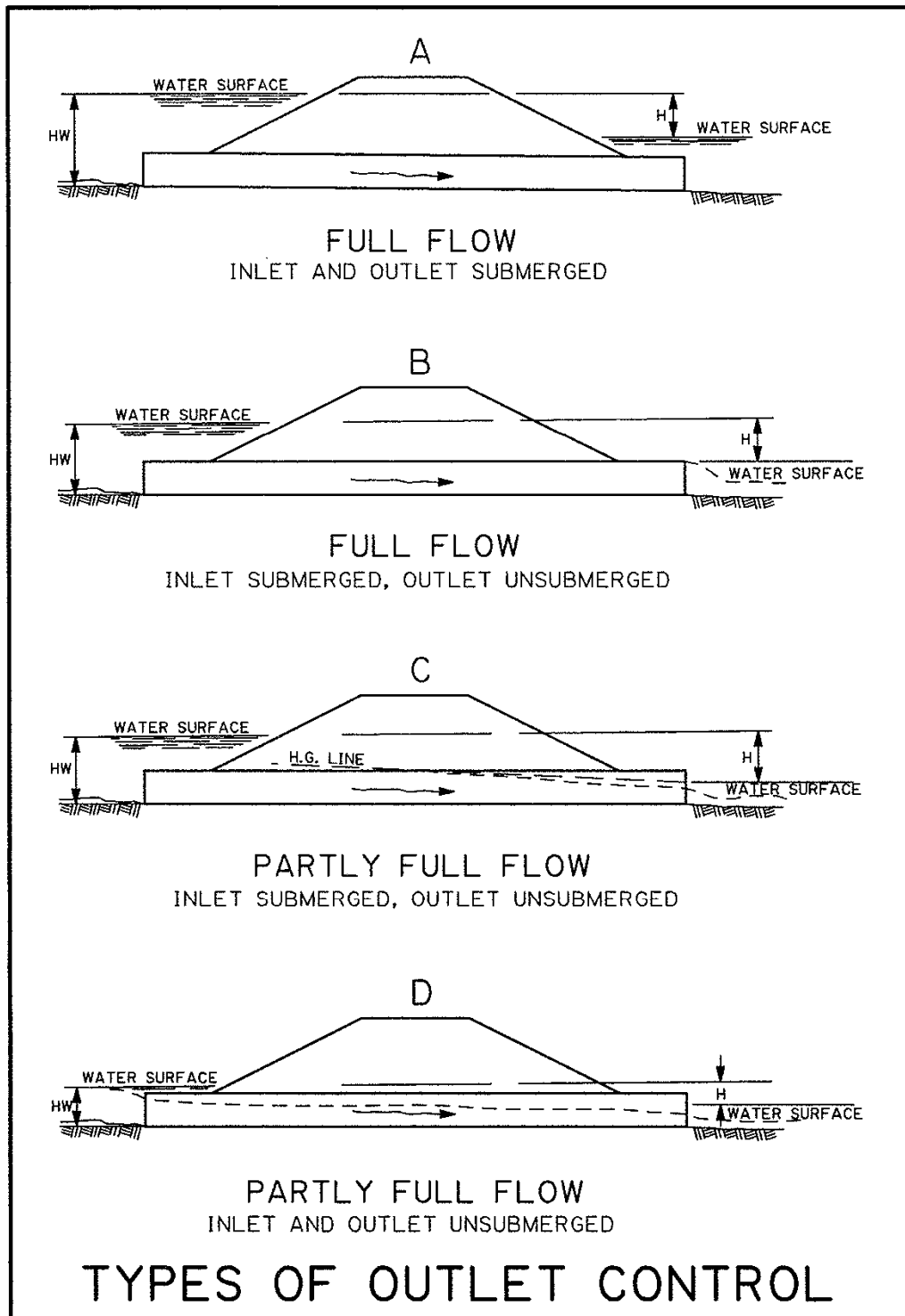


Figure 6-A.7-1 Examples of Outlet Control

6-A.7.2 Head Computation for Outlet Control Conditions, Full Flow

$$H = \left[1 + K_e + \left(\frac{29.14 n^2 L}{R^{4/3}} \right) \right] \left(\frac{V^2}{2g} \right) \quad \text{Eq. 6-A.7-2}$$

Where:

K_e	=	Entrance loss coefficient (Table 6-A.7-1)
n	=	Manning's coefficient of roughness (Table 6-A.4-1)
L	=	Length of the culvert (ft)
R	=	Hydraulic radius = A/p (ft)
A	=	Cross-sectional area of the culvert (ft ²)
p	=	Wetted perimeter of the culvert (ft)
V	=	Full flow velocity = Q/A (ft/s)
Q	=	Discharge (ft ³ /s)
g	=	Acceleration due to gravity (32.2 ft/s ²)

Table 6-A.7-1 Entrance Loss Coefficients, K_e

Types of Structure and Entrance Configuration*	K_e
Concrete Pipe (or Pipe Arch)	
projecting from fill, socket end	0.2
projecting from fill, square cut end	0.5
mitered to conform to fill slope	0.7
headwall, socket end of pipe or rounded	0.2
headwall, square-edge	0.5
Corrugated Metal Pipe (or Pipe Arch)	
projecting from fill	0.9
mitered to conform to fill slope	0.7
headwall, square-edge	0.5
Reinforced Concrete Box	
headwall parallel to embankment,	
square-edge on 3 sides	0.5
bevel-edge on 3 sides	0.2

* For more information on entrance configurations and entrance loss coefficients refer to Hydraulic Design of Improved Inlets for Culverts, Hydraulic Engineering Circular No. 13, FHWA.

6-A.7.3 Critical Depth of Flow Computation for Outlet Control Conditions

Critical depth of flow (d_c) is obtained by balancing the following equation:

$$(\alpha) \frac{Q^2}{g} = \frac{A^3}{T} \qquad \text{Eq. 6-A.7-3}$$

- Where:
- α = Velocity distribution factor (Table 6-A.7.2)
 - Q = Discharge (ft^3/s)
 - g = Acceleration due to gravity ($32.2 \text{ ft}/\text{s}^2$)
 - A = Cross-sectional area of water at any depth of flow (ft^2)
 - T = Top surface of water at any depth of flow (ft)

Table 6-A.7-2 Values of Velocity Distribution Factor, α , for Various Types of Culverts

Culvert Type	α
Reinforced Concrete Box	1.00
Reinforced Concrete Pipe	1.04
Reinforced Concrete Pipe Arch	1.05
Corrugated Metal Pipe	1.12
Corrugated Metal Pipe Arch	1.16

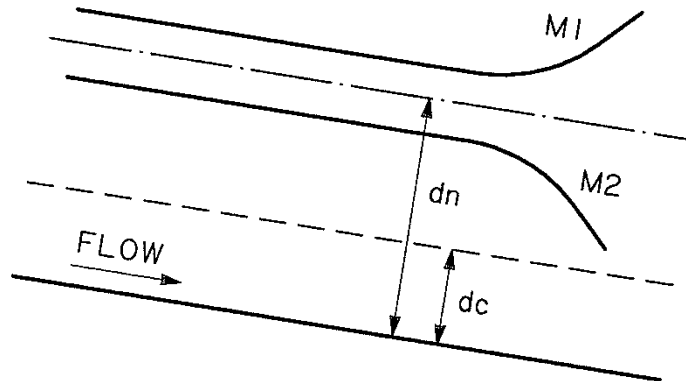


Figure 6-A.7-2 M1 and M2 Curves

6-A.7.4 Part-full Flow in Outlet Control Conditions

Part-full flow condition may occur when critical depth (d_c) falls below the crown of the culvert at the outlet, see Figure 6-A.7-4. To accurately determine headwater (HW) for culverts flowing part-full, backwater computations are required to establish a water surface profile. The water surface profile for normal depth (d_n), greater than or equal to d_c , is computed from the following equations:

- a.) **For Tailwater (TW) $\geq d_n$ (M1 Curve):**

$$\Delta L = \frac{\left(d_1 + \frac{V_1^2}{2g} \right) - \left(d_2 + \frac{V_2^2}{2g} \right)}{(S_{culv} - S_w)} \tag{Eq. 6-A.7-4}$$

- b.) **For Tailwater (TW) $< d_n$ (M2 Curve):**

$$\Delta L = \frac{\left(d_2 + \frac{V_2^2}{2g} \right) - \left(d_1 + \frac{V_1^2}{2g} \right)}{(S_w - S_{culv})} \tag{Eq. 6-A.7-5}$$

- Where:
- ΔL = Distance between two sections of water (ft)
 - d_1, d_2 = Depth at Sections 1 and 2, respectively (ft)
 - V_1, V_2 = Velocities at Sections 1 and 2 (ft/s)
 - g = Acceleration due to gravity (ft/s²)
 - S_{culv} = Slope of the culvert (ft/ft)

$$S_w = \frac{n^2 V^2}{(2.21)R^{4/3}} \quad \text{Eq. 6-A.7-6}$$

Where:	S_w	=	Slope of the water surface (ft/ft)
	n	=	Manning's coefficient of roughness (Table 6-A.4-1)
	V	=	Average velocity of the two sections (ft/s)
	R	=	Average hydraulic radius of the two sections (ft)

The backwater profile starts with a depth (d_1) equal to:

- The tailwater (TW) at the outlet and decreases in depth as the profile extends into the culvert for an M1 curve.
- The tailwater (TW) or critical depth of flow (d_c), whichever is greater, at the outlet and increases in depth (not to exceed the depth of the culvert) as the profile extends into the culvert for an M2 curve.

The depth (d_1) then approaches normal depth (d_n) in the culvert. If the normal depth (d_n) is not reached before the length of the culvert is exceeded, the depth for the last cross section in the culvert is used to compute the headwater (HW). HW is computed by the following equation:

$$HW = d_2 + \frac{V_2^2}{2g} + (K_e) \frac{V_1^2}{2g} \quad \text{Eq. 6-A.7-7}$$

Where:	HW	=	Headwater depth (ft)
	d_2	=	Depth at Section 2 (ft)
	V_2	=	Velocity at Section 2 (ft/s)
	V_1	=	Velocity at Section 1 (ft/s)
	K_e	=	Entrance loss coefficient (Table 6-A.7-1)

6-A.7.5 Outlet Velocity Computation for Outlet Control Conditions

Outlet velocity for culverts operating under outlet control is computed by the following equation:

$$V = \frac{Q}{A} \quad \text{Eqn. 6-A.7-8}$$

Where:	Q	=	Discharge (ft ³ /s)
	V	=	Outlet velocity (ft/s)
	A	=	Cross-sectional area for the depth of flow at the culvert outlet (ft ²). Depth of flow at the outlet is the critical depth of flow (d_c), or the tailwater (TW), whichever is greater, with a maximum equal to the height of the culvert.

6-A.8 DEPTH OF SCOUR

Depth of scour (D_s) can be determined by the following equation:

$$D_s = (0.75DM) \left(\left(40.51 - (F - 5.66)^2 \right)^{1/2} - 2.93 \right) \quad \text{Eq. 6-A.8-1}$$

Where: D_s = Depth of scour, ft
 D = Depth of flow at outlet, ft

For culverts operating under inlet control, D is the normal depth of flow which may be calculated using Manning's formula.

For culverts operating under outlet control, D is the critical depth, or the tailwater, whichever is larger, with a maximum equal to the height of the culvert.

M = Adjustment factor for various NRCS hydrologic soil groups (See Table 6-A.8-1)

$$F = \frac{V}{(gD)^{1/2}}$$

Where: F = Froude number at outlet
 g = Acceleration due to gravity, 32.2 ft/s²
 D = Depth of flow at outlet, ft
(as defined previously)
 V^{**} = Average velocity of flow at the outlet, ft/s

** Average velocity is computed by dividing the design discharge by the cross-sectional area of flow at the depth of flow discussed previously.

Table 6-A.8-1 Values of adjustment factor (M) for various NRCS hydrologic soil groups for the vicinity of the channel

NRCS Hydrologic Soil Group	Adjustment Factor
A	1.0
B	0.7
C	0.6
D	0.5

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HYDROLOGIC SUMMARY TABLE

OUTLET VELOCITY (FT/SEC)																			
DIFFERENTIAL HEAD (FT)																			
ALLOWABLE HEADWATER ELEVATION (FT)																			
HEADWATER ELEVATION (FT)																			
TAILWATER ELEVATION (FT)																			
DESIGN DISCHARGE (CFS)																			
HYDROLOGIC COEFFICIENTS	URBANIZATION																		
	URBANIZATION																		
	PONDING																		
USGS	MAIN CHANNEL SLOPE (FT/MI)																		
NRCS	HYDRAULIC LENGTH (FT)																		
	AVERAGE WATERSHED SLOPE (%)																		
	CURVE NUMBER																		
	SOIL CLASS																		
DRAINAGE AREA (ACRES)																			
STRUCTURE (SIZE & TYPE)																			
STATION																			
STRUCTURE NUMBER																			

1) DESIGN STORM FREQUENCY FOR CROSS DRAINS AND BRIDGES = YEARS

2) 24-HOUR RAINFALL, (- YEAR RETURN INTERVAL) = INCHES

3) ANNUAL PRECIPITATION = INCHES

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CHAPTER 7

SIDE AND MEDIAN DRAINS

7.1 PURPOSE

This chapter presents standards and procedures for the hydraulic design of side drains and median drains carrying low flow.

7.1.1 Exceptions

There will be situations where the side drain or median drain functions more as a cross drain culvert or times when the discharge from a cross drain goes into a roadside ditch and through a side drain before reaching the outfall. In those situations, the methods in this chapter do not apply. The procedures and policy for design of a cross drain culvert as outlined in Chapter 6 should then be followed.

7.2 DESIGN STORM FREQUENCY

Chapter 1 describes the LADOTD criteria for design frequency.

7.3 PEAK DISCHARGE

The methods for determining peak discharge are described in Chapter 3.

7.4 CULVERT SELECTION

Conventional culverts include those commonly installed, such as circular pipes: reinforced concrete pipe (RCP), plastic pipe (PP) and corrugated metal pipe (CMP); arch pipes: reinforced concrete pipe arch (RCPA) and corrugated metal pipe arch (CMPA); and reinforced concrete box culverts (RCB). All such conventional culverts have a uniform barrel cross section throughout.

Whenever possible, the selected culvert should be the most economical alternative which meets the hydraulic, structural, and service life requirements. Generally, a single line of pipe is less expensive than double or multiple lines of smaller pipe; round pipe is less expensive than pipe arch.

7.5 SIZE OF STRUCTURES

Structures shall be sized based on their calculated hydraulic capacity as determined by an approved hydraulic analysis procedure. Culverts should be sized to fit the waterway opening as much as possible, both vertically and horizontally. Where necessary, multiple lines of pipe may be used. See Chapter 5 for installation details. Other specific requirements for side drains and median drains are discussed later in this chapter.

7.5.1 Minimum Culvert Size

Table 7.5-1 shows the minimum size structures used by LADOTD for side and median drains. The minimum round equivalent size for pipe arch is the same as for round pipe.

Table 7.5-1 Minimum Culvert Size

STRUCTURE	LOCATION	MINIMUM SIZE
Side Drains	Private drives and average conditions	18" diameter or round equivalent arch
	Under State & Federal routes or provides an outfall for a cross drain	Same as for Cross Drains – See Chapter 6
Median Drains	Crossing beneath a state route or major roadway	24" diameter or round equivalent arch (Same as Cross Drains)
	Other applications	18" diameter or round equivalent arch (Same as for Side Drains)

7.6 CONSTRUCTION CLEARANCE

For the pipe to not get damaged during construction, a minimum 9" cover is required between the top of the pipe and the lowest part of the subgrade when the pipes are placed underneath the roadway. A minimum 6" cover between the top of the pipe and the ground elevation is required when the pipes are not placed underneath the roadway.

7.6.1 Limited Headroom

In cases of limited headroom, arch pipe may be specified instead of round pipe in order to achieve the necessary hydraulic capacity.

7.7 SIZING PIPES FOR ALTERNATE MATERIALS

Side drains and median drains usually carry such low flows that it is not necessary to specify different diameters between different pipe materials.

7.7.1 Flow Line (FL)

Flow line (invert) is defined as the elevation of the bottom of the inside of a culvert. Invert elevations will usually be the same for all alternates.

7.8 SIDE DRAINS

When an entrance to adjacent property crosses the roadside channel, a drainage structure and driveway or a "dry" ramp (ramp without a drainage structure) will be required. The type of drainage structure required is usually a Side Drain, although Side Drain Arch or multiple lines of pipe may be specified where headroom is a problem.

For ramps in fill (where the ramp elevation at the right-of-way line is lower than at the shoulder), the grade of the ramp between the shoulder and the side drain structure may be 0.00% if necessary to obtain sufficient cover. The ramp grade (in fill) should never be "humped" or sloped to the shoulder in order to achieve sufficient cover.

7.8.1 Size Determination

Table 7.8-1 is used to determine the required diameter of a Side Drain (Arch) for a 5 year return interval only.

The drainage area in Table 7.8-1 is in acres and the 24-hour rainfall storm is rounded to the nearest inch. The terrain classification for the general area of the side drain is based on the roadway centerline grades as follows:

- a.) Flat - centerline grades average between 0.00% and 0.50%.
- b.) Rolling - centerline grades average between 0.51% and 1.50%.
- c.) Hilly - centerline grades greater than 1.50%

Table 7.8-1 Maximum Contributory Drainage Area in Acres per Side Drain Structure (Pipe or Arch Pipe)

Side Drain Structure (Arch*) Diameter (inches)	24 Hour Rainfall						
	6"			7"			8"
	Terrain			Terrain			Terrain
	Flat	Rolling	Hilly	Flat	Rolling	Hilly	Flat
18	5	3	2	4	2	2	3
24	13	7	5	10	5	4	8
30	26	14	10	20	10	7	15
36	49	25	17	36	18	12	28
42	84	42	27	63	29	20	49
48	130	68	42	96	47	30	74
54	194	102	63	142	70	45	110
60	272	149	91	199	102	64	154

* Arch pipe sizes are given as their round equivalent sizes

7.9 MEDIAN DRAINS

There is no separate pay item for median drains. On the plans, median drain pipes are classified as either Cross Drain Pipes or Side Drains. See Figure 7.9-1.

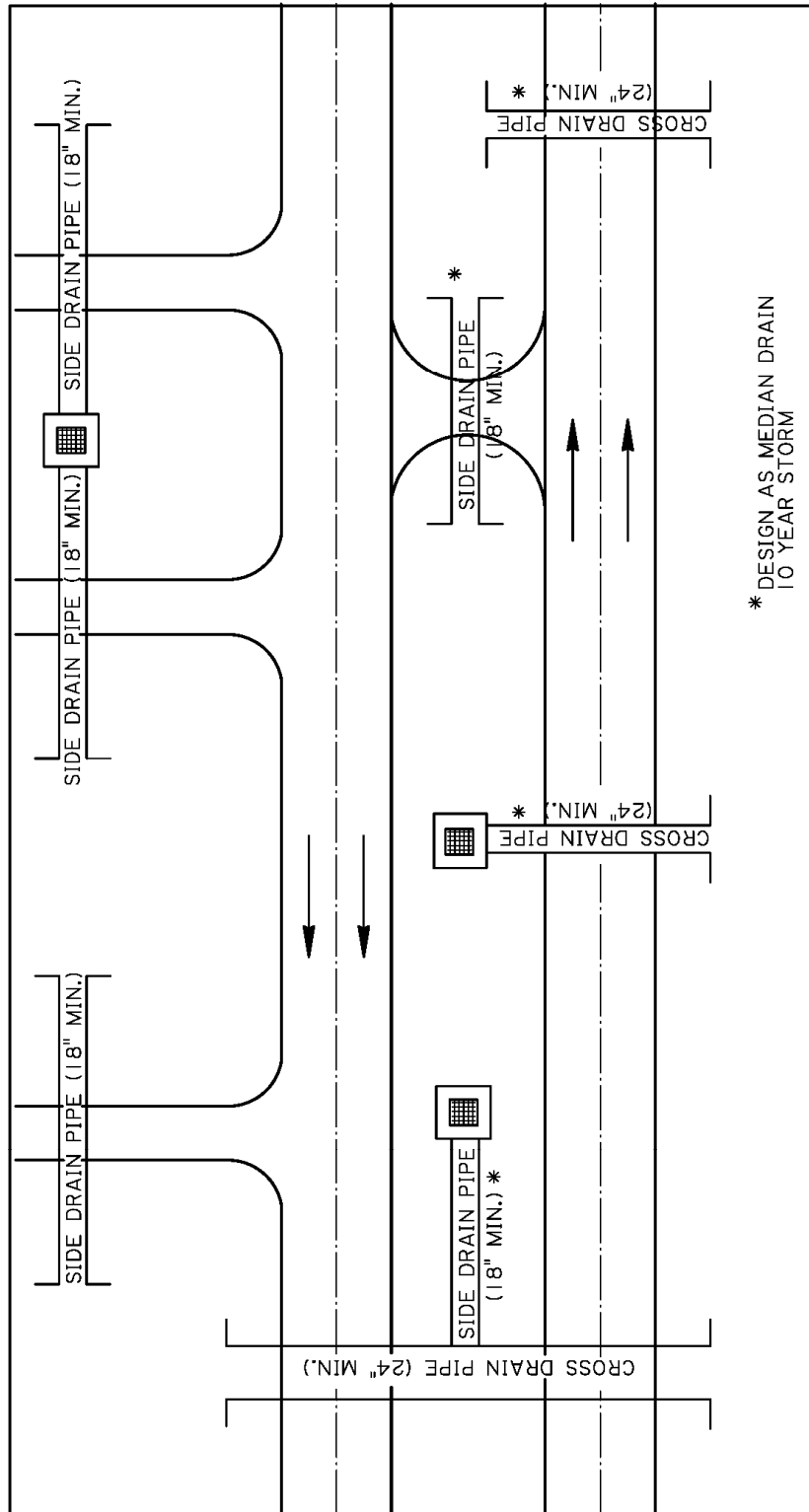


Figure 7.9-1 Pipe Layout Examples

7.9.1 Hydraulic Clearances

Calculated depth of ponding or the hydraulic gradient, whichever is greater, should be one foot below the lowest part of the finish grade.

In situations where a grate inlet is not used, the computed headwater should be one foot below the lowest part of the finish grade.

7.9.2 Ponding

When a grate inlet is used, or there will be a sag, the depth of ponding should be calculated according to the procedure described in Chapter 8 – Part A, Grate Inlets.

7.10 LENGTH OF CULVERTS

Lengths of all pipe and pipe arch culverts will be specified in multiples of 2 ft. Lengths of all reinforced concrete box culverts are rounded off to the whole foot. The barrel length of a box culvert is measured from inside to inside of the headwall.

7.10.1 Side Drain Lengths

Figure 7.10-1 describes the procedure to determine the length of Side Drain pipes and also shows the typical detail and grading section for a residential driveway.

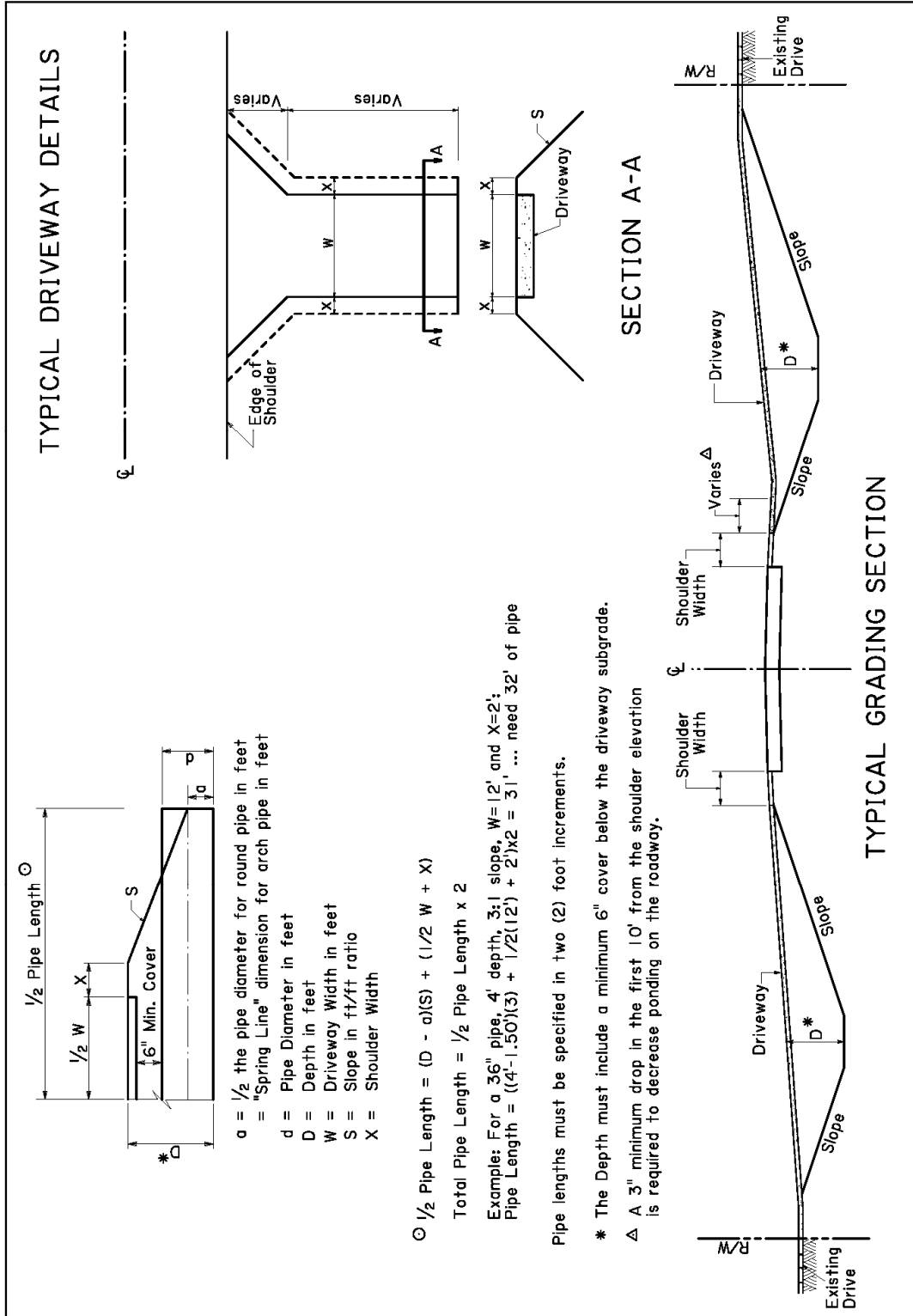


Figure 7.10-1 Side Drain Length Calculations

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CHAPTER 8 STORM DRAIN SYSTEMS

8.1 PURPOSE

A storm drain system consists of curb and gutter drainage that is used for collecting storm water runoff from LADOTD highways and adjacent properties in urban sections and directing that water into a natural stream. The main purpose of storm drain systems is to prevent flooding of streets and highways during the storm by quickly and efficiently transferring rainwater into an outfall. This chapter presents the procedures and guidelines for designing such systems.

8.2 POLLUTION PREVENTION

Storm drain systems are for clean rainwater only. The Environmental Protection Agency (EPA) requires cities, through National Pollutant Discharge Elimination System (NPDES) permits, to monitor the quality of water that flows from storm drain systems into the waterways.

8.3 DESIGN STORM FREQUENCY

Chapter 1 describes the LADOTD criteria for design frequency.

8.3.1 Design Waivers

If unusual conditions warrant, a “design waiver” to use a different frequency will be required. (See Chapter 1)

8.3.2 Lower Service Class Streets

For lower service class streets, if the 10 year design cannot be met, or creates significant design burden, the design may be amended to a 5 year or 2 year storm, with the concurrence of the Chief Design Engineer.

8.3.3 Lateral Ditches & Cross Drain Culverts

If there are any lateral ditches or other storm drainage structures intersecting the project, which are currently picked up in the existing highway system, their discharge will have to be picked up in the new system. When such laterals cross the centerline of the project and a cross drain must be provided, the structure shall be designed in accordance with the methods described in Chapter 6, Cross Drain Culverts. Structures conveying water from catch basins

to trunk lines on the opposite side of the roadway are not considered as cross drains and will be designed as part of the storm drain system by methods described here.

8.4 PEAK DISCHARGE

The methods for determining peak discharge are described in Chapter 3.

8.5 ROADWAY GRADE

Details on roadway grade design are presented in Chapter 1.

8.5.1 Minimum Cross Slope

A minimum cross slope of 0.025 ft/ft shall be used.

8.5.2 Minimum Longitudinal Grade

A minimum longitudinal grade of 0.0040 ft/ft shall be maintained.

8.6 BASIC DESIGN CONCEPTS

Runoff from areas draining toward the highway pavement should be intercepted by roadside channels where practical, or inlets where open channels cannot be used. This applies to drainage from side streets and other areas alongside the pavement. The drainage should be intercepted before it reaches the highway pavement.

The design of storm drain systems consist of two phases. The first phase deals with removal of water from the roadway and involves the spacing and locating of the catch basin inlets. The second phase involves designing the conduit or pipe system to carry this water to an outfall. A storm drain system is designed to receive all the pavement runoff, any rainfall falling in the highway right-of-way, plus any overland flow from adjacent properties while keeping the natural flow patterns of the land.

8.6.1 Preliminary Investigation

Before beginning the design of a storm drain system, the designer should be familiar with the:

- project site
- history of past floods
- effects of floods on the properties near the highway
- effects of floods on the existing drain systems
- natural outfalls near the project

8.7 OUTFALLS

It is LADOTD policy in urban areas to only improve outfalls to the right-of-way line. The local governing authority will be responsible for the construction and maintenance of any improvements beyond the right-of-way, and they should be notified.

Outfalls should only be modified if absolutely necessary for the proper functioning of a subsurface drainage system. If modifications are necessary and performed by parties other than LADOTD or its contractor, particular attention will be given to agreements made. These agreements will be brought to the attention of the Hydraulics Unit and will be noted on the plans. This drainage work will also be covered in any environmental assessment written for the project.

8.7.1 Outfall Pipe

For Storm Drain Pipe (Outfall) materials, refer to EDSM II.2.1.1.

8.8 SPACING AND LOCATION OF INLETS

It is good engineering practice and LADOTD policy to design inlet spacing so that water is removed from the roadway at a rate that prevents the roadway from becoming unusable or unduly hazardous during a storm of design recurrence interval.

8.8.1 Maximum Inlet Spacing

A maximum spacing of 200 ft is to be used between inlets or between a high point in grade and an inlet.

8.8.2 Maximum Width of Flooding

The primary design control for spacing of inlets is the width of flooding of the travel lane. It is the general policy of LADOTD that the system be designed so that at least one half ($\frac{1}{2}$) of the outer travel lane remains free of inundation during the design storm. This should not be deviated from without first consulting with the Hydraulics Unit.

8.8.3 Width of Flooding Reduction

To reduce the width of flooding, inlets may be spaced closer together. Other means to reduce the limits of flooding include: intercepting water behind the curb, providing a gutter section with greater conveyance capacity, etc.

8.8.4 Width of Flooding for Interstates

Width of flooding criteria for interstate systems may be more prohibitive, and should be evaluated on a case-by-case basis with LADOTD's Hydraulics Unit.

8.8.5 Geometric Controls

Pavement drainage inlet locations are often established by geometric features rather than by spread of water on the pavement and inlet spacing capacity. In general, inlets should be placed:

- a.) At all low points in the gutter grade
- b.) Upstream of street intersections
- c.) On both sides of street intersections where water would flow towards the project (water will not be carried across intersections in valley gutters)
- d.) Upstream of driveways, where practical
- e.) On side street at intersections where drainage would flow on to the highway pavement.
- f.) Upgrade of bridges to prevent pavement drainage from flowing on to bridge decks
- g.) Downgrade of bridges to intercept drainage from the bridge
- h.) On low side of super elevated curves
- i.) Crossover points in super elevation transitions
- j.) Behind curb, shoulder, or sidewalk to drain low spots.

8.9 CATCH BASIN TYPES

LADOTD uses three types of drainage inlets:

- A.) Open Top – water enters through a grate located in the top of the catch basin.
- B.) Curb-opening – water enters through an opening in the curb face.
- C.) Combination – a combination catch basin has both a curb-opening inlet and a grate opening in the pavement to catch water.

A guide to selection of catch basins is provided in Table 8.9-1 for inlets used by LADOTD. If an inlet of another type is to be used for roadway drainage, LADOTD's Hydraulics Unit should be consulted. See Part A of this chapter for more information on determining the capacity of LADOTD's catch basins.

8.9.1 Outside of Pavement

Open top catch basins are placed outside of the pavement to catch the water draining behind the curb before this flow is transported into the conduit system. These are typically located in driveways, parking lots and yards. The designer is required to specify the type of grate on the plans. The grate options are: Type B when pedestrian traffic is not expected or Type C when pedestrian traffic is expected. These grates are shown on LADOTD’s Standard Plan MC-01.

8.9.2 Traffic, Pedestrians or Bicycles

Curb-opening inlets are effective in the drainage of highway pavements where flow depth at the curb is sufficient for the inlet to perform efficiently. These inlets are relatively free of clogging tendencies and can be a viable alternative to grates in many locations where grates would be in traffic lanes or would be hazardous for pedestrians or bicycles.

Table 8.9-1 Guide to Catch Basin Selection

CATCH BASIN	TYPE	APPLICATION
CB-01*	Open Top	Outside of Roadway
CB-02*	Open Top	Outside of Roadway
CB-04	Open Top	End of Line (15" Maximum Pipe)
CB-05	Open Top	Yard Drain (8" Maximum Pipe)
CB-06	Curb	General use on a grade or at a low point.
CB-07	Combination	Use when greater capacity than CB-06 is required on a grade or at a low point.
CB-08	Combination	Double catch basin – use when greater capacity than CB-07 is required on a grade or at a low point.
CB-09	Combination	Use when the trunk line has to be placed under the roadway due to limited right-of-way.

* Necessary to specify on the plans what type of grate is required:

- Type B where no pedestrian traffic and no vehicular traffic is expected (ditches, etc.)
- Type C where pedestrian traffic and/or light vehicular traffic is expected (driveways, etc.)

8.9.3 Sag Locations

Combination inlets are required in sags or low points. The efficiency of inlets in passing debris is critical in sag locations because all runoff which enters the sag must be passed through the inlet. Total or partial clogging of inlets in these locations can result in hazardous ponding conditions. Therefore, combination inlets consisting of a grate and a curb-opening are considered advisable for use in sags where hazardous ponding can occur.

8.9.4 Standard Catch Basins and Manholes

LADOTD has many Standard Plans for catch basins and manholes. There are various types to fit different situations, e.g., the location of the trunk line in front of or behind the curb, varying sizes and/or number of pipes, and differing depths of structure. These are included in LADOTD's Standard Plan Booklet which is available through the General Files Section.

8.10 HYDRAULIC DESIGN OF CONDUIT SYSTEMS

Pipes are designed for surcharged-full flow conditions and sized to carry 100 percent of the runoff, even though during the inlet spacing phase of the design it is recognized that a portion of the flow bypasses inlets on a grade.

8.10.1 Pipe Size

A minimum diameter, or round equivalent diameter for pipe arches, of 15" should be used for trunk lines and principal laterals.

In progressing downstream, pipe sizes should never decrease.

8.10.2 Pipe Length

Pipe lengths for storm drains are rounded off to the whole foot. The maximum length of pipe without a manhole or other structure with access is given in Table 8.10-1.

8.10.3 Pipe Slope

To minimize excavation, the pipe slope should conform approximately to existing surface grade.

8.10.4 Flow Lines

Flow line elevations should be set so that pipe centerlines in a manhole will be approximately in the same plane.

Table 8.10-1 Maximum Length of Pipe without Access

Pipe Diameter	Maximum Length for Given Velocity		
	3 – 7 ft/s	8 – 12 ft/s	13 – 20 ft/s
15"	150'	250'	300'
18"	300'	350'	400'
24" – 36"	400'	450'	500'
≥ 42"	600'	650'	700'

8.10.5 Pipe Location

The most desirable location of trunk lines is outside the pavement area, to facilitate future repairs. If not possible due to utility conflicts, the next choice would be under the curb, and the last choice would be under the driving lane.

Acute turns should be avoided between inflow line (or lateral) and outflow line.

8.10.6 Outlet Velocity

Pipes or pipe arches should be sized to operate full with a minimum self-cleansing velocity of 3 ft/s with exception of initial pipes in the system.

Initial pipes in the system may be designed with full flow velocities of lower than 3 ft/s. However, in actuality these pipes will flow part-full with velocities higher than their full flow velocities.

Velocities greater than 20 ft/s should be avoided.

8.10.7 Scour Protection

High outlet velocities will require investigation of the need for outfall scour protection. Outlet scour protection will possibly be required for velocities above 10 ft/s and may be required for lesser velocities in highly erodible soils.

8.10.8 Hydraulic Grade Line (HG or HGL)

The Hydraulic Grade Line should be computed and maintained at least 1 ft below the intake lip of any inlet.

8.11 MANHOLE REQUIREMENTS

8.11.1 Conditions Which Require Manholes

Locations which will require a manhole (assuming a catch basin is not appropriate) are:

- a.) Wherever necessary to keep maximum lengths as noted in Table 8.10-1.
- b.) At points of conflict with utility lines which cannot be moved
- c.) At all angles in storm drains
- d.) At points where grade of storm drain changes
- e.) At points where size of storm drain changes
- f.) At junctions of storm drain lines except for the conditions in Section 8.11.2

8.11.2 Conditions Which Do Not Require Manholes

Locations which will not require a manhole at junction of storm drain lines are:

- a.) When grate inlets (outside the pavement) such as CB-01, CB-02, or CB-04 are used, 15" through 24" pipes may be connected to the trunk line by a fabricated pipe fitting (wye or tee) if the trunk size does not exceed 36" and length of the lateral pipe (between the grate inlet and the fabricated pipe fitting) does not exceed 50 ft .
Fabricated pipe fittings will be noted on Plan-Profile sheets and will be measured per each fitting. When the trunk line is greater than 36", a stub-in pipe may be used instead of a fabricated pipe fitting provided it is less than or equal to half the diameter of the trunk line.
- b.) When stubbing into an RCB, the stub-in cannot be greater than half the rise of the RCB. When setting the flowline of the stub-in, the stub-in should clear the haunch area of the RCB.
- c.) When yard drain (CB-05) is used, 8" yard drain pipe may be stubbed-into the trunk line. Such stubbing of conduits will be noted on Plan/Profile sheets and will not be measured for payment.

8.12 CONSTRUCTION CLEARANCE

A minimum clearance of 9" shall be maintained between the top of the pipe and the lowest part of the subgrade. Minimum clearance should be increased to 12" for pipes greater than 84" in diameter.

8.12.1 Underground Utilities

Approximately 1 ft of clearance should be maintained between storm drains and underground utilities.

8.12.2 Sanitary Sewer Conflicts

Where it is necessary for a sanitary sewer line to pass through a manhole, at least 1 ft of clearance should be maintained between the bottom of the sanitary sewer line and the flow line of the manhole. Ductile iron pipe of sufficient length to insure approximately 2 ft of bearing on compacted soil beyond the walls of the manhole should be required in such cases. Figure 8.12-1 may be referred to for guidance in dealing with conflicts between storm drain and sanitary sewer lines.

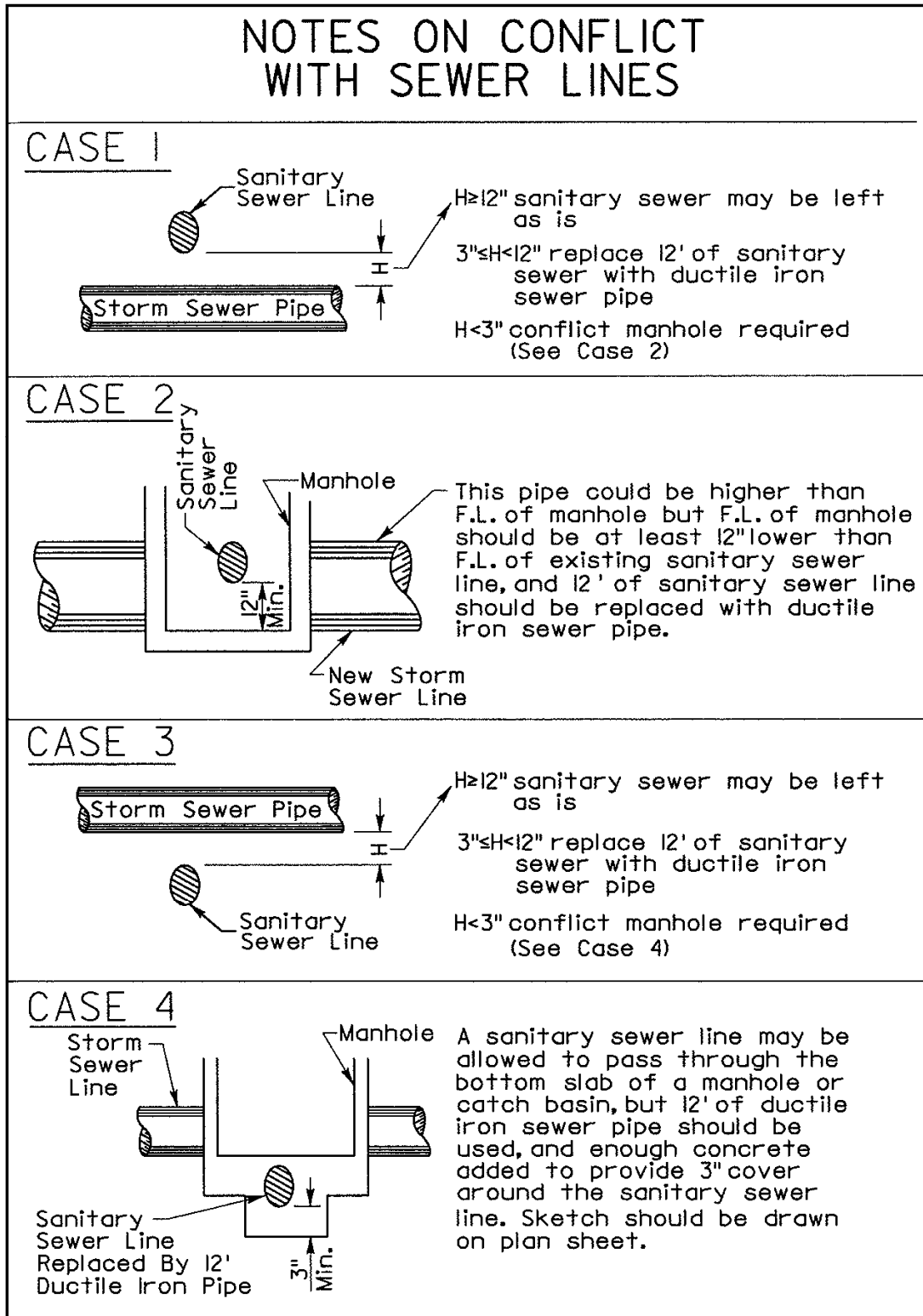


Figure 8.12-1 Dealing with Conflicts between Storm Drain and Sanitary Sewer Lines

CHAPTER 8 – PART A GRATE AND CURB INLETS

8-A.1 INTRODUCTION

The following methodology is to be used in determining the capacity and depth of ponding for grate inlets. For more information, refer to these references or the latest publications/websites from the following agencies:

- A.) Urban Drainage Design Manual Third Edition, Hydraulic Engineering Circular No. 22, FHWA, Washington, D.C., September 2009.
- B.) Drainage of Highway Pavements, Hydraulic Engineering Circular No. 12, FHWA, Washington, D.C., March 1984.
- C.) Pavement and Geometric Design Criteria for Minimizing Hydroplaning, FHWA Report No. FHWA-RD-79-31, FHWA, Washington, D.C., December 1979.

The capacity curves presented in this section were developed from experimental data based on a study conducted by Louisiana State University for the Louisiana Department of Transportation and Development (W. A. Wintz, Jr. and Y. H. Kuo, "A Study of Storm Water Inlet Capacities", 1970).

These graphs are a representation of empirical data which was collected in research of these inlet types only, and they should not be used for any other inlet types.

8-A.2 COMPUTER PROGRAMS

The following programs have been approved for use in the design of curb inlet spacing in storm drain systems for LADOTD highways:

- A.) The LADOTD computer program HYDR6000: "Inlet Spacing" is only good for uniform cross slopes of 0.025 ft./ft. Curb Inlets on any other cross slope including superelevated roadways should be calculated based on the procedures outlined in this section.

There is not a computer program available through LADOTD to calculate capacity or depth of ponding for grate inlets outside the pavement.

8-A.3 HYDROLOGIC ANALYSIS

For curb, gutter and grate inlets operating as weirs, the capacity is calculated using the weir formula. The weir equation is:

$$Q_i = C_w P F d^{3/2} \quad \text{Eq. 8-A.3-1}$$

- Where:
- Q_i = Rate of discharge into the grate opening (ft³/sec)
 - C_w = 3.0
 - P = Perimeter of the grate in feet disregarding bars and the side against the curb (See Figure 8-A.3-1)
 - F = Flow reduction factor for clogging
 - d = Average depth of water at grate or face of curb (ft)

For grate inlets that operate as a weir, the crest length is roughly equal to the outside perimeter (P) along which the flow enters. (See Figure 8-A.3-1) Bars are disregarded in computing P.

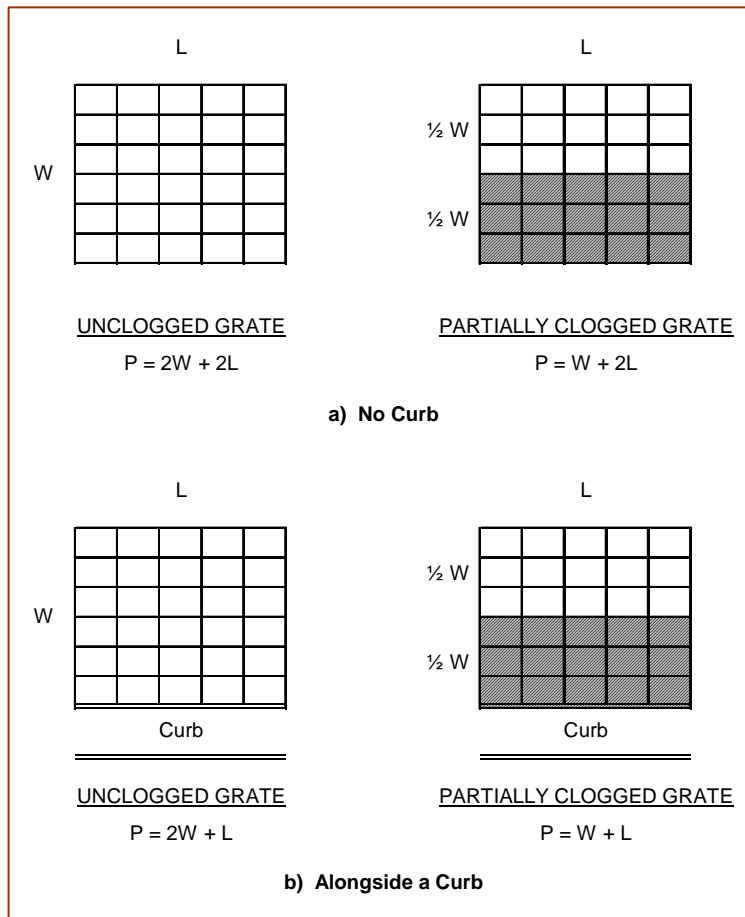


Figure 8-A.3-1 Perimeter (Weir Crest Length) of Grate Inlets

8-A.4 DEPTH OF PONDING FOR GRATE INLETS

The Q intercepted at the inlet will be the runoff generated from the associated drainage basin for the grate plus any bypass to the low point. For depths over the grate of between 0.4 ft and 1.4 ft, the operation of the grate inlet is indefinite due to vortices and other disturbances. The depth of water above the grate is somewhere between that given by Equation 8-A.4-1 (weir conditions) and Equation 8-A.4-2 (orifice conditions).

8-A.4.1 Weir Conditions

Weir operation continues to a depth (d) of about 0.4 ft above the top of the grate. The depth of ponding for a computed runoff (Q) is computed using Equation 8-A.4-1 for a weir.

$$d = \left(\frac{Q}{3.0 PF} \right)^{2/3} \quad \text{Eq. 8-A.4-1}$$

Where: d = Average depth of water at grate (ft)
 Q = Rate of discharge into the grate opening (ft³/sec)
 P = Perimeter of the grate in feet disregarding bars and the side against the curb (See Figure 8-A.3-1)
 F = Flow reduction factor for clogging

8-A.4.2 Orifice Conditions

When the depth at the grate exceeds about 1.4 ft, the grate begins to operate as an orifice and the depth of ponding is determined using Equation 8-A.4-2

$$d = 0.035 \left(\frac{Q}{AF} \right)^2 \quad \text{Eq. 8-A.4-2}$$

Where: d = Depth of water at grate (ft)
 Q = Rate of runoff (ft³/s)
 A = Clear opening of grate (ft²)
 F = Flow reduction factor for clogging (use 0.50)

8-A.4.3 Example Depth of Ponding Calculation

Find the depth of ponding over a median drain that is expected to receive a runoff of 14 ft³/s. The grate is 2 ft by 2.5 ft with a 50 percent clear opening and is located 2.5 ft below the lowest part of the finish grade. The drain pipe is a 18" reinforced concrete pipe.

- a.) Maximum depth of ponding allowed:
 Lowest part of the finish grade minus 1 ft hydraulic clearance
 = 2.5 ft – 1 ft = 1.5 ft

- b.) Find the depth of ponding from Equation 8-A.4-1:

$$Q = 14 \text{ ft}^3/\text{s}$$

$$P = 2 + 2 + 2.5 + 2.5 = 9 \text{ ft}$$

$$F = 0.5$$

$$d = \left(\frac{14}{3(9)(0.5)} \right)^{2/3} = 1.02 \text{ ft}$$

The depth of ponding exceeds 0.4 ft, so use Eq. 8-A.4-2 to calculate the depth.

$$A = (2 \text{ ft})(2.5 \text{ ft}) = 5.0 \text{ ft}^2$$

$$d = 0.035 \left(\frac{14}{(5.0)(0.5)} \right)^2 = 1.10 \text{ ft}$$

- c.) Compute the hydraulic gradient as discussed in Chapter 8, Part B.

$$Q = 14 \text{ ft}^3/\text{s}$$

$$D = 18''$$

$$V = 7.9 \text{ ft/s}$$

$$\frac{V^2}{2g} = 0.97 \text{ ft}$$

$$H_v = \text{entire velocity head to be generated} = V^2/2g = 0.97 \text{ ft}$$

$$H_e = K_e (V^2/2g) = 0.5(0.97) = 0.48 \text{ ft}$$

$$\text{Total loss} = H_v + H_e = 0.97 \text{ ft} + 0.48 \text{ ft} = 1.45 \text{ ft}$$

Assuming that the drain pipe is flowing full, the hydraulic gradient would be 1.45 ft above the crown of the pipe. Assuming that the crown of the drain pipe is 0.5 ft below the top of the grate, a 0.95 ft (1.45' - 0.5') depth of ponding is expected.

- d.) Compare the depths of ponding found in steps b and c. The governing depth of ponding is the one controlled by the grate (between 1.02 ft and 1.10 ft). Since the depth of ponding is less than the allowable (1.5 ft), the design is acceptable.

8-A.5 SPREAD FOR GRATE INLETS OUTSIDE OF PAVEMENT

Using the depth of ponding from Section 8-A.4, the spread can then be determined by Equation 8-A.5-1.

$$T = \frac{d}{S_x} \quad \text{Eq. 8-A.5-1}$$

Where: T = Width of flooding also called the spread (ft)
d = Average depth of water at grate (ft)
S_x = Cross Slope (ft/ft)

If the width of flooding exceeds the allowable value, the inlets on either side of the low point should be moved, if possible. Another solution might be a larger grate at the low point.

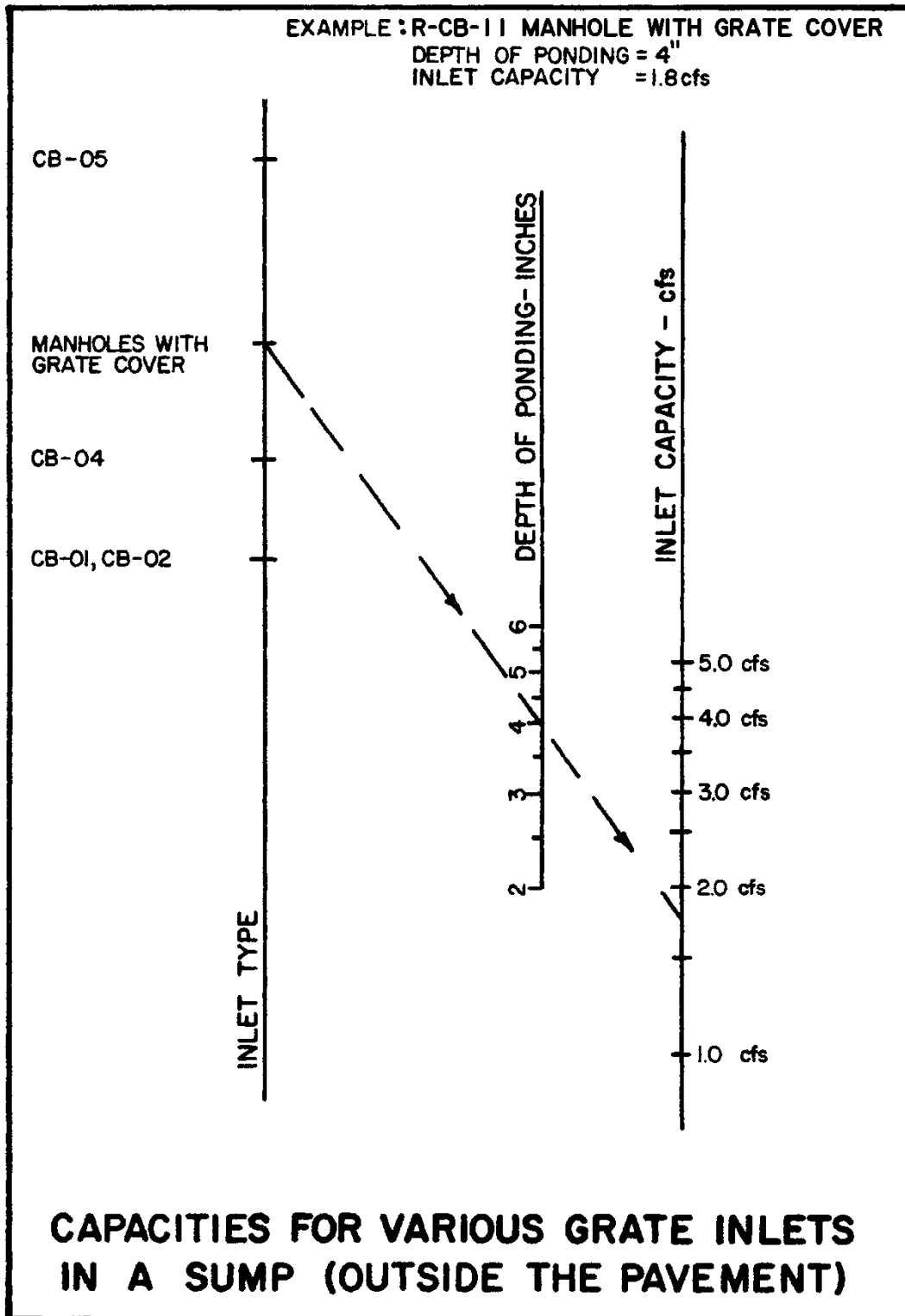


Figure 8-A.6-1 Capacities for LADOTD Grate Inlets Outside the Pavement

8-A.6 INLET CAPACITY – OUTSIDE THE PAVEMENT

LADOTD uses open top grate openings in catch basins and manholes when needing to collect water behind the curb or “outside the pavement” at low spots, sags or sumps. They are:

- A.) CB-01 (grate outside dimensions 2’– 4¼" × 3’– 4"; fits pipes 24"×36")
- B.) CB-02 (grate outside dimensions 2’– 4¼" × 3’– 4"; fits pipes 42"×72")
- C.) CB-04 (grate outside dimensions 2’– 4" × 2’– 4"; end of line, 15" max pipe)
- D.) CB-05 (grate outside dimensions 1’– 1½" × 1’– 1½"; yard drain, 8" max pipe)
- E.) Manhole round grate cover (outside diameter 1’– 11½")

The capacity of these grate inlets depends upon the depth of water at the inlet and the inlet geometry. These inlets most likely function as a weir except in extreme cases of ponding. The capacities for open top grates are given in Figure 8-A.6-1. They were computed from Equation 8-A.3-1 (weir formula) with a flow reduction factor of 0.50 for clogging. Value of P, perimeter of grate opening, in Equation 8-A.3-1 includes all four sides of the grate opening in this case.

Figure 8-A.6-1 may be used to determine the capacity of an inlet by selecting a desirable depth of ponding at the inlet (usually less than or equal to 6 inches) or the depth of ponding at the inlet by knowing the design flow into the inlet. More information on depth of ponding is in Section 8-A.4.

8-A.7 GUTTER CAPACITIES

Capacities of gutters with a uniform cross slope of 0.025 ft./ft. are given in Figure 8-A.7-1.

8-A.7.1 Superelevations

Gutter capacities for other sections with uniform cross slopes or superelevations may be computed from Equation 8-A.7-1 or Equation 8-A.7-2:

$$Q = \left(\frac{0.56}{n} \right) S_X^{5/3} T^{8/3} S^{1/2} \quad \text{Eq. 8-A.7-1}$$

- Where:
- Q = Rate of flow (ft³/s)
 - n = Manning's coefficient of roughness
(use n = 0.015 for concrete or asphalt pavements)
 - S_X = Cross slope (ft/ft)
 - T = Width of flow or Spread (ft)
 - S = Longitudinal slope (ft/ft)

$$Q = 0.56 \frac{z}{n} S^{1/2} d^{8/3} \quad \text{Eq. 8-A.7-2}$$

Where: Q = Rate of flow (ft³/s)
n = Manning's coefficient of roughness
(n = 0.015 for concrete or asphalt pavements)
z = 1/S_x, (S_x = Cross slope (ft/ft))
d = maximum depth of flow at the gutter;
(T = Width of flow or Spread (ft) = zd)
S = Longitudinal slope (ft/ft)

8-A.8 INLET CAPACITY – INSIDE THE PAVEMENT

There are four catch basins used by LADOTD that have either a curb opening or a combination curb and grate opening. The openings are along the curb and for those that have grates, the grates are in the pavement/gutter area. These inlets are considered “inside the pavement” and can be placed either on a grade or in a sag (low point). They are:

- A.) CB-06 (curb opening)
- B.) CB-07 (curb and grate opening)
- C.) CB-08 (double curb and grate opening)
- D.) CB-09 (curb and grate opening – trunk line is under the roadway)

8-A.8.1 Inlets on a Grade

The capacity curves for these four catch basins when they are placed on a grade are given in Figures 8-A.8-1 through 8-A.8-4.

8-A.8.2 Inlets in a Sag

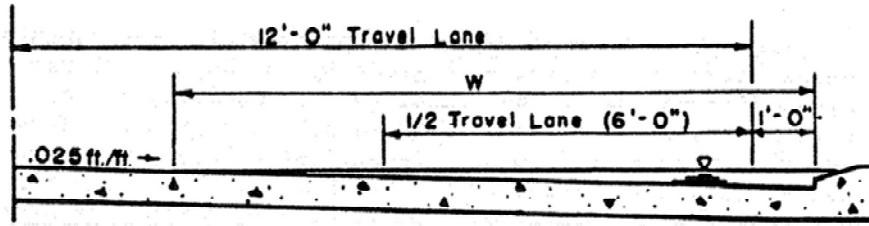
The capacity of curb-opening, gutter, or combination inlets used in a sag, or a low point in grade, depends upon the depth of water at the inlet and the inlet geometry. The inlets operate as a weir until the water submerges the entrance. Capacity curves for the inlet types used by LADOTD are given in Figure 8-A.8-5. They were developed from the weir formula, (see Section 8-A.3). A flow reduction factor of 1.00 was used.

8-A.8.3 Gutter Flow and Interception Ratio

When the appropriate, interception ratio chart (Figure 8-A.8-1, 8-A.8-2, 8-A.8-3, 8-A.8-4) is aligned with the right-hand side of the appropriate gutter flow capacity chart (Figure 8-A.7-1), the gutter flow and interception ratio for a given grade may be determined simultaneously.

Begin by locating the desired longitudinal roadway grade on the horizontal axis of the left-hand chart, and from that point draw a vertical line which will intersect with the curve representing the desired width of flooding. Carry a horizontal line from this point until the vertical axis (ordinate) is intersected, indicating the flow capacity of the gutter under the specified limits of flooding. Extend the horizontal line into the right-hand chart until the curve representing the appropriate longitudinal grade is intersected, and from that point draw a vertical line which intersects the horizontal axis (abscissa) of the right-hand chart, indicating the fraction of the total gutter flow which will be intercepted by the inlet.

FLOW CAPACITY OF STREET & GUTTER



INTEGRAL CURB SECTION WITH 1'-0" ADDITIONAL PAVEMENT

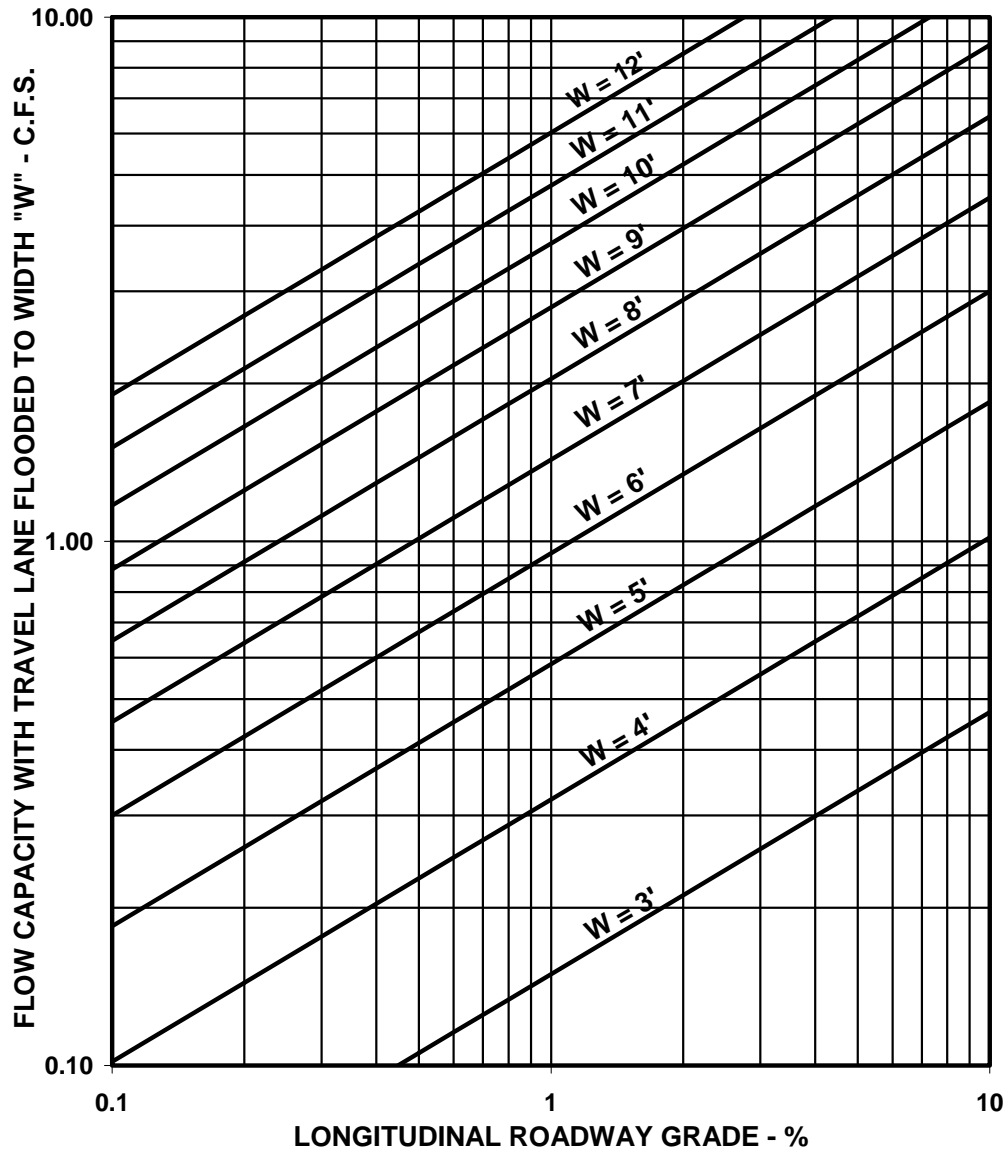


Figure 8-A.7-1 Gutter Capacities For Uniform Cross Slope of 0.025 ft/ft

INTERCEPTION RATIO vs. Q AT VARIOUS GRADES

Q = GUTTER FLOW Q_i = FLOW INTO INLET

CB-06

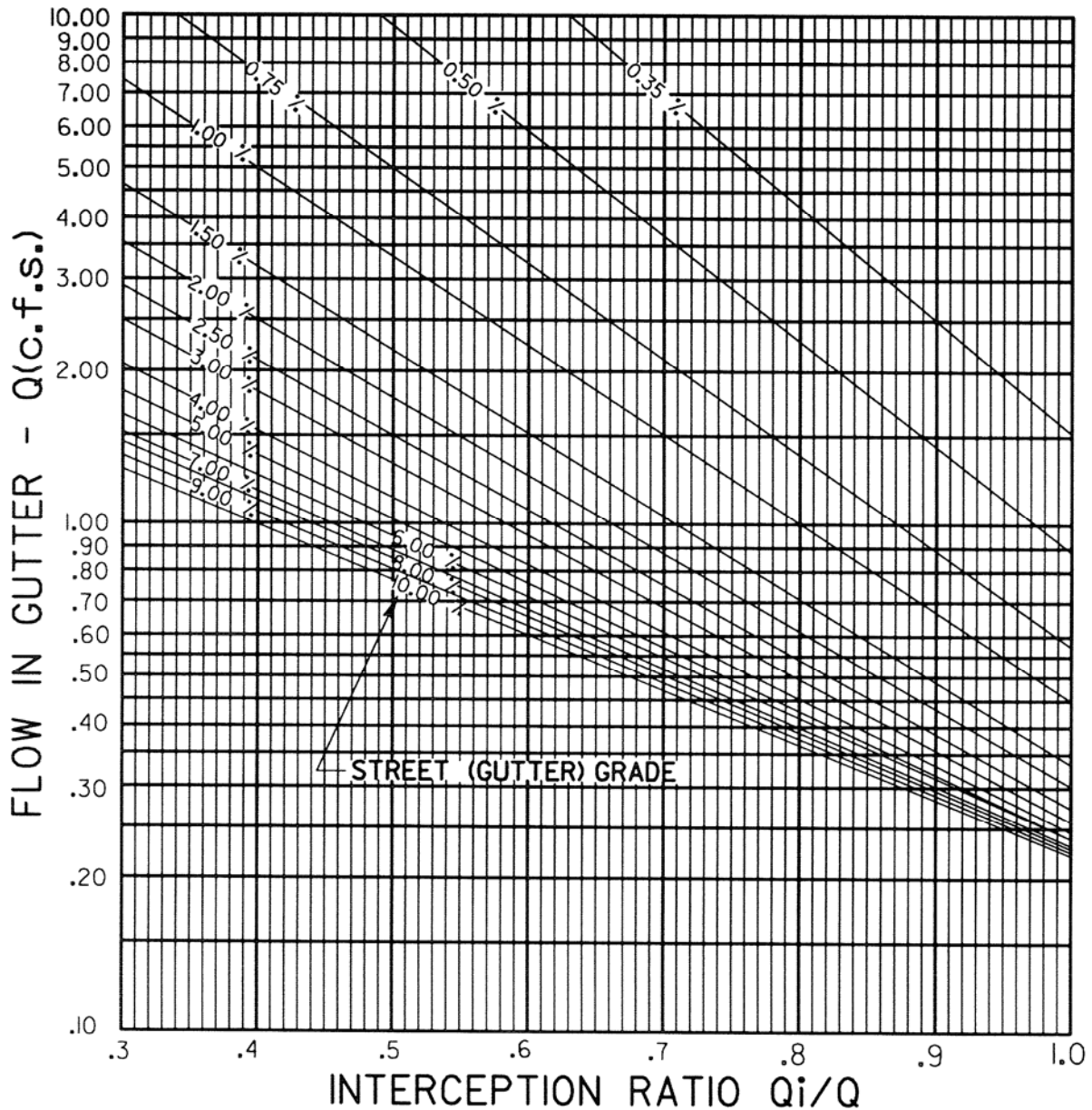


Figure 8-A.8-1 CB-06 Inlet Capacity Curve on a Grade (Inside of Pavement)

INTERCEPTION RATIO vs. Q AT VARIOUS GRADES

Q = GUTTER FLOW Q_i = FLOW INTO INLET

CB-07

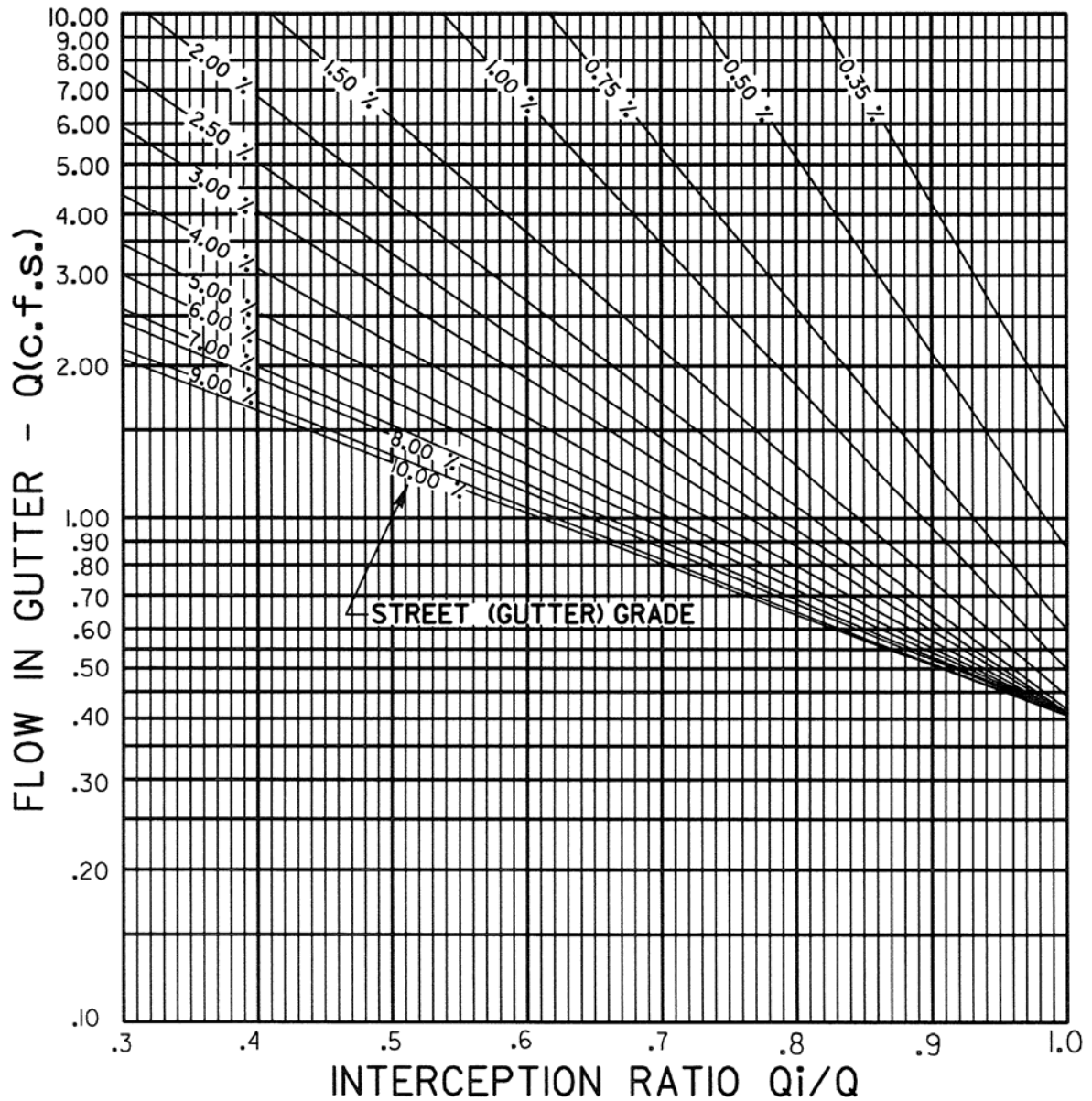


Figure 8-A.8-2 CB-07 Inlet Capacity Curve on a Grade (Inside of Pavement)

INTERCEPTION RATIO vs. Q AT VARIOUS GRADES

Q = GUTTER FLOW Qi = FLOW INTO INLET

CB-08

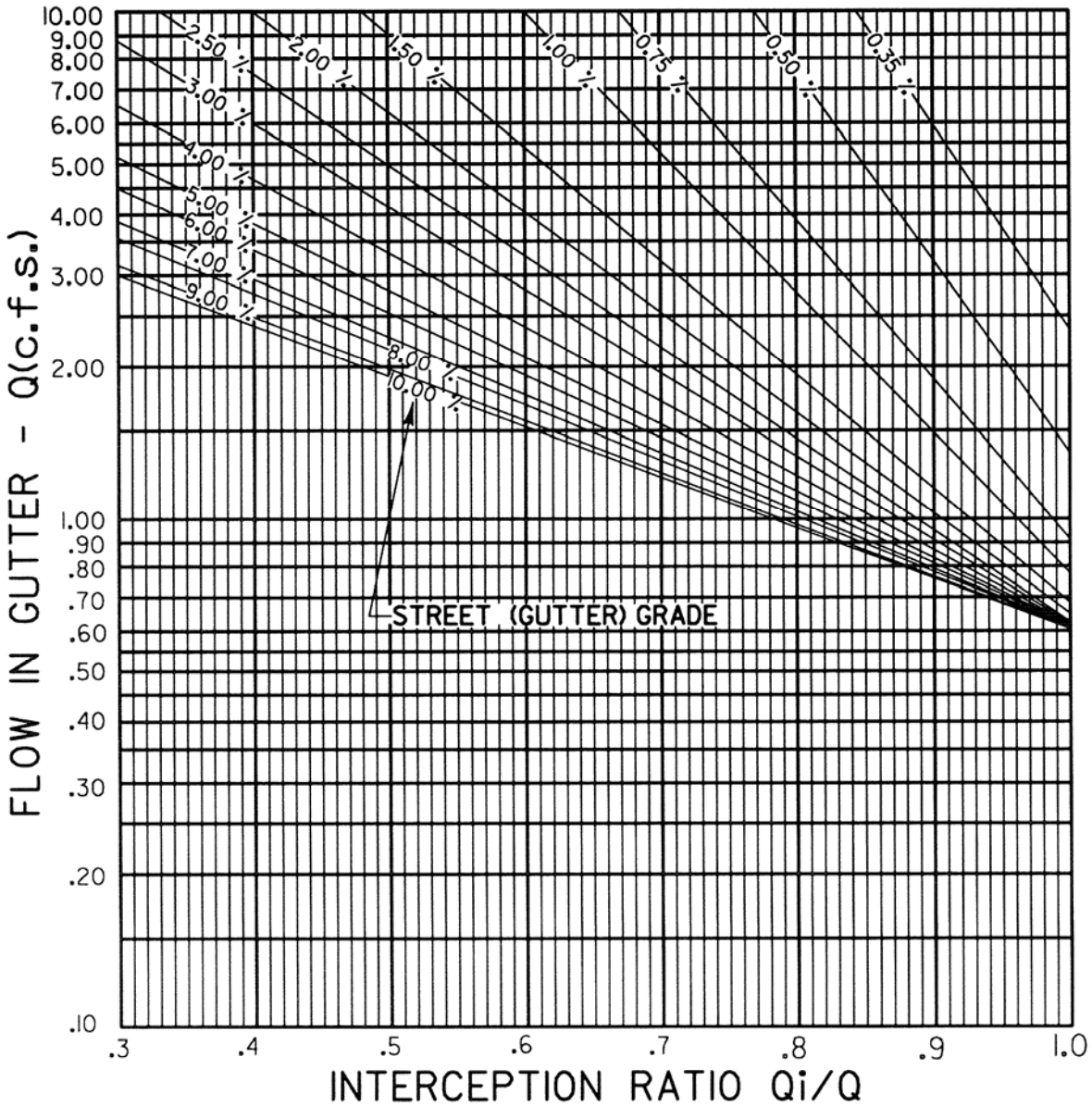


Figure 8-A.8-3 CB-08 Inlet Capacity Curve on a Grade (Inside of Pavement)

INTERCEPTION RATIO vs. Q AT VARIOUS GRADES

Q = GUTTER FLOW Qi = FLOW INTO INLET

CB-09

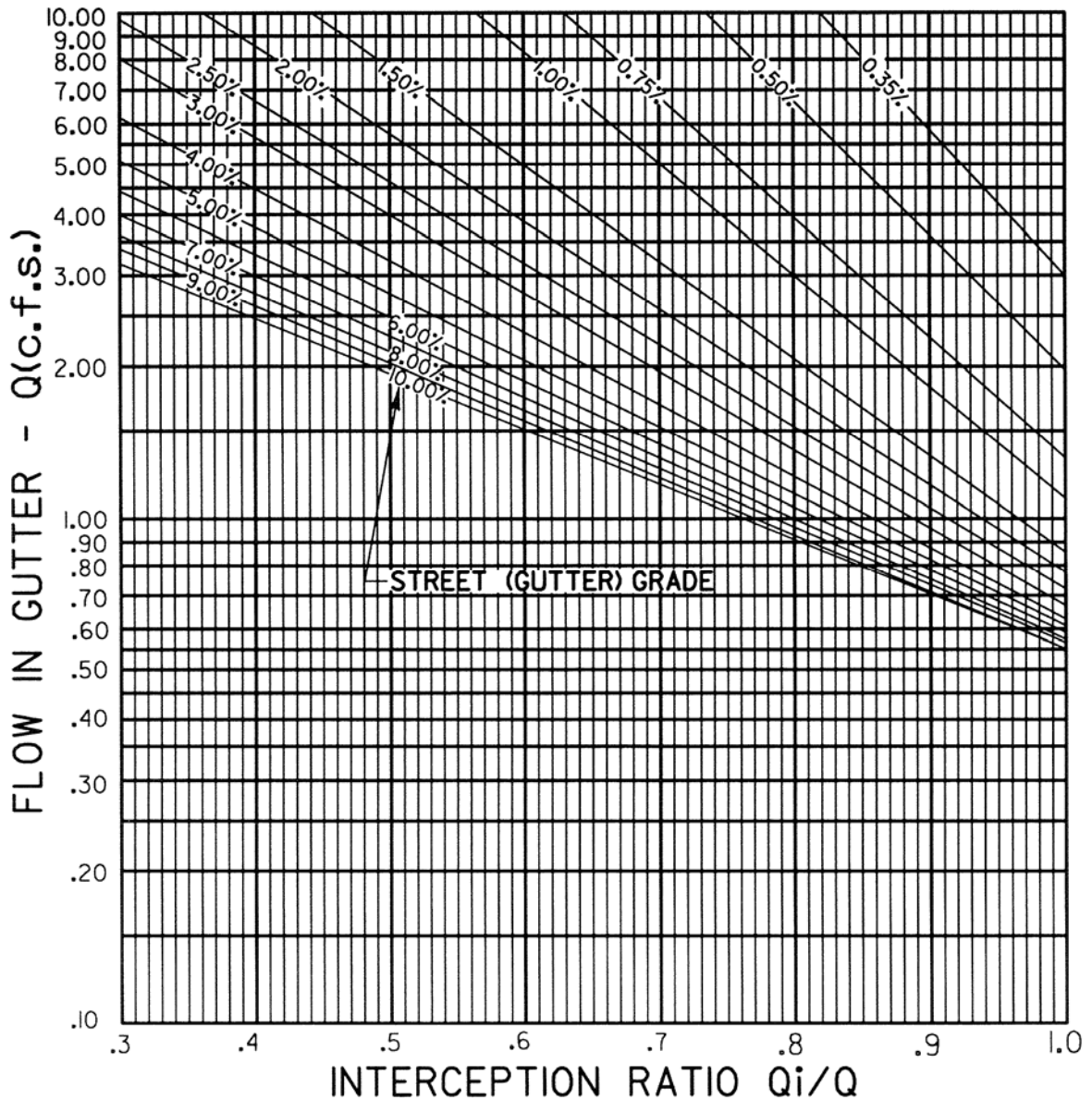


Figure 8-A.8-4 CB-09 Inlet Capacity Curve on a Grade (Inside of Pavement)

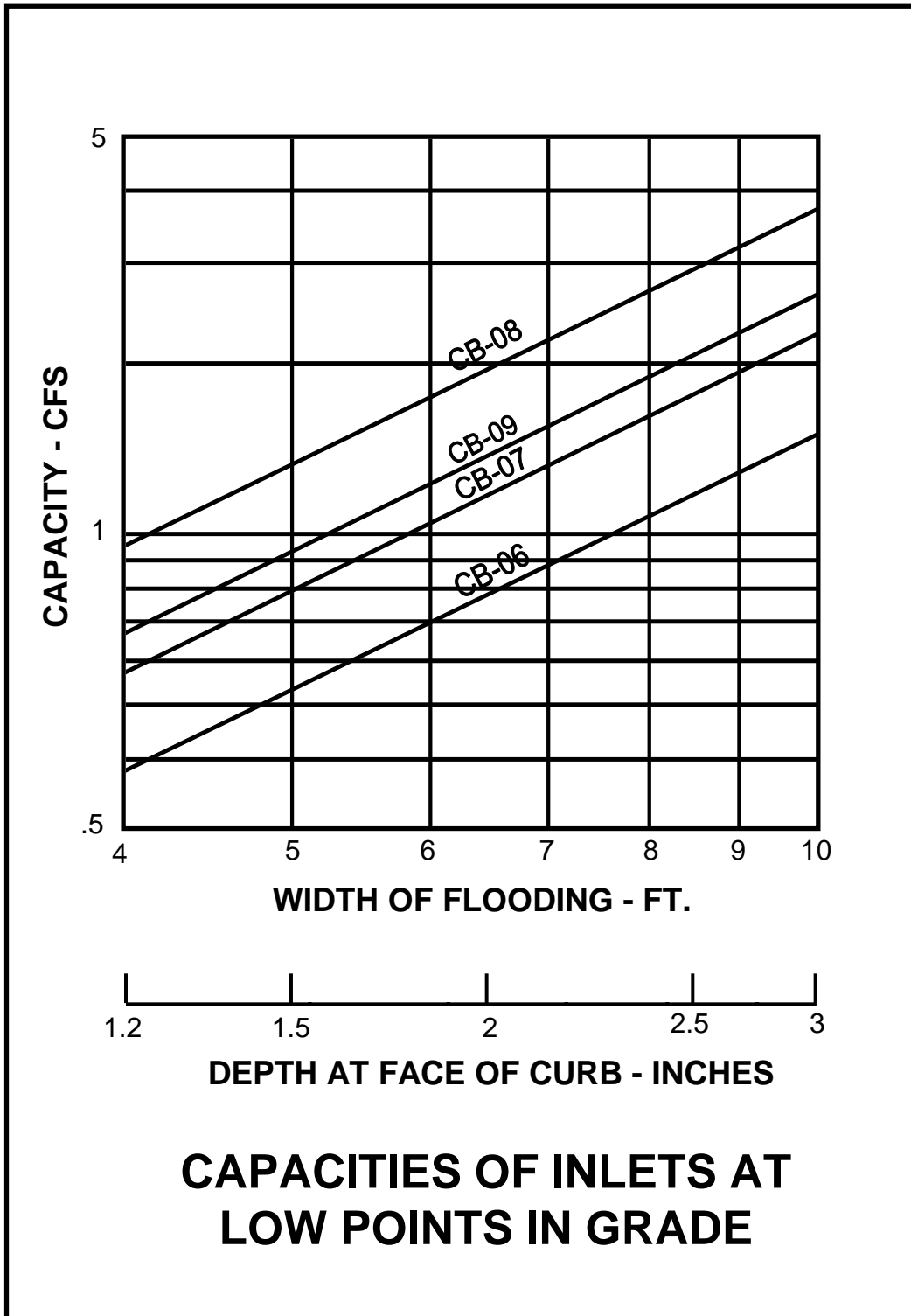


Figure 8-A.8-5 Capacities of LADOTD Catch Basins at Low Points in Grade

8-A.9 INLET SELECTION AND PLACEMENT

To space inlets on a grade, the capacity of the gutter section is determined first. The capacity of the gutter section for a desired flood width is determined using Figure 8-A.7-1 or Equation 8-A.7-1 where applicable. The capacity of the gutter under desirable limits of flooding will vary as the longitudinal slope varies. It is beneficial to compute the conveyance of the section and plot a graph of capacity vs. slope to save work.

Once the amount of runoff necessary to bring the gutter to capacity flow is calculated, the next step is to determine the width (width measured parallel to the roadway) of the drainage area which will have a peak discharge equal to that flow. This is a trial and error procedure. A width of drainage area is assumed, and the runoff from the drainage area so defined is computed according to Chapter 3 – Rational Method. If the runoff computed from this drainage area is equal to or slightly less than the desired amount, then the location of the inlet is set. If, however, the computed runoff is greater, then the width of the drainage area must be reduced by moving the location of the inlet.

The idea is to know how much runoff can be allowed to accumulate in the gutter before placing an inlet to intercept it. On very flat grades or for the first inlet on a grade, this value is equal to the gutter capacity. However, for inlets on a moderate grade, a certain amount of water may be expected to by-pass any given inlet and continue on in the gutter. A design should allow for this by-pass flow which may range from negligible on grades less than 0.5% to as much as forty to sixty percent of total gutter flow for grades of 3.0% or greater. Considering this fact, the permissible accumulation becomes gutter capacity minus residual gutter flow (or by-pass) from the last inlet upstream.

Figures 8-A.8-1 through 8-A.8-4 provide a method of predicting the by-pass flow at various grades for the four inlet types which are most commonly used by LADOTD. These graphs are a representation of empirical data which were collected in research of these inlet types only, and they should not be used for any other inlet types.

The following design procedure is provided for spacing and selection of catch basins. A design example with solution is also illustrated.

8-A.9.1 Design Procedure for Placement and Selection of Inlets

Inlets (catch basins) are used to collect pavement runoff, any rainfall falling in the right-of-way and any overland flow from adjacent properties. The following are the steps that should be taken to determine placement of inlets for a curb and gutter drainage system. At the end of Part A of this chapter is a spread sheet to be used as an aid. (HYDR 6000 calculates most of the information required on this spread sheet, however, it is best to understand the process so as to make good engineering decisions on inlet locations.)

- a.) Roadway Grade – Select a roadway grade and roadway width.
(See Chapter 1.)

- b.) Gutter Profile – Plot the gutter profile and the ground elevation at or near the right-of-way on the plan/profile sheets. Find all low point and high point stations and elevations.
- c.) Allowable Width of Flooding – Determine the allowable width of flooding. (See Section 8.8)
- d.) Catch Basin Location – The first three columns give the location of the catch basin on the plan/profile sheets.

Column 1: Sheet No. of the Plan / Profile Sheet.

Column 2: Structure (inlet) no.

Column 3: Station

- e.) Drainage Area – A width of drainage area is assumed (This is where the inlet will be located. See Section 8.8 for guidelines and restrictions of inlet spacing.)

Columns 4 through 8 are for information of the behind the curb or the paved roadway drainage basin, whichever is greater. (In HYDR 6000, if the drainage length, slope and runoff coefficient are input as 0, it is assumed that there is only roadway drainage and no over the curb drainage.)

Column 4: Drainage Length, L in ft; It is the length of the drainage basin/area perpendicular (\perp) to the roadway. (This is the value input into HYDR 6000. HYDR 6000 computes the hydraulic length based on this value and the width of the drainage area as determined by the stations of the inlets.)

Column 5: Hydraulic Length in feet (the longest length for the water to travel, usually the diagonal length on rectangular shaped drainage basins.)

Column 6: Slope in %, of the drainage basin

Column 7: Area in acres

Column 8: C, runoff coefficient

Column 9: AC, area (Column 7) multiplied by the runoff coefficient (Column 8)

- f.) Drainage area of roadway which goes into the inlet.

Column 10: Drainage Area of Roadway in acres

Column 11: AC for Drainage Area of Roadway,
Area (Column 10) multiplied by C of 0.95 for
pavement.

- g.) Time of Concentration – The time to inlet equals the time of concentration for the drainage basin input into Columns 5 – 8 for behind the curb area or the paved roadway area, whichever is greater.

Column 12: Time to Inlet (TC) in minutes, calculated using Chapter 3 – Rational Method.

- h.) Rainfall Intensity – Find the rainfall intensity for the drainage basin.

Column 13: Intensity (I) with units of in./hour, from the appropriate Rainfall Intensity Curve found in Chapter 3, Section 3.4.3, using the time of concentration from Column 12 converted from minutes to hours. (This is called time of duration.)

- i.) Compute the total AC.

Column 14: Σ AC, add the AC for the area behind the curb and the AC for the paved roadway area.

- j.) Discharge – The discharge is computed by the Rational Method (See Chapter 3)

Column 15: Q in cfs, multiply rainfall intensity (Column 13) by Σ AC (Column 14).

- k.) Gutter Grade information.

Column 16: Gutter Grade in %, enter the actual gutter grade at the location of the catch basin (inlet).

- l.) Catch Basin Type – From Table 8.9-1, select the appropriate catch basin.

Column 22: Low Point? If it is, note that in this column.

Column 23: Required Catch Basin name.

- m.) Determine the interception ratio for the required catch basin.

Column 17: Interception Ratio, Q_i/Q , from Figures 8-A.8-1 through 8-A.8-4.

- n.) Determine the bypass information

Column 18: Bypass Q in cfs; multiply the interception ratio, Q_i/Q , (Column 17) by Q (Column 15) and subtract from Q (Column 15).

Column 19: To Inlet No., the catch basin that the bypass Q goes to

- o.) Total Discharge – The total discharge going into the inlet is computed. This discharge equals the computed Q plus the bypass Q.

Column 20: $Q + Q_{\text{Bypass}}$ in cfs, Add the computed Q (Column 15) plus the bypass Q (Column 18).

- p.) Width of Flooding – The actual width of flooding is determined.

Column 21: Non Low Points – Width of Flooding in ft, is estimated from Figure 8-A.7-1.

OR

Column 21: Low Points – Width of Flooding in ft, is estimated from Figure 8-A.8-5.

Note: ***If the runoff and width of flooding is within acceptable limits, then the required catch basin is set. If not, a larger capacity catch basin or even multiple catch basins may be required or the location of the inlet/catch basin may need to be adjusted.***

- q.) Catch basin profile information.

Column 24: Gutter Elevation

Column 25: Catch Basin Invert Elevation

- r.) Remarks

Column 26: It is reserved for any special note needed for the inlet.

Repeat the process for every inlet required on the job.

8-A.9.2 Example of Inlet Selection and Placement

The following is an example of determining the spacing and required catch basins for a storm drain (curb and gutter) system.

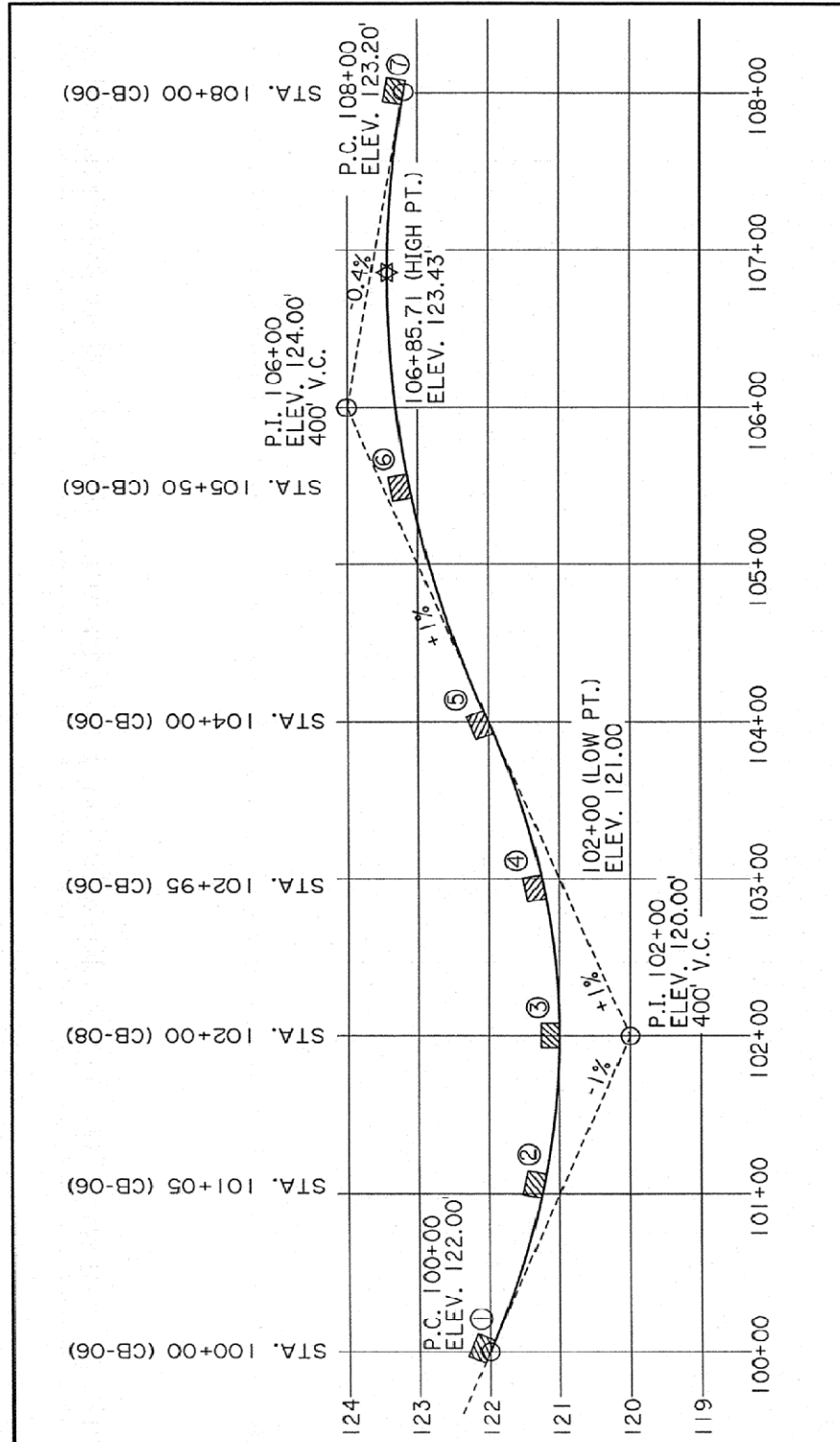
A curb and gutter section is required between Stations 100+00 and 108+00. Travel lanes are to be 14 feet and the cross slope is 0.025 ft./ft. Determine how many catch basins are needed and the size and location of these catch basins.

- A roadway grade was selected.
A plot of the profile is shown in Figure 8-A.9-1(a).

Beginning grade: – 1%
P.I. Station 102+00
P.I. Elev. 120.00 ft.
400 ft. Vertical Curve

Ending grade: – 0.4%
P.I. Station 106+00
P.I. Elev. 124.00 ft.
400 ft. Vertical Curve

INLET SPACING AND SELECTION EXAMPLE



PROFILE

Figure 8-A.9-1(a) Profile of the Roadway for Example 8-A.9.2

- Inlet #1:
 - Distance from beginning of project to first catch basin:
100+00 – 99+00 = 100 ft.
 - Length of Drainage Basin = 100 ft.
 - Hydraulic Length: $\sqrt{(100)^2 + (100)^2} = 141.42$ ft.
 - $T_{Coverland} = 0.7039 \left(L^{0.3917} \right) \left(C^{-1.1309} \right) \left(S^{-0.1985} \right)$
 $= 0.7039 \left(141.42^{0.3917} \right) \left(0.5^{-1.1309} \right) \left(0.3^{-0.1985} \right)$
 $= 13.62$ minutes
 - $T_{pavement} = 0.7039 \left(L^{0.3917} \right) \left(C^{-1.1309} \right) \left(S^{-0.1985} \right)$
 $= 0.7039 \left(100^{0.3917} \right) \left(0.95^{-1.1309} \right) \left(1.0^{-0.1985} \right)$
 $= 4.53$ minutes

Therefore, 13.62 > 4.53, use 13.62

- $I = a (D + b)^c$
 $= (4.016) \left[\frac{13.62}{60} + 0.347 \right]^{-0.826}$
 $= 6.4$ in./hour
- $\Sigma AC = [(0.23)(0.51)] + [(0.08)(0.951)] = 0.19$
- $Q = (I)(\Sigma AC) = (6.4)(0.19) = 1.22$ cfs
- Interception Ratio:

For a CB-06, Figure 8-A.8-1, $Q = 1.214$, $G = 1\%$:

$$\frac{Q_i}{Q} = 0.75$$

- $Q_{\text{bypass to Inlet \#2}} =$

$$Q_{\text{bypass}} = 1.22 - (0.75)(1.22) = 0.31 \text{ cfs}$$

Width of flooding at Inlet #1 (Eq. 8-A-7-1):

$$Q = \left(\frac{0.56}{n} \right) S_X^{5/3} T^{8/3} S^{1/2}$$

$$1.22 = \left(\frac{0.56}{0.015} \right) (0.025)^{5/3} T^{8/3} (0.01)^{1/2}$$

Solve for T: $T = 6.6 \text{ ft.}$

- The width of flooding is below the maximum allowable, therefore a CB-06 catch basin is sufficient.

Repeat the procedure for the remaining inlets. (See Figures 8-A.9-1(b) & 8-A.9-1(c))

INLET SPACING AND SELECTION EXAMPLE

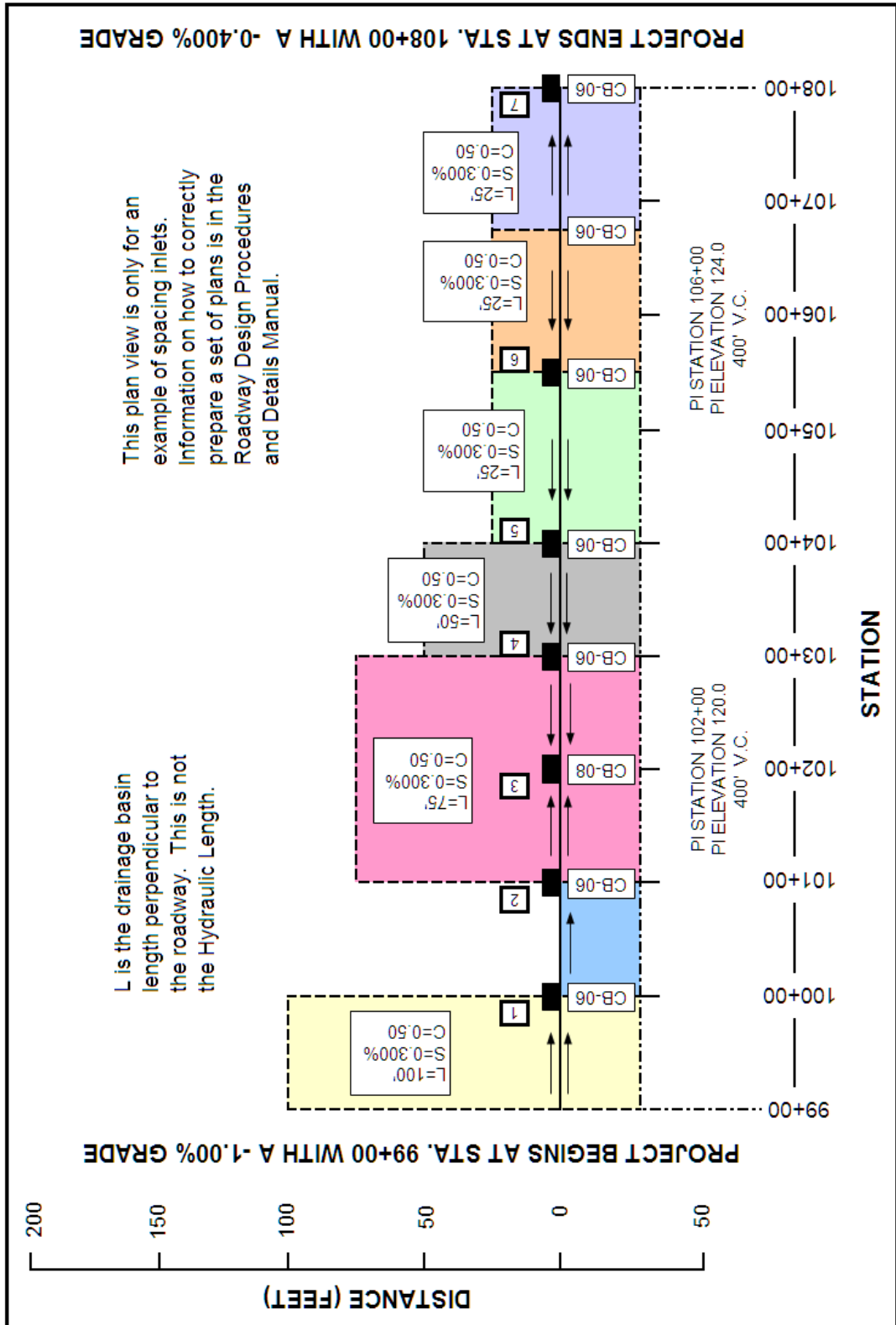


Figure 8-A.9-1(b) Plan view of Inlet Spacing for Example 8-A.9.2

INLET SPACING AND SELECTION - RATIONAL METHOD

1) ROADWAY WIDTH (HIGH POINT TO GUTTER)= <u>35 ft</u> 3) $TC=0.7039(L^{0.3917})(C^{-1.1309})(S^{-0.1985})$	2) REGION I, II, or III = <u>I</u> $a = 4.016$ 4) $I = a(D+b)^c$ $b = 0.347$ $c = -0.826$	DATE: <u>5/28/10</u> PROJECT NO.: _____ COMPUTED BY: _____ DESIGN STORM: <u>10</u> YEAR
---	---	--

SHEET NO.	LOCATION		DRAINAGE AREA					DRAINAGE AREA OF ROADWAY (acre)	AC FOR DRAINAGE ROADWAY	TIME TO INLET (min.)	I (in./hr.)	Σ (AC)	$Q = I \Sigma$ (AC) (cfs)	GUTTER GRADE (%)	INTERCEPTION RATIO Q_i/Q	BYPASS		TOTAL Q (Q + Q _{BYPASS}) (cfs)	WIDTH OF FLOODING (ft.)	LOW POINT YES OR NO	REQUIRED CATCH BASIN	PROFILE		REMARKS	
	INLET NO.	STATION	D.A. LENGTH (ft.) (L)	HYDRAULIC LEN. (ft.) (HL)	SLOPE (%) (S)	AREA (acre) (A)	C									AC	Q (cfs)					TO INLET NO.	GUTTER ELEV.		CATCH BASIN INVERT ELEV.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26
100	1		100+00	100	0.3	0.23	0.5	0.12	0.080	0.08	13.6	6.4	0.19	1.22	1.00	0.75	0.31	2	1.22	6.6		CB-06			
100	2		101+05	0	0.5	0.08	0.95	0.08	0.084	0.08	5.4	8.0	0.08	0.64	0.50	1.00	0.00	--	0.95	6.9		CB-06			
100	3		102+00	75	0.3	0.33	0.50	0.17	0.153	0.15	12.8	6.5	0.31	2.00	--	1.00	0.00	--	2.00	6.6	YES	CB-08			
100	4		102+95	50	0.3	0.12	0.50	0.06	0.084	0.08	12.6	6.5	0.14	0.91	0.50	1.00	0.00	--	1.11	7.2		CB-06			
100	5		104+00	25	0.3	0.09	0.50	0.05	0.120	0.11	14.0	6.3	0.16	1.00	1.00	0.80	0.20	4	1.00	6.0		CB-06			
100	6		105+50	25	0.3	0.08	0.50	0.04	0.109	0.10	13.4	6.4	0.14	0.90	0.50	1.00	0.00	--	0.90	6.8		CB-06			
100	7		108+00	25	0.3	0.07	0.50	0.04	0.090	0.08	12.6	6.5	0.12	0.78	0.40	1.00	0.00	--	0.78	6.6		CB-06			

Figure 8-A.9-1(c) Summary Spreadsheet of Inlet Spacing for Example 8-A.9.2

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CHAPTER 8 – PART B CONDUIT SYSTEMS HYDRAULICS

8-B.1 INTRODUCTION

The following methodology is to be used in determining the capacity and size of the pipes required in a storm drain system. For more information, refer to these references or the latest publications from the following agencies:

- A.) Urban Drainage Design Manual, Hydraulic Engineering Circular No. 22, FHWA, Washington, D.C., August 2001.
- B.) Drainage of Highway Pavements, Hydraulic Engineering Circular No. 12, FHWA, Washington, D.C., March 1984.
- C.) Design and Construction of Sanitary and Storm Sewers, ASCE Manual of Practice No. 37 (WPCF Manual of Practice No. 9), New York, 1976.
- D.) Design of Urban Highway Drainage – The State of the Art, U.S. Department of Transportation, FHWA, Washington, D.C., August 1979.
- E.) Elwyn E. Seelye, Data Book for Civil Engineers, Volume 1 – Design, Second Edition, New York: Johan Wiley and Sons Inc., 1951.
- F.) Concrete Pipe Design Manual, American Concrete Pipe Association, First Edition, 1970.

8-B.2 COMPUTER PROGRAMS

The following programs have been approved for use in the design of storm drain systems for LADOTD highways:

- A.) The LADOTD computer program HYDR6020: “Storm Drain Pipes”.

8-B.3 HYDROLOGIC ANALYSIS

Storm drains are designed to operate under surcharged (pressure) full flow conditions whenever applicable although some parts of the storm drain system may flow part-full even at design discharges.

Most methods of design or analysis of storm drains involve the computation of either the water surface known as the hydraulic grade line (HGL) or energy grade line (EGL). The only advantage one method might have over the other depends upon which elevation is most useful in design and checking.

8-B.3.1 Energy Grade Line

The energy grade line (EGL) is a line showing the total available energy in the system (potential energy plus kinetic energy) and is one velocity head above the hydraulic grade line.

8-B.3.2 Velocity Head

Velocity head is a quantity proportional to the kinetic energy of flowing water expressed as a height or head of water. One velocity head is obtained from the equation:

$$H_v = V^2/2g \qquad \text{Eq. 8-B.3-1}$$

8-B.3.3 Hydraulic Grade Line

The hydraulic grade line (HGL) is the water surface elevation along an open channel. In closed conduit systems flowing under pressure, the HGL is the level to which water would rise in a vertical tube at any point along the pipe. Under this "surcharged" condition, the hydraulic grade line may rise in the drain high enough that water could flow out of manholes and inlets at low points in the highway grade.

Inundation of the highway can be prevented by increasing the size of storm drain pipes downstream from those areas where the elevation of the hydraulic grade line (HGL) exceeds the elevation of the low manholes and inlets. Increasing the size of the pipe reduces the friction loss in the line as well as other head losses associated with flow velocity.

8-B.4 ENERGY LOSSES

Hydraulic grade line computations must account for all energy losses in the storm drain system. Energy losses are functions of flow velocities in drain pipes. Following is a list of the various types of energy losses and the calculations required for their determination.

8-B.4.1 Head Losses/Junction Losses

There are various methods of computing energy losses at the junction of several entering flows. However, when compared to other complex methods, the following guideline seems to provide adequate measure of energy losses at flow junctions:

- a.) one velocity head of the outgoing line for several entering flows
- b.) half a velocity head of the outgoing line for a single entering flow

Use of other values of head loss should be upon the approval of the Hydraulics Design Unit. Manhole losses can be determined by the following equation:

$$H_j = K_j \left(\frac{V^2}{2g} \right) \quad \text{Eq. 8-B.4-1}$$

Where: H_j = head loss at flow junctions, ft
 K_j = head loss coefficient:
= 0.5 for single entering flows with entering and exiting pipes of equal size
= 1.0 for all other cases
 V = average velocity of the outgoing pipe, ft/s
 g = acceleration due to gravity, 32.2 ft/s²

8-B.4.2 Head Loss in Bends and Transitions

Due to the limited use of bends and transitions in storm drain design, head losses for these structures will not be discussed in this section. For details on head loss computations for bends and transitions, refer to FHWA publication "Urban Drainage Design Manual, HEC 22."

8-B.4.3 Entrance Losses

Entrance losses need to be considered in the storm drain design only at those locations where the system originates at a culvert or an inlet. It is calculated using Equation 8-B.4-2. An Entrance Loss Coefficient of 0.5 may be used for simplification regardless of the type of inlet or the culvert entrance configuration.

$$H_e = K_e \left(\frac{V^2}{2g} \right) \quad \text{Eq. 8-B.4-2}$$

Where: H_e = entrance loss, ft
 K_e = entrance loss coefficient
= 0.5
 V = average velocity in inlet pipe, ft/s
 g = acceleration due to gravity, 32.2 ft/s²

8-B.4.4 Exit Losses

A submerged storm drain outfall pipe discharging into a receiving stream will produce an energy loss at its outlet equivalent to one velocity head. Therefore exit loss for storm drains can be computed by the following equation:

$$H_o = 1.0 \left(\frac{V^2}{2g} \right) \quad \text{Eq. 8-B.4-3}$$

Where: H_o = exit loss, ft
 V = average exit velocity, ft/s
 g = acceleration due to gravity, 32.2ft/s²

8-B.4.5 Friction Losses

Friction losses are the most significant energy losses in storm drain systems. Friction loss is the head required to maintain the required flow in a straight alignment against frictional resistance because of pipe roughness. It is determined by the equation:

$$H_f = L \times S \quad \text{Eq. 8-B.4-4}$$

Where: H_f = head loss, ft
 L = length of pipe, ft
 S = hydraulic slope required,
determined by Equation 8-B.4-5:

$$S = \left\{ \frac{V n}{1.486 R^{2/3}} \right\}^2 \quad \text{Eq. 8-B.4-5}$$

Where: S = hydraulic slope required
(not necessarily equal to construction slope
except under certain conditions)
 V = average velocity of flow, ft/s
(flow rate divided by area of flow)
 n = Manning's roughness coefficient
(See Chapter 6, Table 6-A.4-1)
 R = hydraulic radius (area of flow divided by
wetted perimeter)

8-B.5 HYDRAULIC GRADIENT COMPUTATIONS

Storm drain system hydraulic gradient computations must always start at the outlet for the system and proceed upstream. After calculating the friction head in pipe lines and the head loss in manholes and inlets, it is possible to calculate the elevations of the hydraulic gradient from the outlet to the beginning of the storm drain system.

8-B.5.1 Stage Elevation

The stage elevation used in design is usually the stage for the given design storm at the outfall pipe. When the discharge is into a pond, a perennial stream, or a storm water pumping station, the stage for the frequency of the system design should be used as starting grade. Rarely are such data available for ponds or perennial streams. In such instances, the designer usually evaluates the outlet conditions hydraulically. This may require some field information such as alignment, cross sections, and elevations for some practical length downstream. Other hydraulic structures such as culverts, small bridges or other constrictions should also be considered.

8-B.5.2 HGL for Various Pipe Locations

Equations 8-B.5-1, 8-B.5-2 and 8-B.5-3 are used to compute the hydraulic grade line (HGL) at various locations.

- a.) HGL for the lower end of the pipe when it is an outfall

$$HG_{LO} \text{ (for the outfall pipe)} = \text{stage} + \text{exit} \quad \text{Eq. 8-B.5-1}$$

- b.) HGL for the lower end of the pipe when it is not an outfall

$$\begin{aligned} HG_{LO} \text{ (for pipes other than the outfall pipe)} = \\ HG_{UP} \text{ (next pipe)} + \text{manhole loss (next pipe)} \end{aligned} \quad \text{Eq. 8-B.5-2}$$

- c.) HGL for the upper end of the pipe

$$HG_{UP} = HG_{LO} + \text{friction head} \quad \text{Eq. 8-B.5-3}$$

Where: HG_{LO} = hydraulic grade line elevation of lower end of pipe
 HG_{UP} = hydraulic grade line elevation of upper end of pipe
Stage = outlet stage at return period
Exit = exit losses:
 exit losses are 0 if pipe is not submerged,
 exit losses are one velocity head if pipe is submerged

8-B.5.3 HGL for Various Pipe Water Level Conditions

Based on the relationship between the HG_{LO} and the elevation of the lower end of the pipe, there are four possible cases. These are presented in Table 8-B.5-1. For all four scenarios, the variable definitions are as follows:

Where: HG_{LO} = hydraulic grade line elevation of lower end of pipe
 HG_{UP} = hydraulic grade line elevation of upper end of pipe
Stage = outlet stage at return period
Exit = exit losses:
 exit losses are 0 if pipe is not submerged,
 exit losses are one velocity head if pipe is submerged
 Q = actual discharge of the drainage area draining to the pipe
 Q_{cap} = the flow capacity the pipe can carry

Table 8-B.5-1 Four Cases of HGL for Various Pipe Water Level Conditions

A) The Pipe Is Submerged ($H_{G_{LO}} > \text{Top of upper end}$)
<ul style="list-style-type: none"> • $H_{G_{LO}}$ = (as computed in Section 8-B.5.2) • $H_{G_{UP}}$ = (as computed in Section 8-B.5.2)
B) Top of Upper end of pipe > $H_{G_{LO}}$ > Top of Lower end of pipe
<ul style="list-style-type: none"> • If $Q \geq Q_{cap}$ $H_{G_{LO}}$ and $H_{G_{UP}}$ as in Case A except if $H_{G_{UP}}$ below top of pipe then $H_{G_{UP}}$ = top of pipe. • If $Q < Q_{cap}$ $H_{G_{LO}}$ as in Case A $H_{G_{UP}}$ = maximum of $H_{G_{LO}}$ and normal depth + flow line (re-calculate velocity, velocity head, and friction loss based on new depth of flow)
C) The $H_{G_{LO}}$ is between the top and bottom of the lower end of the pipe.
<ul style="list-style-type: none"> • If $Q \geq Q_{cap}$ $H_{G_{LO}}$ = top of pipe (lower end) $H_{G_{UP}}$ = top of pipe (upper end) + friction loss • If $Q < Q_{cap}$ $H_{G_{LO}}$ = max of flow line + normal depth and $H_{G_{LO}}$ (Case A) $H_{G_{UP}}$ = max of flow line + normal depth and $H_{G_{LO}}$ (above) (re-calculate velocity, velocity head, and friction loss based on new depth of flow)
D) The $H_{G_{LO}}$ is below bottom of lower end of pipe.
<ul style="list-style-type: none"> • If $Q \geq Q_{cap}$ $H_{G_{LO}}$ = top of pipe (lower end) $H_{G_{UP}}$ = top of pipe (upper end) + friction loss • If $Q < Q_{cap}$ $H_{G_{LO}}$ = lower flow line elevation + normal depth $H_{G_{UP}}$ = upper flow line elevation + normal depth (re-calculate velocity, velocity head, and friction loss based on new depth of flow)

8-B.6 PIPE SELECTION AND PLACEMENT

After determining the placement of the catch basins, the following design procedure may be followed to design the remaining storm drain system and size pipe conduits for surcharged-full flow conditions. At the end of Part B of this chapter is a spreadsheet to be used as an aid in the design of a storm drain system.

8-B.6.1 Design Procedure for Determining Pipe Capacity

The following steps should be performed for each pipe, with computations progressing to the outfall pipe.

- a.) Inlet Structures – The first four columns give the location of the structure and the inlets on either end. These numbers are to be shown on the plans.

Column 1: Line No. of the Pipe (Same as structure number)
Column 2: Upper End Structure Type; what type of structure the upper end design number refers to; i.e. whether it is a catch basin (CB) or manhole (MH), etc.
Column 3: Upper End Inlet No.
Column 4: Lower End Inlet No.

- b.) Pipe Length –
Column 5: Length of the pipe

- c.) Drainage Area Information – Columns 6 through 12 are for the information of the behind the curb or the paved roadway drainage basin, whichever is greater.

Column 6: Hydraulic Length in feet (the longest length for the water to travel, usually the diagonal length on rectangular shaped drainage basins.)
Column 7: Slope in %, of the drainage basin
Column 8: Incremental Area in acres
Column 9: Total Area in acres; Cumulative values of drainage areas (Column 8).
Column 10: C, runoff coefficient values
Column 11: Incremental AC; Multiply incremental drainage area (Column 8) by runoff coefficient (Column 10).
Column 12: Total AC; Cumulative values of Column 11.

- d.) Time of Concentration – Enter the Time of Concentration.
 Column 13: Travel time in the pipe* (*Calculated later)
 Column 14: Time, mins.; Total time of concentration for the pipe
- e.) Rainfall Intensity –
 Column 15: Using the time of concentration for the first pipe (Column 14), find the corresponding rainfall intensity from the rainfall intensity curves in Chapter 3.
- f.) Discharge –
 Column 16: Q, cfs; Compute the runoff generated by the first drainage area (Column 12 × Column 15) by the Rational Method (see Chapter 3). For the first pipe in the system, the required capacity is equal to the runoff generated by the first drainage area (assume that each inlet has 100% interception ratio).
- g.) Pipe Size –
 Column 17: Pipe Diameter, in.; Enter the first estimate of pipe size. Assume full flow for the first trial. Use a slope roughly parallel to the existing surface grade and desirable velocity range in guiding judgment. In flat areas, keep pipe velocities as low as practical to ensure minimum head losses in structures. Flow rate through a sloping conduit or open channel is given with practical accuracy by Manning's formula:

$$Q = A \left(\frac{1.486}{n} \right) R^{2/3} S^{1/2} \quad \text{Eq. 8-B.6-1}$$

- Where: Q = rate of flow, ft³/s
 A = area of flow, ft²
 n = Manning's roughness coefficient
 (see Chapter 6, Table 6-A.4-1)
 R = hydraulic radius
 (area of flow/wetted perimeter)
 S = slope of conduit, ft/ft

Figures 8-B.6-1 and 8-B.6-2 provide solutions to Manning's Equation for full flow when the value of Manning's Roughness Coefficient, n, is equal to 0.012. Plotted on the horizontal scale is the slope in percent. The vertical scale is the design flow. Diagonal lines represent pipe diameter (or round equivalent diameter for pipe arches) in inches. Velocity lines are also plotted with values along the bottom diagonal line. The required pipe size can be

obtained by projection of a horizontal line from the design flow, Q, and a vertical line from the slope, S.

- h.) Hydraulic Slope –
Column 18: The required hydraulic slope is computed from Eq. 8-B.4-5.
- i.) Velocity –
Column 19: Using the total runoff of Column 16 and the estimated pipe size of column 17, compute the velocity.
- j.) Friction Loss –
Column 20: Friction loss in ft; Friction loss is computed by multiplying the length of pipe (Column 5) by the required hydraulic slope (Column 18).
- k.) Velocity Head –
Column 21: Enter the Velocity Head, $V^2/2g$.
- l.) Junction Loss –
Column 22: Enter the Junction Loss (the head loss through inlets or manholes). For typical head loss computations, see example problem solution.
- m.) Construction Slope –
Column 27: Enter the construction slope of the pipe.
- n.) Travel Time In Pipe –
Column 13: With the length of reach in Column 5 and the velocity in Column 19, determine the time of travel in the pipe in minutes from the following relationship:
$$[t = (L/V)(1 \text{ min}/60 \text{ sec})]$$
- o.) Repeat – Repeat procedure for next run of pipe using cumulative product of drainage area and runoff coefficient (Column 12), and the time of concentration equal to the time of concentration for the second drainage area or the time of concentration for the first drainage area plus time of flow for the first pipe (Column 13), whichever is greater. As the computation progresses, pipe flow is computed with a cumulative "AC" multiplied by a rainfall intensity derived by using a time of concentration equal to the largest time obtained by combining the overland flow time for any drainage area on the system with the pipe flow time from that drainage area to the pipe under construction.

8-B.6.2 Hydraulic Gradient Computations

System hydraulic gradient computations must always start at the outlet for the system. With the friction loss (Column 20) and the junction loss (Column 22), it is possible to calculate the elevations of the hydraulic gradient (Columns 23 and 24) from the outlet to the beginning of the storm drain system.

- p.) Gutter or Inlet Top Elevation –
Column 25: Enter the street elevation at the upper end of the reach. This will be the top of manhole, top of inlet grate, flow line of gutter or other pertinent elevation.

- q.) Hydraulic Clearance –
Column 26: Compute the hydraulic clearance; the difference between the street elevation (Column 25) and the hydraulic grade line.

8-B.6.3 Flow Line Determination

- r.) Construction Slope –
Column 27: Enter the construction slope of the pipe.

- s.) Upper Flow Lines –
Column 28: The upper flow line elevation is determined.

- t.) Lower Flow Lines –
Column 29: Flow lines or invert elevations are determined after satisfactory hydraulic grades have been established.

- u.) Remarks –
Column 30: Enter any pertinent remarks here.

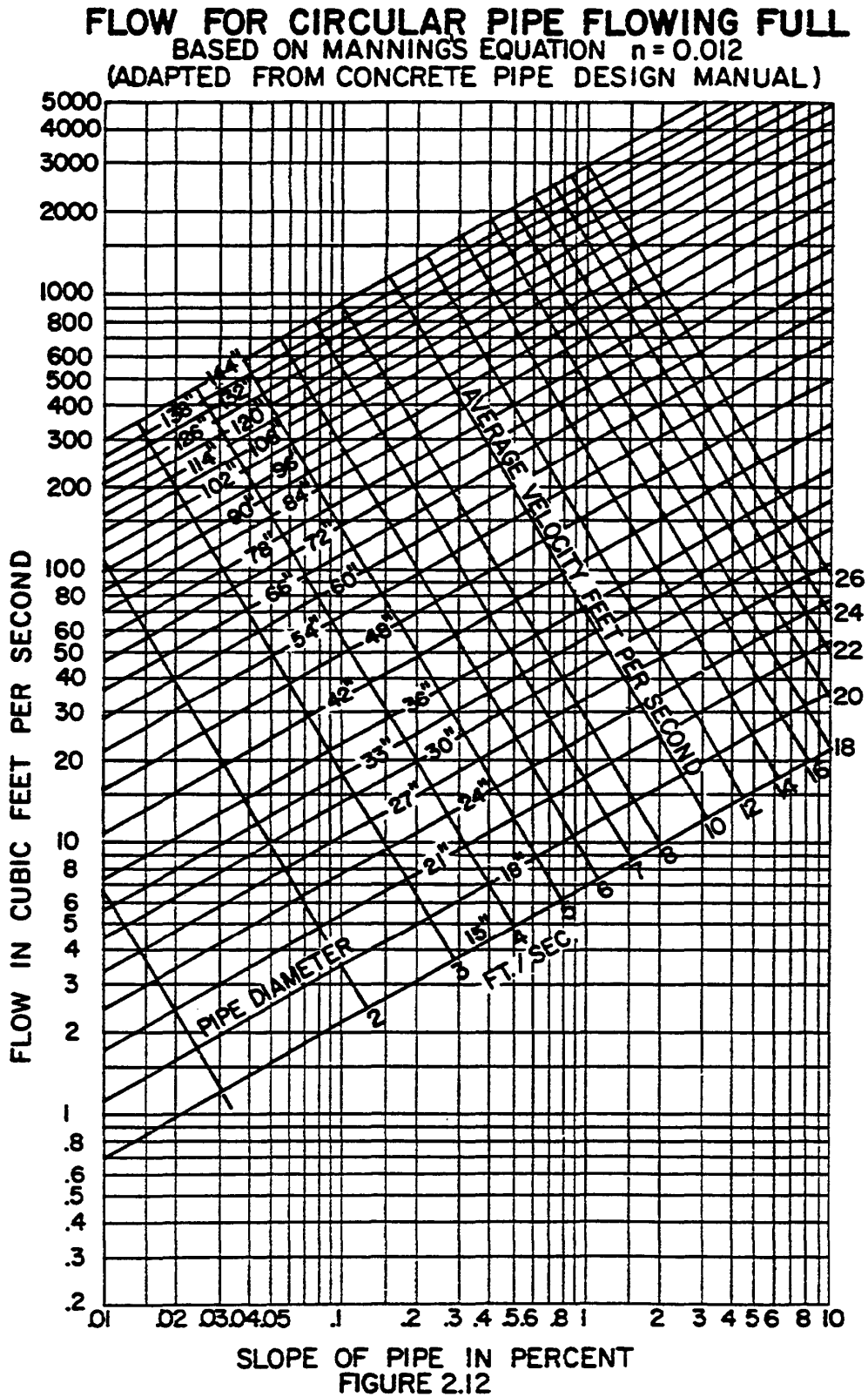


Figure 8-B.6-1 Flow for Circular Pipe Flowing Full

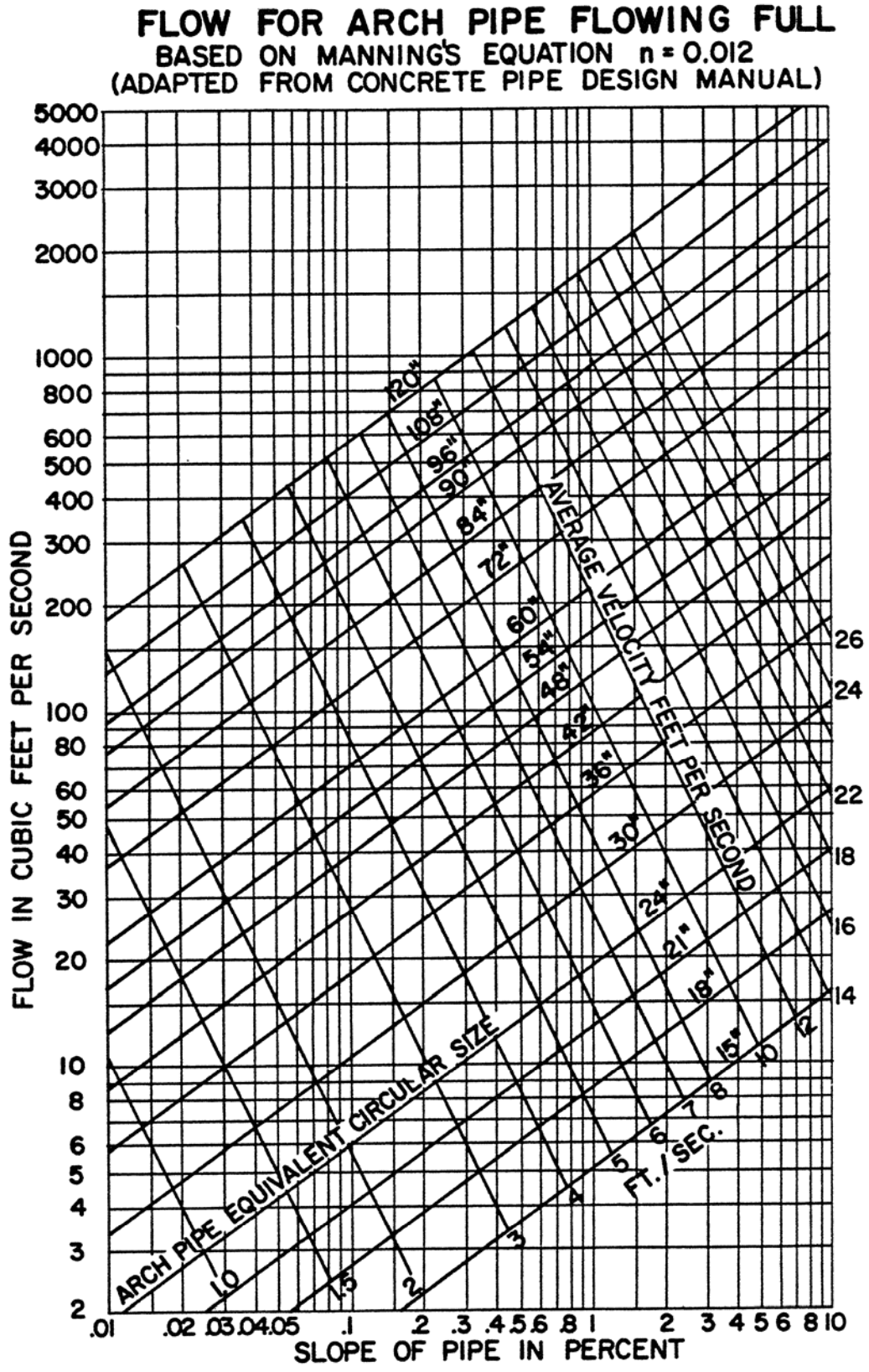


Figure 8-B.6-2 Flow for Arch Pipe Flowing Full

8-B.6.4 Example of Pipe Size Selection and Placement

The following is an example of determining the size of pipes to provide adequate hydraulic clearance for a storm drain (curb and gutter) system. Given the information as shown in Figure 8-B.6-1(a), determine the size and flow line elevations for the pipes in the storm drain system that will meet design guidelines as discussed in this chapter. The 10 year storm elevation is 56.20 feet. The average surface grade equals 0.20%. The gutter grade profiles for the following inlets are: Inlet No. 1 = 60.20 ft, Inlet No. 7 = 60.00 ft, Inlet No. 11 = 59.80 ft, and Inlet No. 13 = 60.00 ft.

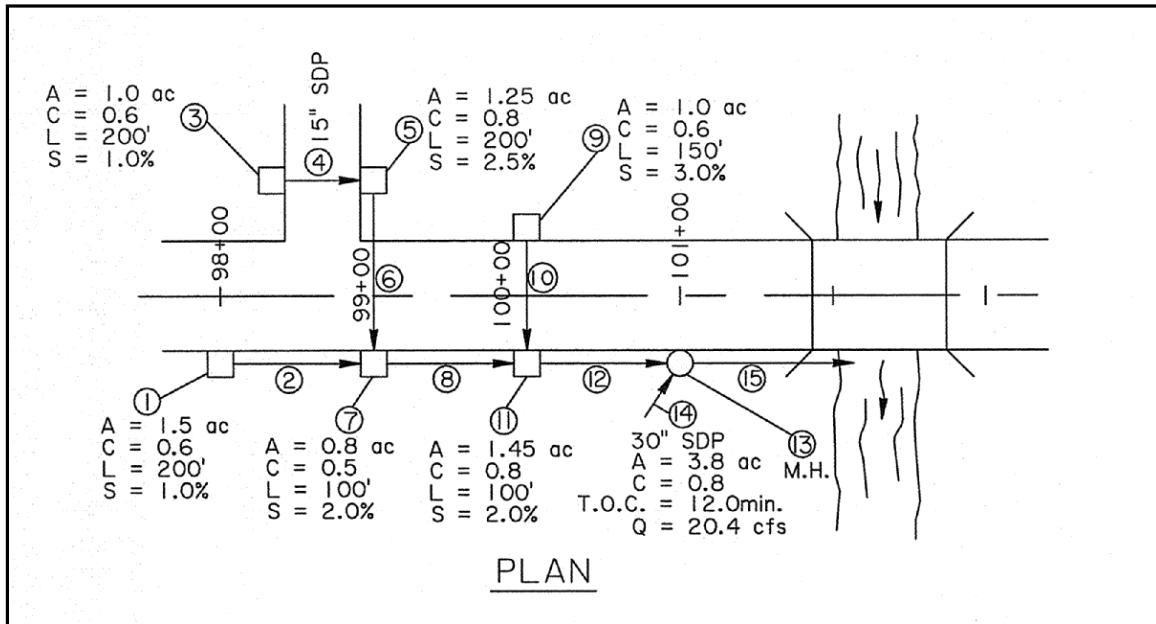


Figure 8-B.6-1(a) Plan View of Example of Pipe Size Determination and Placement in a Storm Drain System Design

- Pipe Selection Process Computations

A plot of the profile is shown in Figure 8-B.6-1(b)

For Line No. 2 and Inlet No. 1:

$$L \text{ or } HL = 200$$

$$C = 0.60$$

$$S = 1.0\%$$

- $TC = (0.7039)(HL^{0.3917})(C^{-1.1309})(S^{-0.1985}) = \underline{10.0 \text{ min}}$

- $I = a(D+b)^c = (4.016)[(10/60) + 0.347]^{-0.826} = \underline{7.0 \text{ in/hr}}$

- $Q = (\sum AC)(I) = (0.90)(7.0) = \underline{6.3 \text{ ft}^3/\text{sec}}$

Assume full flow:

- Estimated Pipe Size = 18"
 - Q = 6.3 cfs
 - n = 0.012
 - A = 1.77 ft²
 - R = 0.375

Solve for S:

$$Q = A \left(\frac{1.486}{n} \right) R^{2/3} S^{1/2}$$

- S = Hydraulic Slope = 0.0031 ft/ft

Velocity:

- V = Q/A = 6.3/1.77 = 3.6 ft/sec

Friction Loss:

- H_f = (Pipe Length)(Hydraulic Slope) = (100)(0.0031) = 0.31 ft

Velocity Head:

- H_v = V²/2g = (3.6)²/(2)(32.2) = 0.20 ft

Junction Loss:

- H_j = Head Loss + Entrance Loss
 - = V²/2g + k (V²/2g) = 0.2 + 0.5(0.20) = 0.30 ft

Travel time in the pipe:

- t = Pipe Length/Velocity
 - t = (100 ft / 3.6 ft/sec) (1 min / 60 sec) = 0.50 min

- Repeat Pipe Selection Process Computations for Each Line of Pipe
- Below is a summary of the some of the calculations for each line of pipe:

- Inlet No. 1 Pipe No. 2
 - Q = 6.3 cfs D = 18" V = 3.6 ft/s
 - H_v = Entire velocity to be generated = V²/2g = 0.20'
 - He = K_e(V²/2g) = 0.50(0.20) = 0.10'
 - H_j = H_v + He = 0.30'

- Inlet No. 3 Pipe No. 4
 - Q = 4.2 cfs D = 15" V = 3.4 ft/s
 - H_v = Entire velocity to be generated = V²/2g = 0.18'
 - He = K_e (V²/2g) = 0.50(0.18) = 0.09'
 - H_j = H_v + He = 0.27'

- Next, calculate the upper and lower hydraulic gradients for each line of pipe, starting with the outfall pipe:
 - o Line No. 15
$$\begin{aligned}\text{Lower} &= \text{HG}_{\text{LO}} = 10 \text{ year storm stage} + \text{Exit Loss} \\ &= 56.20 + 0.44 = 56.64 \\ \text{Upper} &= \text{HG}_{\text{UP}} = \text{Lower} + \text{Friction Loss} \\ &= 56.64 + 0.22 = 56.86\end{aligned}$$
 - o Line No. 14
$$\begin{aligned}\text{Lower} &= \text{HG}_{\text{LO}} = \text{HG}_{\text{UP-15}} + \text{Hj-15} \\ &= 56.86 + 0.44 = 57.30 \\ \text{Upper} &= \text{HG}_{\text{UP}} = \text{Lower} + \text{Friction Loss} + \text{Junction Loss} \\ &= 57.30 + 0.22 + .13 = 57.45\end{aligned}$$
 - o Line No. 12
$$\begin{aligned}\text{Lower} &= \text{HG}_{\text{LO}} = \text{HG}_{\text{UP-15}} + \text{Hj-15} \\ &= 56.86 + 0.44 = 57.30 \\ \text{Upper} &= \text{HG}_{\text{UP}} = \text{Lower} + \text{Friction Loss} \\ &= 57.30 + 0.19 = 57.49\end{aligned}$$
 - o Line No. 10
$$\begin{aligned}\text{Lower} &= \text{HG}_{\text{LO}} = \text{HG}_{\text{UP-12}} + \text{Hj-12} \\ &= 57.49 + 0.30 = 57.79 \\ \text{Upper} &= \text{HG}_{\text{UP}} = \text{Lower} + \text{Friction Loss} + \text{Junction Loss} \\ &= 57.79 + 0.29 + 0.32 = 58.40\end{aligned}$$
 - o Line No. 8
$$\begin{aligned}\text{Lower} &= \text{HG}_{\text{LO}} = \text{HG}_{\text{UP-12}} + \text{Hj-12} \\ &= 57.49 + 0.30 = 57.79 \\ \text{Upper} &= \text{HG}_{\text{UP}} = \text{Lower} + \text{Friction Loss} \\ &= 57.79 + 0.20 = 57.99\end{aligned}$$
 - o Line No. 6
$$\begin{aligned}\text{Lower} &= \text{HG}_{\text{LO}} = \text{HG}_{\text{UP-8}} + \text{Hj-8} \\ &= 57.99 + 0.25 = 58.24 \\ \text{Upper} &= \text{HG}_{\text{UP}} = \text{Lower} + \text{Friction Loss} \\ &= 58.24 + 0.20 = 58.44\end{aligned}$$
 - o Line No. 4
$$\begin{aligned}\text{Lower} &= \text{HG}_{\text{LO}} = \text{HG}_{\text{UP-6}} + \text{Hj-6} \\ &= 58.44 + 0.19 = 58.63 \\ \text{Upper} &= \text{HG}_{\text{UP}} = \text{Lower} + \text{Friction Loss} + \text{Junction Loss} \\ &= 58.63 + 0.25 + 0.27 = 59.15\end{aligned}$$
 - o Line No. 2
$$\begin{aligned}\text{Lower} &= \text{HG}_{\text{LO}} = \text{HG}_{\text{UP-8}} + \text{Hj-8} \\ &= 57.99 + 0.25 = 58.24 \\ \text{Upper} &= \text{HG}_{\text{UP}} = \text{Lower} + \text{Friction Loss} + \text{Junction Loss} \\ &= 58.24 + 0.31 + 0.30 = 58.85\end{aligned}$$
- The Hydraulic Clearance = Street Elevation at the Gutter – HG_{UP}

- o Line No. 2 = 60.20 – 58.85 = 1.3 ft
- o Line No. 4 = 60.20 – 59.15 = 1.0 ft
- o Line No. 6 = 60.00 – 58.44 = 1.6 ft
- o Line No. 8 = 60.00 – 57.99 = 2.0 ft
- o Line No. 10 = 59.80 – 58.40 = 1.4 ft
- o Line No. 12 = 59.80 – 57.49 = 2.3 ft
- o Line No. 14 = 60.00 – 57.45 = 2.5 ft
- o Line No. 15 = 60.00 – 56.86 = 3.1 ft

The profile view showing the above results plus the flow lines for the pipes is shown in Figure 8-B.6-1(b). This is an example of a storm drain system only. Information on how to correctly prepare a set of plans is in the Roadway Design Procedures and Details Manual. All the values given and calculated for this example are summarized on the spreadsheet shown in Figure 8-B.6-1(c).

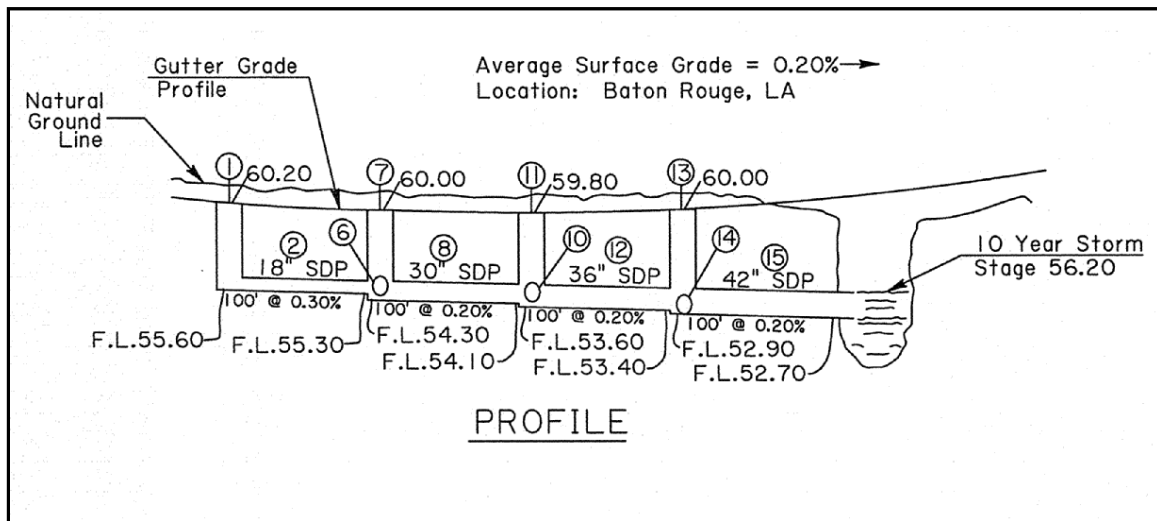


Figure 8-B.6-1(b) Profile View of Example of Pipe Size Determination and Placement in a Storm Drain System Design

DATE: 7/1/09
PROJECT NO.: 999-99-9999
COMPUTED BY: WMR
DESIGN STORM: 10 YEAR

**STORM DRAIN SYSTEM DESIGN
(RATIONAL METHOD)**

LINE NO. (STRUCTURE NO.)	INLET STRUCTURES			PIPE LENGTH (ft.)	DRAINAGE AREA				TIME (mins.)		I (in./hr.)	Q (cfs)	PIPE DIAMETER (in.)	REQUIRED HYDRAULIC SLOPE (ft./ft.)	V (ft./sec.)	FRICTION LOSS (ft.)	$\frac{V^2}{2g}$ (ft.)	JUNCTION LOSS (ft.)	HYDRAULIC ELEVATION		GUTTER OR INLET TOP ELEVATION	HYDRAULIC CLEARANCE (ft.)	CONSTRUCTION SLOPE (ft./ft.)	FLOWLINE ELEVATION		REMARKS			
	UPPER END STRUCTURE TYPE	UPPER END	LOWER END		AREA (acres)	RUNOFF COEFFICIENT (C)	AC	TOTAL	TOTAL	TOTAL									UPPER	LOWER				UPPER	LOWER				
2	CB	1	7	100	200	1.0	1.50	0.6	0.9	0.9	0.5	10.0	7.0	6.3	18	0.0031	3.6	0.31	0.20	58.85	58.24	60.20	1.3	0.003	55.60	55.30	OK		
4	CB	3	5	70	200	1.0	1.00	0.6	0.6	0.3	10.0	7.0	4.2	15	0.0036	3.4	0.25	0.18	59.15	58.63	60.20	1.0	0.003	55.96	55.75	OK			
6	CB	5	7	100	200	2.5	1.25	0.8	1.0	1.6	0.5	10.3	6.9	11.0	24	0.0020	3.5	0.20	0.19	58.44	58.24	60.00	1.6	0.002	55.00	54.80	OK		
8	CB	7	11	100	100	2.0	0.80	0.5	0.4	2.9	0.4	10.8	6.8	19.7	30	0.0020	4.0	0.20	0.25	57.99	57.79	60.00	2.0	0.002	54.30	54.10	OK		
10	CB	9	11	70	150	3.0	1.00	0.6	0.6	0.6	0.3	7.2	7.5	4.5	15	0.0041	3.7	0.29	0.21	58.40	57.79	59.80	1.4	0.004	54.58	54.30	OK		
12	CB	11	13	100	100	2.0	1.45	0.8	1.2	4.7	0.4	11.2	6.7	31.2	36	0.0019	4.4	0.19	0.30	57.49	57.30	59.80	2.3	0.002	53.60	53.40	OK		
14	—	Stubin	13	12	—	—	3.80	0.8	3.0	3.0	0.0	12.0	6.6	20.1	30	0.0021	4.1	0.02	0.26	57.45	57.30	60.00	2.5	0.003	52.94	52.90	OK		
15	MH	13	outfall	100	—	—	0.00	10.80	0	0.0	7.7	0.3	12.0	6.6	50.8	42	0.0022	5.3	0.22	0.44	56.86	56.64	60.00	3.1	0.002	52.90	52.70	Exit Loss=0.44'	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30

Figure 8-B.6-1(c) Summary Spreadsheet of Pipe Selection

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CHAPTER 9 DRAINAGE DESIGN IN WETLANDS

9.1 PURPOSE

This chapter describes general drainage design policy for wetland areas. This policy is designed to fit average conditions; therefore, some areas or projects will require modification of this policy. The Hydraulics Unit should be consulted when unusual situations require such modification.

The purpose of this policy is to provide drainage which will accommodate a wide range of flow conditions, from everyday sheet flow to large major storms, and do so with negligible changes to existing conditions.

A thorough examination of aerial photographs, quadrangle maps, plans, gage records, etc., is required to give the drainage designer a minimum comprehensive overview of a project. This should be supplemented by a field trip to further familiarize the designer with the terrain.

9.2 TERMINOLOGY

- A.) Wetlands: Lowlands covered with shallow and sometimes temporary or intermittent waters, which are valuable to the aquatic food chain because of its nutrient output, and whose high nutrient productivity and supply depends to some extent on sheet flow. Wetlands may be salt marsh, freshwater marsh, tree swamp or combinations and variations thereof, and may or may not be subject to tidal action. Slopes are very slight and drainage slow.
- B.) Sheet Flow: The type of water flow taking place in most areas of the wetlands. Flow not contained in a defined or discernible waterway (such as bayou) but rather flowing overland in a "sheet" which is of a relatively constant depth when compared to varying depths of flows occurring in streams, rivers, etc.
- C.) Major Drainage Structure: One in, or over, a defined, discernible waterway such as a bayou, ditch, or canal; hereinafter referred to as a major structure; may be a culvert or a bridge.
- D.) Minor Drainage Structure: A culvert which does not serve a defined, discernible stream; hereinafter referred to as a minor structure.
- E.) Drainage Area: The area contributing runoff to a particular drainage structure.

9.3 DESIGN STORM FREQUENCY

The minimum design storm frequency used for major structures will be 50 years, regardless of the class of highway. For minor structures, a 5-year return interval design storm shall be required.

9.4 DRAINAGE AREA

Because of the extremely flat terrain, there are many uncertainties associated with wetlands drainage design. For example, in other terrains, determining the drainage area is relatively easy. In wetlands areas, the drainage area is much more difficult to determine. Hilly country has many easily discerned ridge lines which defines the drainage area readily. In wetlands areas, there are usually no ridge lines, and a line which may accurately define a drainage area today may be incorrect tomorrow due to wind shifts, tide, and/or rainfall changes, etc.

Drainage areas may be assigned in wetlands areas with acceptable accuracy, considering the following criteria.

9.4.1 Stream Size

In determining the placement of a drainage area boundary between two waterways remember that the larger the stream, the larger the area of influence during major floods. Also, drainage areas in this terrain should tend to have their width equally divided by their waterways in most cases.

9.4.2 Direction of Sheet Flow

Inside a particular major drainage area, the influence of the central waterway on sheet flow direction decreases as the distance from the waterway increases. For example, if the general or average sheet flow direction is from north to south and the general direction of flow in the waterway is from north to south, a drop of water falling near the stream may actually flow southeast or southwest until reaching the stream.

9.4.3 Channels

Man-made or improved channels usually offer some impediment to sheet flow intersecting them. This is due to the combination of two effects; the spoil banks, and the lesser resistance to flow in the channel. Often such channels may be a portion of a drainage area's boundaries.

9.5 CHANNEL REQUIREMENTS

In order to minimize disruptions to sheet flow, collector (upstream) and distributor (downstream) channels are required. These channels should parallel the roadway and be constructed outside of the spoil bank or embankment (if no spoil bank). These channels are to be blocked near the major structures. It should be remembered that due to hurricanes, abnormally high tides, etc., collector and distributor channels may switch functions. These channels should be designed accordingly.

9.5.1 Channel Depth

In designing channel depths, locations, etc., it should be remembered that their purpose is to help maintain sheet flow as close to its original pattern as possible.

Channels should neither raise nor lower preconstruction water levels.

9.5.2 Required Channel Dimensions

Normally, a four foot bottom channel, two feet deep, (below average natural ground line) with 4:1 side slopes will usually suffice, as the everyday flow serviced is quite slow.

9.5.3 Channel Grade

A 0.00% channel grade is acceptable.

9.6 CULVERT REQUIREMENTS

9.6.1 Minimum Spacing

Minor structures should be spaced between 500 and 1500 feet apart.

Spacing should be varied (by changing the number of structures) to achieve structures which yield differential heads near the allowable.

9.6.2 Flow Lines

Flow lines may be set below distributor and collector channel bottom elevations or waterway bottom profiles by a dimension equal to one-third the height of circular, square or rectangular culverts, or one quarter the rise of arch pipes. This dimension is not to exceed one foot.

9.6.3 Structure Size

The minor structure sizes will depend upon their design flow (which depends on the number of structures), the allowable differential head, structure type, and the vertical distances between channel bottoms and normal water surface.

9.6.4 Differential Head

Major Drainage Structures:

- All major drainage structures shall be designed for 0.5 to 1.0 feet of differential head for a storm with a fifty year return interval.

Minor Drainage Structures:

- Minor structures shall be designed for approximately 0.5 feet of differential head for a five year return interval.

9.6.5 Soffits

Major Drainage Structures:

- The soffits of all major structures may be a maximum of one foot above the average water level.

Minor Drainage Structures:

- The soffits of all minor structures should be no higher than 6" above the average water level.

9.7 WETLANDS DRAINAGE DESIGN EXAMPLE

The following are the steps in designing the necessary structures for the conditions shown in Figure 9.7-1. Table 9.7-1 displays the final structure information.

9.7.1 Flow Characteristic

The area is in what is considered wetlands. It is assumed to have sheet flow.

9.7.2 Drainage Area

Major drainage areas, as well as minor drainage areas, are not actually defined by ridge lines, but are arbitrarily determined considering:

- a.) The larger the waterway, the greater its drainage area.
- b.) The shorter the distance to a waterway, the greater the effect that waterway has on sheet flow direction.
- c.) Man-made or improved channels usually offer some impediment to sheet flow. The older they are the less likely they are to contain breaks. Additionally, the canal itself offers a path of less resistance.

Major Drainage Structures:

- The major structures will be designed for these areas: structure Number 5 for the area bounded by A-B-J-D-E-F-A, and structure Number 12 for the area bounded by F-E-D-J-H-I-F. (See Figure 9.7-1)

Minor Drainage Structures:

- The remaining minor structures will have portions of the major drainage areas reassigned to them by the following approach:

Upper boundaries will be assigned by approximating a line separating areas which would drain to the waterway, from areas which would drain to the roadway. These lines are represented in the drawing by B-C so that a drop of rain falling slightly to the west of it will flow in such a direction as to intersect the roadway instead of the waterway. Conversely, a drop of rain falling slightly to the east of B-C would intersect the waterway before draining across the roadway.

The procedure for locating such a line is to set it at an angle Δ to the average waterway alignment near the roadway (see Figure 9.7-1). In setting this angle, the relative effect that the waterway has on the hydraulic gradient within its drainage area should be considered. In general, the narrower the drainage area and the larger the waterway, the greater the waterway's influence and the larger the angle Δ should be. Other parameters are: stream flow direction relative to sheet flow direction, magnitude of stream's natural levee or manmade spoil bank, and number of openings in those levees and/or spoil banks. Angles of 30 and 45 degrees will usually be descriptive.

The total drainage area for the minor structures should now be determined.

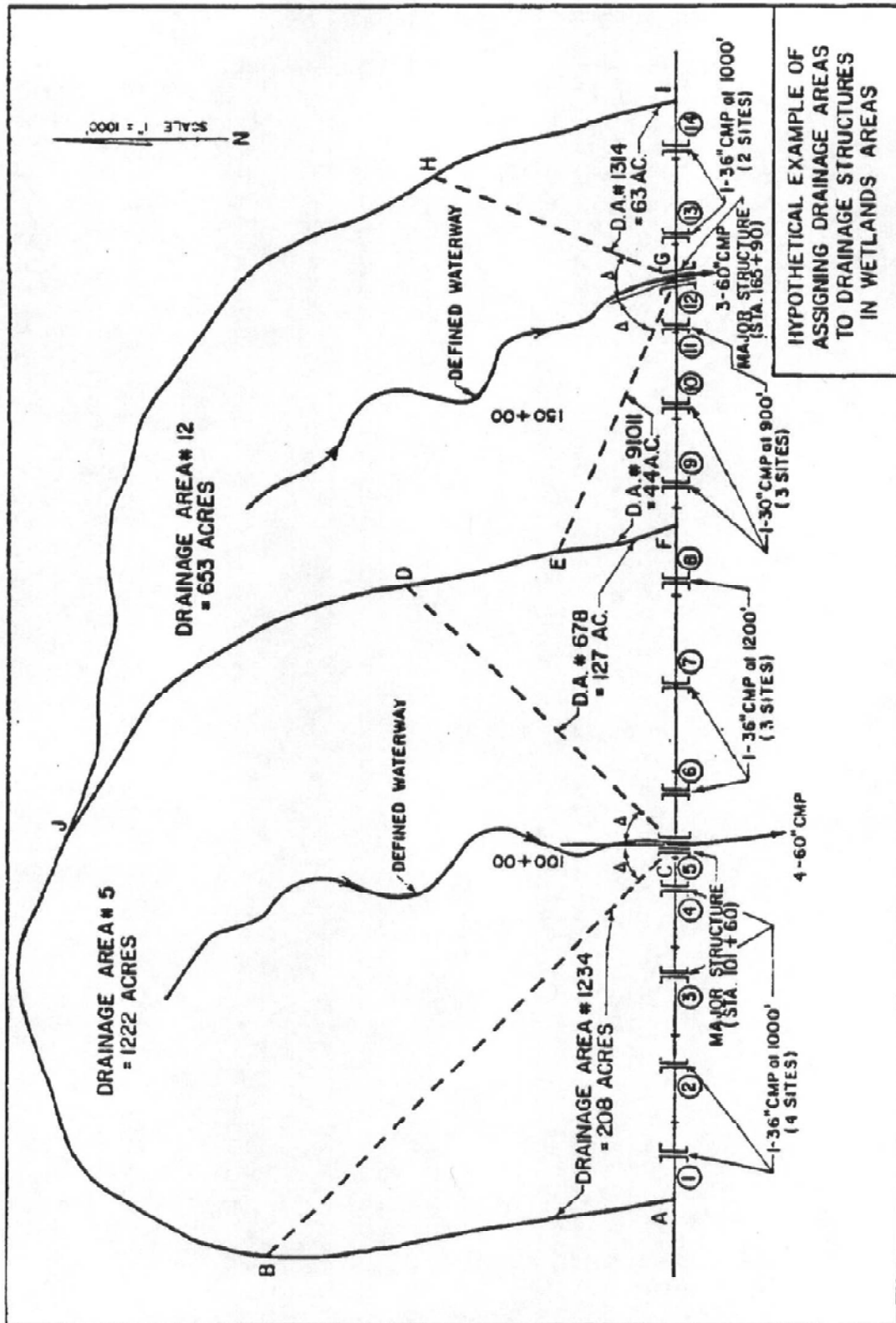


Figure 9.7-1 Wetlands Drainage Design Example

9.7.3 Discharge

Major Drainage Structures:

- The design Q is for a 50-year return interval. Use the NRCS method to determine the runoff with the following:

$$\begin{aligned} \text{CN} &= 80 \\ \text{I} &= 12" \text{ (24 hours)} \\ \text{S} &= 0.1\% \end{aligned}$$

Multiply the design Q by 0.55 (wetlands reduction factor for 50-year return interval).

Entire area "above" centerline considered, no reduction for equalizers

Minor Drainage Structures:

- The design Q is for a 5-year return interval. Use the NRCS method to determine the runoff with the following:

$$\begin{aligned} \text{CN} &= 80 \\ \text{I} &= 8" \text{ (24 hours)} \\ \text{S} &= 0.1\% \end{aligned}$$

Multiply the design Q by 0.45 (wetlands reduction factor for 5-year return interval).

Areas to be determined as shown in Figure 9.7-1, $\Delta = 30^\circ - 45^\circ$. The more the expected effect of the major waterway on gradients, the greater the angle Δ .

The runoff (Q) is calculated for the total drainage area for minor structures. This total runoff should be divided into n equal portions, n being the number of minor structures. (Q each structure = (Q total)/(No. Of structures)) For example, D.A. Number 1234 has a total area of 208 acres, which produces 68 cfs for a five year storm. This runoff of 68 cfs divided by four yields 17 cfs, the assigned Q for each minor structure within D.A. Number 1234 (defined by A-B-C-A).

9.7.4 Structure Size

Major Drainage Structures:

- Major structures are sized for a differential head (ΔH). $\Delta H = 0.5'$ to $1.0'$ assuming:
 - Cross Drain Pipe, no headwalls
 - Length = 180'
 - Tailwater = height of the structure
 - Outlet control
 - S = 0.00001 feet per feet

Minor Drainage Structures:

- Minor structures sized same as major structures with the exception that $\Delta H = 0.5'$.

9.7.5 Structure Spacing

Major Drainage Structures:

- There are no limits for major structure spacing. Major structures are to be placed where the discernable waterways are located.

Minor Drainage Structures:

- Limits:
Minimum = 500'
Maximum = 1500'
- Spacing:
Distance between edge of assigned drainage areas and major structures divided by the number of pipes
- Exterior structures (outside structures in any particular drainage area) are spaced at one half spacing to major structure or drainage area boundary.

9.7.6 Collector and Distributor Channels

- Collector channels on upstream side, plugged at major structures.
- Distributor channels on downstream side, plugged at major structures.
- Standard channel (collector and distributor):
Bottom width = 4'
Depth = 2'
Side slopes = 4:1
Grade = 0%

Table 9.7-1 Major and Minor Structures Required

Structure Number	Drainage Area (acres)	Q ₅ (cfs)	Structure
1	208	17	1-36" CMP
2		17	1-36" CMP
3		17	1-36" CMP
4		17	1-36" CMP
5	1222		4-60" CMP (major)
6	127	17	1-36" CMP
7		17	1-36" CMP
8		17	1-36" CMP
9	44	9	1-30" CMP
10		9	1-30" CMP
11		9	1-30" CMP
12	653		3-60" CMP (major)
13	63	16	1-36" CMP
14		16	1-36" CMP

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CHAPTER 10 STORM WATER PUMPING STATIONS

10.1 GENERAL

The quantity of storm water runoff reaching a pumping station is determined by hydrologic analysis of the regional rainfall events, characteristics of the watershed, and the hydraulics of the collection system (gutters, inlets, ditches, channels, and conduits).

In the design of storm water pumping stations, it is necessary not only to determine the peak flow, but also to define the variation of flow rate with time (inflow hydrograph). The inflow hydrograph is the basis for the economical design (pump sizes and wet well capacity) of pumping stations.

When two or more drainage basins drain into a pumping station, a composite hydrograph should be developed and used as the inflow hydrograph. The incoming main drain should also be large enough to carry the design peak flow for the pumping station.

It is generally not economical to size the pumps to accommodate the peak inflow rate or to provide variable pumping capacity to match exactly the inflow hydrograph. The use of available storage volume allows for the pumping capacities which are smaller than peak discharge rates.

The basic relationship on which design is based is: $\text{Outflow} = \text{Inflow} - \text{Storage}$.

10.2 DESIGN FREQUENCIES

The recommended design frequencies are:

- For storm drains in residential areas, 5 to 10 years.
- For storm drains in commercial and high-value districts, 10 to 50 years.
- For flood protection works and depressed roadways, 50 years or more.

Depending on economic justification, the proper storm frequency should be selected at the pre-design conference.

10.3 INFLOW HYDROGRAPH

There are several methods for the development of inflow hydrographs. Refer to FHWA publication HDS-02 for examples. The Natural Resource Conservation Service (NRCS) developed one of the methods of estimating the runoff hydrograph. The NRCS method may be used for drainage areas up to 2000 acres. For drainage areas larger than 2000 acres,

hydrographs developed by the U.S. Geological Survey (USGS) or other general hydrograph procedures may be used.

10.3.1 Future Development

Future development of the watershed should be considered and incorporated into the design in deriving the inflow hydrograph.

If the watershed or collection system will be modified by future development, it is recommended to plan and design for both present and anticipated conditions, over the life of the pumping station. One way of providing increased future capacity is to allow space for additional pumps in the station.

10.4 WET WELLS

To size the wet well for storm water pumping stations, the inflow hydrograph must be routed through the wet well. Methods of reservoir routing are adequately covered in most hydrologic texts (FHWA publication HDS-02). The general equation for reservoir routing is:

$$(\text{Rate of inflow} - \text{Rate of outflow}) = \text{Rate of change in storage} \quad \text{Eq. 10.4-1}$$

10.4.1 Pump Capacity

In a pumping station situation, the rate of inflow is derived from the inflow hydrograph, the rate of outflow is the pumping rate, and the rate of change of storage is a direct function of the shape and volume of the wet well and the piping system leading to the wet well. Therefore, the total pump capacity may be balanced with the required storage volume to obtain the most economical balance between the two.

10.4.2 Wet Well Volume

A minimum of two pumps with one being able to discharge the design flow is recommended for small pumping stations. For large pumping stations two pumps can be used, but efficiency of operation over varying ranges of flow usually dictates three or more pumps of the same capacity. Capacities should be selected so that with any pump out of service, the others can handle the design flow. It is not unusual to use one small pump (low-flow pump) to start first, followed by larger pumps to handle the design flow.

10.5 STORM WATER PUMPING STATIONS HYDRAULICS

For more information on the design of storm water pumping stations, equipment and accessories, refer to these references or the latest publications or websites from the following agencies:

- A.) Highway Stormwater Pump Station Design, Hydraulic Engineering Circular No. 24, FHWA, Washington, D.C., February 2001
- B.) Urban Hydrology For Small Watersheds, Technical Release No. 55, USDA Natural Resources Conservation Service, June 1986.
- C.) Highway Hydrology, Hydraulic Design Series No. 2, FHWA, Washington, D.C., 2nd Edition, October 2002.

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CHAPTER 11

BRIDGE HYDRAULIC DESIGN

11.1 PURPOSE

This chapter is prepared to provide guidance in bridge hydraulic design through appropriate policies and design criteria. For more in-depth fundamental details, the hydraulic design engineer should refer to provisions of AASHTO LRFD Bridge Design Specifications, Hydrology and Hydraulics Section; the AASHTO Highway Drainage Guidelines; and various Hydraulic Engineering Circulars published by the Federal Highway Administration. Unless specified otherwise, the hydraulics design engineer should consult the most recent edition of these references for procedures and guidelines.

Adhering to these guidelines does not relieve the hydraulics design engineer from the responsibility of applying sound engineering principles and judgment. While the guidelines apply to the majority of bridge hydrologic and hydraulic design of new and replacement bridges, they are not intended to be exhaustive.

If the bridge is in a coastal environment, the design should be in accordance with procedures set forth in HEC-25 – Highways in the Coastal Environment and AASHTO LRFD Bridge Design Specifications.

11.2 DEFINITION

Bridges are defined as:

- structures that transport traffic over waterways 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 20 feet or more. However, structures designed hydraulically as bridges as described above are treated as bridges in this chapter, regardless of length.

11.3 ANALYSIS/DESIGN

Proper hydraulic analysis and design is as vital as the structural design. Bridges should be designed for:

- minimum cost subject to criteria,
- desired level of hydraulic performance up to an acceptable risk level,
- mitigation of impacts on stream environment,
- accomplishment of social, economic, and environmental goals and

- full compliance with the requirements of existing Federal Emergency Management Agency (FEMA) or other officially delineated or regulatory floodplains

11.4 DESIGN POLICY

11.4.1 General Policy

The hydraulic design of all bridges shall be done in accordance with good engineering practice and comply with all applicable Federal, State, local government and local flood control districts statutes and regulations. Specifically, the hydraulic design shall comply with FHWA's FAPG 23 CFR 650 Subpart A, and FEMA's National Flood Insurance Program.

11.4.2 FEMA Floodplain Compliance

- The final design selection should be in full compliance with the maximum backwater allowed by the National Flood Insurance Program.
- The final design should not significantly alter the flow distribution in the floodplain.
- Where design considerations permit, the "crest-vertical curve profile" should be considered as the preferred highway crossing profile when allowing for embankment overtopping at a lower discharge.
- A specified clearance should be established to allow for passage of debris.

11.5 DESIGN CRITERIA

11.5.1 AASHTO General Criteria

Design criteria are the tangible means for placing accepted policies into action and become the basis for the selection of the final design of bridges. Criteria are subject to change when conditions so dictate as approved by LADOTD.

Following are certain American Association of State Highway Transportation Officials (AASHTO) general criteria adopted by LADOTD related to the hydraulic analyses for bridges as stated in AASHTO's Highway Drainage Guidelines:

- Backwater will not significantly increase flood damage to property upstream of the crossing.
- Velocities through the structure(s) will not damage either the highway facility or increase damages to adjacent property.
- Maintain the existing flow distribution to the extent practical.
- Pier spacing and orientation, and abutment designed to minimize flow disruption and potential scour.

- Foundation design and/or scour countermeasures to avoid failure by scour.
- Freeboard at structure(s) designed to pass anticipated debris.
- Minimal disruption of ecosystem and values unique to the floodplain and stream.

11.5.2 LADOTD Criteria

LADOTD criteria augment the general criteria by providing specific, quantifiable values that relate to local site conditions. Evaluation of various alternatives according to these criteria can be accomplished by using the water surface profile programs such as FHWA's WSPRO or U.S. ACOE's HEC-RAS.

- a.) Design Floods/Frequency:
Design floods for roadway inundation are discussed in Chapter 1. Design frequencies may be lower or higher when justified by risk analysis and local site conditions. The flood frequencies used for scour analysis differ. (See Section 11.7)
- b.) Backwater/Increases over Existing Conditions:
Backwater created by the structure shall conform to FEMA regulations. It is LADOTD's policy not to allow any backwater increase for sites covered by the National Flood Insurance Program (NFIP) and for delineated floodplains established under the NFIP. For sites not covered by NFIP the backwater increase shall not exceed one foot during the passage of the 100-year flood. Any increase in backwater shall not significantly change land use values, unless flood rights are acquired.
- c.) Clearance:
The freeboard, which is defined as the clearance between the low chord of the bridge and the design approach water surface elevation, should not be less than one foot if there is no debris problem at the bridge site. In addition, the low chord should be set to clear the FEMA base flood (100-year flood) water surface elevation. In cases where debris is considered to be a problem, a minimum one foot additional freeboard should be provided to allow for passage of debris. The minimum freeboard requirements are as follows:
 - 1 ft for design WSE & clear for 100-year WSE (without debris site)
 - 2 ft for design WSE & 1 ft for 100-year WSE (with debris site)

Where the minimum requirement is not practical, the freeboard should be established based on the type of stream and level of protection desired as approved by LADOTD.

- d.) Scour:
Design for bridge foundation scour should consider the magnitude of flood that generates the maximum scour depth. It is LADOTD's practice to evaluate bridge scour from the 500-year flood, or overtopping flood if the overtopping flood is less than the 500-year flood.

11.6 DESIGN PROCEDURE

11.6.1 Hydraulic Performance of Bridges

The stream-crossing system is subject to either free-surface flow or pressure flow through one or more bridge openings with possible embankment overtopping. These hydraulic complexities should be analyzed using a water surface profile computer program. FHWA's WSPRO and U.S. ACOE's HEC-RAS are acceptable computer programs to analyze the hydraulic performance of bridges over riverine waterways. The Watershed Modeling System (WMS) program is another tool available for performing hydrologic and hydraulic computations. Generally, the hydraulics design engineer should refer to the user's instruction manual and other documentation of the specific computer program as sources for the theory employed and detailed information on using the computer model.

11.6.2 Design Procedure

A detailed hydrologic and hydraulic analysis should be performed for designing all new bridges over waterways, bridge widening, bridge replacement, and significant lateral encroachments resulting from the placement of highway fill embankments within a floodplain. It is necessary to do this to be able to ensure that LADOTD construction is in compliance with FHWA and FEMA rules and regulations.

Typically, the basic procedural sequence of the hydrologic and hydraulic analysis process should include Data Collection, Hydrologic Analysis, Hydraulic Analysis, Scour Estimates, Selection of Final Design, and Documentation.

- a.) Data collection:
The data and resources necessary to perform a hydrologic and hydraulic analysis usually include, but would not be limited to: topographic maps, aerial photographs, roadway plan/profile sheets, and typical cross-sections to cover the width of the floodplain in the vicinity of the crossing. Stream gage records and high water marks are useful information for calibration and verification of computer model results. Careful consideration should be given to existing studies, such as floodplain studies that may have been performed by FEMA. In the event a FEMA floodplain (or other officially delineated floodplain) is involved, it will be necessary to have available flood maps, flood profiles, and hydraulic model data. If existing flood studies are used,

validity of assumptions and accuracy of the results of such studies should be verified.

b.) Hydrologic Analysis:

Discharges at a crossing site are computed by procedures detailed in Chapter 3 of this Manual. The referenced methods are used in conjunction with gage data, when available at the design site, as found in Reference "Floods in Louisiana, Magnitude and Frequency". Annual publications such as "Water Resources Data" published by the United States Geological Survey and "Stages and Discharges" published by the Corps of Engineers may also be used when additional information is required. Several computerized water resources databases are also accessible through the United States Geological Survey.

For gaged sites (or gages in the proximity of the site) with good discharge records, a Log-Pearson Type III frequency distribution (prorated upstream or downstream as appropriate) would be the preferred method for determining peak discharges.

If the site is covered by a FEMA (or other officially delineated) floodplain, the peak discharges employed in making the official floodplain delineation should be used in the subsequent hydraulic analysis. In situations where the officially delineated study considers only the 10, 50, 100, and 500-year flood events, it will be acceptable to estimate the magnitude of intermediate frequency events such as 2, 5, and 25-year flood events.

In all cases, the 2, 5, 10, 25, 50, 100, and 500-year flood discharges will either be computed or obtained from appropriate sources, and the discharge-frequency curve can be plotted. Ample documentation should be made throughout the analysis process, especially where judgments are made and where deviations from the normal procedures are necessary to obtain these discharges.

c.) Hydraulic Analysis:

LADOTD prefers WSPRO and HEC-RAS computer models for performing the hydraulic analysis. An existing conditions model and a proposed conditions model should be established for each bridge site.

In order to ensure convergence of the computed profile, the water surface profile used in the hydraulic analysis of a bridge should extend from a point downstream of the bridge that is beyond the influence of the constriction to a point upstream that is beyond the extent of the bridge backwater. These additional cross-sections may be obtained via survey data or topographic maps. Successful application of a computer

program to a specific bridge and proper interpretation of output from the computer program require experience and engineering judgment to supplement the quantitative analysis.

- National Flood Insurance Program (NFIP) Model:

It is necessary to establish the effective hydraulic conditions through a process that will be referred to as a computer model calibration. This calibration is to be accomplished by attempting to reconcile the historical records of discharges vs. water surface elevations. When a FEMA floodplain is involved, the conditions effective at the time of the FEMA floodplain study (including the flows modeled in the original NFIP model, plus any subsequent letters of map change) should be used to calibrate the computer model. Current (existing) development may have increased the flood depths, but remained less than the allowable one foot above flood depth from the legal NFIP map (including all LOMRs). The proposed project should be modeled with the current (existing) conditions to ensure the proposed project will not go over the allowable one foot increase, which would require a letter of map revision (LOMR) to inform the public and FEMA of increased risk in the area.

- Existing Conditions Model – Floodplains not Regulated by NFIP:

Computer model calibration is to be accomplished by attempting to reconcile the historical records of discharges vs. water surface elevations. The existing conditions model should be adjusted to match any known elevation information, such as survey data or gage information. The design, 100, and 500-year (or overtopping) flood events should be included in the model.

- Proposed Conditions Model:

The proposed conditions model should include any and all proposed replacement structures (superimposed on the existing conditions model). The design, 100, and 500-year (or overtopping) flood events should be included in the model.

If a FEMA regulated floodplain does not exist, compare the proposed conditions model with the existing conditions model. The difference in water surface elevation (backwater) should be one foot or less.

For FEMA regulated floodplains with or without floodways, compare the proposed condition model with the NFIP model. For models of FEMA regulated floodplains without floodways, the difference in water surface elevation (backwater) should be one foot or less.

If the FEMA regulated floodplain has a floodway, then there should be no change in water surface elevation between the proposed condition and NFIP model.

If the water surface elevation or backwater limits stated above are exceeded, the proposed replacement structures are to be considered unacceptable and must be adjusted until these limits are met.

d.) Final Design Selection:

The final design recommendation will include the bridge length, deck elevation, position in the floodplain (where applicable), abutment slopes, any recommended excavation, any recommended channel alignment, and any special features such as guide banks, erosion or scour protection measures, etc. The final design selection must meet LADOTD policies and design criteria.

11.7 SCOUR

11.7.1 Scour Estimates

Scour is most likely to occur where velocities are high and the river bed is a non-cohesive soil. Evidence of scour should be investigated at project sites. The location and severity of scour should be noted. It should also be noted whether the stream bed is movable or stable. Any historical scour data on other bridges or similar facilities along the stream should also be considered carefully.

Bridge foundations shall be designed to withstand the effects of scour resulting from the lesser of the following two events:

- The 500-year event, or
- The event causing overtopping of the approach roadway or the bridge deck.

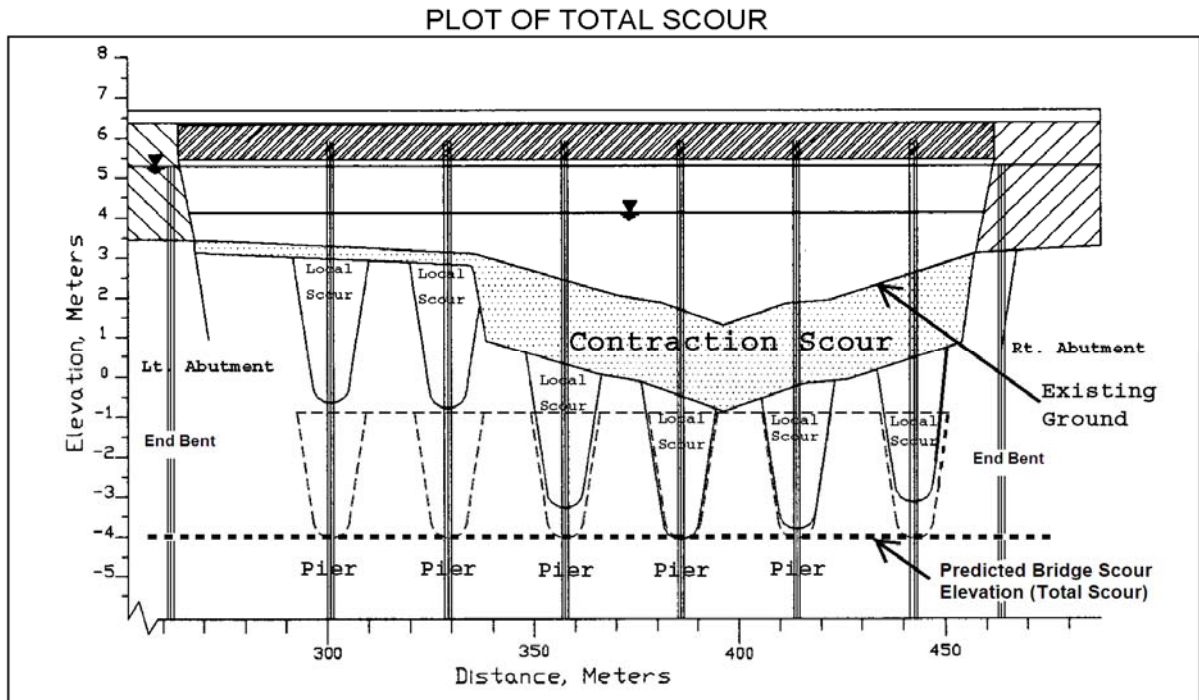
LADOTD employs the procedures and guidelines presented in the FHWA's "Evaluating Scour at Bridges" (HEC-18) and "Stream Stability at Highway Structures" (HEC-20) to determine and counteract the impact of scour on bridges. FHWA's WSPRO computer program has incorporated the scour equations from HEC-18. The total scour depth can be estimated by WSPRO program or directly from HEC-18 equations. Sound engineering judgment should then be exercised to determine the appropriateness and validity of the estimated total scour depth.

Degradation or aggradation of the river should be estimated, contraction and local scour should be determined, and appropriate positioning of the foundation should be set below the scour elevation across the bridge opening if practicable.

11.7.2 LADOTD Policy For Predicting The Scour Elevation For Bridges

Below is the established LADOTD guidelines and policy for predicting the scour elevation for bridges. This elevation is designed to withstand the worst case scour depths, calculated according to FHWA's recommendations, as stated in HEC-18, *Evaluating Scour at Bridges*. This procedure shall be used for "On-System" and "Off-System" bridges.

- 1) The Predicted Bridge Scour Elevation is based on contraction scour and local scour at piers. The Predicted Bridge Scour Elevation is determined by subtracting the combined contraction scour depth and local scour depth from the lowest point in the channel below the proposed bridge. See sketch on Page 11-9 showing *Plot of Total Scour*.
- 2) The Predicted Bridge Scour Elevation applies to all piers including the end bents in the main channel section. Refer to Sections 2.3.2 and 8.11 in HEC-18.
Reasons: Louisiana streams have a great potential for flooding and migrating. Retrofits, scour countermeasures and bridge replacements are costly and disrupt road service to the traveling public.
- 3) The Predicted Scour Elevation for the main bridge and relief bridge(s) is to be the same.
Reason: Louisiana has a history of relief structures having greater scour problems than the structure(s) in the main channel. Even if there is no stream under the relief structure, flooding can cause severe erosion and scour.
- 4) Abutment scour is not usually calculated. Instead, as per FHWA's recommendations in HEC-18, abutments are protected with some type of revetments or riprap.
 - (a.) Abutment scour protection should be designed and installed to account for any long term degradation and contraction scour at the abutment toe.
 - (b.) When revetment or rip rap cannot be used, the abutment scour must be determined in order to design the abutment foundation.
- 5) Long term degradation should be considered in situations where stream bed may be affected by upstream gravel/sand mining operations or in upland areas where there is a potential for headcutting.



11.7.3 Scour Protection and Countermeasures

Every effort shall be made to minimize the effects of scour, such as placing piers outside the main channel, aligning piers to the direction of flow, and using round piers or columns. The following preventive/protection measures should be applied to all proposed new and replacement bridge sites:

- A minimum abutment slope of 3:1 armored with flexible revetment or riprap is required to eliminate the abutment scour estimate; when the design velocities are 3.0 fps or more, wrapping the revetment around the embankment to the limits of the approach slab is recommended.
- A minimum total scour depth of 5 ft is required.
- The predicted scour elevation is conservatively assumed to be constant across the bridge opening and should be the same for both the main bridge and relief bridge(s).

To comply with the National Scour Evaluation Program, a plan of action, which should include timely installation of stream instability and scour countermeasures, should be developed for each scour critical bridge. Monitoring structures during and/or after flood events as a part of a plan of action can be considered an appropriate countermeasure.

Hydraulic countermeasures are primarily designed to resist erosive forces or modify the stream flow. Examples of hydraulic countermeasures include the placement of riprap at

abutments or piers and the installation of river training structures such as guide banks (spur dikes).

a.) Riprap:

Riprap is considered an acceptable scour countermeasure for protection of bridge abutments. The use of riprap at bridge piers, on the other hand, is not acceptable for new bridges and is considered only as a temporary countermeasure in the case of scour repairs. LADOTD employs the riprap design guidance and procedures presented in the FHWA's "Bridge Scour and Stream Instability Countermeasures" (HEC-23) and "Design of Riprap Revetment" (HEC-11).

b.) Guide Banks (Spur Dikes):

The purpose of guide banks is to align flow from the floodplain with the bridge opening and to minimize scour at the abutment by moving the scour-causing turbulence to the upstream end of the guide bank. The design guideline for guide banks should refer to HEC-23.

11.8 DECK DRAINAGE

11.8.1 Spread Standards

Effective bridge deck drainage is necessary to minimize the possibilities of vehicular hydroplaning and corrosion of the bridge structure. It is more difficult to drain bridge decks than approach roadways.

In general, for traffic safety and life of the structure, the spread of bridge deck drainage should not encroach on any portion of the designated traffic lanes. The design of the deck drainage system should conform to the guidelines and procedures described in FHWA's publication "Design of Bridge Deck Drainage" (HEC-21).

11.8.2 Scupper and Slot Drains

Scupper or slot drains must be adequately sized and spaced to remove rainfall-generated runoff from the bridge deck before it encroaches onto the traveled lanes.

11.8.3 Bridge End Drains

Because of the vulnerability of approach roadway shoulders and abutments to erosion from concentrated flow, sufficient bridge end drain capacity should be provided. The bridge-end drainage system comprises the inlets and the outfall pipes or ditches. The inlets should have sufficient capacity to intercept all flow from the bridge and intercept all flow from curbed roadways before it reaches the bridge. A closed conduit is often preferable to an open chute down to the abutment slope. Bridge end drain systems are pavement drainage devices.

HEC-21 provides detailed design methods and equations. Refer to EDSM II.2.1.1 for allowable materials.

11.9 DOCUMENTATION

Formal documentation of all design considerations, the general design process, and computations and analysis will be presented in the Bridge Hydraulics Report. At a minimum the Bridge Hydraulics Report will include: a brief summary; a general description of the watershed and the hydrologic characteristics of the area; a general statement of the scope and the reasons for the proposed project; documentation of any coordination with other agencies or governments; brief discussion of any local or regional ordinances influencing design; historical flood information and general site data used in hydraulic design; general discussion of design analysis (all hydrology and hydraulic calculations are documented in Appendices); final design recommendations; Hydraulic Data Table for the proposed structure (the design-year, 100-year, and overtopping floods are to be included in the Hydraulic Data Table, 500-year or overtopping flood for scour); and a general flood hazard summary. A typical Table of Contents of a Bridge Hydraulics Report is shown in Section 11.9.1.

11.9.1 Sample Table of Contents of a Bridge Hydraulics Report

Bridge Hydraulics Report

Preface

I. Summary

II. General Description of the Area

III. Site Data

IV. Interagency Coordination

V. Design Analysis

A. Hydrology

B. Hydraulics

(1) Existing Bridge Hydraulics

(2) Proposed Bridge Hydraulics

C. Stream Stability and Scour Analysis

VI. Recommendations

VII. Flood Hazard Summary

Appendix A USGS Gage Information

Appendix B FEMA Studies and FIRM panel

Appendix C Hydrology

Appendix D WSPRO for Existing Structure

Appendix E WSPRO for Proposed Structure

Appendix F Stream Stability and Scour Analysis

11.10 REFERENCES

- A.) Model Drainage Manual, Third Edition, AASHTO, 2005
- B.) Location and Hydraulic Design of Encroachments on Floodplains, FHWA, FAPG 23 CFR 650A, Federal-Aid Policy Guide, 1994
- C.) FEMA, National Flood Insurance Program and Related Regulations, 1987
- D.) Highway Drainage Guidelines, Fourth Edition, AASHTO, 2007
- E.) Bradley, J.N. Hydraulics of Bridge Waterways, HDS-1, FHWA, 1978
- F.) Shearman, J.O., User's Manual for WSPRO, FHWA, 1998
- G.) U.S. Army Corps of Engineers, HEC-RAS River Analysis System, User's Manual, Version 3.0, 2001
- H.) Highway Hydrology, Hydraulic Design Series No. 2, FHWA, Washington, D.C., 2nd Edition, October 2002.
- I.) Ensminger, P.A., Floods in Louisiana, Magnitude and Frequency, Fifth Edition, 1998
- J.) Richardson, E.V., Evaluating Scour at Bridges, HEC-18, FHWA, 2001
- K.) Lagasse, P.F., Stream Stability at Highway Structures, HEC-20, FHWA, 2001
- L.) Lagasse, P.F., Bridge Scour and Stream Instability Countermeasures, HEC-23, FHWA, 2001
- M.) Brown, S.A., Design of Riprap Revetment, HEC-11, FHWA, 1989
- N.) Young, G.K., Design of Bridge Deck Drainage, HEC-21, FHWA, 1993